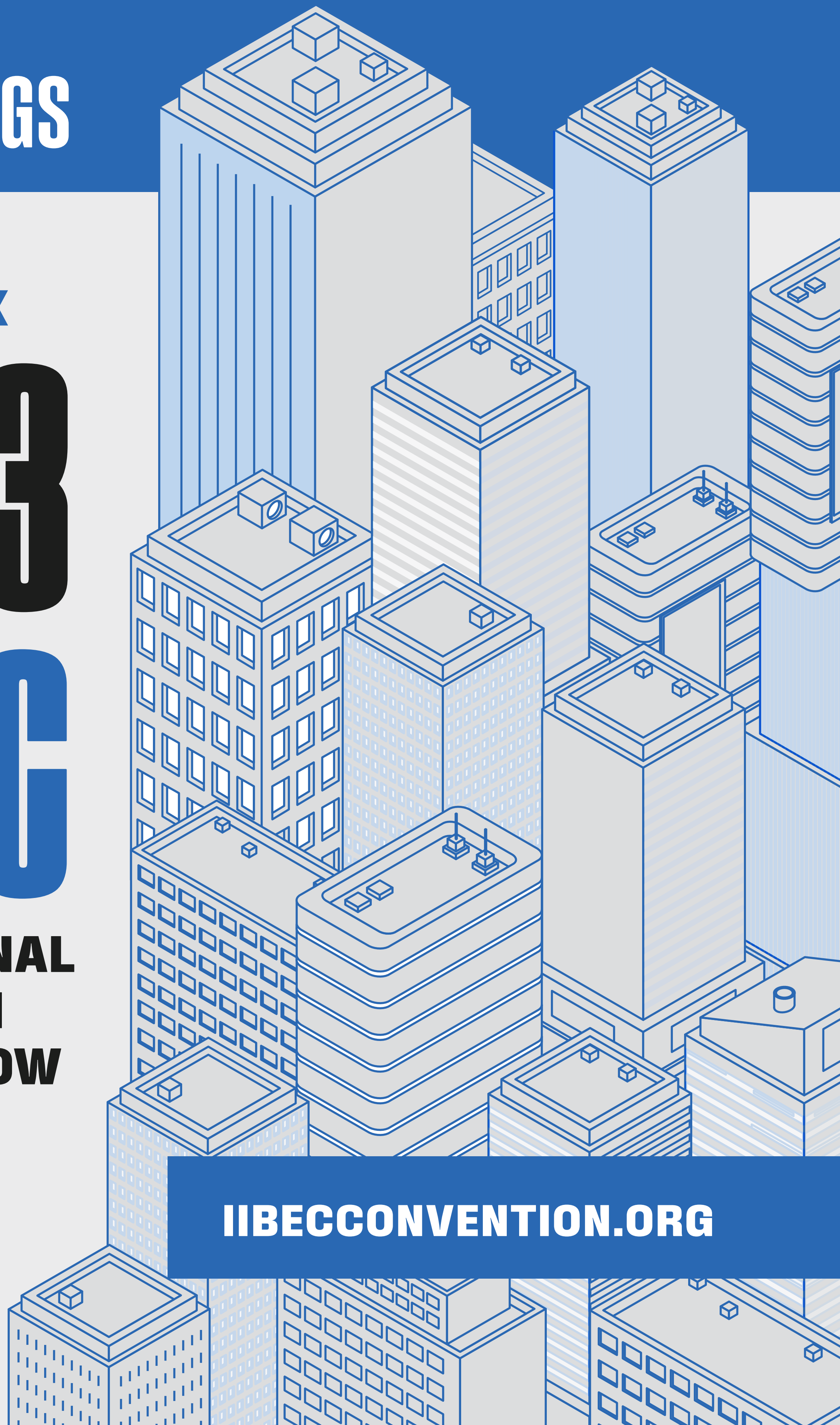
PROCEEDINGS

MARCH 3-6
HOUSTON, TX

2023 IIBEC

INTERNATIONAL CONVENTION & TRADE SHOW



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2023 IIBEC INTERNATIONAL CONVENTION & TRADE SHOW

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STUCCO ON AN ISLAND FAR, FAR AWAY: LESSONS LEARNED THROUGH INVESTIGATING STUCCO FAILURE

ABSTRACT

Labor shortages are having a global effect on construction efforts. For example, according to Michael Bellaman, president and chief executive officer of the Associated Builders and Contractors, the US construction industry "desperately needs qualified, skilled craft professionals to build America."1 The United States has a population of over 320 million. By comparison, the population of the British Virgin Islands (BVI) is about 30,000. In 2018, the BVI Department of Trade, Investment, Promotion and Consumer Affairs held a construction industry workshop with local a Architectural Firm and Businessman, who stressed "If certain practices are not improved, the void in skill levels and professionalism that exist will be filled by people from the outside. As a result, the workshop is self-preservation"2. The limited size of the skilled labor force within this smaller population, which has fewer resources for proper training, significantly affects the overall delivery of projects in the BVI. In construction, proper design and installation are essential to ensure that the building systems function properly. The Director of Trade, Mrs. Karia Christopher also recognized this challenge and stated: "The aim of the workshop was to have the total process and the understanding that the entire team has to work in tandem. For instance, the architectural services, the construction, trucking and heavy equipment, everything needs to work together to have a satisfied customer; not limited to residential, but commercial as well"2. For the building in this case study, the use of skilled laborers and professionals from outside the islands could have been a partial solution to supplement available local resources. However, the expenses for travel and lodging from the design phase through construction for the project team members may have been a restricting factor.

LEARNING OBJECTIVES

- Discuss the challenges of fieldwork outside of the United States.
- Understand petrographic analysis uses in facade evaluation.
- Summarize the investigation and evaluation of Portland cement stucco.
- Recommend ways to repair and replace Portland cement stucco using industry-accepted best practices.

SPEAKERS

Vu T. Nguyen, EI, RRC, RWC, BECxP, CxA + BE

Vu Nguyen is a Senior Project Manager with 26 years of experience in the Building Enclosure industry. Vu has provided design, consultation, commissioning, survey and management of various types of building enclosure projects. His daily responsibilities include maintaining design production schedules and construction budgets, coordinating pre-bid and pre-construction conferences, progress meetings and evaluating submittals. In addition to his role as a Senior Project Manager, Vu also holds a position as an Authorized Project Reviewer, which is a selective in-house quality assurance role to ensure consistency throughout Terracon projects. As an APR, Vu is a key team member in many projects from beginning to end, however, he is most heavily involved at critical junctures. Vu most enjoys forensic projects when he is able to investigate long term building enclosure failures by tearing structures apart, detect the source of the problem and ultimately offer an innovative design solution that best suits his clients' needs.

AUTHORS:

Vu T. Nguyen, EI, RRC, RWC, BECxP, CxA + BE

Stewart Swing, RRC, RWC

2023 IIBEC INTERNATIONAL CONVENTION & TRADE SHOW

In 2010, a five-story landmark office building was constructed using local skilled labor in Road Town on the island of Tortola. For the purposes of this paper, this building will be referred to as the Tortola Office Building. Shortly after its construction, the building's portland cement plaster (stucco) facade was observed to be failing (**Fig. 1**).

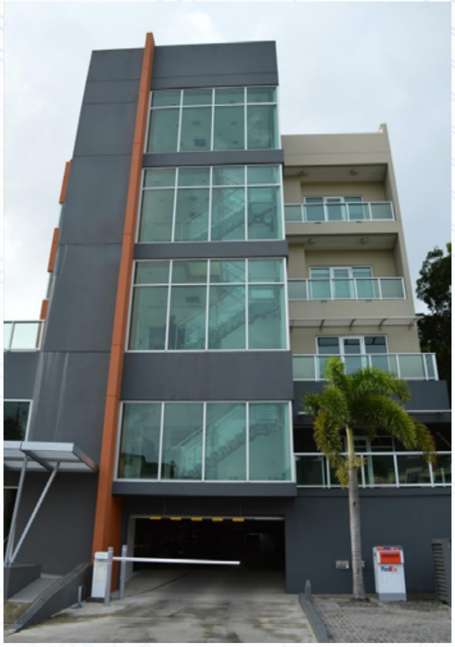
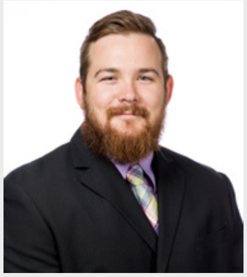


FIGURE 1. Overview of the Tortola Office Building.

Efflorescence, discoloration, and heavy water stains were prominent on the surface of the wall cladding, indicative of water infiltration behind the stucco. Map cracking and shrinkage cracks were observed throughout the building facade, and the finished coating was blistered. The most hazardous evidence of failure was the debonding and separation of the plaster from the building, which led to large pieces of falling debris over pedestrian pathways. After numerous attempts were made to repair the failing facade, Terracon Consultants, Inc. was requested in 2018 to investigate the facility to determine the cause of failure, perform a survey assessment, and provide recommendations for remediation. Terracon is a consulting firm that specializes in Building Enclosure Systems. The group that performed the investigation was based out of Charlotte, North Carolina.

There are many factors that contribute to the successful delivery of a project, and perhaps many more that can lead to disastrous results. To ensure success on a project using a portland cement plaster cladding system, stakeholders must carefully plan and execute product selection, design detailing, substrate preparation, and mixing and application of the materials that make up the system. In this paper, our discussion of the Tortola Tortola Office Building will focus on the challenges the investigative team encountered while working outside of the United States, investigation techniques used to evaluate the stucco on the structure, best practices for proper design and installation of directly applied portland cement, and the repair recommendation strategies to remedy the structure.



Stewart Swing, RRC, RWC

Stewart Swing is an instrumental senior staff engineer with strong communication, time management and leadership skills with 8 years of field experience. His first year in the field, he worked with a foreman learning the trades of waterproofing. Since then, he has developed a notable enthusiasm for waterproofing and has continued to accumulate knowledge of building enclosures. His daily responsibilities include preparation of specifications and drawings, contract administration, and construction observations. Swing most enjoys forensic projects when he has the opportunity to investigate long term building enclosure failures by tearing structures apart, detect the source of the problem and ultimately offer an innovative design solution that best suits his clients' needs.

DEFINITIONS: PORTLAND CEMENT PLASTER AND STUCCO

The terms "plaster" and "stucco" are often used interchangeably, but the stucco is a hardened state of plaster that is used specifically for exterior cladding. Stucco can be a desirable option as a cladding system. It is a durable material that can be aesthetically pleasing, available in a variety of colors and textures. Some may say it is a simple system, both in its design detailing considerations and in its application in the field. But designers, building enclosure consultants and others working with stucco still require an understanding of the properties and behavior of the materials that make up the cladding as well as an understanding of the various types of building construction substrates and the transitions that can occur between them.

STUCCO IN THE BRITISH VIRGIN ISLANDS

In locations such as the BVI where the climate is hot and humid, stucco is a very common system for building cladding, even though the climate conditions present construction and maintenance challenges. The materials needed for stucco application are readily available on the island. The warmer and humid climate provides an environment favorable to mixing, application, and curing of the stucco. Colder and dryer conditions require extra measures, like maintaining wet surfaces during application, and temperature application limits. (Note: Stucco can be also applied in cold climates, but guidelines that should be followed for those climate conditions are beyond the scope of this paper.) Given that stucco is a common system for buildings in the BVI, it might be presumed that there should be an abundance of "stucco know-how" among the professionals who design for the area, as well as those who erect buildings with stucco installations. But if that is the case, what went wrong on this project, and what caused the failure of the stucco at the building? The factors that contributed to the failure of the stucco appeared to be related to poor mixing of the stucco material components and substrate conditions that were not properly prepared. As Terracon did not review any documents related to the construction administration aspects of the project, quality control and assurance is at the very least suspect. Proper quality control, assurance, and oversight could have been the difference in achieving the proper preparation and installation.

STUCCO STANDARDS

Several organizations, such as the Portland Cement Association[3], National Concrete Masonry Association[4], and Stucco Manufacturers Association[5], have set standards recognized in the United States for the design and installation of stucco cladding systems. ASTM International standards include ASTM C926, *Standard Specification for Application of Portland Cement-Based Plaster*,[6] and ASTM C1063, *Standard Specification for Installation of Lathing and Furring to Receive Interior and Exterior Portland Cement-Based Plaster*.[7]

Stucco is typically composed of portland cement–based materials and sand and/or aggregates mixed with water. Directly applied stucco relies on the continuity of the material as weather-resistive barriers are not generally installed on the substrate before the stucco materials are applied.

Stucco systems are generally installed as either two- or three-coat systems. A two-coat system is composed of a base coat and a finish coat, whereas a three-coat system has a base coat, a brown coat, and a finish coat. The plaster for a two-coat system is typically about ½ in. (12.7 mm) thick.

Directly applied stucco systems are typically applied over cast-in-place concrete, concrete masonry, or clay masonry. Stucco can also be applied to metal lath that spans open framing, or metal lath over sheathing material. The stucco assembly for the Tortola International Pancake House consists of a two-coat system constructed over cast-in-place concrete with concrete masonry unit infill. The stucco is applied directly to those substrate materials and does not have a built-in drainage plane.

DIRECTLY APPLIED STUCCO AS A BARRIER SYSTEM

Stucco in a directly applied configuration is considered a barrier system. This type of application has been successful for many buildings for many years. As building design and construction become more complex, the requirements for air- and watertightness become even more critical, and a barrier system without means for water management becomes increasingly at risk for water infiltration problems. Given the issues encountered if a stucco system is not properly designed, detailed, and installed, designers are increasingly recognizing that providing a drainage plane behind the stucco is beneficial for water management if the cladding material cracks. In a system that is installed with a drainage plane, a means for the water to exit the cladding system is typically provided through weeps at the base of the material terminations. In the absence of a drainage plane, water that penetrates the stucco will result in moisture intrusion, which may find its way into the building interior. Where stucco is installed as a barrier system (that is, without a water-resistant barrier at the drainage plane), the stucco material itself functions as the waterproofing system. When redundancy is not provided by a drainage plane for water management, it becomes more critical for stucco barrier systems to be designed, detailed, and installed properly.

STUCCO PROPORTIONAL MIXTURES

The "ingredients" of stucco should be considered carefully because the proportions of portland cement, sand, aggregates, and water determine the behavior of the stucco system within various climates. ASTM C926[2]and the National Concrete Masonry Association[5] provide the guidelines for proper mixing, the types of components to use, and correct proportions. In addition to climate conditions, the type of substrate should also be considered when selecting the materials. For example, concrete masonry units are cement-based components and are highly compatible with portland cement

FIGURE 2. Concrete beam between the first and second floor where metal lath was used with stucco.

stucco applications because natural "suction" promotes bonding between the stucco and the concrete masonry unit substrate.

Substrates such as brick absorb less moisture than concrete masonry units and therefore require different types of plaster coats. In cases where metal lath is engineered as part of the system, requirements for the plaster mixture components and proportions can also vary. In addition to the proportions of cement and sand, the volume of water must also be properly controlled during the design and installation. Water proportions in the mixture design can affect the curing, strength, and workability of the stucco, as well as other aspects of stucco performance. Where stucco systems are improperly designed or installed on a project, the risk of premature failure increase. In our experience, material shrinkage caused by improper proportioning, mixing, or curing is generally one of the largest contributors to stucco system failure.

STUCCO JOINT PLACEMENT

One of the most critical considerations for proper stucco installation is placement of joints. Without proper joint placements, cracks can develop due to shrinkage stresses, building movement, foundation settlement, differences between the behaviors of the stucco and the material substrate, weakened sections in walls (such as at fenestrations), and thermal changes. For directly applied systems, the rule of thumb is to install control joints in the stucco directly in line with control joints on the building. In addition, stucco directly applied to a consistent masonry base usually performs well and minimizes the potential for cracks to develop unless there are bonding issues. In the original design drawings for the Tortola International Pancake House, control joints were shown on the elevation plans for the stucco cladding, primarily at the substrate transition where concrete masonry units were used as infill between the structural concrete floor slabs. The installed stucco system included horizontal reveal joints placed along those areas where materials varied. The building design did not have expansion joints in the structure, and vertical joints shown in the drawings for the stucco cladding were not installed. ASTM C1063[7] states that stucco systems should have joints placed for dimensions exceeding 18 ft (5.5 meters), no panel size should be more than 144 ft^2 (1.4 m^2), and the ratio of length to width should not exceed 2.5 to 1. At some of the locations on the Tortola Office Building, we observed metal lath behind the stucco and found that it was not continuous. Where the lath was discontinuous, the stucco cladding was continuous but transition details such as reveal joints or accessories for movement were not provided. Application of the stucco over metal lath did not seem to comply with the ASTM C1063[7] requirements, particularly where the concrete beam exists between the first- and second-floor levels (see **Fig. 2**).

STUCCO EVALUATION

When evaluating stucco systems, a thorough review of the exterior should be performed to identify and locate evidence of cracking and discoloration. These types of deficiencies can be signs of distress, and, if not properly addressed, they have the potential to multiply exponentially throughout the rest of the facade over time. Where cracking and discoloration are observed, further investigation is necessary to determine the cause of the distress. As deficient conditions are identified, proper sounding of the stucco should be performed to determine whether the material has become debonded. Debonding on the structure can generally be attributed to poor substrate preparation, out-of-plane substrates, improper mixture proportions, poor application of the stucco, or a combination of these factors.

To properly evaluate stucco, it is critical to have access to areas with noted distress. Unfortunately, gaining safe access is more difficult in places such as Tortola where worksite safety is not as regulated as in the United States. During our research, we were unable to find an agency in Tortola that governs the safety aspect of construction sites in a manner comparable to the US Occupational Safety and Health Administration.

In the BVI, it can also be challenging to locate a properly trained operator for equipment such as lifts to access high areas, and coordinating equipment requires greater effort than in the United States since phone or online communication with your favorite contractor or equipment supplier is not always feasible. In Tortola and similar places with limited resources, it is therefore critical to ensure that those staff members from your firm who are tasked with performing on-site work are thoroughly trained in safe equipment operations and to recognize hazardous conditions. Extra time and coordination may be needed to secure the resources to safely access areas of interest. Personal safety equipment such as harnesses and lanyards, hard hats, gloves, protective eyewear, and

FIGURE 3. Efflorescence and discoloration of stucco facade.

FIGURE 4. Cracks observed in the stucco facade.

steel-toed shoes is not necessarily readily available at a local hardware store. Next-day delivery service is also not a certainty. Therefore, generating a checklist of supplies and having a backup plan can mean the difference between sitting pretty and being a sitting duck.

As noted earlier, the defects observed on the Tortola Office Building included cracks, leaching of white deposits (efflorescence), blisters, and discoloration (**Fig. 3** and **4**).

Efflorescence is usually a sign that water entered behind the waterproofing barrier and then found a pathway back out, transporting salts contained within the stucco mixture to the exterior. The salt formation can also add to the stress on the bond line.

Leaks to the interior space, primarily around windows, were also reported. Previous repairs to the stucco seemed to have addressed some of those leaks. However, following these localized repairs, areas of cracking continued to progress to other sections of the building. Therefore, spot repairs would be an impractical long-term strategy.

During the investigation of the Tortola building, the team closely examined the stucco at locations where deficiencies occurred. Building elevation drawings were used to map the locations of the defects and identify the various types of deficiencies and their general locations (**Fig. 5**). Large sections of stucco were sounded and determined to be debonded; also, in isolated sections, the cladding had fallen off the structure.

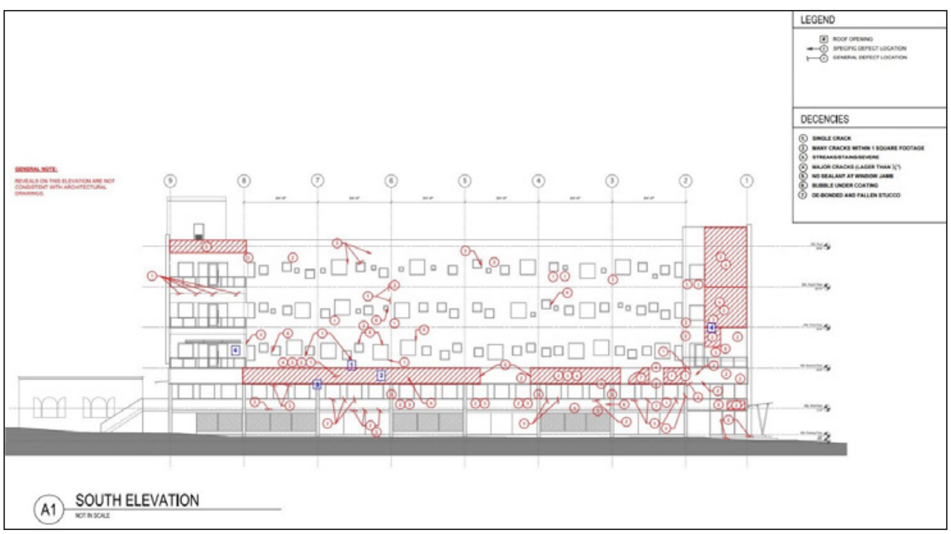


FIGURE 5. Elevation drawings were developed to document the locations and various different types of defects observed in the stucco.

Destructive openings (generally 12 × 12 in. [305 × 305 mm]) were performed at 11 locations. At the destructive openings, several observations indicated that substrate conditions partially contributed to the debonding of the stucco. The cast-in-place concrete beam at floor slabs was extremely smooth to the touch, which is not ideal for achieving a good bond with the stucco. Several methods can be used to enhance bonding, including the use of bonding agents or surface preparation techniques that mechanically roughen the substrate such as sandblasting and high-pressure water blasting. Bonding agents can be applied to the substrate surface or integrally mixed into the stucco. It is unclear whether bonding agents were used for this project. A pink residual material was observed on the concrete surface at some of the locations where debonding had occurred, but that material could not be identified. As shown in **Fig. 6**, the concrete substrate was also out of plane by as much as ½ in. (12.7 mm). ASTM C926[6] states that concrete substrates "shall be straight and true within ¼ inch in 10 ft." The stucco material thickness (**Fig. 7** and **8**) was also observed to vary from 1 to 1½ in. (25.4 to 38.1 mm).

The inconsistency of the material thickness likely caused differential shrinkage, which in turn led to cracking at some of the areas observed. Cracking typically occurs at the thinnest portion of the stucco cladding. At the concrete bands between the second and third floors of the Tortola Office Building, an approximately 12-in.-wide (305-mm) water table exists. The stucco

FIGURE 6. A level/straight edge was used to show the precast concrete substrate behind the stucco was out of plane.

FIGURE 7. Stucco thickness was measured to be approximately 1 inch thick.

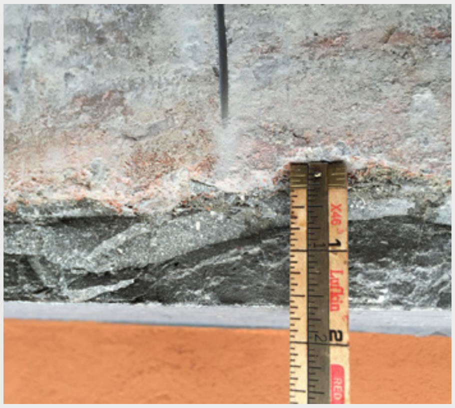


FIGURE 8. Stucco thickness was measured to be approximately 1.5 inches thick.

application in the water table area was thicker than that at the vertical face. One of the worse areas of observed cracks was located where varying thicknesses of stucco combined with a change in plane at a building corner. Thicker areas of stucco tend to move differently from thinner areas, and the same applies to different planes (**Fig. 9**).

In field testing, sounding with a hammer was conducted. A hollow sound was heard at cast-in-place concrete substrate areas, and various sounds ranging from hollow sounds to ringing occurred at the concrete masonry substrate locations. Hollow sounds indicate that the concrete is likely not well bonded, and a ringing sound typically indicates that the stucco material is likely bonded.

At most of the 11 destructive openings, the stucco was removed in large, whole pieces with relative ease, indicating that failure was occurring at the bond line. No joint accessory was observed at the reveals. Joint accessories can be used to divide wall areas and control cracking as well as to control the stucco thickness (**Fig. 10**). ASTM C926[6] provides a list of waterproofing materials behind control joints for consideration, as this is where cracking is likely to occur.

PETROGRAPHIC ANALYSIS

As part of the analysis of the building, investigators used samples removed from destructive openings to perform petrographic analysis in accordance with ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete*,[8] and chemical testing in accordance with ASTM C1324, *Standard Test Method for Examination and Analysis of Hardened Masonry Mortar*.[9] The testing objectives were to determine the condition, composition, and probable causes of failure of the installed materials.

A lesson learned from the Tortola experience is that the number of samples to remove and retain should be planned when investigating projects in remote locations. Stucco weighs roughly 10.5 lb/ft^2 (51.25 kg/m^2), and the 11 samples taken on this investigation from the destructive openings weighed over 100 lb (45 kg). Packing this extra weight in luggage and lugging it around airports proved to be a tiresome and physically exhausting task.

After the samples were delivered to the laboratory, they were prepared for testing by using diamond polishing discs to grind and polish the sawn

FIGURE 9. Stucco on water table is cracked.

FIGURE 10. Destructive opening at a reveal of the stucco. No joint accessories were observed.

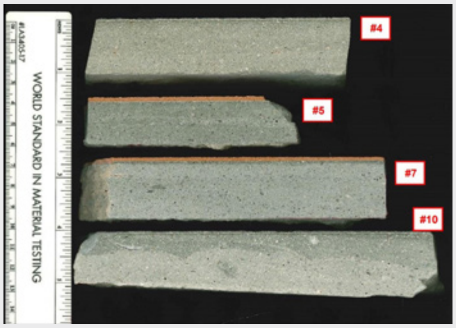


FIGURE 11. Laboratory testing: samples were prepared by using diamond polishing discs to grind and polish. *Report of Stucco Evaluation and Coating Testing March 9, 2017 Office Building (Tortola - British Virgin Islands) TEC Services Project No. 17-0369.13*

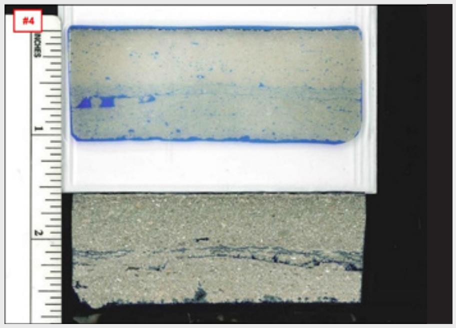


FIGURE 12. Laboratory testing: thin sections were cut as part of the evaluation procedures. *Report of Stucco Evaluation and Coating Testing March 9, 2017 Office Building (Tortola - British Virgin Islands) TEC Services Project No. 17-0369.13*

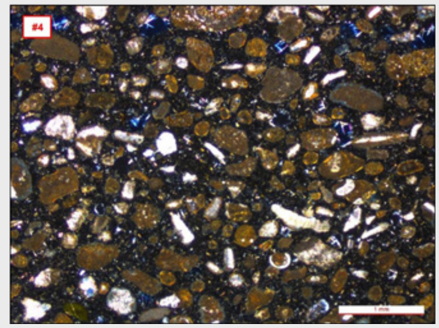


FIGURE 13. Laboratory testing: magnified samples using a polarized light microscope to view stucco components. *Report of Stucco Evaluation and Coating Testing March 9, 2017 Office Building (Tortola - British Virgin Islands) TEC Services Project No. 17-0369.13*

surface or prepared as thin sections (**Fig. 11** and **12**).

Four samples were evaluated. Three samples were from locations where precast concrete was the substrate, and one sample was from a location where concrete masonry unit was the substrate. The evaluation was focused on the scratch coat of the stucco samples to determine whether the quality of bonding was affected by the stucco composition.

The thin-section samples were encapsulated in a blue dyed epoxy and allowed to cure. Following the sample curing, the sawn surface was ground flat and adhered to a glass slide where the material was saw cut from the slide surface and polished. The materials were then examined using a polarized light microscope capable of magnifications of 400X and a digital microscope capable of magnifications up to 200X (**Fig. 13**).

In review of the samples, it was noted that the base coat was thicker than the nominal plaster thickness of $\frac{1}{4}$ in. (6.4 mm) specified in ASTM C926[6] for three-coat work applied over precast concrete. In the samples reviewed, the base coats varied in thickness from $\frac{3}{8}$ to $\frac{5}{8}$ in. (9.5 to 15.9 mm). Trowel marks and bonding agents were not observed in the samples.

The samples were composed of various materials. Two types of sand were observed: (a) natural sand primarily consisting of quartz, and (b) marine limestone consisting of fossiliferous limestone, which contained forams. The natural sand particle-size distribution fell within the allowable limits of ASTM C144, *Standard Specification for Aggregate for Masonry Mortar.*[10] However, the marine limestone had finer particles that fell outside of the ASTM C144 limits. For both samples, the composition of the base coat was determined to be under-sanded (less sand than specified) and not well graded.

The binder used in each sample was determined to be a blend of portland cement and ground limestone. The hardness of the samples indicated the masonry to be consistent with a Type M mortar mixture. Samples were compared against control samples, and the comparisons showed that the sample materials had a higher than typical volume of paste. Paste carbonation was also present on numerous samples, indicating that an airway was present at the bond line of the material and the substrate. Samples also exhibited microcracking, which can occur when material is applied over a dry substrate, causing the material to dry too quickly.

No entrained air voids were found in the stucco samples. The air voids noted in the stucco were irregular entrapped voids. The average air content of the stucco scratch coat was 3.4%, with a range from 2.5% to 4.4% (based on point-counting data). Interconnected voids were occasionally present at the bond lines of the individual stucco layers. Most of the voids in the stucco, including voids near the bottom of the scratch coat, were partially filled with secondary deposits. The secondary deposits were typically calcium hydroxide and ettringite. These deposits are an indication that water was passing through the stucco.

SUMMARY OF FINDINGS

The following is a summary of the on-site and laboratory test findings:

- Deficiencies observed in the stucco included cracks, discoloration/efflorescence, and debonding.
- Thickness of the stucco varied significantly.
- The sand proportions in the stucco mixture were not sufficient.
- The substrate was not properly prepared.

Cracking led to water intrusion, which led to degradation of the stucco and damaged its adhesion to the substrate. Over time, the cracking continued due to shrinkage, debonding, and differential movements, and, eventually, issues related to water intrusion into the building interior space became highly problematic: efflorescence, blisters, and discoloration affected the building aesthetics, and debonding of the material to the substrate became so severe that stucco pieces started falling off the building.

RECOMMENDATIONS

The client desires a uniform appearance, a watertight cladding system with redundancy in resisting and managing water, and a long-term solution to minimize the maintenance required. The recommendations made for this project therefore include the complete removal and replacement of the existing stucco. The exposed substrates, the concrete masonry units, and the precast concrete should all be properly prepared for application of a new two-coat, directly applied stucco system with a fluid-applied water-resistant layer as a secondary drainage plane. High-quality ready-mix packaged materials are recommended to minimize the use of inferior or incompatible materials in constructing the system. Accessories with flashing at terminations, reveals, and intersections should be installed, and weeps should be incorporated for water management at the drainage plane. Control joints should be evaluated and used at substrate material changes. All sealants at joints and window perimeters should be replaced with the new cladding system. Coping caps at parapet walls will also need to be replaced (reuse of the existing caps may not be feasible since damage can occur during removal and reinstallation). Finally, a new finish should be applied to match the general appearance of the current system.

CONCLUSION

If this is your first time reading a case study involving stucco, you might conclude that stucco is a risky system to have as an exterior building cladding. However, a better conclusion is that redundancy in a waterproofing system as well as proper design, material selection, preparation and installation are all crucial to a successful installation, even for a system as "simple" as stucco. A project is underway to implement the recommendations.

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DIGGING DEEP: WATERPROOFING DEEP FOUNDATIONS FOR NEW CONSTRUCTION

ABSTRACT

The deeper the foundation, the higher the risk for water infiltration. Below-grade waterproofing systems are a critical part of the overall building enclosure and should be carefully selected to properly protect the structure and interior below-grade space. This presentation will discuss the challenges specific to deep foundation waterproofing as well as provide best practices for various below-grade elements with which the waterproofing will interface. New construction project examples and scenarios will be included to elaborate on the challenges, discussions, and solutions.

LEARNING OBJECTIVES

- Apply key principles used in determining below-grade waterproofing selections.
- Identify project decisions that are critical to the development of below-grade waterproofing systems for deep foundations.
- Describe an appropriate below-grade waterproofing system based on varying site conditions and structural foundation types.
- Predict construction challenges that may arise during the installation of below-grade waterproofing systems for deep foundations.

FIGURE 1. Waterproofing at a deep foundations site.

SPEAKER

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Below-grade waterproofing systems can be critical features of building enclosure design, particularly when the structure has a deep foundation. As the foundation goes deeper, there is a greater likelihood that it will encounter the groundwater table and hydrostatic conditions, which makes choosing the right below-grade waterproofing system even more important (**Fig. 1**). This paper discusses several noteworthy design considerations for below-grade waterproofing for new construction, the types of below-grade systems available, and specific concerns associated with waterproofing deep foundations.

DESIGN CONSIDERATIONS

When selecting and designing a below-grade waterproofing system, the primary factors to consider include the type of foundation system, the project's site conditions, product performance properties, and construction sequencing. The design specifications for a building's foundation are influenced by the building size and structural system, as well as the site's soil and geological composition. Foundation systems are typically either categorized as shallow foundations or deep foundations. Shallow foundations are used when the structural loads can be adequately distributed to a relatively shallow level of soil. Examples include structural slab-on-grade foundations, footings, and grade beams. Deep foundations are used when there are more significant loads that need to be transferred to deeper soil or bedrock. These types include piles, caissons, and drilled shafts, among others.

While the structural engineer is responsible for the foundation design, the waterproofing consultant needs to understand the type of foundation being used to anticipate the various interfaces and conditions the below-grade waterproofing design will need to reflect. The proximity of the foundation to the groundwater table and potential presence of soil contaminants are critical factors to evaluate, and they can substantially influence decisions about the type of waterproofing membrane to select for the foundation elements.

To obtain a general idea of how high the groundwater table can be at the project site, one should review the geotechnical investigations report. These investigations are typically performed at the early stages of the project and can involve various subsurface and soil assessment methods, including borings. The report may document whether groundwater was encountered in any of the borings, and it may identify the historic high-water-table elevation or provide a recommended design groundwater table elevation. This elevation should be compared with the building's foundation elevation, as well as the lowest of any foundation elements (such as the grade beam, footing, elevator pit, or sump) to ascertain whether hydrostatic conditions should be assumed for this project. When reviewing the geotechnical investigations report, one should stay cognizant of the fluctuating nature of groundwater elevations and understand that the geotechnical investigations typically capture one specific moment in time. While the report can provide good insights, it does not always paint the full picture of the site's hydrogeology.

The geotechnical investigations report can also provide insight into past uses of the site, and whether contaminated soils are present. For example, former gas stations and auto repair shops are likely to have petroleum or methane contaminants in the soil. Other contaminants can include acid and alkaline water, insecticides, and fertilizers.[1] In coastal regions, subsurface saltwater is another consideration; the presence of subsurface saltwater may warrant the use of specific versions of waterproofing membranes designed for these conditions.

When selecting the appropriate below-grade waterproofing system for the project, one should also consider the owner's budget and what they consider to be a level of acceptable risk. This information will inform your decisions and recommendations regarding certain waterproofing product features, such as whether membranes are fully welded, adhered, or taped. Construction sequencing may influence items like backfilling and soil retention systems, and by understanding the construction project's timeline, you can guide the selection of waterproofing products with application methods appropriate for the sequencing. This issue will be further discussed in the following section.

BELOW-GRADE WATERPROOFING MEMBRANE SYSTEMS

Before diving into the different below-grade waterproofing product options, it is necessary to understand the difference between damp-proofing and full waterproofing membranes. Damp-proofing is defined as "the treatment of a of a surface or structure to block the passage of water in the absence of hydrostatic pressure."[1]

FIGURE 2. Pre-applied blindside waterproofing installed prior to pouring of concrete wall.

FIGURE 3. Installation of post-applied waterproofing membrane.

Some vapor retarders are considered a form of damp-proofing. In below-grade applications, vapor retarders are most commonly thinner sheets of polyethylene plastic intended to prevent the transmission of moisture vapor. Vapor retarders offer varying degrees of protection from vapor transmission, depending on the material's vapor permeability. Vapor retarders with the lowest range of permeability are often referred to as vapor barriers, with vapor permeance values of 0.1 perm or less.[2] ASTM E1745, *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*, provides three classes for vapor retarders, all of which share a maximum water vapor permeance of 0.1 perm but have different levels of tensile strength and puncture resistance.[3]

The 2021 *International Building Code*[4] (IBC) specifies under-slab damp-proofing to be polyethylene sheets of a minimum thickness of 6 mil with joints lapped a minimum of 6-in. (152 mm). These vapor retarders are typically recommended in non-hydrostatic conditions where there is a low probability of direct contact with groundwater. Per the IBC, the parameters of hydrostatic conditions occur when the "existing groundwater table is above or within 5 feet (1524 mm) below the elevation of the *lowest floor* level where such floor is located below the finished ground level adjacent to the foundation." [4] Seams between sheets of vapor retarder are usually taped. Flashing at penetrations and terminations may use accessory tape, mastics, and/or termination bars.

Below-grade waterproofing membranes are intended to protect the foundation elements from groundwater intrusion, especially in hydrostatic conditions. While waterproofing membranes can be vapor retarders or even vapor barriers, vapor retarders may not always act as waterproofing membranes capable of resisting water migration. Most below-grade waterproofing membranes can be classified in the following categories: bentonite, thermoplastic, composite, self-adhering, or fluid applied. For sheet waterproofing membranes, the membrane seams may be taped, adhered, hot air welded, or torch applied, depending on the product technology. The membrane manufacturers typically have specific accessory products to be used with the waterproofing membrane to form a complete below-grade waterproofing system. The IBC requires under-slab waterproofing membranes to be either "rubberized asphalt, butyl rubber, fully adhered/fully bonded HDPE [high-density polyethylene], or polyolefin composite membrane or not less than 6-mil polyvinyl chloride."[4] IBC requires that waterproofing on below-grade walls extend a minimum of 12 in. (305 mm) above the maximum elevation of groundwater table.

In most cases, the 2021 IBC requires as a minimum a 6-mil polyethylene vapor retarder under concrete floor slabs where hydrostatic conditions do not occur. [4] Hydrostatic conditions are considered when the groundwater table is above or within 5 feet (1524 mm) below the lowest subsurface floor elevation. Waterproofing is required per the IBC when the geotechnical investigation indicates hydrostatic conditions and a groundwater control system is not provided in the project design. For shallow foundations in non-hydrostatic conditions, a vapor retarder may be all that is needed. For deep foundations, there is a greater likelihood that hydrostatic conditions can be encountered, and full waterproofing membrane systems would therefore need to be used. Even in deep foundations with non-hydrostatic conditions, it may be prudent to include waterproofing membrane on vertical foundation elements such as walls, as most polyethylene vapor retarder products are primarily intended for horizontal under-slab applications.

In new construction projects, below-grade waterproofing membranes can be installed before foundation elements are constructed; these systems are referred to as pre-applied or blindside applications (**Fig. 2**). Alternatively, the below-grade waterproofing membranes can be installed after foundations are in the ground; these systems are called postapplied systems (**Fig. 3**). When choosing between preapplied and postapplied membranes, the decision is usually driven by construction logistics, site constraints, and foundation construction methods. Installation of postapplied membranes requires soil excavation to keep foundation elements exposed after the concrete has been cast. That may be a reason to choose preapplied membranes for some sites because deeper foundations would require a wider footprint of soil excavation if post-applied membranes were to be used. If shotcrete is being used on foundation elements in contact with the below-grade waterproofing membrane, preapplied membrane systems are the only option.

PREAPPLIED (BLINDSIDE) BELOW-GRADE WATERPROOFING MEMBRANES

There are several types of preapplied (blindside) waterproofing membranes, including bentonite, thermoplastic, and composite. Bentonite membranes typically consist of sodium montmorillonite clay granules integrated into geotextiles. Some products include an HDPE layer and a polypropylene layer for additional waterproofing protection. A distinguishing feature of bentonite membranes is that bentonite swells when it comes into contact with water, creating a monolithic barrier against the foundation. Bentonite can swell up to 15 times its original dry volume, and it has the ability to repeatedly re-swell when dried and hydrated again. To be fully effective, bentonite waterproofing membranes must be compressed against the concrete foundation elements to form a mechanical bond. Seams can be mechanically fastened, or adhered using adhesive or mastic products, depending on the manufacturer.

Thermoplastic waterproofing membranes for foundations are made from materials similar to those used in thermoplastic roofing products. Polyvinyl chloride (PVC), ketone ethylene ether (KEE), and thermoplastic polyolefin (TPO) along with reinforcing fabrics are key components of the waterproofing membranes. The below-grade thermoplastic membranes tend to be in thicker forms than their roofing counterparts. Supplemental layers such as polymers or butyl adhesives provide the bonding mechanism that allows the membrane to integrally bond with the concrete foundation cast against it. Some thermoplastic membranes may also use HDPE. Seams are either heat welded or taped.

Composite membranes consist of several layers combined in one membrane, typically including an HDPE layer, a protective coating layer, and an adhesive layer. Seams are self-adhered together. While each layer serves an intended purpose in the waterproofing membrane, there is a possibility of delamination between the individual layers, as the component in contact with the foundation is the only one fully bonded to the concrete.

Other preapplied below-grade waterproofing products include modified bitumen-based membranes, such as styrene-butadiene-styrene (SBS) polymer, often with a polyester or fiberglass reinforcing layer. These tend to be thicker membranes, and their seams are heat welded.

POSTAPPLIED BELOW-GRADE WATERPROOFING MEMBRANES

Postapplied waterproofing membranes can be either self-adhered or fluid applied onto the foundation. Since the waterproofing system is usually designed to be continuous around the foundation, installers may need full access to the exterior surfaces, and achieving that access may involve a significant amount of excavation. Self-adhering waterproofing membranes designed for below-grade applications typically consist of rubberized asphalt with an HDPE film. When using self-adhering membranes, primers may still be required to achieve proper adhesion to the concrete. Membranes should be kept clean and free of dust and debris, particularly at laps, as the waterproofing system relies on strong membrane-to-membrane bonding at seams to maintain full effectiveness.

Fluid-applied waterproofing designed for foundations can be trowel grade or formulated for spray applications, and they are asphalt emulsion based and polymer modified. Because these products form monolithic waterproofing layers during installation, fluid-applied systems do not share the risk of seam failure that sheet membranes inherently possess. Maintaining a consistent, uniform thickness throughout the application process is the primary challenge with fluid-applied waterproofing membranes.

DEEP FOUNDATION CHALLENGES

When site conditions and project design warrant the use of deep foundation systems, the structural components that the design team chooses may increase the complexity of the below-grade waterproofing design. Soil retention systems and any related elements that may disrupt the continuity of the waterproofing should be addressed and detailed during the design phase. Even supplementary items, such as concrete protection slabs, should be accounted for in the overall waterproofing strategy and discussed with the waterproofing manufacturer to ensure all items with which the membranes come into contact have been comprehensively reviewed and approved. The following sections review elements that can be sources of deviations from an even and consistent substrate for the waterproofing membranes. Manufacturers will typically have system-specific guidelines and accessory products for these conditions, but it is still recommended to verify with their technical representatives whether the correct approach is being pursued on your project as the manufacturer will likely have specific warranty requirements.

LAGGING WALLS

A lagging walls system is a common method of soil retention. The lagging walls consist of steel H-member soldier piles with infill material (either precast concrete, steel boards, or, most commonly, wood planks or boards) stacked between the steel flanges to prevent soil from entering adjacent excavated areas. These systems can be front lagged, which means the wood planks are positioned at the front of the soldier piles in the direction of the excavated area (**Fig. 4**), or they may be middle lagged (with the wood planks positioned in the center of the soldier piles) or back lagged (with wood planks positioned in the back). For the purposes of below-grade waterproofing installation, front lagging is the most preferable of the three options because

FIGURE 4. Example of front-lagged lagging wall.

it provides the most even and continuous surface for the waterproofing system. Middle or back lagging introduces gaps between the outer face of the wood planks and the soldier pile's outer flange. Some waterproofing manufacturers may not allow gaps in the substrate over a certain width for the waterproofing membrane or drainage board (if being used), and infill such as spray foam insulation may be needed to reduce the gap widths.

FOUNDATION PILES

Pile foundations are often used when larger loads must be transferred to a deeper level of soil or rock. When designing the below-grade waterproofing system around piles, it is important to consider the following questions:

- What is the pile material? Concrete and steel piles tend to be the most common. Verify with the waterproofing membrane manufacturer which substrates are acceptable to them.
- Will the pile be composed of more than one material? For example, concrete piles in a steel casing can present different risks than piles made of concrete alone. Waterproofing consultants may need to coordinate between the waterproofing manufacturer and the structural engineer to determine whether the risks can be avoided with a pile flashing detail that is acceptable to all project stakeholders.
- Will the piles connect to pile caps?

  The shape of pile caps (in section) can have implications on the waterproofing membrane installation. A minimum dimension for the pile embedment into the pile cap will be required by the engineer for proper load transfer, but there should also be sufficient pile height for the waterproofing membrane under the pile cap to lap and flash onto the pile surface, as required by the waterproofing manufacturer. Between these two considerations, the greater dimension should be used.

These items can sometimes require several rounds of discussion and coordination involving the waterproofing manufacturer, structural engineer, and pile designer or engineer (if included) to satisfy each party's requirements and to consider alternative design approaches. It is best to start these discussions early to prevent costly change orders or delays in the schedule.

GROUND ANCHORS AND SOIL NAIL WALLS

Both ground anchors (also referred to as tiebacks) and soil nail walls are examples of soil retention systems that use steel elements (rods, bars, etc.) inserted into the ground and acting in tension.[5] Ground anchors are prestressed before installation, whereas soil nails are not. Most of the steel rod or bar length does not interface with the waterproofing, but both systems typically have an exposed head of the steel elements with a bearing plate that faces the site excavation (**Fig. 5**). Waterproofing manufacturers may have a prefabricated cover to be installed over each anchor head, and the prefabricated cover is then integrated with the below-grade waterproofing membrane. If the specific conditions of the ground anchors or soil nails do not permit the use of the prefabricated covers, it is advisable to consult with the waterproofing manufacturer as it will likely be necessary to generate custom or project-specific details and accessories.

INTERNAL BRACING

When tiebacks cannot be used due to property line or site constraints, alternative options for supporting the shoring wall, such as internal bracing, may need to be pursued. Bracing within the excavated site can involve rakers and/or struts. Rakers are diagonal components placed vertically against the shoring wall and on the ground, whereas struts are placed horizontally at excavation corners bracing two sides of the shoring walls. If the foundation walls are poured around the raker and strut penetrations, the concrete and the below-grade waterproofing system will need to be patched once the bracing is removed. The waterproofing membrane patch should be integral with the rest of the below-grade waterproofing as indicated in the manufacturer's instructions. Before the concrete patches are poured, water stops should be installed continuously along the perimeter of the patch, as each bracing penetration would have a cold joint with the rest of the concrete wall.

SHOTCRETE WALLS

Shotcrete presents several challenges for effective below-grade waterproofing that stem from the nature of the application process. These issues primarily arise from overspray of the shotcrete as well as the potential for more voids and cold joints than would occur in cast-in-place concrete construction. Experienced shotcrete applicators and contractors are essential, as excessive overspray can affect the overall adhesion between the shotcrete and the waterproofing membrane. Shotcrete

FIGURE 5. Soil nail wall with exposed nail heads and bearing plates.

is typically installed vertically in "lifts," which may introduce more cold joints than traditional cast-in-place concrete. Additionally, in blindside applications, the shotcrete applicators should take extra care in ensuring full coverage around reinforcing steel to minimize the amount of voids between the foundation and waterproofing membrane. The positioning and frequency of reinforcing steel can occasionally block shotcrete from reaching and bonding to the waterproofing membrane, which could create weak points in the below-grade waterproofing system. Some waterproofing manufacturers have blindside membrane products specifically designed for shotcrete applications and specify that these products must be used on the shotcrete portions of the foundation. These topics should be thoroughly discussed with the design and construction teams, with the available options for waterproofing the shotcrete portions of the foundation evaluated against the owner's risk tolerance.

SUPPLEMENTAL CONCRETE SLABS

In addition to the concrete slab-on-grade foundation, contractors may also propose providing a rat slab or protection slab. Rat slabs, also referred to as mud slabs, are placed over the excavated soil, often to provide a uniform surface for the waterproofing membrane installation. While the use of rat slabs can be an alternative to meeting the grading and compaction requirements specified by the waterproofing manufacturer, it is important to ensure that the rat slabs are smooth and free from surface inconsistencies that can puncture the membrane. The manufacturer may also have concrete surface profile requirements for the exposed face of the rat slab that serves as the membrane substrate.

Protection slabs are placed over the waterproofing membrane and are intended to protect the membrane from construction traffic and other potential damage before the slab-on-grade foundation is poured. In this case, the waterproofing membrane would become directly bonded to the protection slab instead of the structural slab-on-grade foundation, which is the more important component to be waterproofed. A cold joint would now exist between the protection slab and the structural slab-on-grade foundation. For protection slabs to be effective instead of a potential liability, they should be properly designed with sufficient thickness, reinforcing, and cover. If the protection slab is too thin, it may be especially susceptible to cracking, which could compromise the waterproofing membrane's performance. It is recommended to provide an additional layer of blindside waterproofing membrane over the protection slab, with this layer to be fully bonded to the structural slab-on-grade foundation, particularly in hydrostatic conditions.

FIGURE 6. Dobie block placed between rebar and waterproofing membrane.

REINFORCING BAR CONSIDERATIONS

As concrete foundations go deeper, more reinforcing steel is installed. To keep the reinforcement in the correct position, reinforcing bar ties, which connect to metal clips, are often used. Each clip is a penetration through the waterproofing membrane and must be addressed in a method approved by the manufacturer. Another risk is that reinforcing bars are too close to the waterproofing membrane. That could pose structural issues, such as insufficient cover on the concrete foundation element, as well as waterproofing issues since the waterproofing membrane should be in direct contact with concrete to achieve proper adhesion. Dobie blocks, which are small concrete cubes with metal ties, can be used as a simple way to position the reinforcing bars a standard distance away from the waterproofing membrane (**Fig. 6**).

CONCLUSION

With any foundation, whether shallow or deep, it is critical to properly design and install the below-grade waterproofing system as the system will be exponentially more difficult to access or repair as construction progresses. Although deep foundations are more challenging to waterproof, early discussions and collaboration with the design team and engineers can be beneficial in developing a foundation design that is waterproofing friendly and minimizes risk.

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ASSESSMENT OF THERMAL BRIDGING OF FASTENERS THROUGH INSULATED ROOF ASSEMBLIES

ABSTRACT

Roof fastener systems are comprised of metal screws and plates used to attach roof membranes, cover boards, and insulation. These systems can have an adverse impact on the thermal performance of roof assemblies, as the components create thermal bridges that bypass the thermal resistance of insulation in the roof assembly. This in turn allows heat to escape at an accelerated rate, flowing outward in cold weather and inward in warm weather. While the thermal performance of 3-D thermal bridges can be numerically simulated with software tools, such simulations are time-consuming and need to be verified by laboratory tests to validate the underlying assumptions made during the simulation.

During this presentation, participants will learn how the research team used a series of laboratory tests to compare the thermal performance of physical models of simple roof assemblies under controlled laboratory environmental conditions with computer simulations of the same conditions. Assemblies were comprised of high-density polyisocyanurate cover board, polyisocyanurate insulation, and steel deck, tested both with and without #12 and #15 fasteners and plates. In this session, the results of both physical models and computer simulations are presented and compared. The outcome is an experimentally validated computer simulation approach that will enable consultants to evaluate a broader range of roof assemblies and roof fastener configurations.

LEARNING OBJECTIVES

» Understand the physics behind the adverse impact of fasteners and plates on the thermal performance of roof assemblies.

» Identify the limitations of both physical models and computer simulations of real-world roof assemblies.

» Describe the impacts of fasteners and plates on the thermal performance of roof assemblies as identified in physical models and computer simulation studies.

» Recognize the potential application of this study's conclusions to the evaluation of additional roof assemblies and modification of existing codes and standards.

SPEAKERS

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Dr. Georg Reichard is a professor and department head of building construction and an associate director in the Myers-Lawson School of Construction at Virginia Tech. His research deals with experimental and numerical methods, simulation, and data modeling, in particular in the area of building sciences related to building enclosures and environmental systems. In his current research, he focuses on building performance, enclosure durability, disaster resilience, energy efficiency, and integrated decision-making for retrofit solutions in connection with different control strategies and building materials. Reichard holds a master's and a doctoral degree in civil engineering from Graz University of Technology, Austria.

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INTRODUCTION

Fasteners through roofing assemblies, composed of metal screws and plates, are thermal bridges that bypass the thermal insulation and create points of increased heat flow. **Figure 1** shows an example of this effect on snow-covered roofs.

As energy code requirements for thermal insulation have become more stringent, thermal bridges, such as fasteners, are a more significant contributor to the overall heat flow through building enclosure systems. For this reason, some energy codes and performance standards (e.g., Passive House) require documenting thermal bridges and quantifying their influence through detailed analysis. The impacts of point thermal bridges (e.g., fasteners) can be numerically simulated with software tools; however, such simulations are often time-consuming and sometimes need laboratory tests as validation.

This study provides a relative comparison of various roofing configurations with and without fasteners. The authors compare the thermal performance of a physical assembly, tested under controlled laboratory conditions, with a detailed three-dimensional (3D) computer simulation of the same assembly. By incrementally increasing the complexity of the assemblies in the tests and simulations, the authors seek to better understand the limitations of simulations, with the ultimate goal of developing an experimentally validated computer simulation approach that will enable the evaluation of a broader range of roof assemblies and roof fastener configurations.

The authors have approached this in two phases, the first of which is covered within this paper. This preliminary phase focused on the change in heat flux or flow (converted into a thermal resistance or R-value) through a series of test roof assemblies with and without fasteners. The authors did not compare surface or internal temperatures of the test assemblies to computer simulations and so cannot yet comment on the validity or accuracy of the simulations and the experimental results. The authors intend to publish a separate follow-up study comparing the experimental data to two- and three-dimensional

FIGURE 1. Examples of thermal bridging at fasteners on snow-covered roofs.

computer simulations to review the accuracy of computer modeling methods commonly used for calculating and accounting for thermal bridges in the design and construction industry.

RELATED STUDIES

Several simulation studies, discussed in further detail below, have estimated the thermal penalty attributable to fasteners in roofing assemblies.

A Heat Transfer Analysis of Metal Fasteners in Low-Slope Roofs

In an early finite-difference method simulation, Burch et al. found an increase of 3% to 8% in heat loss due to fasteners. They examined fasteners modeled as cylinders with metal caps in low-slope roofs with metal and wood decks at a density of 0.5 fasteners/ft^2 (5.3 fasteners/m^2) in insulation ranging from 1 to 6 in. (25 to 150 mm) thick. The increase in heat loss rose with insulation thickness; the 8% increase in heat loss corresponded to the assembly with 6 in. of insulation. They found that burying fasteners below the top layer of insulation reduced their thermal effect to one-fourth that of the case where fasteners penetrated both layers of insulation. Further, fasteners had twice the effect in metal decks as compared to wood decks, and replacing the metal fastener caps with plastic caps reduced thermal loss per fastener by 44%.

Effects of Mechanical Fasteners and Gaps between Insulation Boards on Thermal Performance of Low-Slope Roofs

Petrie et al. conducted laboratory experiments on roof assemblies incorporating three different types of fasteners through two 2-in. (5.1-cm) layers of polyisocyanurate (polyiso) insulation and found that fasteners, on average, reduced the thermal resistance of the roof assembly by 7% at a mean insulation temperature of 75°F (24°C) compared to the same roof assembly with no fasteners. Petrie et al.'s steady-state simulation with HEATING 7 showed a 12% reduction in roof R-value for fasteners with steel plates, while only a 3% reduction in roof R-value when using specially designed steel fasteners with plastic heads extending through the top layer of insulation. Findings were extended to determine their impact on simulated heating and cooling loads in six locations representing varying climates in the United States.

Roofing Research and Standards Development: ASTM STP1590

In a simulation of a roof assembly with fastener plates placed above the cover board and fasteners penetrating the insulation into a steel roof deck, Olson et al. found that while the system nominally met the insulation requirements of the International Energy Conservation Code, it failed to meet these requirements when the conductive effect of industry-standard fasteners was considered. Olson et al. showed that, in a simulated roof assembly, the thermal penalty of fasteners in a temperate climate may exceed that of other penetrations such as roof drains, equipment supports, and roof vents in a typical installation. They explained that this was due to the large number of fasteners as compared to a typically smaller number of other, larger roof penetrations.

Olson et al. used the 3-D, finite-difference software HEAT3 to find a roughly 17% increase in heat loss caused by fasteners, assuming exposed metal plates over gypsum cover board at 1 fastener per 2 ft^2 (0.2 m^2) or 0.5 fasteners/ft^2 (5.4 fasteners/m^2).

Olson et al. explored the protective effect of using an insulating cover board in lieu of a gypsum cover board, and also the impact of placing fasteners below the cover board and fully adhering the cover board with adhesive. Their simulations showed that even when adhering an insulating cover board over fastened insulation, there was still a 10% reduction in effective thermal resistance compared to a roof assembly with no fasteners.

Towards Codification of Energy Losses from Fasteners on Commercial Roofing Assemblies & Development of Chi-Factors Towards Codification of Thermal Bridging in Low-Slope Roofing Assemblies

Moletti and Baskaran and Moletti et al. tested a range of common roofing assemblies in a horizontal guarded hot box apparatus and found that thermal bridging attributable to roof fasteners increased with fastener density and with increasing thermal resistance of the insulation they penetrate. They reported a loss in effective R-value ranging from 4.4% to 13.3% across design assemblies rated R-26 through R-36. They also found that covering the fastener heads with the top layer of insulation led to 30% to 70% reductions in thermal bridging compared to fasteners extending through the cover board and insulation, with more favorable results derived from a thicker top layer of insulation.

Building Envelope Thermal Bridging Guide V. 1.6

Developed by Morrison Hershfield and industry partners, this guide includes a catalog of common building enclosure details incorporating thermal bridges. The reported values were calculated using a 3D finite-element analysis (FEA) heat transfer software package developed by Siemens PLM Software. The catalog includes multiple roof details, two of which (10.1.9 and 10.1.13) incorporate exterior-insulated, low-slope, mechanically fastened roof assemblies over metal decks similar to (although not directly comparable to) the assemblies included in this study. Detail 10.1.9 includes a fastener density of 0.3 fasteners/ft^2 (3.4 fasteners/m^2) with #10 and #14 fasteners embedded at different depths of a roofing assembly with various insulation thicknesses. Detail 10.1.13 includes a fastener density of 1 fastener/ft^2 (10.8 fasteners/m^2) with #14 fasteners through the entire depth of the roofing assembly with various insulation thicknesses.

Significance

As evidenced by the range of conclusions garnered from these studies, more physical experiments and computational simulations addressing fastened roof components in their various permutations are needed to understand how thermal bridging from fasteners numerically impacts the overall thermal performance of roofing assemblies. These studies are necessary to support design efforts and the future development of building codes, industry standards, and energy performance certifications. Figures or tables containing data on the point

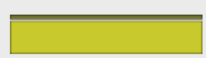
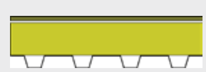
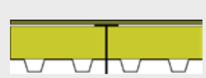
TABLE 1. Naming protocol for study.

Fastener Code	Fastener Configuration
A	No fastener
B	#12 fastener, 6 in. long

Assembly Code	Assembly Type
I	Single 4 - in. polyiso board
II	4 in. - polyiso covered with 0.5 in. high-density polyiso cover board
III	4 in. - polyiso on steel deck
IV	4 in. - polyiso on steel deck covered with 0.5 in. high-density polyiso cover board

Abbreviations	Full Term
PIR	polyiso board
HDB	High-density polyiso cover board
SD	Galvanized steel deck

TABLE 2. Assembly cases.

Fastener Code	Assembly Code	Case	Assembly Components	Diagram
A	I	A-I	4-in. PIR	
A	II	A-II	0.5-in. HDB 4-in. PIR	
B	I	B-I	#12 fastener 4-in. PIR	
B	II	B-II	0.5-in. HDB #12 fastener 4-in. PIR	
A	III	A-III	4-in. PIR SD	
A	IV	A-IV	0.5 in. HDB 4-in. PIR SD	
B	III	B-III	#12 fastener 4-in. PIR SD	
B	IV	B-IV	0.5 in. HDB #12 fastener 4-in. PIR SD	

transmittance of roof fasteners, based on their dimensional characteristics and the parameters of the roof assemblies in which they are used, such as those published within the *Building Envelope Thermal Bridging Guide* by Morrison Hershfield, allow a practical and simple way to estimate overall thermal performance. These data could, in turn, lead the roofing industry to develop more thermally efficient assembly technologies.

METHODOLOGY

The following section summarizes the methodology used as a basis for this study in both the physical experiment and the 3D computer simulation.

Study Setup

The roofing assembly builds in complexity, in the stepwise fashion shown in **Tables 1** and **2**.

In the B-cases, a 6 in.- (150-mm-) long #12 fastener penetrated the insulation layer and the top flute of the SD, (where applicable). This resulted in an approximately 2-in. (51-mm) portion of the fastener that was exposed below the top flute of the SD Insulation retention plates with a 3 - in. (76-mm) diameter and 0.019-in. (0.48-mm) thickness were used with the fastener.

Physical Experiment

The authors tested the simplified roof assemblies as depicted in Tables 1 and 2 in a controllable climate test chamber.

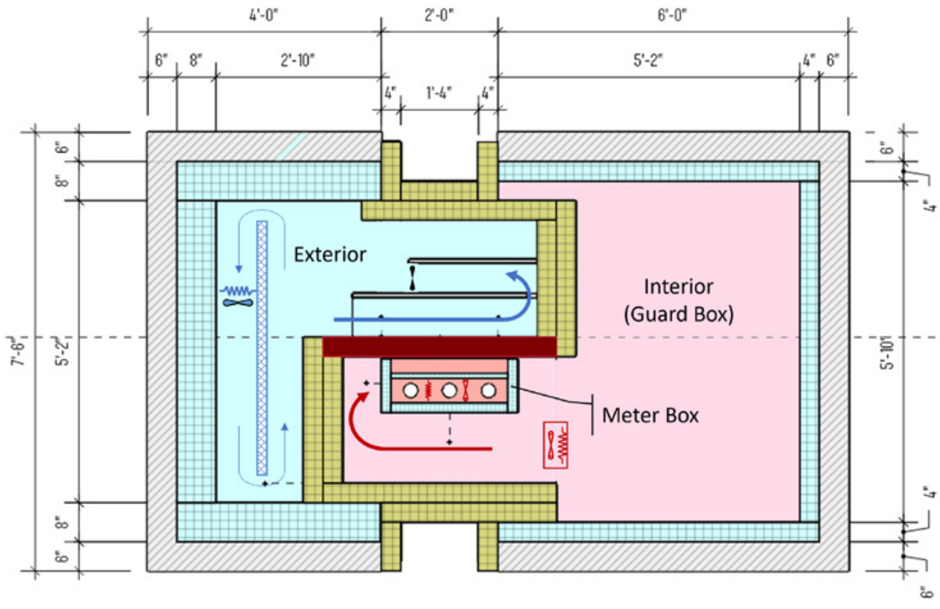


FIGURE 2. Climate chamber configuration for roof system testing.

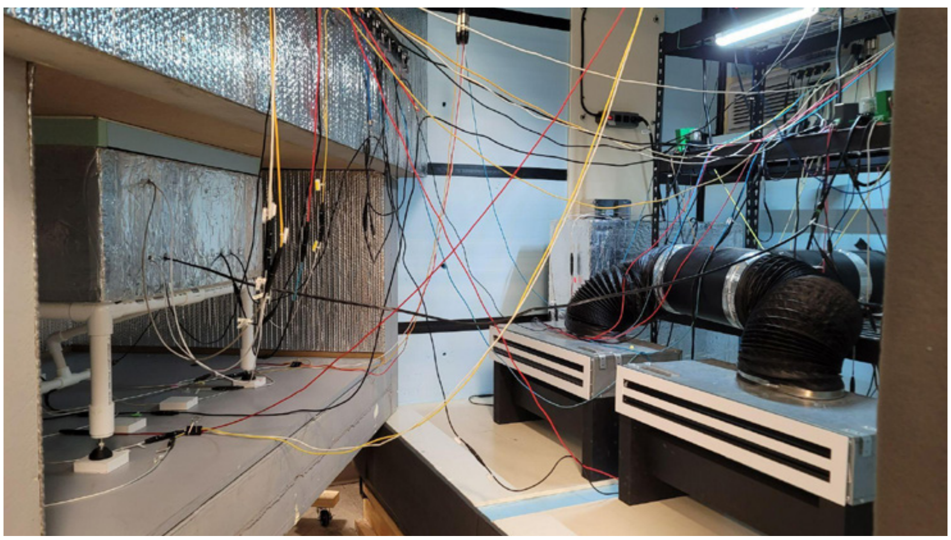


FIGURE 3. Open view of assembly frame and meter boxes (left) and guard box with climate control (right) within the climate chamber.

The climate chamber configuration is shown in **Figure 2**. Experimental tests were conducted in triplicate series to permit a baseline for statistical evaluation of measurements.

The climate chamber consists of a warm side (interior condition) chamber and a cold side (exterior condition) chamber. It allows for controlling temperature and relative humidity on both sides of the test assembly and capturing temperature, relative humidity, heat flux, and air velocity measurements as needed depending on study requirements. The climate chamber was customized with an assembly frame to allow for horizontal mounting of the test assembly to include gravitational impacts. Since relatively small local heat flux differences had to be assessed, a guarded meter box approach was designed for these experiments. **Figure 3** shows an open view of the climate chamber including the meter box within the lower guard box (interior chamber).

All tests were conducted under steady-state conditions and did not consider the temperature dependence of the insulating materials. A 2-ft x 2-ft (0.6-m x 0.6-m) area of the test assembly was monitored and the heat flux across the test assembly was measured. The exterior chamber was held at 50°F (10°C), and the interior chamber was held at 100°F (38°C), resulting in a mean insulation temperature of 75°F (24°C). For the I-and II-cases, the near-surface airflow was maintained at 50 fpm (0.25 m/s) in the exterior chamber and 70 fpm (0.36 m/s) in the interior chamber. For the III-and IV-cases, the near-surface airflow was maintained at 50 fpm (0.25 m/s) in the exterior chamber and 40 fpm (0.20 m/s) in the interior chamber. These velocities were used to create a homogeneous condition across both sides of the test assembly. Adding the SD in Cases III - and IV- cases changed the airflow rate in the interior chamber.

The test sequence was developed to minimize the number of times the test chamber needed to be opened and closed and the samples manipulated, and to enable the same 4- in. (102-mm) polyiso board (PIR) specimen to be used throughout an entire series of tests, thereby eliminating variation in PIR as a potential error source. The roof assemblies studied in this analysis incorporated the following modifications/simplifications from a typical roofing assembly that may be observed in the field (i.e., on a construction site):

1. The roofing membrane was omitted since the membrane's contribution to thermal resistance is negligible and adhering a membrane could introduce potential error between assemblies.
2. The adhesive layer (e.g., low-rise spray foam adhesive) was omitted between the cover board (HDB) and PIR to facilitate removing the HDB between tests. Foam weatherstripping tape was applied to the top perimeter of the PIR (beneath the HDB) to achieve an air seal between the two layers, which resulted in an air gap of approximately 0.1-in. (2.5 mm) between the two layers.
3. One layer of 4-in. (100-mm) PIR was used in lieu of multiple PIR board layers to avoid discrepancies caused by imperfect contact between the layers and between staggered boards. These imperfections are

not considered in computational simulations and are also difficult to replicate with each test case. The 4-in. (100- mm) PIR does not meet current prescriptive energy code requirements in most of the continental United States (e.g., per the 2021 International Energy Conservation Code). However, 4-in.-thick boards are consistently produced and can therefore be expected to have a reliable R-value.

4 Foam weatherstripping tape was applied to the top perimeter of the SD, beneath the PIR, to air seal between the two layers, which resulted in an air gap of approximately 0.19-in. (4.76- mm) between the two layers. Foam flute plugs were also utilized at the open ends of the metal deck.

The above-noted modifications were included in the corresponding detailed 3D computer simulation (see next section).

Computer Simulation

The authors performed a detailed steady-state thermal analysis of the same roof assemblies tested in the physical experiment (see **Table 2**) using the 3D FEA tool ANSYS, developed by ANSYS, Inc. ANSYS simulates heat flow through materials, components, and systems based on a defined geometry and interior/exterior environmental conditions, referred to as boundary conditions.

Geometry

The finite-element method utilized in the detailed ANSYS computer simulation allows for a more accurate representation of the fastener geometry than the finite-difference method used in past research (reference Related Studies above) since it can mesh irregular (i.e., non-rectilinear) shapes.

The model geometry (**Figure 4**) was developed as described in Tables 1 and 2 above, with the following clarifications:

» The fastener manufacturer provided a detailed 3D SolidWorks model of the fastener and fastener plate geometry, including ribbed plate and fastener threads. Several minor simplifications, which are considered

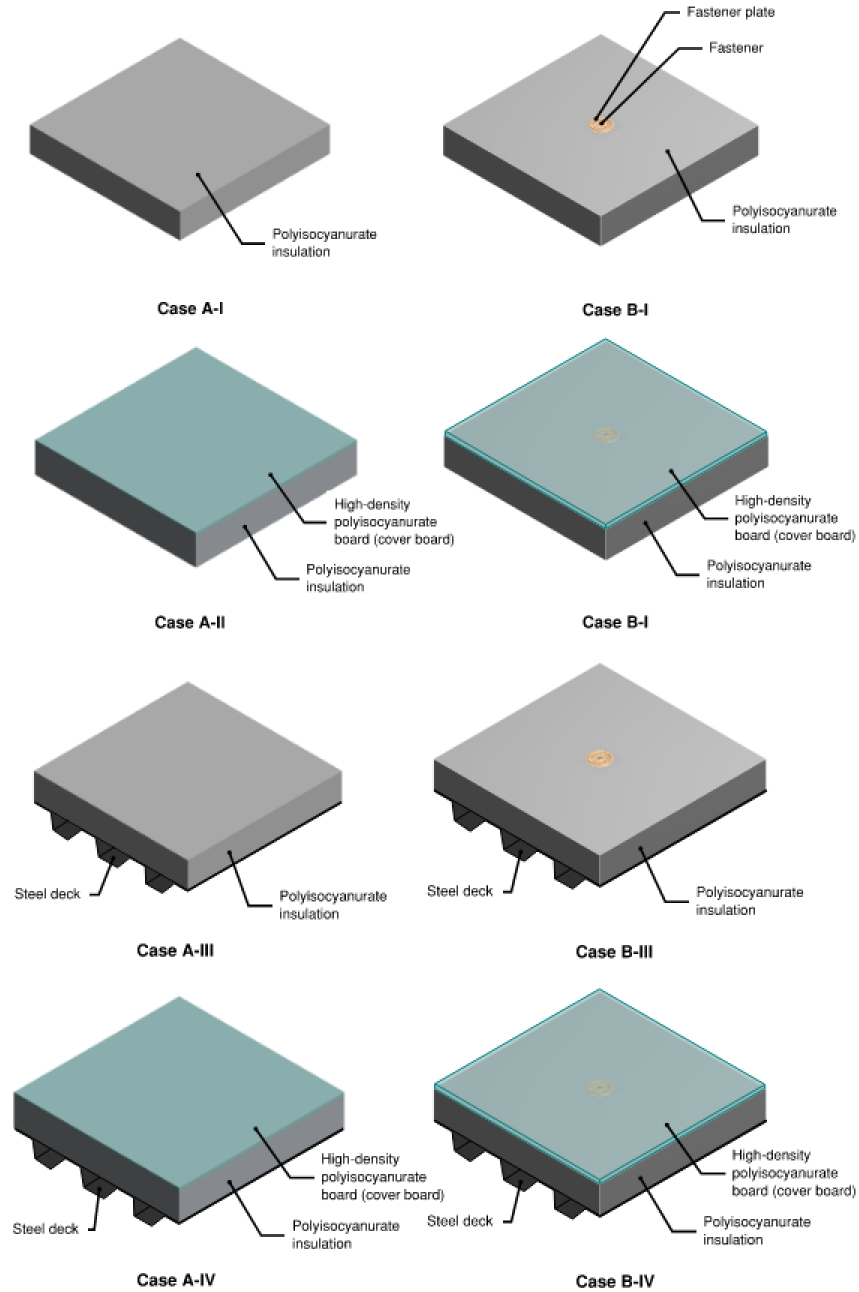


FIGURE 4. General geometry of 3d simulated assembly configurations.

to have negligible impacts on the overall heat flow, were made to the fastener threads and head to facilitate meshing.

Material Properties

Thermal conductivities and their sources for the solid model components are as follows:

1 High-density polyiso board (HDB): 0.017 Btu/hr-ft-°F (0.029 W/m-K), from manufacturer's published product data (GAF)

2 Polyiso board (PIR): 0.015 Btu/hr-ft-°F (0.026 W/m-K), from manufacturer's published product data (GAF)

3 Fastener and plate (carbon steel): 29 Btu/hr-ft-°F (50 W/m-K), from fastener manufacturer (OMG)

4 Galvanized steel deck (SD): 36 Btu/hr-ft-°F (62 W/m-K), from THERM material database (LBNL)

5 Air cavities: vary, from THERM model

The authors utilized a two-dimensional (2D) FEA tool, THERM by the Lawrence Berkeley National Laboratory (LBNL), to determine the effective thermal conductivity of the small, enclosed air cavities within the model (e.g., between

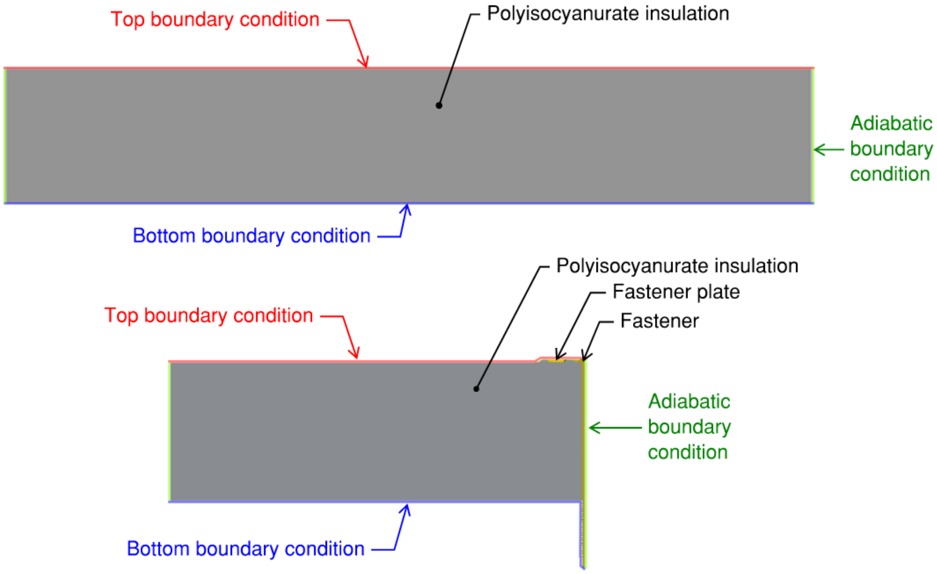


FIGURE 5. Example boundary conditions for cases A-I (top) and B-I (bottom).

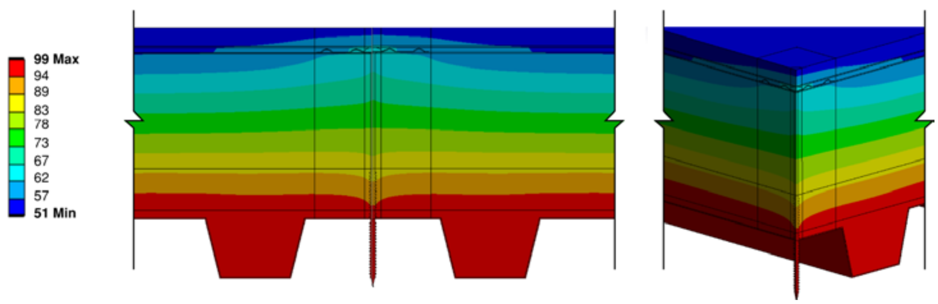


FIGURE 6. Color temperature output for case B-IV at fastener (section and isometric views).

the top of the insulation and the bottom of the cover board) for input into ANSYS.

Boundary Conditions
Each case was modeled with steady-state boundary conditions applied to the outermost surfaces (**Figure 5**). The side faces of the assembly (i.e., cut surfaces at the perimeter of the assembly) were assigned as adiabatic boundary conditions, which represent boundaries across which there is no heat flow. Simulations were based on 2-ft. x 2-ft. (0.6-m x 0.6-m) assembly dimensions.

The boundary conditions (indicated in **Table 3** below) incorporate near-surface temperatures and air flows and the emissivity of the adjacent visible surfaces (i.e., interior surfaces of the testing chamber) measured in the physical experiment. The authors matched the computer models' boundary conditions to the experimental setup rather than utilizing standard ASHRAE boundary conditions, to eliminate a possible source of difference between the experimental and computer simulation results.

To calculate the convective film coefficient, the authors followed the methodology for forced convection (utilizing external flows over a flat plate) outlined in chapter 4 of the *ASHRAE Handbook 2017: Fundamentals*. Properties of air were obtained from papers by Baumgartner et al. and Kadoya et al. The convective film coefficient does not incorporate natural convection as it is expected that the size of the test chamber limits the ability for natural convection to develop.

TABLE 3. Boundary conditions used for computer simulation in ANSYS.

Surface	Temperature		Convective Film Coefficient		Emissivity
	°F	°C	Btu/hr-ft2-°F	W/m2-K	
Bottom – warm side (I-and II-Cases)	100	38	0.54	3.08	0.95
Top – cold side (I-and II-Cases)	50	10	0.44	2.49	0.95
Bottom – warm side (III-and IV-Cases)	100	38	0.41	2.33	0.95
Top – cold side (III-and IV-Cases)	50	10	0.44	2.49	0.95
Sides – adiabatic	N/A	N/A	N/A	N/A	N/A

Simulation
The simulated heat flow in ANSYS was converted into a U-factor (and associated R-value) using the projected area of the assembly in the horizontal (i.e., projected-X) plane. **Figure 6** shows the typical temperature output from ANSYS.

RESULTS

The following section summarizes results from both the physical experiment and the 3D computer simulation.

Physical Experiment Results

Figure 7 shows the average calculated thermal resistance R-values and the range of individual test results from the three laboratory tests for each test assembly configuration. **Figure 8** shows the percent change from the A-case R-values to the B-case R-values.

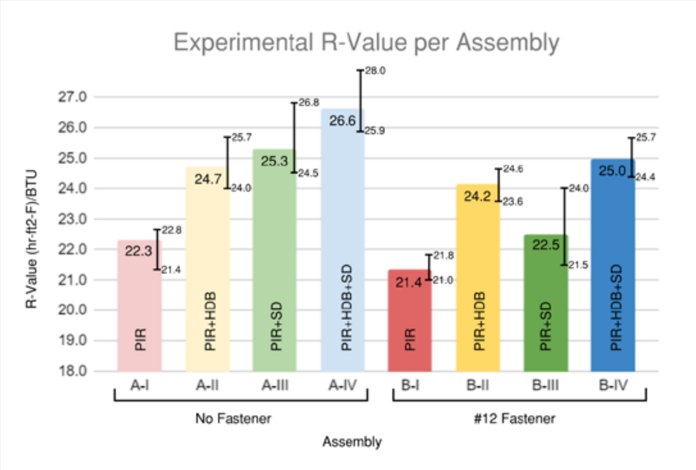


FIGURE 7. Physical experiment R-value for each assembly.

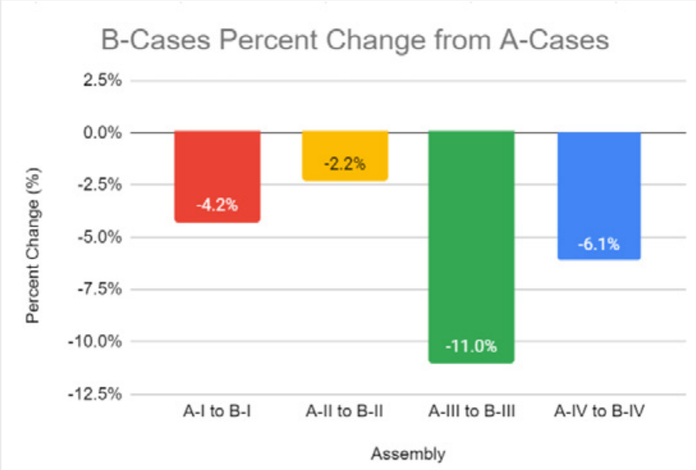


FIGURE 8. Physical experiment R-value Percent change from A-cases to B-cases.

The cases with no fastener (A-cases) show the following trends:

1 **Case A-I to A-II:** The R-value increased by 10.8% when adding the cover board HDB to the PIR.
2 **Case A-I to A-III:** The R-value increased by 13.5% when adding the steel deck SD to the PIR.
3 **Case A-I to A-IV:** The R-value increased by 19.4% when adding the HDB and SD to the PIR.
4 **Case A-II to A-IV:** The R-value increased by 7.7% when adding the SD to the PIR and HDB.
5 **Case A-III to A-IV:** The R-value increased by 5.2% when adding the HDB to the PIR and SD.

The cases with a #12 fastener (B-cases) show the following trends:

1 **Case B-I to B-II:** The R-value increased by 13.1% when adding the HDB to the PIR.
2 **Case B-I to B-III:** The R-value increased by 5.4% when adding the SD to the PIR.
3 **Case B-I to B-IV:** The R-value increased by 17.0% when adding the HDB and SD to the PIR.
4 **Case B-II to B-IV:** The R-value increased by 3.4% when adding the SD to the PIR and HDB.
5 **Case B-III to B-IV:** The R-value increased by 11.0% when adding the HDB to the PIR and SD.

The cases with no fastener (A-cases) and the cases with a #12 fastener (B-cases) show the following trends relative to one another:

1 **A-cases to B-cases overall:** Adding a #12 fastener in the B-cases reduced the thermal resistance by a range of 2.2% (A-II to B-II) to 11.0% (A-III to B-III) when compared to the same condition in the A-cases with no fastener.
2 **III-cases vs. I-cases:** The III-cases with PIR and an SD had a greater relative drop in thermal resistance when the #12 fastener was added (11.0%) compared to the I-cases with just PIR (4.2%).
3 **IV-cases vs. II-cases:** The IV-cases with PIR, an HDB, and an SD also had a greater relative drop in thermal resistance when the #12 fastener was added (6.1%) compared to the II-cases with just PIR and an HDB (2.2%).
4 **II-cases vs. I-cases:** The II-cases with PIR and an HDB had an lesser relative drop in thermal resistance when the #12 fastener was added (2.2%) compared to the I-cases with just PIR (4.2%).
5 **IV-cases vs. III-cases:** The IV-cases with PIR, an HDB, and an SD also had a lesser relative drop in the thermal resistance when the #12 fastener was added (6.1%) compared to the III-cases with just PIR and SD (11%).

Computer Simulation Results

Figure 9 below shows calculated R-values from the computer simulation of each test assembly configuration, and **Figure 10** shows the percent change from the A-case R-values to the B-case R-values.

The cases with no fastener (A-cases) show the following trends:

1 **Case A-I to A-II:** The R-value increased by 10.2% when adding the HDB to the PIR.
2 **Case A-I to A-III:** The R-value increased by 0.8% when adding the SD to the PIR.
3 **Case A-I to A-IV:** The R-value increased by 11.4% when adding the HDB and SD to the PIR.
4 **Case A-II to A-IV:** The R-value increased by 1.2% when adding the SD to the PIR and HDB.
5 **Case A-III to A-IV:** The R-value increased by 10.5% when adding the HDB to the PIR and SD.

The cases with a #12 fastener (B-cases) show the following trends:

1 **Case B-I to B-II:** The R-value increased by 11.0% when adding the HDB to the PIR.
2 **Case B-I to B-III:** The R-value decreased by 0.4% when adding the SD to the PIR.
3 **Case B-I to B-IV:** The R-value increased by 10.5% when adding the HDB and SD to the PIR.
4 **Case B-II to B-IV:** The R-value decreased by 0.4% when adding the SD to the PIR and HDB.

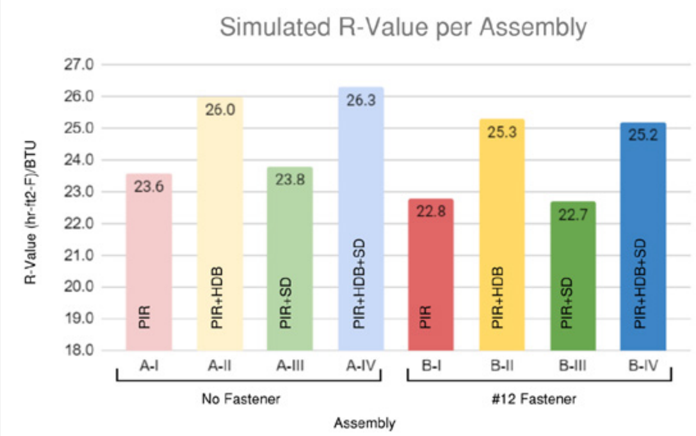


FIGURE 9. Computer simulation R-value results for each assembly.

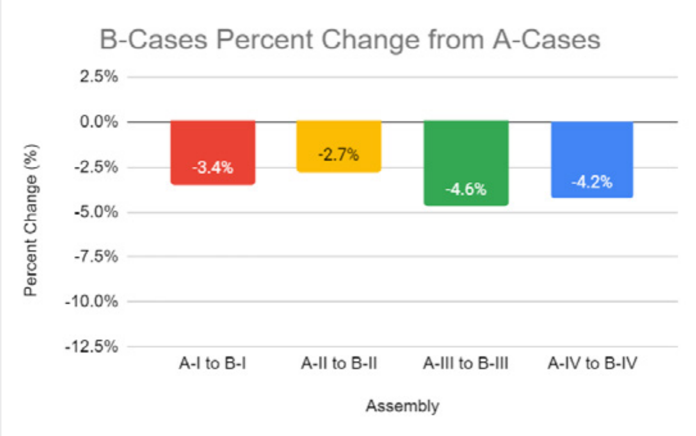


FIGURE 10. Computer simulation R-value Percent change from A-cases to B-cases.

5 **Case B-III to B-IV:** The R-value increased by 11.0% when adding the HDB to the PIR and SD.

The cases with no fastener (A-cases) and the cases with a #12 fastener (B-cases) show the following relative trends to one another:

1 **A-cases to B-cases overall:** Adding a #12 fastener in the B-cases reduced the thermal resistance by a range of 2.7% (A-II to B-II) to 4.6% (A-III to B-III) when compared to the same condition in the A-cases with no fastener.

2 **III-cases vs. I-cases:** The III-cases with PIR and SD had a greater relative drop in thermal resistance when the #12 fastener was added (4.6%) compared to the I-cases with just PIR (3.4%).

3 **IV-cases vs. II-cases:** The IV-cases with PIR, an HDB, and an SD also had a greater relative drop in thermal resistance when the #12 fastener was added (4.2%) compared to the II-cases with just PIR and an HDB (2.7%).

4 **II-cases vs. I-cases:** The II-cases with PIR and an HDB had a lesser relative drop in the thermal resistance when the #12 fastener was added (2.7%) compared to the I-cases with just PIR (3.4%).

5 **IV-cases vs. III-cases:** The IV-cases with PIR, an HDB, and an SD also had a lesser relative drop in the thermal resistance when the #12 fastener was added (4.2%) compared to the III-cases with just PIR and SD (4.6%).

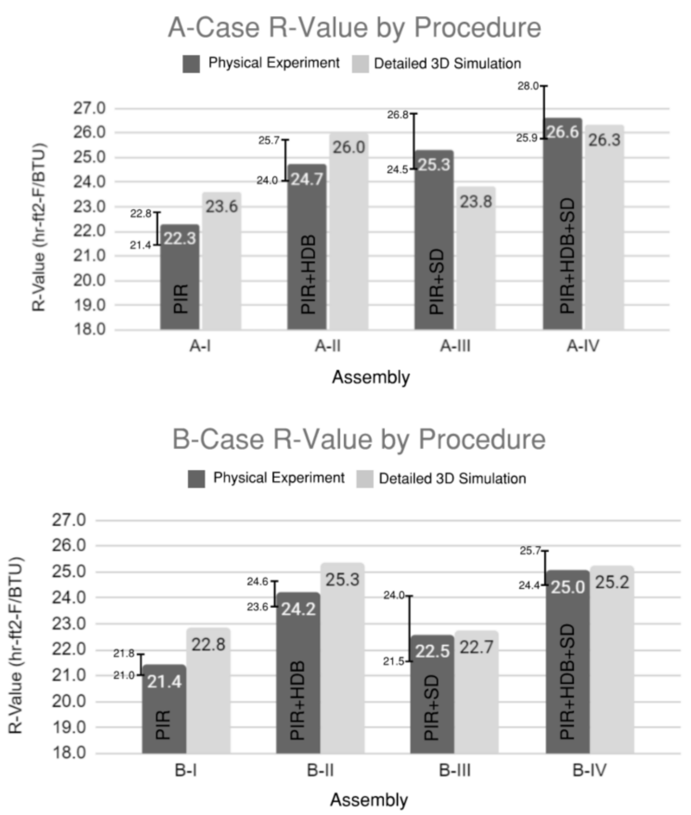


FIGURE 11. Comparative R-value results.

Comparative Results

Figure 11 shows calculated R-values from the physical experiment compared to the computer simulation for each test assembly configuration, and **Figure 12** shows the percent change from the A-cases' R-values to the B-cases' R-values for the two procedures.

The two procedures show the following notable differences for the cases with no fastener (A-cases):

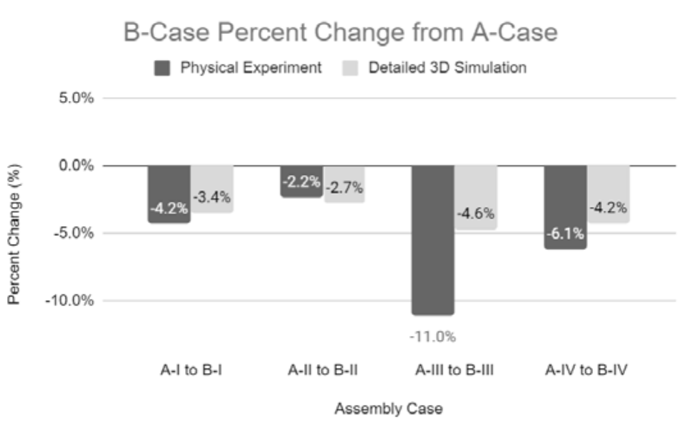


FIGURE 12. Comparative percent change from A-cases to B-cases.

1 **Case A-I to A-III:** The R-value increased by 13.5% in the physical testing when adding the SD to the PIR, while the corresponding R-value in the computer simulation stayed relatively constant (0.8% increase).

2 **Case A-II to A-IV:** The R-value increased by 7.7% in the physical testing when adding the SD to the PIR and HDB, while the corresponding R-value in the computer simulation increased only slightly (1.2% increase).

3 **Overall comparison:** The difference between the physical experiment results and the computer simulation results varies between 1.2% and 5.9% by case (utilizing the physical experiment data as a baseline).

The two procedures show the following notable differences for the cases with a #12 fastener (B-cases):

1 **Case B-I to B-III:** The R-value increased by 5.4% in the physical testing when adding the SD to the PIR, while the corresponding R-value in the computer simulation stayed relatively constant (0.4% decrease).

2 **Case B-II to B-IV:** The R-value increased by 3.4% in the physical testing when adding the SD to the PIR and HDB, while the corresponding R-value in the computer simulation stayed relatively constant (0.4% decrease).

3 **Overall comparison:** The difference between the physical experiment results and the computer simulation results varies between 0.8% and 6.7% by case (utilizing the physical experiment data as a baseline).

Generally, the physical testing and computer simulation show similar trends in the relative change in thermal resistance between the A-cases and B-cases (with the addition of a #12 fastener). However, the two procedures show the following notable differences:

1 **III-cases:** In the assembly with the PIR and SD, the physical testing showed a much greater relative drop in the thermal resistance (11%) compared to the computer simulation (4.6%).

2 **IV-cases:** In the assembly with the PIR, HDB, and SD, the physical testing showed a somewhat greater relative drop in the thermal resistance (6.1%) compared to the computer simulation (4.2%).

DISCUSSION AND CONCLUSIONS

In this section, the results from the previous section are discussed in detail. The discussion is divided into three sections: physical experiment conclusions, computer simulation conclusions, and conclusions related to the comparison between computer simulation and experimental results. A discussion on the comparison to past research by others (i.e., related studies) is also included.

Physical Experiment Conclusions

The experimental results show that adding a fastener reduces the thermal resistance of the roofing assembly in all cases. By incrementally adding layers, the results show the following:

1 **The insulating effect of HDB:**

- In cases without a fastener, adding the SD to the PIR (A-III relative to A-I) has a better R-value than adding the HDB alone (A-II relative to A-I). However, in the cases with a fastener, adding the HDB to the PIR (B-II relative to B-I) is more effective than adding the SD alone (B-III relative to B-I) because the HDB reduces the thermal bridging from the fastener.
- In cases with a fastener, in assemblies with PIR and SD alone compared to PIR alone (B-III relative to B-I), there is increased radiant exchange to the interior because the SD with a fastener acts as a radiator. Adding the HDB (B-IV) insulates the fastener, increasing the fastener and SD temperature and reducing the radiant heat exchange to the interior.
- In cases with a fastener, both cases that include the HDB (B-II and B-IV) had a much smaller drop in R-value relative to their corresponding A cases than those without a HDB (B-I and B-III).

2 **The insulating effect of SD air spaces**:

- In cases with and without a fastener, the addition of an SD (A-III and B-III relative to A-I and B-I, respectively, and A-IV and B-IV relative to A-II and B-II, respectively) increases the thermal resistance, which is likely due to the enclosed air pockets within the flutes, since trapped air is an insulator.
- In the cases adding an SD to PIR without a fastener (A-III), the SD with air pockets has a higher R-value than HDB and PIR (A-II). This trend is not the same when a fastener is added, as B-III has a lower R-value than B-II. The fastener may introduce enough thermal bridging to counteract the benefit of the insulating air spaces with this specific configuration.

3 **The impact of SD on thermal bridging:**

- The SD cases without an HDB had a greater relative drop in thermal resistance compared to their respective A-cases when fasteners were added (B-III relative to A-III) than those with PIR alone (B-I relative to A-I). This is likely because the SD acts as a thermal radiator.
- The SD cases had a greater relative drop in thermal resistance compared to their respective A-cases when fasteners were added (B-III and B-IV relative to A-III and A-IV, respectively) than did the cases without an SD (B-I and B-II relative to A-I and A-II, respectively).

In summary, the physical experiment results demonstrate that a roof assembly with an HDB adds insulating value from the board itself while also reducing thermal bridging from the fastener. Adding an SD also adds insulating value from the enclosed air pockets, but it concurrently amplifies the thermal bridging from fasteners.

It is worth noting, however, that various aspects of the experimental setup proved difficult to maintain and replicate, which likely impacted the results to an extent (as indicated by the variation of R-values across samples for each assembly reported in Figure 7). Additional testing (i.e., gathering of additional data points to serve as the basis for a statistical analysis) needs to be performed to evaluate potential outliers in the dataset.

Computer Simulation Conclusions

The computer simulation results also show that adding a fastener reduces the thermal resistance of the roofing assembly in all cases. By incrementally adding layers, the results show the following:

1 **The insulating effect of HDB:**

- In cases without a fastener, adding the HDB to the PIR (A-II relative to A-I) is more effective than adding the SD alone (A-III relative to A-I). The same trend can be seen when a fastener is added (B-II relative to B-I versus B-III relative to B-I). This diverges from the experimental result trend for the same cases.
- In cases with a fastener and PIR, with or without SD alone (B-III relative to B-I), there is minimal difference in R-values. Adding the HDB (B-IV) insulates the fastener, increasing the fastener and SD temperature and reducing the radiant heat exchange to the interior.
- In cases with a fastener, both cases that include the HDB (B-II and B-IV) had a smaller drop in R-value relative to their corresponding A-cases than those without an HDB (B-I and B-III).

2 **The insulating effect of SD air spaces:**

- Adding the SD changes the R-value minimally with or without a fastener (A-III relative to A-I, A-IV relative to A-II, B-III relative to B-I, and B-IV relative to B-II). This indicates that air space modeling and contact resistance between layers requires further study. Note that the metal deck and insulation were not modeled in contact with one another since the physical experiment incorporated a small (0.19-in [4.76-mm]) air gap between the two layers.

3 **Impact of SD on thermal bridging:**

- The SD cases without an HDB had a greater relative drop in thermal resistance compared to their respective A-cases when fasteners were added (B-III relative to A-III) than those with insulation alone (B-I relative to A-I). This may be because the SD acted as a thermal radiator.
- The SD cases had a greater relative drop in thermal resistance compared to their respective A-cases when fasteners were added (B-III and B-IV relative to A-III and A-IV, respectively) than did the cases without SD (B-I and B-II relative to A-I and A-II, respectively).

In summary, similar to the physical experiment, the computer simulation results demonstrate that a roof assembly with an HDB adds insulating value from the board itself while also reducing thermal bridging from the fastener. Also, adding SD amplifies the thermal bridging from fasteners. In contrast to the physical experiment, the computer simulation demonstrates, perhaps incorrectly, that adding an SD has minimal impact on overall thermal resistance rather than increasing the thermal resistance, indicating that the way the models account for air space should be further reviewed.

Comparison of the Results of Physical Experiments and Computer Simulations

When comparing the results of the physical experiments and computer simulations on a case-by-case basis, the difference between them ranges from 0.8 to 6.7%. The authors intend, through ongoing work, to review the correlations in more detail. As shown in the discussion above, some trends observed by both approaches were similar. The diverging trends that warrant further review include the following:

- The trends when adding the SD do not match. This may indicate that assumptions with air space modeling and contact resistances are inaccurate. Computer models assume each layer is in perfect contact.
- The nature of the physical experiment introduces a potential for outliers; however, it is difficult to perform statistical analyses on small sample sizes.

General Comparisons to Past Work

On the experimental side, Moletti et al. (2021) reported a 4.4% decrease in effective thermal resistance for an R-26 system on a steel deck with #14 fasteners penetrating a fiberglass mat gypsum roof cover board and two layers of insulation at a density of 0.25 fasteners/ft (2.69 fasteners/m^2). While not an exact match, case B-III in this study (R-23.6 insulation on steel deck with #12 fasteners at a density of 0.25 fasteners/ft^2 ([2.69 fasteners/m]^2) and no cover board) is similar and showed an 11.0% R-value reduction in the physical experiments.

On the simulation side, the results from the present study can be compared in general terms to Olson et al.'s (2015) finite-difference method simulation. Case B-IV in the present study is the most similar, with Olson et al.'s study utilizing a simplified representation of the metal deck, fastener, and fastener plate and including an additional gypsum substrate board between the metal deck and 4.5-in. (110-mm-) thick PIR. With a #12 fastener head and plate buried below the HDB cover board, at the same fastener density as considered in this study, Olson et. al found a 5.9% reduction in effective R-value compared to an assembly with no fasteners. This can be loosely compared to the 4.2% R-value reduction found in this study for case B-IV relative to case A-IV.

General Conclusions

The authors conducted physical experiments and computer simulations in a stepwise fashion to isolate the influence of the different layers in the assembly and to see where physical modeling and computer simulation converge and diverge. Both physical experimentation and computer simulation are simplifications of reality, and there are errors inherent in both approaches. The results of this study identify some diverging trends that warrant further analysis. The value of computer simulation, once validated by physical experimentation, is its ability to quickly extend results to a wide range of possible scenarios.

CONTINUATION

The next steps of this study include a review of the temperatures at different locations of each test assembly and the determination of point transmittances for the roof fasteners. With these data, the authors can determine more precisely where the computer simulations are diverging from the physical experiment. A sensitivity analysis can also be performed to determine the relative impact of air space modeling and the effect of contact resistances on the computer simulation results. Future work may include performing additional physical testing to produce a statistically significant sample size.

The authors also intend to perform simplified 2D FEA evaluations, which are often used by practitioners for determining overall roof assembly thermal performance (due to increased efficiency and overall lower cost to perform the analysis), to review the potential negative consequences inherent with analyzing a 3D problem in 2D.

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HISTORIC INDUSTRIAL BUILDING REUSE AND THE BUILDING ENCLOSURE

ABSTRACT

Prior to the pervasive use of mechanical interior conditioning, building enclosures were designed and constructed where building finishes accommodated drying the building enclosure to both the exterior and interior environments. This is especially true in historic industrial buildings, namely mass masonry structures. In the past couple of decades, many of these building types have been converted into modern living spaces. Conditioning the interior environment and the application of moisture-sensitive interior materials can result in undesirable condensation and the potential for biological growth on building interiors. The existing building enclosure construction can create conditions where raising the enclosure R-values to modern code-required values can be problematic. Installed components can conflict with the locations in which building science indicates the insulation should be installed. Existing construction can present conditions that, due to the manner of assembly or time-induced deterioration, are difficult to seal against air infiltration.

In this presentation, case studies will be discussed to demonstrate how these project issues were addressed. Attention will be given to the continuity of the air, water, and thermal control layers given the existing, historic building conditions. Designers, researchers, contractors, and building owners will be especially interested in the diagnostic testing procedures and, design-driving results that will be presented and discussed.

LEARNING OBJECTIVES

- Identify the types of historic building enclosure construction that may present issues with modern interior environmental needs.
- Compare how the air, water, and thermal layers may need to be assessed differently when designing repurposed historic industrial buildings.
- Examine ASTM testing procedures to ensure the successful application of specific details are recommended on historic repurpose projects.
- Assess which details need special attention when designing repurposed, historic industrial buildings.

SPEAKERS

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William G. Lehne, PE, CIT, attended Clemson University from 2011 to 2014, graduating with an undergraduate degree in civil engineering with an emphasis in structures. He received his professional engineering license in the states of North and South Carolina in 2020 and received his Level 1 – Certified Infrared Thermographer certification in 2021. He has received training on whole building air testing through Retrotec as well as training on utilizing WUFI Pro. Lehne's work experience includes the

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The industrial revolution in the United States brought about large-scale industrial facilities throughout the country. During the late 19th and early 20th century, these industrial facilities were generally constructed using mass masonry. Mass masonry structures are composed of multiple-wythe brick masonry and may include stone, concrete masonry units, cast stone, and terra cotta. Mass masonry was common for industrial structures during this period, as brick masonry could be fabricated from local materials and could be manufactured in a size that was manageable for the labor force. In addition, this type of construction was adequate for the industrial use requirements of the period, as the load-bearing masonry walls had a long service life (**Figure 1**).

Over time, these facilities were often abandoned, whether it was from industry shifting toward production overseas; requirements for a new, modern industrial space; or closure of businesses. With a shifted focus on sustainability and the preservation of historic structures over the last couple of decades, preservation and reuse of historic industrial facilities has become more prevalent. However, these facilities are often inadequate for the requirements of modern-day industrial use, which results in an adaptive reuse approach. Adaptive reuse often changes the occupancy type of the structure and requires restoration of the facility along with physical modifications to accommodate the intended occupancy and to meet the current design requirements for the new occupancy type. Due to the location, history with surrounding communities, desired industrial aesthetic appearances, and sustainable use of existing materials, adaptive reuse projects have been presented as viable alternatives to new construction. Reuse of existing building stock in-situ is a sustainable building option for multiple reasons: the structure and exterior enclosure are existing and the embodied carbon/energy within the existing on-site materials is retained; there is a reduction in the materials which must be provided and transported to the site for construction; and the historic integrity of the structure can be preserved. While the cost of new materials is avoided and historic tax credits are often granted, owners/developers should be prepared for specialized costs, such as including additional evaluation and design needs stemming from working with existing conditions, unexpected existing problems that will require solutions during construction, the cost of skilled restoration labor for the mass masonry, and the cost of replacement materials which will maintain the historical integrity of the structure.

Design considerations for restoration and adaptive reuse involve knowledge outside what is typically understood for new, modern cavity wall construction. A lack of understanding regarding the building science related to the original construction and the design considerations for the correct performance of the building enclosure for its new occupancy type can result in multiple challenges during construction and future occupancy of the structure. It is important for the design/consulting team to communicate an understanding of the cost, effort, and design of the building enclosure

FIGURE 1. Overview of previous industrial mass masonry structure during construction of adaptive reuse project.

system which is required for adaptive reuse of historic mass masonry structures. An understanding of and investment in design and evaluation for adaptive reuse projects can prevent costly errors during construction and potential errors that will affect the structure's ability to meet the requirements of its new occupancy.

RESERVOIR SYSTEMS

When compared with modern-day cavity wall construction, a mass masonry building enclosure functions differently. While cavity wall systems are intended to function as a barrier system with a drainable cavity, a mass masonry building enclosure is a reservoir system that is designed to interact with moisture. A mass masonry wall (reservoir) system impedes bulk water from reaching the interior of the structure during a wetting event by shedding a majority of the water at the exterior surface. Water absorbed past the exterior surface into the masonry is retained and subsequently released once the wetting event has passed. Depending on the exterior and interior environmental conditions, water absorbed within the masonry wall has the potential to dry to the interior or exterior of the wall.

Originally, the potential for moisture drive to the interior was accommodated, intentionally or unintentionally, by leaving the interior masonry exposed or by installing interior finishes with high permeability. This was conducive to drying that may occur through the interior masonry face. Additionally, when compared with modern-day structures, these structures did not have the same requirements or considerations for energy efficiency. Often, during the cold months, excess heat was provided to the interior environment of industrial facilities through radiant heaters, while in hot months, fenestration with large, operable panels provided ventilation.

FIGURE 2. View of moisture-related damage to interior finishes.

FIGURE 3. View of low permeability barrier installed at wall interior.

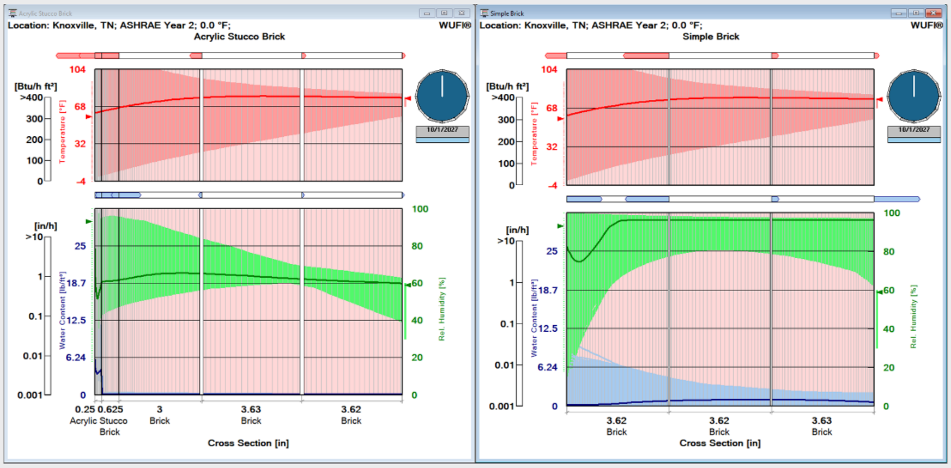

FIGURE 4. WUFI analysis results in animation still.

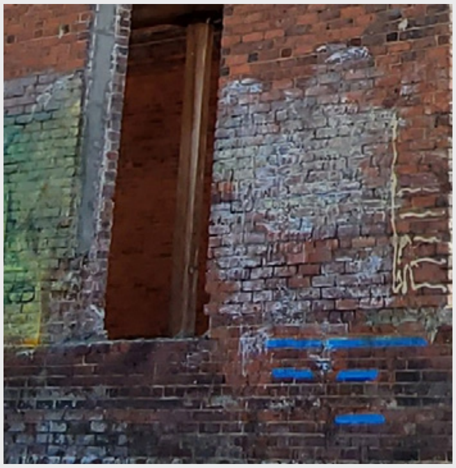

FIGURE 5. Representative view of masonry requiring restoration.

The excess heat during the cold months resulted in drying of the reservoir systems to the exterior, while the lack of air conditioning during the hot months mitigated colder interior surfaces' propensity for condensation. Modern HVAC systems, which heat, cool, and often dehumidify buildings, along with requirements for higher R-values of the wall assembly, were not factors for these structures during the period when they were constructed; thus, they are important factors to consider for adaptive reuse.

In the past few decades, an increasing number of these mass masonry industrial structures are being converted to modern living spaces. Often, these renovations include the addition of insulation to the wall assembly; less permeable interior finishes, which are frequently moisture sensitive (**Figures 2** and **3**); and a conditioned interior environment. Conditioning the interior environment, installing insulation, and applying moisture-sensitive interior materials create conditions which can result in moisture-related damage. This elevated moisture within the interior finishes presents a potential for biological growth.

Adding insulation to the exterior walls to meet modern and code-required R-values requires special consideration for mass masonry (reservoir) systems. Historic preservation requirements, requirements of the intended occupancy, and design analysis of the hygrothermal response changes to the building enclosure system and changes to the building enclosure system requirements are integral parts of a mass masonry adaptive-reuse project.

BUILDING LAYERS AND DESIGN ASSESSMENT

Assessing historic structures for adaptive reuse requires a multifaceted approach. As a first step, the designer/consultant should compile and review the structure's existing history and documentation. Expectations for historic integrity and allowable modifications should be identified and agreed upon, as they will guide design decisions. Once existing information regarding the structure is known, an evaluation of the code requirements for that occupancy and the modifications of the existing structure required to meet them can be performed.

It is important for the designer to properly assess the modifications to the building envelope system that are necessary for the new occupancy requirements. Understanding the performance of the in-place mass masonry structure and any potential adjustments to the system(s) is critical to ensure the system(s) will function as intended. This is accomplished through an understanding of building science, use of hygrothermal analysis, and proper detailing. The Glaser method, known as a dew-point analysis, provides the location of the condensation temperature. However, this method is limited in the information it can provide, as it does not take into account multiple factors, such as built-in moisture and driving rain, and is based on a steady-state condition. Fraunhofer IBP developed Wärme Und Feuchte Instationär (WUFI) software to perform dynamic hygrothermal analysis. WUFI is a valuable tool for understanding modifications to mass masonry construction that are required for adaptive reuse projects. WUFI provides the designer with an evaluation tool to understand how the modifications to the reservoir system will affect its hygrothermal performance (**Figure 4**).

SITE EVALUATION

Masonry restoration and evaluation will be required as part of adaptive reuse. Over its lifetime, a mass masonry reservoir system will have a number of maintenance-related items, including but not limited to repairs of cracking within the masonry; repair of spalled masonry units; repointing mortar joints that have cracked, recessed, or separated from the brick masonry units; removal of biological growth; removal of abandoned fasteners or abandoned steel elements; and repairing abandoned penetrations (**Figure 5**). Cracking must be evaluated to determine if there are underlying conditions that need to be resolved to prevent the cracking from reappearing after the masonry has been repaired and to identify potential structural issues.

In addition to general maintenance items, the porosity of the brick masonry may need to be evaluated. Over time, brick masonry expands from its kiln-

FIGURE 6. Representative view of unadhered pockets of interior coating with efflorescence behind unadhered coating.

FIGURE 7. Representative view of damp portion of wall with water weeping from anomalies at the interior masonry surface.

fired volume and increases in porosity. Increased porosity allows for additional water to be absorbed into the masonry and for the water to travel further within the reservoir wall system.

Increased porosity of the brick and anomalies within the brick masonry can prevent the masonry from properly impeding the movement of moisture to the interior during a wetting event and can result in overloading of the reservoir system or in bypasses through the reservoir system. This can allow for bulk water intrusion or for an increase in the moisture present at the interior face of the masonry.

EVALUATION CONSIDERATIONS

On-site testing is recommended as part of the mass masonry evaluation and adaptive reuse design process. Because mass masonry reservoir systems behave more as an integrated unit compared with modern veneer cavity wall systems, evaluation of mass masonry walls for water intrusion can be more difficult than evaluation of a cavity wall system. Testing is often limited to a few components at a time and may not capture the multiple components contributing to overloading of the mass masonry wall during a single test. The familiar testing approach of starting at the base of the area of concern, moving horizontally, and then moving vertically is also applicable to testing mass masonry. This should include care to isolate testing areas and prevent overspray and runoff from impacting locations outside of the test. However, documentation of the condition and locations of the areas tested prior to the current test is a key factor in understanding what conditions are contributing to the overloading of the masonry system. There is a potential that the previously tested areas will contribute to the overloading of the system, resulting in water intrusion observed during a later test.

When evaluating mass masonry walls for water intrusion, two key factors help to interpret the results and guide the recommendations:

- system vs. condition and
- failure mode.

Water intrusion through a mass masonry reservoir system may be the result of an inability of the building envelope system to meet the requirements demanded of the system as designed and/or it may be the result of degradation or anomalies within the building envelope system. It is important to distinguish which aspect of the system is being evaluated during testing: its adequacy or its condition. A system that has performed well in the past may have additional components installed that result in moisture-related failures of the assembly or may be introduced to additional requirements that are outside of its original design. These can include installation of new interior finishes, installation of additional thermal resistance, installation of new exterior applications to the mass masonry, or changes to the interior environmental conditions. Evaluation of the interior environmental conditions, destructive testing to evaluate components which may have been added to the mass masonry system, evaluation of site and roof drainage, evaluation of observed cracking to determine if there are underlying conditions, and evaluation of potential rising damp provide information on whether the existing building envelope system needs to be modified to adequately meet the current performance requirements. Evaluation of the condition of the masonry for maintenance items such as cracking, debonding of the mortar from the masonry, recessed mortar joints, biological growth or foreign materials, condition of the fenestration and the installation, porosity of the masonry, and spalling provides insight into the restoration efforts which will be required to restore the system back toward its original level of performance.

The failure mode observed while testing provides valuable information to assist the designer/consultant with recommendations and repairs. The quantity of water intrusion observed should be noted to inform the designer of the severity of the issue they are attempting to address. Water intrusion through mass masonry may be observed as a damp portion of the masonry wall that does not result in bulk water intrusion, a damp portion of the wall that results in bulk water intrusion through multiple gaps and cracks within a masonry wall area, or bulk water intrusion that is confined to a condensed entry point. While a damp interior masonry surface may not result in bulk water intrusion, it can contribute to failure of interior coatings and finishes by increased moisture drive to the interior and by increasing the moisture content at the interior finish–to–masonry interface. When water passes through masonry, or cementitious materials such as mortar, it can transport minerals from within the masonry and deposit them on the face of the masonry; this is known as efflorescence. Coating failure from elevated moisture or vapor drive typically consists of unadhered pockets of the interior coating (**Figure 6**). When opened, efflorescence within the unadhered coating pocket is often observed.

Water intrusion that starts as a damp portion of the masonry and results in bulk water intrusion through multiple anomalies within an area of the masonry wall points to overloading of the masonry reservoir system, which may require greater effort to restore the mass masonry.

Water intrusion at a condensed interior entry point indicates a more direct pathway through the masonry wall (**Figure 7**).

SITE EVALUATION PROCEDURES

The following are evaluation procedures that can be utilized when assessing mass masonry reservoir systems.

ASTM E2128

ASTM E2128, *Standard Guide for Evaluating Water Leakage of Building Walls*,[1] provides an outlined standard for evaluating water leakage of building walls. This includes, but is not limited to, references to AAMA 501.2, *Quality Assurance and Diagnostic Water Leakage Field Check of Installed Storefronts, Curtain Walls and Sloped Glazing Systems* (Monarch Type B-25 brass nozzle testing);[2] ASTM E1105, *Test Method for Field Determination of Water Penetration of Exterior Windows, Skylights, Doors, and Curtain Walls, by Uniform or Cyclic Static Air Pressure Difference* (spray rack testing);[3] and AAMA 511, *Voluntary Guideline for Forensic Water Penetration Testing of Fenestration Products*.[4] The primary purpose of ASTM E2128 is to "recreate leaks that are known to occur," not to "demonstrate code compliance or compliance with project documents unless such deviations are actually related to the leakage problem."[1]

Modified ASTM E1105

A modified ASTM E1105 procedure is available for testing of masonry (**Figure 8**). It is titled *Using Modified ASTM E1105 to Identify Leakage Sources in Building Wall Systems*.[5] This modified procedure recommends utilization of a similar water application rate, but instead of a 15-minute test duration, a 30-minute test duration is specified. This modified method lists construction of a negative pressure chamber on the interior as optional.

Masonry Absorption Testing In-Situ

When evaluating masonry for absorption of water at the exterior surface, two methods currently exist: RILEM tube testing and ASTM C1601, *Standard Test Method for Field Determination of Water Penetration of Masonry Wall Surfaces*.[6] The data obtained from this testing provides information regarding the porosity of the brick masonry and whether options to reduce porosity should be considered. This test method is also useful in evaluating coating materials as previously described to determine the efficacy of the material being used in conjunction with the masonry of the structure. A baseline reading of the material must first occur, followed by application of the coating material and subsequent testing to compare the results to determine if the material is functioning as intended.

RILEM Tube Testing

Technical committees, through *Reunion Internationale des Laboratoires d'Essais et de Recherches sur les Materiaux et des Constructions* (RILEM), developed a testing method for evaluating water absorption rates of masonry exposed to water at the exterior surface. This testing method is commonly called the RILEM tube test. This procedure involves using a graduated cylinder with an open end attached to the wall using putty to ensure a tight seal. Water is filled to the

FIGURE 8. Representative view of Modified ASTM E1105 testing of mass masonry.

appropriate level (the water level loosely correlates to wind loading) and is allowed to remain within the cylinder for a measured period of time. At the end of the intended test time, the water level is measured, and an absorption rate is calculated from the change in water level over time and from the surface area of the wall that is exposed to the water in the RILEM tube. When utilizing RILEM tube testing, different sample locations should be tested, including the face of brick, mortar T-joints, mortar bed joints, and mortar head joints.

ASTM C1601

ASTM C1601[6] measures water penetration of an in-situ masonry surface. This test involves mounting a minimum 12 square foot, closed chamber to the exterior side of the masonry specimen. The chamber is positively pressurized, and water is introduced within the chamber as a sheet flow down the face of the masonry. The water applied to the masonry is drawn from a well with an initial water volume and is returned to the well through an outlet at the bottom of the chamber, creating a closed testing system where water can only escape by absorption into the masonry. The change in water volume over time and the change in water level at the end of the test correlate to the water absorbed into the masonry through application to the exterior surface.

Both RILEM tube testing and ASTM C1601 have benefits and limitations. RILEM tube testing is easier to set up and less time-consuming compared with ASTM C1601 testing. A RILEM tube test can be completed by one person in the span of 15 to 30 minutes, with multiple RILEM tube tests running at the same time. An ASTM C1601 test can take between four and six hours to complete and often requires two people to set up. However, the exposed masonry surface area during a RILEM tube test is 0.88 square inches, compared with the minimum exposed masonry surface area during an ASTM C1601 test of 12 square feet. According to an article by the National Concrete Masonry Association, "A study conducted at the University of Wyoming concluded that 1,665 tests would need to be conducted for every 12 ft^2 (1.11 m^2) of wall surface being evaluated in order to achieve a sample error of 10% or less [8]. Hence, drawing any conclusions about the water penetration characteristics of an entire wall assembly based on 50, 100, or even 500 tests can be speculative at best."[7] This does not mean that data from RILEM tube testing is not useful, but rather that there are additional limitations on the conclusions that can be drawn from the data. RILEM tube testing provides a simple and portable evaluation tool to make relative inferences regarding the absorption performance of the masonry. However, if the designer/consultant desires to measure and report in-situ surface water absorption rates for masonry, ASTM C1601 should be utilized. Both tests are best used to test before and after results for a mass masonry wall. While an absorption rate is obtained from ASTM C1601, the test does not include pass/fail criteria.

Air Leakage Testing

ASTM E779, *Standard Method for Determining Air Leakage Rate by Fan Pressurization*[8] is a quantitative test for measuring building air leakage. ASTM E1186, *Standard Practices for Air Leakage Site Detection in Building Envelopes and Air Barrier Systems*[9] is a qualitative test for identifying potential sources of air leakage. Both test methods use a mechanically produced pressure differential across the building envelope (pressurization and/or depressurization). ASTM E1186 has multiple methods for utilizing tools to identify air leakage locations, including infrared thermography, hand-held or theatrical smoke, pressure chambers, and bubble gun testing.

Detailing

As mentioned earlier in this paper, mass masonry walls do not conventionally contain drainage planes and do not have cavities containing insulation or open cavities where insulation may be installed. When designing repairs or retrofit conditions needing bulk water and water vapor resistance, the wall must be treated as a barrier system. As will be discussed, this simply written requirement can create other iss ues for the designer which also must be addressed.

Because mass masonry walls provide water shedding and reservoir retention, the first approach to reusing mass masonry walls is to repair the existing masonry components back to original conditions, or as close as is possible with modern methods. This includes items discussed previously. The specifics of masonry repair can be found in many different publications and are not covered in detail within this paper. However, the proper design and specification of the mortar joints will impact how well the wall can shed water.

As noted earlier, the design of the new mortar joint is critical. Not only the joint shape, but also the pointing mortar's composition should be carefully selected. Depending upon the historic nature of the wall and whether the original wall design is legally protected by historic regulations, a designer should specify the mortar joint geometry to be concave. Many historic masonry walls were constructed with joints struck flush with the wall face or raked back from the wall face. In some cases, the authors have seen joints raked as much as ½ inch back from the wall face. Numerous studies have been performed and results published regarding how these types of mortar joints have far inferior resistance to water absorption compared with a concave struck joint. During repair design, the use of a concave struck mortar joint should be considered. The concave shape helps the joint resist moisture intrusion. The mortar composition should be soft or softer and should have porosity similar to or greater than the existing mortar. Throughout history, mortar joints have been the "sacrificial" portion of a mass masonry wall and were softer than the surrounding brick. If any conditions were to place stresses on the wall, including the natural expansion of the brick masonry units over their lifetime, the mortar joints would degrade to prevent the alternative face spalling of the brick masonry units. To replace historic mortar with a more modern, harder mortar may cause internal stresses to fracture the now softer masonry units, rather than the mortar joints. To match the existing mortar strength, petrographic and/or chemical tests should be performed on wall samples. At a minimum, mortar much softer than modern mortars should be selected for historic masonry mortar pointing. This is a durability design decision, though, and should be thoughtfully considered. If the reader wishes to know more about selection of specific historic mortar mixes and why concave joints resist bulk water better than other joint geometries, numerous articles and research results can be found within the industry and academia addressing the specifics of these topics.

The historic nature of mass masonry walls predicates that most were constructed prior to the mechanical conditioning of interior spaces. The reuse of buildings built with mass masonry walls creates a state where the mass of the wall alone must separate interior and exterior environmental conditions. These environmental conditions may often be on opposite ends of the environmental spectrum, such as hot/humid outside and cold/dry inside. Also, given the mass of a mass masonry wall, there is intrinsically some insulative value, but not to a degree which would help prevent condensation on the colder side of the wall. In addition, masonry, being an absorptive material, will naturally allow water vapor to be transported from the high-pressure side of the wall to the low-pressure side. This makes the design of a water-resistant exterior wall using existing mass masonry a difficult endeavor. No matter the final design solution, the wall design and expected wall performance should be coordinated with the design of the HVAC system. It is likely the HVAC system may have to accommodate thermal and humidity conditions affected more by the exterior environment than would occur in a more modern building. A discussion about HVAC design is beyond the scope of this paper, though, and will only be touched upon, as mass masonry walls are impacted by the differences between the interior and exterior environments.

Given that the nature of mass masonry requires the mass of the wall to respond as a barrier to air and vapor transmission, design options are rather limited. Bulk water must be controlled at the exterior masonry face. Vapor transmission could be controlled at the interior or exterior face but, given that the bulk water should be controlled at the exterior face, the design should not develop a condition where any moisture could be trapped within the mass of the wall between interior and exterior control layers.

There are multiple ways to make a mass masonry wall perform better as a barrier system to liquid water. These are typically in the form of coatings which are applied to the exterior surface of the building. The desired efficacy and aesthetic results will influence a designer's decision as to which method is selected. Any method selected may affect the final appearance of the building and could impact any historic designation the building may carry.

More difficulty is imparted to a mass masonry renovation project when designing approaches to thermal barriers. Raising the thermal resistance of a mass masonry wall requires adding insulation to the wall. Adding it to the interior side of the wall, which is the only place to physically locate it without changing the exterior aesthetics, changes the thermodynamics of the wall. The interior masonry face, which was once exposed to the conditioned or tempered interior air, is now thermally separated from the interior, making it colder. Depending upon the geographic building location, this could make the interior masonry face reach temperatures where condensation of interior water vapor could occur, should that vapor be allowed to pass through the insulation. Interior vapor barriers may be used but could also create a condition where moisture within the masonry which evaporates toward the building interior could become trapped within the wall behind the vapor barrier. This approach may influence the selection of the liquid water barrier design for the exterior masonry face. In this case, it would be imperative to use a liquid water barrier with high permeability, allowing any water vapor within the masonry to dry to the exterior of the building and not become trapped behind the interior vapor barrier. Interior vapor barrier selection should also be very carefully considered with some consideration toward "smart" vapor barriers, which can change permeability depending upon the level of humidity present.

As can be expected at this point, the hygrothermal changes stemming from modern exterior wall renovations are complicated and difficult to determine through general knowledge of thermal movement from hot to cold and vapor movement from high pressure to low pressure. This is where WUFI analyses performed on the original wall design and then on various design options can greatly help the designer better understand how a new wall design may respond to the environmental conditions and whether over time, it will have an opportunity to dry and remain within the parameters where condensation and the possible biological growth associated with moisture and many building materials do not form.

Water-Repellent/Waterproof Coatings

Often, water repellents and waterproof coatings are used interchangeably; however, there are important differentiating factors that should be understood prior to approaching restoration. The National Parks Service brief "Assessing Cleaning and Water-Repellent Treatments for Historic Masonry Buildings"[10] describes water-repellent coatings as breathable, meaning they allow vapor to pass through the system while keeping liquid water from penetrating the surface. Conversely, waterproof coatings are intended to seal the surface from liquid water and vapor.

While the first line of defense against water intrusion should be properly repointed and repaired masonry, often water intrusion may still appear, whereby alternative options such as coatings as described may be considered. If moisture intrusion continues following proper repairs, consultation with an architectural conservator should be made to determine applicable systems and approach strategies.

These coating systems are often inaccurately prescribed to remedy bulk water intrusion without understanding the function of the wall system. Most

FIGURE 9. View of cotton mill constructed in 1897.

historic masonry structures have survived hundreds of years without the use of coating materials and, if properly maintained, should continue to function as designed.

Detailing coatings around wall openings, such as windows and doors, can be rather difficult. In a wall containing a drainage plane, fenestrations can be sealed to the barrier creating the drainage plane. This creates continuity of the water barrier from the drainage plane to the fenestration. With coatings applied to the building exterior, or those which are absorbed into the masonry units, creating a continuous system requires removal of the sealants around the fenestrations and application of the coating to a point beyond where a proper seal may be made between the fenestration and the wall. Ideally, all fenestrations would be removed prior to installing the coating. This allows the coating to wrap the entire fenestration opening. However, water-repellent coatings are often the type which is designed to penetrate the masonry and can be relatively transparent. Ensuring the fenestration sealant is continuously sealed to the coating around the perimeter of all fenestrations cannot be ensured without water testing the fenestrations following the completion of the coating and installation of the fenestration sealant. It should be noted that these systems often have wind-driven rain warranty limitations, are limited in their warranty duration, and generally require maintenance after 10 years.

CASE STUDIES

Terracon has been involved with several projects where historic industrial buildings, namely mill buildings in the southeastern US, have been repurposed for multifamily residential or office buildings. The historic appearance of the building was desired to be retained while providing a conditioned interior for the occupants. With most of these projects located in the southern US, the design cooling load was high, and the vapor drive was predominantly from the building exterior to the interior. The locations of the air and water management planes required scrutiny. Selection of building enclosure improvement materials and detailing of transitions between components went through several iterations and reviews to determine the best solution for the given conditions.

Case Study 1

A former cotton mill constructed in 1897 was reimagined as a high-end apartment complex (**FIgure 9**). The two-story structure consisted of brick mass masonry wall construction with stucco applied over the brick at various locations. The spaces include primarily residential units, leasing office, recreational space, fitness room, and clubhouse. Construction consisted of interior and exterior renovations, including window replacement. The existing window systems were 12 feet tall with segmental arched tops replaced as part of the renovation. The arched head and jamb interfaces were mass masonry with the sill finished in concrete (**Figure 10**). Shortly after the building opened, leaks were reported by residents at windows. Water testing was performed on the assemblies isolating the fenestration system and the surrounding construction independently. Windows installed in mass masonry construction require particular attention to detail, as the interfaces surrounding the windows provide opportunities for moisture to penetrate the masonry and migrate beyond the system and into the interior space. Upon investigation, it was determined that water intrusion was a combination of the assembly construction including sealant joints as well as migration through the masonry adjacent to the fenestration. Recommendations provided to the client first included the repair of the fenestration assemblies. Because the fenestration assemblies were customized, limited modifications could be provided to mitigate the moisture

FIGURE 10. Representative view of new fenestration installed in existing mass masonry wall opening.

FIGURE 11. Representative view of cracking in stucco.

FIGURE 12. Representative view of forensic water testing.

surrounding the opening; therefore, the perimeter of the assembly was detailed using masonry sealer.

Case Study 2

A firehouse constructed circa 1890 comprising a load-bearing, multiple-wythe brick masonry structure with a wood-framed interior was remodeled in the 1930s, including the installation of cement stucco on the brick masonry wall exterior. At some point in the building's history, cement stucco was also applied to the interior. A second coat of softer, possibly gypsum-based stucco was added to the interior over the cement stucco. When the building was adapted to be a conference center, modern HVAC systems were added and due to its location in the southeast US, and the numbers of people who can fill the conference center, it is often in cooling mode, which is also drying the interior environment.

In recent years, the building owner had repaired numerous problems with the exterior and interior stucco (**Figure 11**). The exterior cracks had been repaired in 2010 and 2016, yet the interior continued to experience water damage in the form of the soft stucco coat spalling and the interior paint bubbling. Terracon was contracted to do visual observations and water testing to determine the source of the water intrusion and interior damage (**Figure 12**).

The visual observations and testing revealed numerous locations where liquid water was infiltrating the stucco and getting into the brick masonry substrate (**Figure 13**). Through capillary flow and the drying of the exterior wall to the building interior, a significant amount of liquid water and water vapor were damaging interior finishes (**Figure 14**). Difficulty arose in understanding which exterior conditions created which interior damage. Water from testing was revealing itself on the interior in locations unexpected by the testing team.

For this reason and to ensure a continuous barrier system could be installed around the entire building, a waterproof coating with high permeability was specified for the exterior. All the windows were specified to be replaced, which allowed the coating to wrap the window openings and the perimeter sealant to bridge between the window frame and the coating at every window.

Because the application of the coating would likely trap some of the existing moisture within the wall and create a condition where the path of least drying resistance was toward the interior, the owner was advised the interior repairs would need to wait for a couple of months. This was to ensure most of

FIGURE 13. Representative view of original fenestration.

FIGURE 14. Representative view of soft stucco wrapped into the window rough opening.

the walls would be dry enough to not create further problems with interior finishes. The interior finishes were also addressed. The soft stucco was recommended to be removed and replaced with cement stucco, which would be less susceptible to moisture damage. To help allow the wall to dry to the interior, high-permeability paint was specified.

In addition, the roof was nearing the end of its serviceable life. This allowed the design team to develop a parapet cap solution, making the new coating continuous with the roofing system. Therefore, the entire building enclosure would be continuous and tight from the grade to the roof.

CONCLUSION

In closing, historic industrial mass masonry buildings are a popular candidate for adaptive reuse, such as multifamily dwellings and commercial buildings. These new uses within historic enclosures present design and performance challenges which can create conditions detrimental to interior construction and air quality and have negative impacts upon the users. By understanding the building science behind how mass masonry walls perform and how the changes to the interior environment influence performance, the designer can better make repair decisions and material selections, which can extend the life of the building and provide a sustainable and financial benefit for the building owner.

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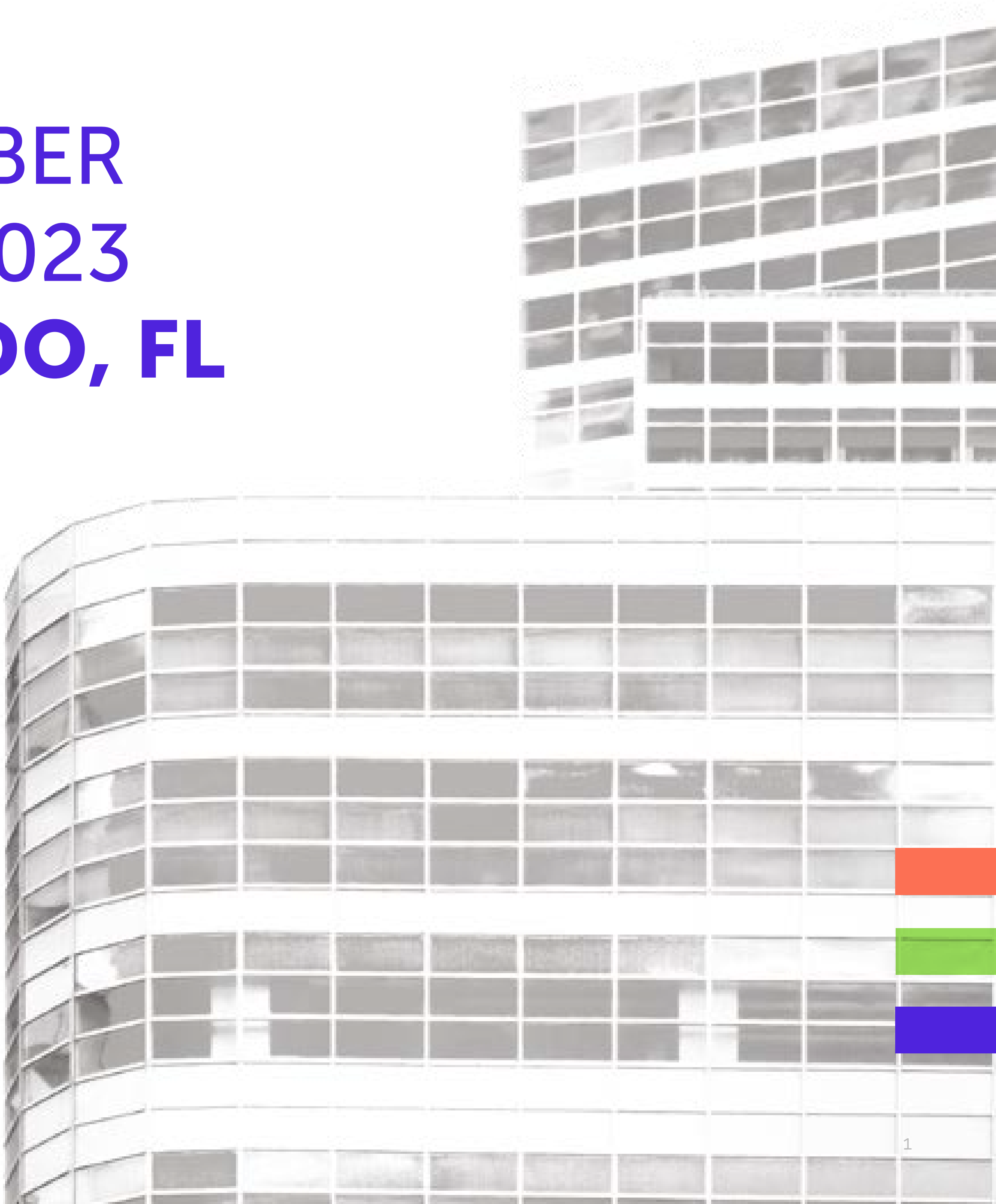

BRICK BY BRICK: TRADITIONAL AND UNCONVENTIONAL MASONRY RESTORATION STRATEGIES

ABSTRACT

Exterior masonry wall design and construction practices have evolved to include mass, transitional, barrier, and cavity walls. As the inventory of these buildings age into the future, repair and/or restoration will be required. It's not a question of if, but rather when and how.

Although time-tested, traditional repair strategies are suitable for many projects, other lesser-established unconventional strategies can be considered to improve exterior wall performance. Over-cladding or exterior coating application can be implemented to fundamentally transform the exterior wall into a cavity wall or barrier wall, respectively. Unconventional interior repairs, including the use of crystalline waterproofing technologies, urethane foam, or variable vapor retarders in conjunction with insulation can also be considered to improve wall performance with respect to water leakage, air infiltration, and/or thermal properties. This article covers both traditional options and "outside-the-box" strategies for masonry restoration and repair projects. This article also includes discussions related to building science, air and vapor transport related to traditional and unconventional strategies, and several case studies.

LEARNING OBJECTIVES

- Define masonry wall types as mass, transitional, cavity, or barrier walls that can be constructed of many different materials.
- Demonstrate an understanding of building science associated with masonry walls and the impacts of various restoration strategies.
- Review various traditional and unconventional masonry restoration options to overcome issues associated with water leakage, air infiltration, and thermal performance.
- Describe advanced technologies that can be applied to renovations associated with exterior masonry walls.

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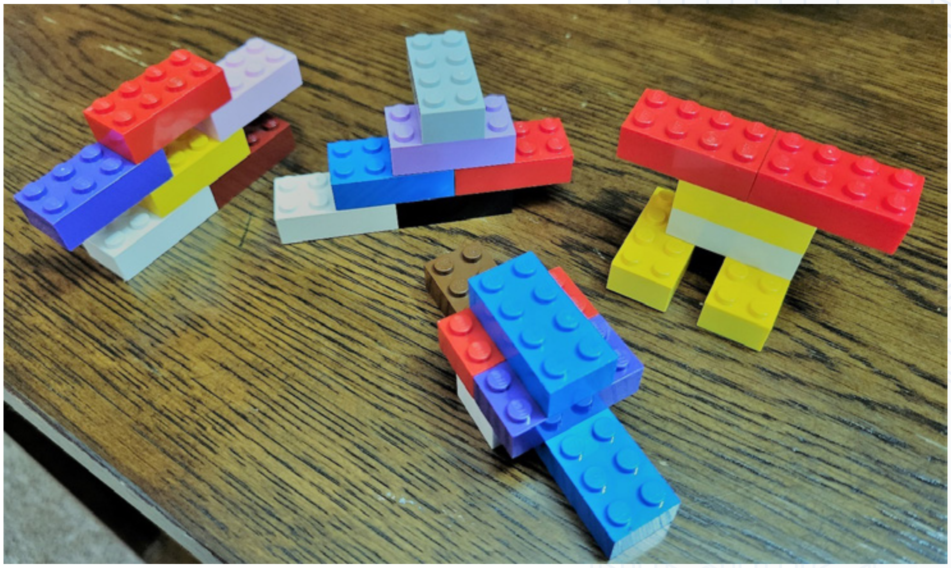

FIGURE 1. Four of 915,103,765 possible ways to combine 6 eight-stud LEGO bricks.

Exterior masonry walls are designed and constructed using a variety of materials and with various strategies to limit air movement into and out of buildings, manage moisture, and provide thermal control. Even if buildings are constructed similarly, exterior walls will differ with respect to material properties, conditions during fabrication and construction, and workmanship. Given that there are 915,103,765 ways to combine six 8-stud LEGO bricks (**Fig. 1**), there must be an unlimited number of ways to configure different exterior masonry walls. No two exterior masonry walls will be exactly the same.

Although buildings are often designed for useful service lives in excess of 50 years, and exterior masonry walls can be expected to last for more than 100 years if properly maintained, buildings begin to age immediately. Aging exterior masonry wall components and systems will need to be maintained and repaired over time. It is not a question of if but rather when. This paper explores opportunities to improve exterior masonry wall performance with respect to moisture management, air infiltration considerations, and thermal properties. In addition to traditional restoration strategies such as repointing that are generally well known by structural engineers, building enclosure consultants, and qualified restoration contractors, this paper explores unconventional restoration strategies as a series of case studies.

BUILDING BLOCKS (MASONRY UNITS)

Fabrication of masonry units began millennia ago with materials that were available to local populations. As manufacturing technology and transportation infrastructure advanced over time, masonry units became readily available around the world. Due to its versatility and durability, masonry remains popular as a construction material today. Common types of masonry units include clay and concrete units, natural stone, calcium silicate units, and glass block, among many others. The specifications for each type of unit are based on several properties, including compressive strength, absorption characteristics, saturation coefficients, and others that can be evaluated by means of standards available through ASTM International and industry organizations. For exterior masonry wall assemblies, masonry units are typically bound together with mortar and, in some instances, with grout. The characteristics of several common masonry unit types are briefly summarized in the following sections.

Natural Stone Units

The first natural stone units used in exterior wall construction were crudely stacked. As craftsmanship improved and tools advanced, natural stone units were shaped into polygonal or square units so that close-fitting joints could be achieved. Common types of stone used in exterior wall construction include granite, limestone, sandstone, and marble. These units today are available in a wide range of sizes, shapes, textures, and finishes achieved by polishing or machine tools. The specific properties of each stone vary, and the absorption properties are typically dependent on the density of the stone. Many natural stone units can be used in load-bearing wall assemblies, as a veneer, or as part of a rainscreen cladding system.

FIGURE 2. Various forms of exterior masonry wall deterioration.

Clay Masonry Units (Brick)

Clay masonry units have been in use for at least 10,000 years. Originally, these units would often be air- or sun-dried for five years or more. Today, the entire brick-making process can be completed in less than a week with a kiln, which allows for the firing of bricks in a continuous process. Immediately after firing, clay masonry units begin to absorb moisture from the environment, and the accumulation of moisture within the units results in slow, irreversible expansion. Clay masonry units can be hollow, solid (units that are more than 75% solid), or 100% solid. They are currently classified by three grades: severe weathering (SW), moderate weathering (MW), and negligible weathering (NW). Grade SW units are the most durable with respect to exterior conditions.

Calcium Silicate Units

Calcium silicate units are manufactured using sand, lime, and water. They are air dried, but unlike clay masonry units, they are exposed to steam under pressure to cure. The manufacturing process attempts to emulate how stone is formed within the earth, though in a much more rapid manner. During the manufacturing process, raw materials chemically react to form a calcium silicate hydrate binder, resulting in integrally bonded units. Unit strength depends on the quality of the binder, the pressure of the press, and autoclaving conditions. Calcium silicate units exhibit shrinkage over time and deform when loaded, but they are rarely subjected to high enough stress levels in service that creep becomes significant. There are currently two defined grades for calcium silicate units: those appropriate for severe weathering (SW) and moderate weathering (MW) conditions.

Concrete Masonry Units

Concrete masonry units (CMUs) are fabricated with portland cement, aggregate, and water. Additives and pigments can also be included to aid with moisture resistance, curing, coloration, and finish properties. CMUs derive their strength from the cement hydration process, and much of concrete technology is applicable to CMUs. CMUs can be fabricated as concrete blocks or concrete bricks. Concrete blocks are used in both load-bearing and non-load-bearing applications, whereas concrete bricks are more typically used within non-load-bearing veneers. Block CMUs are classified as Type I (moisture controlled) or Type II (non-moisture controlled), and Brick CMUs are classified as Grade N (architectural veneer) or Grade S (general use). CMUs exhibit shrinkage over time due to drying shrinkage, carbonation shrinkage, or both drying and carbonation shrinkage. Repeated drying and wetting of units can also result in reversible shortening and expansion, respectively.

MASONRY DETERIORATION MECHANISMS

Like most construction materials, masonry is subject to deterioration over time in the presence of moisture, other environmental factors, and loading (**Fig. 2**). The following are several types of masonry distress:

- **Cracking:** Cracking is defined as a splitting within masonry units, mortar joints, or both, due to one or many internal or external stresses.
- **Delamination and spalls:** Delamination involves debonding of the exterior surface of a masonry unit and can present a potential fall hazard. A delamination that has separated from the unit, revealing the inner surface of the masonry unit to the elements, is classified as a spall.
- **Bond line separation:** This type of masonry distress is a failure in the bond between masonry units and mortar joints.
- **Mortar washout:** Mortar washout is defined as mortar deterioration and erosion of the mortar from within the joint.

Cracking, delamination, bond line separation, and mortar washout all can allow water to intrude into an exterior masonry wall. Water can pass through imperfections or cracks as small as 0.005 in. (1.3 mm; slightly thicker than a human hair), and it can enter through even smaller cracks when it is subjected to a pressure differential. Once water enters beyond the exterior face of a masonry wall, it can cause additional distress including the following:

- **Deterioration from freezing and thawing cycles:** When water freezes and expands within the pores of a masonry unit, internal tensile stresses within the material can lead to cracking, delamination, and possibly spalled units (**Fig. 3**).
- **Efflorescence:** Efflorescence is generally a benign form of distress in which light-colored minerals are deposited on the surface of masonry units after water evaporates from within the walls. Efflorescence can occur on both the interior and exterior sides of the walls.

FIGURE 3. Deterioration of brick veneer from freezing and thawing cycles.

- **Corrosion of metal components:** Water provides an ideal environment for steel corrosion if chlorides or other corrosion-promoting chemicals are present. Depending on the function of metal components within wall systems, the formation of corrosion products can exert expansive stresses on masonry, leading to cracking and spalls. Additionally, masonry veneer can become unstable if masonry ties corrode and can no longer provide resistance to out-of-plane loads.

Displacement, bowing, and bulging can also occur due to compression stresses, inadequate lateral support, lack of adequate movement joints, or a variety of other reasons (**Fig. 4**). Dimensional changes can also occur due to creep of CMUs or a backup concrete structure, volume changes (masonry walls are constantly expanding or contracting), and other factors. Displacement and dimensional changes can cause masonry to deteriorate, especially in areas of restraint where the natural movement of exterior wall components is restricted.

TYPES OF MASONRY WALL SYSTEMS

The design of masonry exterior wall systems has evolved over time. Most of these systems primarily fall within the categories of mass, transitional, cavity, and barrier walls.

Mass Masonry Walls

Mass masonry exterior wall systems were commonplace in buildings constructed before the 1950s. Buildings supported by load-bearing mass masonry walls are generally limited in their height to approximately four

FIGURE 4. Two examples of masonry wall bowing due to lack of movement joints.

FIGURE 5. A thin layer of insulation provided on the interior side of a mass masonry wall.

stories, although taller exceptions are common. The tallest modern load-bearing mass masonry building on record is the 16-story Monadnock Building in downtown Chicago, Illinois, constructed in 1891.

The large thermal mass of mass masonry structures assists in reducing temperature fluctuations within buildings. As such, insulation was typically not provided within or inboard of the walls. In some cases, a thin layer of insulation was provided on the interior side of mass masonry walls (**Fig. 5**).

Multi-wythe mass masonry wall systems rely on their thickness and solid construction to absorb water and evaporation to discharge water that accumulates within the walls. The performance of mass masonry walls depends on several factors, including the ability of the masonry and mortar joints to reject and shed most water during precipitation events. When masonry units and mortar joints exhibit cracking and deterioration, more water can penetrate into the walls, potentially exceeding the absorptive capacity of the walls. Over time, water intrusion and cycles of freezing and thawing further deteriorate the masonry units and erode mortar within interior wythes of the masonry. As the mortar and masonry units within the walls deteriorate, water passes more easily through the walls. Eventually, restoration and repairs will be required to address water leakage issues and potential structural concerns.

In general, thick mass masonry walls (approximately 15 in. [380 mm] and thicker) are significantly more reliable with respect to resisting water leakage than thinner mass masonry walls because the thicker walls have greater water storage capacity.

Today, mass masonry walls can be constructed of single-wythe CMUs. Walls with fully grouted cores will generally provide better moisture resistance than walls constructed with cores only grouted at locations containing steel reinforcement. Integral water repellants are often included in single-wythe CMU walls to provide additional resistance to water penetration.

Transitional Walls

Transitional masonry walls encompass many wall systems developed over a short period during the late 19th and early 20th centuries following the advent of iron, steel, and concrete structural framing. In this type of wall construction, masonry and steel components were typically constructed side by side within a wall assembly. Transitional masonry structures could be built taller than their mass masonry predecessors, and they included many of the first US skyscrapers. One famous transitional structure is the Rookery Building in Chicago (1888). The proportions of the vertical loads supported by iron and steel building components and by masonry components vary widely depending on the detailing implemented.

Moisture management in transitional masonry structures is similar to that in mass masonry structures: the masonry walls absorb water and release that moisture to the exterior environment or to the building interior via evaporation. The marriage of materials in transitional walls is potentially problematic because iron or steel framing within a wall assembly is vulnerable to moisture-related deterioration. Embedded iron and steel elements that exhibit corrosion impart expansive forces on the surrounding wall elements, resulting in cracking of masonry and mortar joints. Transitional masonry walls are also susceptible to distresses caused by differential movement and thermal expansion of the multiple construction materials incorporated into the same wall.

Similar to mass masonry walls, transitional walls typically were not insulated. However, limited insulation was sometimes provided on the interior side of transitional walls.

In the mid-20th century, designers developed newer types of transitional walls with masonry veneer constructed outboard of CMU backup walls. In many cases, these walls were designed and constructed with a fully or partially grout-filled collar joint to connect the veneer to the backup walls. Originally, header courses were used to mechanically connect the veneer to the CMU. Eventually, mechanical ties and anchors were introduced. These types of walls with no air-and-water barrier (AWB) or clear drainage plane between the brick and CMU often are susceptible to water leakage.

Cavity Walls

The cavity wall system became widely used by the 1980s and is the most prevalent type of exterior masonry wall construction today. Properly designed and constructed modern cavity wall systems are often more effective at limiting water leakage when compared with mass masonry and transitional walls. Cavity wall design assumes that masonry veneer joints will allow water penetration beyond the exterior wall surface under certain conditions. A water management system consisting of a water-resistive barrier, through-wall flashing, weeps, and accessory components is required to manage and discharge water that enters the wall drainage cavity.

In modern wall construction, water-resistive barriers are constructed over solid substrates (concrete, CMUs, plywood sheathing, oriented strand board sheathing, or exterior gypsum sheathing); however, it should be noted that a water-resistive barrier is not required over concrete or CMU backup walls in all jurisdictions. In many cases, insulation is provided within the drainage cavity, although insulation provided on the building interior or between exterior wall stud framing remains common practice in certain locales. Emerging technologies that incorporate insulation and water-resistive barriers into a single product are also becoming more commonplace.

Since masonry veneer is nonstructural, it must be anchored to the structure or backup wall to resist out-of-plane loads. In cases where thick continuous exterior insulation is required, engineered masonry tie assemblies may also be required.

Barrier Walls

Barrier walls can be constructed of precast or cast-in-place concrete, insulated and formed metal panels, and exterior insulation and finish systems (EIFSs) and stucco applied directly over a backup substrate without a drainage plane. They offer only a single line of defense against bulk water penetration and are considered by some as a zero-tolerance wall system. Water that penetrates beyond the exterior surfaces of the wall and sealant joints will penetrate into the building and can cause water-sensitive concealed materials to deteriorate.

Typically, masonry has not been used as part of barrier wall systems due to the nonhomogeneous nature of such walls. However, thin masonry units can be embedded into precast concrete wall panels to provide the concrete barrier wall with a masonry aesthetic. Single-wythe CMU walls can also essentially be changed from a mass wall to a barrier wall by applying an elastomeric coating to the exterior face of the CMUs.

CONVENTIONAL MASONRY RESTORATION REPAIR STRATEGIES

Conventional masonry restoration repair strategies have been described in many previous technical publications and discussions of such repairs are not the main subject of this paper. Typical masonry restoration strategies for exterior walls include, but are not limited to, the following:

- Repointing of mortar joints
- Replacement of unit masonry materials
- Routing and sealing of cracked masonry units and mortar joints
- Application of sealant at joints between dissimilar materials and within skyward-facing joints
- Application of a penetrating water repellent to exterior wall surfaces
- Restoration or replacement of corroded steel elements such as lintels and shelf angles
- Installation of through-wall flashing at localized areas such as above lintels and below copings
- Providing supplementary anchorage or employing stabilization techniques

However, without proper design and industry-standard construction methods, conventional masonry restoration strategies can have limited benefits or result in aesthetic concerns. As an example, repointing is a common repair practice that requires removing deteriorated mortar to a uniform depth and placing new mortar within the joint. The deteriorated mortar should be removed to a uniform depth that is a minimum of twice the joint width, generally ¾ in. (19 mm), or until sound mortar is reached. If mortar is not removed to an adequate depth, deficiencies in the joint within the depth of the wall may not be uncovered (**Fig. 6**). Repointing performed to a limited depth is likely to provide only minimal benefits when compared with grinding and repointing to at least a ¾ inch depth (**Fig. 7**).

FIGURE 6. Voids in a mortar joint were uncovered following grinding during a repointing project.

FIGURE 7. Mortar joint repointing to an insufficient depth of approximately ¼ in. (6.4 mm).

FIGURE 8. Water-repellent staining on an exterior masonry wall surface.

Water repellents should not replace or be considered equivalent to essential details that resist water penetration such as through-wall flashing and weeps in masonry cavity wall construction. Additionally, only products that permit evaporation and the passage of water vapor, such as siloxanes and silanes, should typically be applied to exterior masonry walls. Although water repellents are widely used as part of exterior masonry wall restoration projects, they typically do not provide protection at crack locations in masonry units and mortar joints. Additionally, water repellents must be reapplied at regular intervals of approximately 5 to 10 years to remain effective. In cases where water repellents are incorrectly applied, staining can occur (**Fig. 8**).

BUILDING SCIENCE OF DIFFERENT WALL TYPES

Until masonry cavity walls became prevalent, AWBs, through-wall flashing, and cavity drainage systems were not typically included in the design and construction of exterior masonry walls. Today, although modern building codes typically require flashing at various locations, the use of a dedicated AWB in cavity wall assemblies is still not always required for some wall types in certain types of buildings. Although the use of vapor retarders has become commonplace on the interior side of frame walls in cold climates, vapor retarders are often misused because some designers and tradespeople do not fully understand building science related to air movement, vapor drive, and moisture management.

Similarly, exterior masonry wall assemblies were traditionally constructed without the use of insulation. In cases where insulation was provided as part of the exterior wall assembly, the insulation was usually placed on the interior face of the masonry wall, or in cases where wood stud walls or cold-formed steel back-up walls were used, batt insulation was placed between the studs. The placement of insulation between exterior wall studs remains a common practice today, primarily in light commercial and residential structures. Due to thermal bridging, such insulation only provides partial thermal benefit when cold-formed steel framing is used. It should be noted that continuous exterior insulation is now required by many energy codes, especially in cold climates.

The concept of a "perfect wall" has been around for many years. Theoretically, a perfect wall would have exterior cladding to shed water and protect the control layers (rainwater control layer, air control layer, vapor control layer, and thermal control layer) that are located on the exterior of the building structure. Also, a perfect wall could be constructed in any climate, although claddings and control layers will need to be selected accordingly.

The inventory of existing exterior masonry walls is immense and varies widely. While it may not be possible to construct a "perfect wall" when dealing with existing conditions in a restoration capacity, there are means available to improve exterior wall properties with respect to water penetration, air infiltration, and thermal performance.

CASE STUDIES

The following case studies illustrate traditional and unconventional methods that can be considered to mitigate problems with walls that fail to meet design or performance requirements. Several of the approaches described within can also change the exterior aesthetics of the building, which is a primary concern for some owners.

Case Study 1: Transitional Masonry Wall → Properly Detailed Cavity Wall at Localized Areas

The subject residential building is a four-story steel structure constructed in 1980 in a cold climate. The exterior walls consist of brick veneer over CMU backup walls and include elements of both cavity and transitional wall types. An investigation revealed that reported water leakage at window locations was due to water infiltration through the masonry exterior walls above the fenestration. To address these issues, a repair program including through-wall flashing and weeps above lintels was developed. A new AWB above the through-wall flashing was also installed to ensure a continuous drainage plane above the through-wall flashing.

The condition of the backup masonry varied throughout the building and included areas of out-of-plumb masonry, loose masonry units, and significant voids in the backup CMUs. Project specifications required repairs to the backup wall in the form of repointing, parging, and unit replacement to ensure a suitable substrate for the AWB and through-wall flashing (**Fig. 9**). Although

FIGURE 9. Preparation of a backup wall prior to installation of the AWB and through-wall flashing.

traditional through-wall flashing repairs are typically limited to the three or four courses above steel lintels, the additional repairs performed for this project were intended to limit the possibility of water leakage through deficient areas of the backup wall structure above the areas of through-wall flashing repairs (**Fig. 10**).

Case Study 2: Masonry Cavity Wall without AWB → Overclad with Drainable EIFS

The exterior walls for this building were constructed in 1981 as an addition to an existing medical facility located in a cold climate. Exterior walls at this area of the building include brick veneer over a CMU backup wall, glass-and-aluminum storefront systems, exposed concrete columns, and precast concrete wall panels at roof-to-wall transition locations. Hospital staff had complained of cold interior temperatures and condensate formation near exterior walls during winter months for many years. An investigation revealed that the extent of exterior wall insulation within the building ranged from minimal to nonexistent. Additionally, the windows were offset from the interior insulation, and their placement within the wall assembly rendered the windows "heat starved" and susceptible to condensation during periods of cold exterior temperatures.

The project team had originally considered an interior insulation strategy that would involve the application of spray polyurethane foam (SPF) on the interior side of exterior walls. However, this strategy was complicated by access restrictions, the presence of steel spandrel beams that would limit the efficacy of SPF application at top of wall conditions, and other concerns; therefore, the team ultimately implemented an exterior insulation strategy using drainable EIFS as a rainscreen. This solution also allowed for

FIGURE 10. Installation of brick veneer following installation of the AWB and through-wall flashing.

FIGURE 11. Mock-ups installed to evaluate adhesion of the AWB to substrates and the EIFS insulation to the AWB.

FIGURE 12. Completed overclad area of the EIFS adjacent to the existing masonry exterior wall.

a change in exterior aesthetics.

To achieve a rainscreen design with continuous exterior insulation, the exterior of the existing masonry walls was restored by means of localized brick replacement and limited repointing to allow for application of a continuous AWB on the exterior face of the masonry. Mock-ups were used to verify adhesion of the AWB to existing substrates and the EIFS insulation to the AWB (**Fig. 11**) before work on the overclad commenced (**Fig. 12**). Thermal modeling was also performed to verify adequate thermal performance at window locations and at roof-to-wall transitions.

Case Study 3: Masonry Cavity Wall → Overclad with Metal Panel Rainscreen System

Located in a moderate climate near the Atlantic Ocean, the subject building is a multistory medical building constructed in 1995. Lower levels of the building are constructed of brick veneer, an air space, spunbonded polyethylene building wrap, exterior gypsum sheathing, and cold-formed steel stud framing with batt insulation between the studs. Performance issues with the exterior wall assembly had not been reported during the building's service life, but the owner wanted to make aesthetic changes so this existing building would more closely match the architecture of newer buildings constructed by the hospital system.

As the building enclosure consultants for the project, the authors reviewed existing building drawings, architectural drawings and specifications, and shop drawings for the proposed exterior wall overclad using a metal panel open-joint rainscreen assembly. Because the new metal panels were a delegated design item, the subcontractor's specialty design engineer was responsible for providing engineering calculations for anchoring the metal panels to the building structure. The design for this overclad also included a new AWB applied over the brick veneer that would render the existing building wrap redundant.

Various options for attaching the metal panel rainscreen cladding were considered (**Fig. 13**). An investigation that involved the making of exterior investigative openings determined that the cold-formed steel stud vertical framing was installed at an irregular spacing. Therefore, the specialty design engineer worked with the project team and anchor manufacturer to perform a series of in situ tests to verify the in-plane and out-of-plane resistance

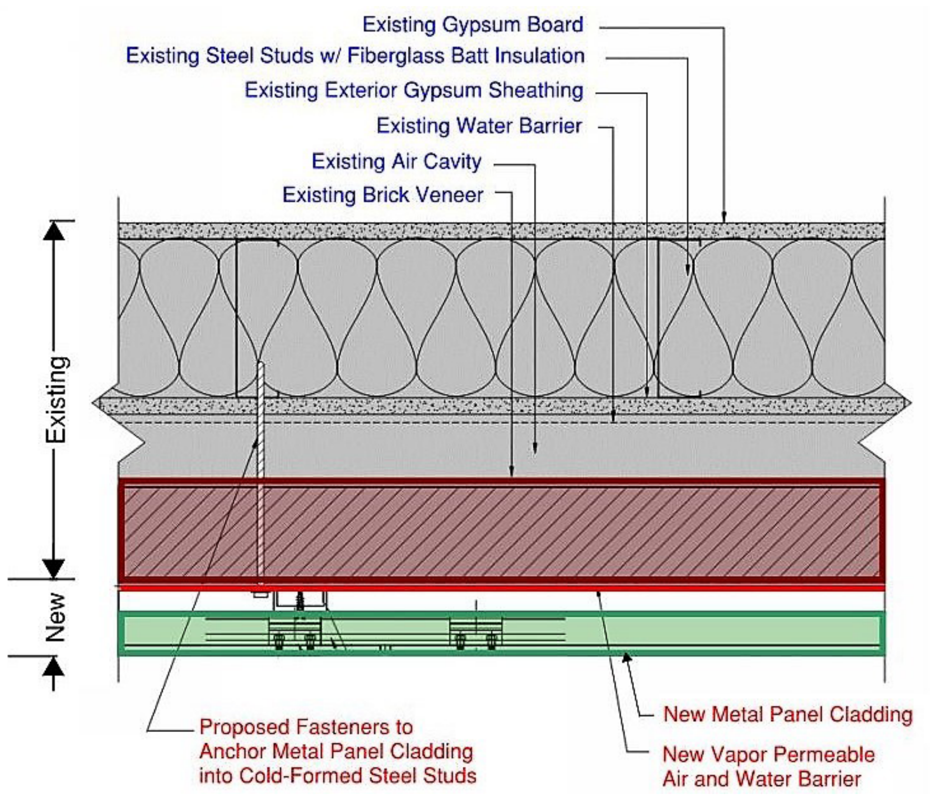

FIGURE 13. Schematic detail depicting metal panel rainscreen installed over a brick-veneer cavity wall.

of the existing veneer to support new loads imparted to it from the new metal panel cladding system. Ultimately, a solution was developed so that the new metal panel cladding could be installed directly into the brick veneer, with supplementary anchors provided into the existing framing to ensure redundancy of load paths.

Case Study 4: Masonry Cavity Wall without AWB → Interior Barrier Accomplished Using Crystalline Waterproofing

The exterior walls that are the subject of this case study were constructed in 1992 as an addition to an existing medical complex located in a cold climate. Before the interior spaces within this area of the hospital were renovated, the authors were retained to perform a building enclosure condition assessment at the property.

Exterior walls are constructed of brick veneer, an air gap, extruded polystyrene insulation, and CMU backup walls. Copper through-wall flashing is provided above lintels and at the base of the wall. No AWB had been provided on the exterior face of the CMUs. The interior spaces had previously been used for storage and light administrative uses, but the renovated spaces were designed to be used for medical purposes; therefore, a higher-performing exterior wall assembly was required. Given the deficiencies of the through-wall flashing, gaps in the backup CMU walls, and lack of a continuous AWB, the preferred solution would have been to remove the brick veneer and install a new AWB and through-wall flashing system. However, the owner deemed such a recladding solution to be not practical due to budget and schedule constraints.

Following demolition of interior finishes, water leakage through the field of the walls and at through-wall flashing locations was documented on several occasions during precipitation events. Therefore, the authors recommended a hybrid repair strategy that would incorporate traditional masonry repairs and window replacement in conjunction with application of a crystalline waterproofing system on the interior face of the CMU walls (**Fig. 14**). In

FIGURE 14. Crystalline waterproofing application on the interior side of a CMU wall.

general, the crystalline waterproofing application required the following:

- Cleaning the interior faces of CMUs and mortar joints so the surfaces would be free of foreign materials.
- Repointing cracked and deteriorated mortar joints on the interior face of the wall.
- Wetting the wall to a saturated surface damp condition and rewetting continuously until water was no longer accepted.

Applying the crystalline waterproofing system in accordance with the manufacturer's approved installation instructions. The final thickness of the interior waterproofing system was approximately ¼ in. (6.4 mm).

After repairs were completed, the interior of the building was monitored for approximately three months until new interior finishes were installed. No water leakage was documented during precipitation events or during field quality control testing after the repairs were completed and the windows were replaced.

Case Study 5: Mass Masonry Wall → Exterior Barrier Accomplished Using Translucent Vapor-Permeable Coating

Constructed in 1972 in a cold climate, the subject building is a five-story residential structure with two-wythe mass masonry exterior walls. The two masonry wythes are connected with header courses every sixth course. The building had a long history of water leakage and exterior wall performance issues. An investigation revealed that water leakage was prevalent throughout the building because the header courses that extend from the exterior to the interior of the building provide a direct path for water leakage once water penetrates the exterior surface of the walls.

The authors determined that traditional repairs alone would unlikely resolve the water leakage issues at the building because the exterior wall system lacked sufficient mass. The recommended repair project involved localized brick replacement, 100% repointing, and sealant replacement. In addition, a translucent vapor-permeable coating was applied to the restored exterior wall surfaces. The translucent coating was applied in two thin layers to ensure that the coating would remain vapor permeable after repairs were completed, thus allowing for evaporation of water that may penetrate through coating imperfections over time. This strategy essentially changed the wall behavior from that of a mass masonry wall to a barrier wall system, thus improving the performance of the exterior wall with respect to water penetration. Although the translucent

FIGURE 15. Translucent silicone coating applied to the exterior face of a two-wythe mass masonry wall.

FIGURE 16. Interior insulation and a variable vapor retarder installed on the interior side of a mass masonry wall.

silicone coating has resulted in a slight sheen that was not present in original conditions, the exterior masonry remains visible through the coating (**Fig. 15**).

Case Study 6: Mass Masonry Wall → Mass Wall with Interior Insulation and Variable Vapor Retarder

The subject university building was constructed in 1911 in a cold climate near the Atlantic Ocean. Exterior mass masonry walls have an ashlar granite facing and granite rubble on the interior side of the wall. Wood lath and an interior plaster finish had originally been provided throughout the building. These conditions remained in place for over 100 years until a comprehensive restoration was undertaken beginning in 2018. As part of this masonry restoration and window replacement project, the university requested that the building be upgraded to improve its energy efficiency.

Exterior insulation was not permitted on this historic structure, so a repair approach was developed that provided an air gap on the interior face of the wall, 3 in. (76 mm) of mineral wool insulation, a variable vapor retarder (a "smart" air barrier), and interior drywall finishes (**Fig. 16**). A variable vapor retarder exhibits low permeance during seasons of low humidity (winter), and high permeance during periods of high humidity, thus allowing for vapor diffusion and limiting moisture accumulation within the wall assembly over time. To vet this potential solution, the architect's building enclosure consultant used WUFI Pro 6.2 software to calculate the transient, one-dimensional, heat and moisture transport to determine the increase in moisture accumulation over time, percent saturation in the granite, and freezing and thawing potential of the masonry. The analysis compared results of a variable vapor retarder with that of a traditional vapor retarder over a 10-year period on various building elevations. The results indicated that the variable vapor retarder approach was superior to the approach using a traditional vapor retarder.

Additional thermal modeling was undertaken to vet detailing associated

TABLE 1. Items to consider before implementing unconventional masonry wall repair strategies

Properly detailed cavity wall at localized areas
» Areas not addressed during the repair program will still include deficiencies and will be susceptible to air infiltration/exfiltration and water leakage.
» Repair of the backup walls is required to ensure a sound substrate for the AWB and through-wall flashing.
» New brick and mortar may not match existing adjacent areas, resulting in potential aesthetic concerns following completion of repairs.
Overclad with drainable EIFS or metal panel rainscreen
» Overcladding provides an opportunity for aesthetic changes with respect to the existing walls.
» Overcladding includes a new water drainage plane on the exterior face of the veneer for redundancy and water penetration resistance.
» Localized repointing and unit masonry replacement will likely be required to ensure a suitable substrate for AWB application.
» An EIFS overclad will provide continuous exterior insulation. A metal panel rainscreen overclad can also be designed to included continuous exterior insulation.
» If an EIFS is used, mock-ups are recommended to verify adhesion characteristics of the AWB to the substrates and the EIFS insulation to the AWB.
» If a metal panel rainscreen is used, the load path for attaching metal panels must be established by means of calculations and/or testing.
» Thermal modeling is recommended to evaluate interface conditions at fenestration and at roof-to-wall transitions.
Interior barrier accomplished using crystalline waterproofing
» Traditional masonry repair strategies should be implemented in tandem with interior crystalline waterproofing repairs.
» Interior finishes must be removed to access the repair area. Repointing the interior face of CMU joints will likely be required.
» Application of the interior waterproofing will result in a slightly thicker wall, possibly reducing interior space within the building.
» Field quality control testing is recommended following implementation of repairs and before installation of new interior finishes.
Exterior barrier accomplished using translucent vapor-permeable coating
» Localized brick replacement and 100% repointing may be required before coating is applied.
» Evaluation of aesthetic and performance mock-ups is recommended before a building-wide repair program is established.
» Applying the coating too thickly can cause a chalky appearance and can inadvertently result in vapor-retarding properties.
» Application of a translucent coating will change the appearance of the building and result in a sheen.
» Subsequent recoating projects will need to be performed using compatible materials.
Mass wall with interior insulation and variable vapor retarder
» Hygrothermal and thermal analyses are recommended before repairs are implemented on a building-wide scale.
» Traditional masonry repair strategies should be implemented in tandem with these repairs to limit moisture penetration into the walls.
» Evaluation of mock-ups is recommended to allow for review of variable vapor retarder integration and termination detailing.
» The additional materials will result in a thicker wall, thus reducing interior space within the building.

AWB = air-water barrier; EIFS = exterior insulation and finish system.

with the new aluminum-clad wood windows and aluminum frame windows that were installed during the restoration project. Several mock-ups were implemented to review detailing and integration of the windows with the new interior smart air barrier. Air site leak detection field quality control testing using theatrical fog was also performed to verify continuity of the smart air barrier at interface conditions with the windows.

CONCLUSION

Unconventional masonry repair strategies can assist with addressing concerns associated with water leakage, air control, vapor diffusion, and energy efficiency. In many cases, these strategies can be successfully implemented by combining traditional repair strategies with the unconventional strategies. **Table 1** lists some of the items that should be considered before implementing the unconventional strategies described in this paper.

Just as there are countless variations of masonry walls, an extensive array of possible maintenance and repair strategies is available for consideration by knowledgeable engineers and building enclosure professionals. Although many buildings will eventually experience performance issues related to water leakage, air control, or thermal issues, conventional and unconventional repair strategies can be used to maintain and repair both historic and relatively modern masonry exterior walls.

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MYSTERIOUS MOISTURE MARKS: ASSESSMENT OF WATER STAINS AT WINDOW GLAZING

ABSTRACT

The threat of water intrusion within a building enclosure will raise different levels of concern, including aesthetic, environmental, and threats to the structural integrity. The appearance of water stains upon the interior surfaces of the window glazing system may raise alarms for the unit owner or building stakeholders. However, not all signs of water stains are the same. An assessment of the reported area of staining should include evaluations of all reasonable sources. Whether condensation from the inside or active water intrusion from the outside, the presentation will touch upon the most common scenarios and sources. A discussion regarding the conditions that may or may not be attributed to window glazing will take place. Expectations of the ability of watertightness performances based on the window type or configurations will be presented. Additionally, a discussion regarding the invasive and emerging noninvasive methods such as thermal imaging analysis via drone technology will occur.

LEARNING OBJECTIVES

- Summarize waterproofing specifications and detailing of glazing systems, particularly those in common with window glazing types and configurations.
- Identify typical signs and causes of water stains found at window glazing systems that may or may not be related to water intrusion.
- Describe the methodology of window inspections with a focus on the use of drones, and analyze the use of thermal imaging.
- Evaluate common repair practices and emerging new products for watertightness / weathertightness methodology.

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Every year, approximately 1 in every 50 homeowners in the United States claims water damage.[1] During extreme weather events such as hurricanes and tropical storms, the claimed damage to the interior contents due to wind driven rain can range from 50% to 100% of the overall damage claims. [1] Moisture intrusion is a substantial concern because biological growth may potentially cause structural deterioration, serviceability disruptions, and damage to the interior contents. Even during normal weather conditions, condensation deposits can affect the performance of buildings by affecting the overall heat, air, and moisture (HAM) transfer phenomenon and energy consumption.[2] There are also life-safety concerns. Too much moisture within the building enclosure can lead to forms of biological growth that are detrimental to human health. It is critical to mitigate moisture through proper building management that maintains acceptable indoor air quality.

When an anxious homeowner or other stakeholder discovers staining around windows, a qualified building investigator can assess the situation and provide guidance. A thorough assessment of the reported area of staining should include evaluations of all likely causes of moisture intrusion.

According to the US Environmental Protection Agency (EPA), the only way to eliminate mold and mold spores in an indoor environment is by controlling the moisture entering into the building.[3] Mold and other types of organic growth are often observed near openings such as windows and doors. This growth may be related to poor insulation practices, which lead to a buildup of condensation on the surfaces of a window assembly ("sweating") and affects the overall HAM of the building. Compared with the rest of the building enclosure, openings such as these are the most susceptible to condensation, which transfers excessive moisture to adjacent finishes.

There are multiple building codes and standards available for wind-driven-rain testing during extreme weather events (**Table 1**).[4-14] In contrast, fewer codes and standards address moisture intrusion under normal weather conditions.

Building enclosure issues related to moisture can be briefly classified into three categories, design, construction, and maintenance.

- Design-related problems may include improper specification of insulation or improper window design. Examples of poor design include an exterior sill with no slope or improperly designed drainage mechanisms.
- During the construction phase, improper installation of an air/vapor barrier, a lack of rain penetration tests, and improper flashing tend to lead construction defects and potential water and moisture intrusion.
- Inadequate maintenance and a lack of operational awareness can lead to premature failures and reduced life-cycle performance for building enclosure components and systems. Failure to perform timely inspections, deferred maintenance, and an insufficient preventive maintenance plan all increase the risks for an array of water and moisture intrusion opportunities.

WINDOW TYPES

Windows are a critical part of a building enclosure, but they are vulnerable to age- and weather-related damage. Because water intrusion within a building enclosure can cause aesthetic and structural problems, the appearance of water stains on the interior surfaces of the window glazing system may raise alarms for the unit owner or building stakeholders. However, not all water stains are the same.

When evaluating water staining, it is important to begin with an understanding of window types and performance expectations. Each window type discussed herein serves a distinctive purpose, and the distinct styles and configurations require specific approaches toward investigation and repair.

Single-Hung Windows

Typically, a single-hung system consists of two glass panels (sashes). Single-hung windows open vertically, with one window panel or sash moving up and down and the other sash remaining stationary. Thus, when you open the window, the upper sash is covered on the inside. How these sections move is the major difference between single-hung and double-hung windows. If the operable sash of the window is impeded from functioning properly, that could lead to gaps and seams that allow water intrusion. Field consultants who perform or oversee water penetration field-testing procedures such as ASTM E331[6] should be knowledgeable at the vulnerability of each window type. ASTM E331 diagnostic water intrusion testing delivers water via spray nozzles directed

TABLE 1. Items to consider before implementing unconventional masonry wall repair strategies

Standard	Type of load	Specified load	Specified number of cycles	Notes
ASTM E283[4]	Static	299 Pa	N/A	Laboratory test Infiltration must be less than 0.06 CFM per square foot of glazing and 0.09 CFM/ft^2 of projected window.
ASTM E330[5]	Static	Design wind pressure(DP)	N/A	Laboratory test Interstory drift and deflection must be within serviceability limits for an applied 10-sec load. No gasket disengagement or structural failures.
ASTM E331[6]	Static	Largest of 20% DP or 718 Pa	N/A	Laboratory test When a rain spray rate of 3.4 L/m².min (5.0 U.S. gal/ft².h) is used for 15 minutes, no water infiltration must be observed.
ASTM E1105-05(A)[7]	Static	Largest of 20% DP or 718 Pa	N/A	Field test When a rain spray rate of 3.4 L/m².min (5.0 U.S. gal/ft².h) is used for 15 minutes, no water infiltration must be observed.
ASTM E1105-05(B)[7]	Cyclic static	Largest of 20% DP or 718 Pa	Minimum of 3	Field test When a rain spray rate of 3.4 L/m².min (5.0 U.S. gal/ft².h) is used for 15 minutes, no water infiltration must be observed.
ASTM E547-00[8]	Cyclic Static	137 Pa	Unspecified	Laboratory test When a rain spray rate of 3.4 L/m².min (5.0 U.S. gal/ft².h) is used, no water infiltration must be observed.
BS EN 12155[9]	Static	Depends on rating pressure	N/A	Laboratory test When a rain spray rate of 2 L/m².min is used, no water infiltration must be observed.
BS EN 13050[10]	Dynamic	37.5% of design pressure	Unspecified	Laboratory test When a rain spray rate of 2 L/m².min is used, no water infiltration must be observed.
BS EN 13051[11]	Static	No loads; Annex B suggests the use of BS EN 12155 loadings if air pressure is required	N/A	Field test When a rain spray rate of 5 L/m².min is used, no water infiltration must be observed.
BS EN 12865[12]	Pulsating load	Incremental steps of 150 Pa	As many as needed	Laboratory test (limit of watertightness) When a runoff rate of 1.2 L/m².min and a driving rain rate of 1.5 L/m².min are used, no water infiltration must be observed.
AAMA 501.1-17[13]	Dynamic	300.0 Pa, 380.0 Pa, 480.0 Pa, 580.0 Pa, and 720.0Pa	One 15-min cycle at a time	Laboratory test/ field test When a rain rate of 3.4 L/m². min (5.0 U.S. gal/ft².h) is used, no water infiltration must be observed.
CSA A440[14]	Static	150 Pa, 200 Pa, or 250 Pa	Four cycles of 5 min, each with air pressure, and 1 min with no pressure	Field test When uniform water film on the outside of the window is used, no water infiltration must be observed.

Note: N/A: Not applicable, 1 L/m^2 = 0.0245 US gal/ft^2; 1 Pa = 0.000145 psi; 1 CFM = 1 ft^3/min = 0.028 m^3/min.

ASTM: American Society for Testing and Materials, **AAMA:** American Architectural Manufacturers Association, **BS EN:** British Standards European Norm, **CSA:** Canadian Standards Association within a grid pattern, with the water uniformly sprayed and directed at the vulnerable areas. Within properly installed single-hung widows, vulnerable areas likely include exposed fasteners, gaskets, and the operable sash.

Double-Hung Windows

Like a single-hung window, a double-hung window has two sashes; however, in a double-hung system, both the lower sash and the upper sash can move up and down, and the sashes usually tilt out for easy cleaning and maintenance. A double-hung system has twice the moving parts of single-hung system, and if one or both of the operable sashes of the window are impeded from functioning properly, that could lead to gaps and seams to allow water intrusion. During ASTM E 331 testing,[6] water is applied to the exterior of the test window while the pressure inside is lowered by means of an air chamber built on the inside or opposite side of the test window. The vulnerable areas should be observed. If water intrudes within the vulnerable areas, the test can be redirected, recalibrated, and refocused to pinpoint the source of origin. Within properly installed double-hung widows, vulnerable areas likely include exposed fasteners, gaskets, and one or both of the operable sashes.

Casement Windows

Casement windows swing out to the side or up to open. This mode of opening allows the window to be constructed of solid glass; therefore, compared with a single- or double-hung window, a casement window offers a less-obstructed view overall. Casement windows usually come with one casement windowpane on the left and one on the right. Defects at any screws, bolts, springs, or hand crank could impede the casement from functioning properly, creating gaps and seams that allow water intrusion.

Since casement windows operate differently than the sliding mechanisms of other window types, they require a slightly different verification testing method. When preforming ASTM E331[6] testing, the investigators must ensure that the hand crank or locking mechanism is completely engaged

FIGURE 1. Base of Sliding Window was previously blocked and allowed water accumulation.

during the time that the calibrated spray apparatus is applying water and uniform static pressure is simultaneously applied to opposite sides of the test area.

Awning Windows

Awning windows are ideal for climates with a lot of rain because the windows create water-resistant awnings when opened. Awning windows swing open on the outside by being pushed outward with the latch or handle. This design makes awning windows more weatherproof, but they are not invulnerable. If an awning window is left open, updrafts or wind-driven rain could lead to moisture accumulation within the interior space.

Sliding Windows

Sliding windows have a minimum of two sections or sashes, and one of the sections slides horizontally outside/inside of the other to open or close. Similar to double-hung windows, if one or both operable sashes of the window are impeded from functioning properly, that could lead to gaps and seams that allow water intrusion. Debris, long-term wear, or distortion of the track may impede the window from closing or sealing properly. Water may drain off the base of the track during rain events or when other water accumulates from the outside environment. Sliding windows and other window types may have drainage systems or weep holes to keep water out of the window, but the following factors can impede any water entrapment precautions:

- » Improper installation: If the window frame, track, or base is misaligned, that can prevent water from flowing toward or out of drains.
- » Impeded drainage: Debris on the lower track can cause obstructions.
- » Lack of coordination among trades: Paint contractors, stucco installers, or other vendors may unintentionally apply construction materials that cover or obstruct the weep holes.

Fixed Windows

Fixed windows include arched, picture, and geometric-shaped windows that do not open or close. These types of window are often installed above standard windows that provide ventilation. Some fixed windows can open the same way that a casement window does. They can also be installed in a multiarch structure with square or rectangle windowpanes on the side and arched curved windows. Picture windows are fixed windows that are inoperable, but they are often paired with operable windows. They are large window types that do not have any breaks or visible frames. Fixed windows have no operable sashes, panels, or mechanisms with the no potential for gaps, seams, or misalignments that create ideal paths for water intrusion. Therefore, if there is water intrusion, attention should be focused on the condition and construction of the window frame.

The condition of the window frame material is a significant concern during the evaluation of water intrusion, and forensic water testing may be warranted. Deterioration or distortion of the window frame can be the source of the mysterious water stains.

In window frames made wood, deterioration from rotting or warping is a commonplace culprit in water intrusion. Absorbed moisture causes wood rot and creates ideal conditions for further rot, biological growth, and pest infestation. Termites and other wood burrowing insects can further damage the wood and create additional pathways for water intrusion.

When the window frame is made of untreated wood, moisture can become trapped and long-term cycles of expansion and contraction can lead to permanent bends in the frame that distort the window's appearance, making it look crooked, twisted, cupped, or bowed. In addition to the aesthetic effects, such distortion can adversely affect the window system's operation or weathertightness.

PERFORMANCE EXPECTATIONS

The industry standard specification for evaluating fenestration products is AAMA/NWWDA 101/I.S. 2-08 *Voluntary Specification for Aluminum, Vinyl and Wood Windows and Glass Doors*.[22] It establishes the following performance requirements for a window assembly:

- Structural ability to resist wind loads or wind pressure standards
- Resistance to air leakage
- Resistance to air infiltration
- Resistance to forced entry

Products that with the certification under AAMA/NWWDA 101/I.S. 2-08 are designated by a four-part code that denotes the type of window, the performance class, and performance grade. For example, the code C-R15 indicates a casement window (C) recommended for residential applications (R), with a performance grade of 15. How well a window performs when subjected to heavy rains and high wind pressures reflects its performance grade and design pressure. The window design pressure (lb/ft^2) is typically provided based on the on structural rating only. However, a strong structural assembly prevents the risks of component displacement and further water intrusion. In addition to this design pressure, the performance grade indicates that a window has met the water resistance and air infiltration standards for that grade.

The minimum recommended design pressure for residential windows is 15 lb/ft^2 (73.24 kg/m^2). A design pressure of 15 lb/ft^2 means a window has been tested to withstand sustained wind pressures of 22.5 lb/ft^2 (109.85 kg/m^2), roughly equivalent to a 95 mph (42.5 m/s) wind (depending on the pressure coefficient), applied to either side of the window, simulating either positive or negative wind pressures. The test pressure is always 150% of the rated design pressure to provide a safety factor. To earn a performance grade of 15, a window must also pass a water pressure test of 2.86 lb/ft^2 (13.96 kg/m^2), which simulates rainfall of 8 in. (203 mm) per hour with a wind speed of 34 mph (15.2 m/s).

In coastal areas or other areas prone to intense rain events or hurricanes, higher-performance-grade windows exceeding minimum code requirements are recommended. Window design pressure ratings combine the window's resistance to (a) water leaks, (b) air leaks, and (c) actual structural loading. Points are assigned for the window's ability to resist each type of force and then a total window performance grade rating number is calculated. Higher ratings indicate better performance in preventing common causes of water intrusion. Thus, a high rating describes a window that is significantly more resistant to water and air leaks than the threshold performance criteria.

FIGURE 2. Aged sealant conditions around a window.

A consideration when setting performance expectations and investigating window conditions will be the perimeter conditions. All window systems, regardless of their condition at the time of assembly, are susceptible to the passage of time and exposure. Sealants that are vulnerable to age can dry and crack, leaving passageways for water to enter the wall structural enclosure (**Fig. 2**). Unpainted areas of the exterior wood window frame components will retain moisture, potentially subjecting the frame to accelerated rot and decay. Over time, framework for both wood and aluminum windows can expand and contract with temperature changes, thus creating space at the perimeter of the window system where water intrusion can occur. Fluctuations between daytime and nighttime temperatures cause repeated movement of window glass that expands in warm weather and contracts in cool weather. This movement can cause glass to fracture to allow water intrusions. In double-paned windows, flexing motion can cause the seals between the panes to fail, resulting in window condensation fog.

On rare occasions, windows assemblies have defects due to their original manufacture. For example, newly fabricated window frame materials may exhibit cracks or splits either at the manufacturer or after transport to the site.

FIGURE 3. Improper assembly at the joining of window systems.

Although defects may be difficult to observe in an installed window system (**Fig. 3**), exposed defects may be noted during a visual inspection. Outside the wall, gaps and unsealed elements might be noted in the exposed window joinery and miter joints. Separation and other defects tend to occur at the 90-degree angles of the corner miter joints. The lack of a firm seal at mull bars or at the joining of window assemblies may also allow water intrusions.

Water stains may occur early if windows are improperly installed and allow water intrusion through gaps, voids, and separations. When evaluating window installations, investigators should assess whether corrosion-resistant flashing and the watertightness methodology are sufficient to divert water away from the building enclosure and prevent water intrusion within the wall cavity and frame components of the structure. The investigator should be familiar with the window assembly and associated building construction and be capable of recognizing whether fasteners are missing, components are misaligned, or waterproofing is improperly installed.

Recent weather history at the site is also an important influence on the assessment. Different types of severe weather events can affect window components in different ways. Hail can cause physical damage to not just window frames and glass but also the surrounding exterior cladding. Hail can break glass panes, allowing water intrusion during and after a rain event. Impact dents from hail on the window frames may affect the way that the window operates, thus compromising the watertightness. The impact of windborne debris adjacent to window openings can also create openings for water to intrude through the building enclosure.

TYPES OF STAINING

Water stains on windowsills and at the perimeter of the openings can be a source of anxiety for building owners and occupants. The cause of a stain might be a simple drop or splash from watering a nearby plant, or the stain may be the tip of the iceberg, indicating a larger structural issue. This is why it is so important to determine whether stains are the sign of a major leakage problem or not.

Water Accumulation—Clear

The appearance of standing or pooling water around or at the base of a window is often reported as a leak. It is important to investigate and figure out where the moisture is coming from. If the window is open, you should be able to close the window, mop up the mess, and not worry much further. But if a closed window allows water infiltration, that suggests faulty installation or failure of materials. Such situations warrant further investigation.

Stains and Discoloration—Amber

Another sign of a window leak is the appearance of stains or discoloration. The area could be dry or wet, and the stains may be copper, yellow, or brown residue. Growth of the stain over time is a likely sign of a leak. Reddish staining can also be a sign of corrosion of metal fasteners or structural rebar reinforcement (**Fig. 4**).

Biological Growth—Green or Black

Biological growth often occurs in areas of excess moisture such as bathrooms, kitchens, and basements. If it appears on or around windows in an area without any plumbing fixtures or any obvious sources of running water, the cause is likely a leaky window. Biological growth can look

FIGURE 4. Red stains reveal compromised conduit not window assembly.

FIGURE 5. Example of Bio growth.

spotty and fuzzy (**Fig. 5**). In addition to being an eyesore, airborne spores can adversely affect health and well-being or produce musty odors. The odor often stems from areas where moisture has been accumulating for long periods. Interior finishes such as drywall are absorbent when saturated with moisture, and as low-air-circulation environments, they can serves as petri dishes for biological growth and detectable odors.

Finish Distortion—Fading

When drywall absorbs water, this can cause paint to fade and wallpaper to lose adhesion. Therefore, if the interior finishes adjacent to the window assembly begin to distort and peel away from the wall surfaces, leaks in the window assembly should be suspected. Water intrusion through the window assembly can also lead to fading or flaking of the window finishes. Distortions of the window frame such as warping will also compromise the window integrity (**Fig. 6**). Walls around the windows may also exhibit signs of significant separation gaps as the materials warp. Soft spots and spongy materials will sag because of the weight of entrapped water and as building materials deteriorate. The buildup of biological growth, wood rot, and pest infestation can add mass beneath the surface. Extensive warping could be the result of structural damage. These conditions merit additional review and are expensive to investigate and repair.

FIGURE 6. Distortion at window.

Window Sweating

The fundamentals of window sweating are simple. When the air within the building enclosure forms a convection current cycle against the cold surfaces of a window, the colder air sinks and warm air replaces it. As warm, moist air encounters the colder interior glass surface, the air drops below dew point, depositing moisture on the glass. As the convection current process continues over time, additional moisture leaves deposits on the glass. Adjacent surfaces may eventually become stained as built-up condensation on the surface of glass drips onto the windowsill and other surfaces. If an individual is not present to witness this process, the cause of staining may be a mystery.

When investigating “sweating” windows and related staining, it is important to keep in mind that condensation on windows within an enclosed occupied space may not necessarily be a sign of water infiltration. Human activities such as showering, cooking, and simply breathing can affect the dew point and convection current cycles, leading to condensation on glass.

Understanding the types of window glass installed in the building enclosure is important to assess the appearance of water and its significance. Single-paned windows are less energy efficient and less insulated than their double-paned counterparts. Therefore, the use of single-pane glass can lead to more condensation on the window surface, increasing the risk for water staining of the window trims, surrounding walls, and floor surfaces below.

FIGURE 7. Example use of aerial drone.

Condensation can be the result of poor thermal bridging design. Thermal insulation acts as barrier to regulate temperature as per the design intent. When thermal insulation is interrupted by a window, condensation can build up, resulting to areas of water staining and distortion.

The window assembly material may further affect the risk for moisture condensation buildup and staining. High-conductivity materials such as aluminum have low thermal resistance relative to insulated materials, which means they allow heat to bypass the thermal barrier. Investigators should understand which window materials carry higher risks that may factor in the assessment and mitigation strategies going forward. Guidelines such as *Voluntary Test Method for Thermal Transmittance and Condensation Resistance of Windows, Doors and Glazed Wall Sections* (AAMA 1503-09)[23] can be used to evaluate certain window elements and provide expectations for condensation resistance. The AAMA condensation-resistance factor(CRF)[23] indicates the magnitude of the temperature-driven vapor that can take the form of condensation. That type of condensation can potentially mislead an observer to conclude that the window is defective. Determining a window's CFR and taking note of the surrounding environment are key to a proper assessment of the source of water stains.

FACADE ASSESSMENT

In addition to window condensation, other causes of moisture stains can range from steam from a teapot to more serious causes such as an underperforming HVAC system. Properly maintained and balanced mechanical ventilation systems are needed to control the moisture levels within the enclosure. Depending on the airtightness of a design and the performance of operable natural ventilation, the rate of moisture may fluctuate dramatically and result in dew point moisture accumulation and staining of surfaces at the window area.

With recent advancements in technology, several noninvasive methods have been developed to gain insight to mysterious moisture stains near the window. These innovative methods can be employed to analyze a variety of construction defects, wall coating failures, and potential structural issues that can be hidden behind moisture stains.

An investigator's initial observations of the exterior facade may involve the use of long-range binoculars or a high-power camera lens. These tools are beneficial when you have a direct line of sight. However, there are times when the target area of concern is in an obscured location and costly mobilization of equipment would be required to gain a clear vantage point.

Commercial unmanned aerial vehicles (drones) can be used to avoid the need for lift booms and scaffolding as staging equipment. When equipped with a high-quality camera lens, a drone may help the investigator visually note deviations in the facade (**Fig. 7**). Deleterious facade conditions may correlate with the moisture staining, water-related distortion, or biological growth witnessed within the building enclosure. The photographic survey performed by the drone can document from a variety of angles, positions, and heights any threats of structural failure, loss of facade elements, and other potential issues responsible for the water stains.

As discussed earlier, failures of installed window sealant (or other building components) can take the form of distortions, voids, and displacements that allow water intrusion and lead to interior staining. Sealant failures may be due to insufficient sealant adhesion, incorrect sealant cure times, or sealant joint discontinuities. In some cases, inconsistent quality control measures or improper product specifications for the sealants may be the likely cause of water stains. Visual inspection may narrow the universe of potential causes to sealant issues instead of window assembly defects. If so, the costly and unnecessary endeavor of building permitting and purchasing and installing window assembly replacements can be avoided.

Not all defects are visible to the seasoned investigator's naked eye or via the ocular lens of a typical drone camera. It is therefore fortunate that not all cameras are the same. Specialized drones with dedicated cameras with infrared (IR) thermography capabilities are available. The IR thermography can capture the temperature distribution on surfaces and relay that information on a visual spectrum. The drone can be maneuvered across the facade and over large areas in the search of abnormalities. It can also focus on specific areas that may correlate with locations of reported interior water staining. IR imagery may identify discontinuities of the building enclosure's facade that are not visible by standard methods (**Fig. 8**). The speed of the IR assessment via drones allows the investigator to view "invisible" conditions within inaccessible areas in an

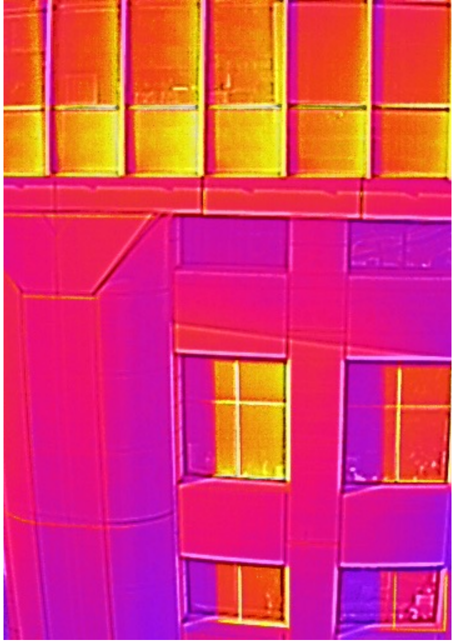

FIGURE 8. IR facade survey via drone.

expedited manner, thereby preventing future water damage. Thermal modeling of the facade while using fixed-measure temperatures as point of calibration can be ideal to understand, analyze, and provide recommendations for mitigation.

CONCLUSION

When investigating mysterious moisture stains, an effective strategy is to determine the logical steps of the investigation based on facts, reasonable expectations, and precedents based on scientific research. The investigation should include the following:

- Identifying the physical evidence around the window system without first making presumptions about the source or causality of the staining
- Gathering and documenting the visual information from both the interior space and the exterior environment about the window system and adjacent conditions
- Interviewing owners, tenants, and other stakeholders about the installation, use, and maintenance of the window system
- Reviewing past and present information about neighboring window systems and similar adjacent conditions

Often, the goal of a moisture distress assessment is to determine whether the concerns are justified, identify the sources of identified problems, determine the causality, and assess any life-safety risks. After the initial assessment and observations, investigators should communicate their findings, risks, and recommendations to the stakeholders. The assessment should be used to determine whether interior staining indicates detrimental water intrusion or superficial surface condensation and then form an appropriate plan of action to mitigate and address the sources of water stains. The action plan may involve a systematic approach of targeted water testing as established in AAMA 511[24] or ASTM E2128.[25] Test protocols vary based on specific conditions and components. An alternative approach to water testing involves selective demolition to investigate the water stain areas and repair the issues discovered. These approaches vary in terms of costs, durations, and interruptions, which is why it is important to conduct a preliminary assessment before choosing what actions to take. Poor workmanship, failure of window systems, and simple user error require different approaches and resolutions. Simple reasoning can shed light to the mysterious appearance of moisture stains and provide proper direction. **Figure 9** provides an overview of typical moisture stain assessment by a building investigator and future steps.

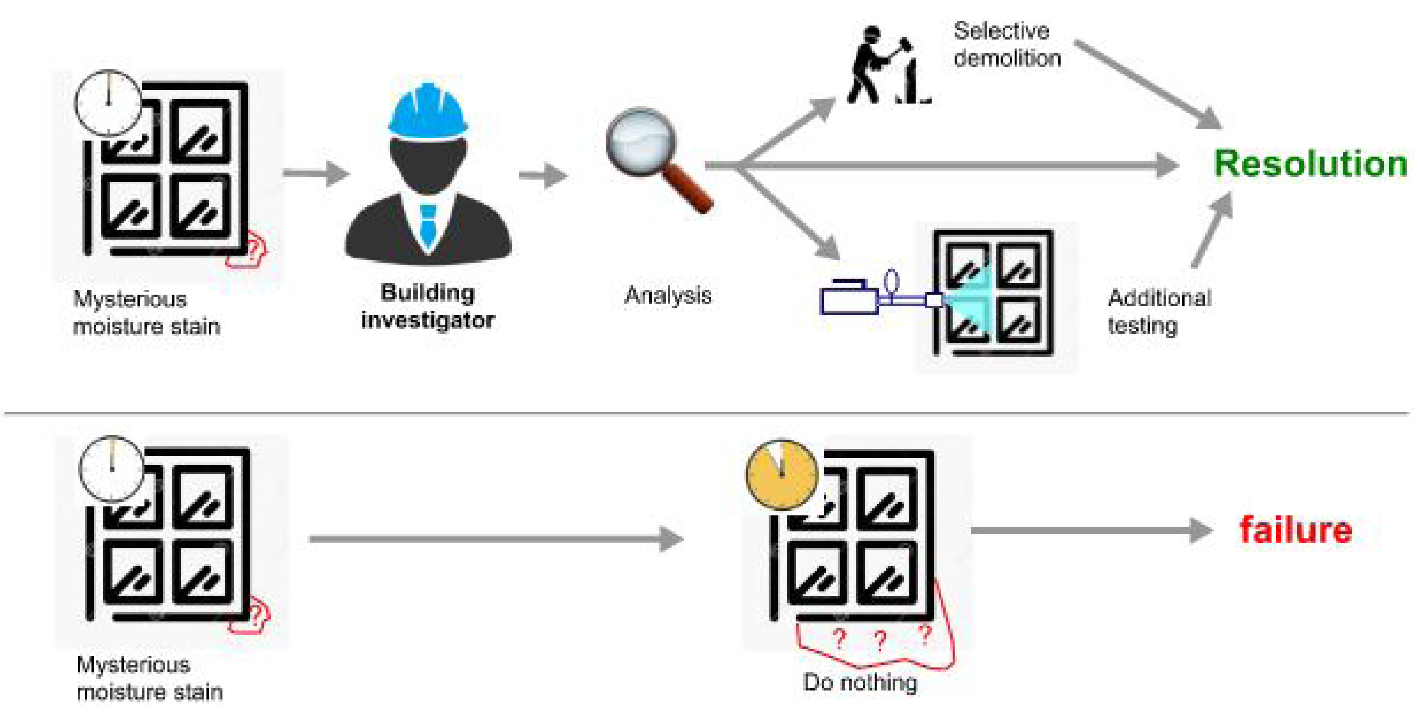

FIGURE 9. The path of assessment.

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ASTM: American Society for Testing and Materials, AAMA: American Architectural Manufacturers Association, BS EN: British Standards European Norm, CSA: Canadian Standards Association

BASICS OF THIN BRICK WALL SYSTEMS

ABSTRACT

Several thin brick wall systems have been developed for the construction industry, ranging from composite thin brick precast panelized walls to rail-supported thin brick rain screens. Each of the thin brick wall systems has benefits and drawbacks that require careful consideration when considering climate, code, and adjacent wall assemblies. When the wrong thin brick system is chosen for a specific application or the system is composed of using the wrong elements, problems can occur during or soon after construction. Our experience investigating problems with thin brick wall systems and navigating design challenges during construction has shown the importance of understanding the variety of systems that exist and when one system may want to be selected versus another. Understanding how each system functions is also essential to understanding how the detailing of the systems varies and the importance of matching the right details to the right system.

LEARNING OBJECTIVES

» Evaluate the basic material properties of thin brick systems and how they are similar and different from conventional brick systems.

» Learn to differentiate between barrier thin brick and rain screen thin brick systems and how this affects the selection of other system components, i.e., weather barriers and insulation.

» Discuss climate considerations for thin brick system selection.

» Determine how codes such as NFPA 285 can drive system selection..

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1. INTRODUCTION

Proprietary thin brick veneer (TBV) systems utilizing either clip or panel-supported systems are growing in popularity as an economical alternative to traditional brick cavity wall construction. This is partly because TBV systems provide the outward appearance of conventional brick masonry, facilitate continuous exterior insulation, and due to their lightweight nature, do not require the supplemental structure necessary to support a traditional brick veneer cavity wall. Although TBV looks like traditional brick veneer on its surface, the system differs fundamentally from brick veneer. Multiple proprietary TBV systems are available in the marketplace today, and understanding the pros, cons, and detailing intricacies of each is challenging.

Thin brick systems in generic form have been used for years. These systems typically embed the thin brick in a cementitious material, either precast concrete or portland cement plaster (stucco). Some, such as thin brick embedded in precast concrete, function as barrier wall systems, with all water being shed at the wall's surface. Others, including thin brick applied over stucco and lath, or many proprietary systems used today, incorporate a drainage plane to manage water behind the thin brick cladding. Some proprietary systems also function as drained and back-vented rainscreen systems with mechanical attachments and air spaces to help promote drying. Selecting the appropriate system for a project requires an understanding of available systems and their track records of successful use, and then matching the right system with consideration of project parameters such as climate, the height of the wall system, and budget. Thoughtful system selection and detailing of TBV can provide a practical, lightweight facade system and help minimize the risk of costly problems.

This paper describes different types of thin brick systems, how they are designed to perform, and the limitations and potential of the systems.

2. TYPICAL BRICK PROPERTIES

Thin brick units, or "tiles," are manufactured like traditional full-depth clay brick units. To understand the tiles used in TBV, it is helpful first to consider the properties of conventional full-depth clay brick units, especially those that instill durability. The manufacturing of clay brick units is the result of forming the raw materials of surface clays, fire clays, shales, or a combination of these through extrusion, molding, or dry pressing. After forming, the brick is fired in a kiln, creating a process of vitrification or melting which solidifies the brick. Through the vitrification process, the clay slowly softens and consolidates. As the temperatures increase, the clay becomes more fluid, and the mass becomes tighter and more solid. This reduces the material's porosity and lowers absorption. The properties of the brick resulting from this process will vary depending on the raw materials and the temperatures at which they are fired.

Durability, color, texture, size variation, compressive strength, and absorption are the brick properties most affected by the firing process, and they are generally considered to be the most important. These properties are closely monitored through the production process and have standards defined and developed through ASTM International (ASTM).

Texture and absorption are essential considerations when selecting brick for two reasons. First, they play a critical role in the bond strength of brick and mortar. Second, absorption can be directly tied to water penetration resistance and freeze-thaw durability. ASTM has developed specifications to guide the selection of the appropriate brick quality based on exposure conditions. The most durable brick is classified as severe weathering and given a grade of SW. SW brick is considered and recommended for outdoor use in all climates. Understanding the properties of traditional brick, we can now discuss how these properties present themselves in thin brick and why they must be considered.

"Thin brick" refers to the tiles formed and fired thinner than traditional brick. Exterior-face dimensions are generally the same as those available for conventional brick, but the brick is much thinner – a traditional brick is 35/8 in. thick, whereas thin brick is typically ½ in. to 1 in. thick or up to 13/4 in. thick. Thin brick is often manufactured with textured, ribbed, or dovetailed backs to provide increased surface area and allow mechanical keying of adhesives, improving the bond of the thin brick to its substrate.

Much like traditional bricks, thin bricks can be manufactured to the

FIGURE 1. Panel-mounted, proprietary TBV with formed corner tiles.

size required for a project. Many vendors of proprietary thin brick systems now require their thin brick to be used in their thin brick support systems, partly because of the required keying mechanisms. This can limit the available selection of brick colors and textures. It is possible to cut (i.e., remove faces from) traditional bricks for use in thin brick systems, but manufacturer approval should be obtained before exploring this option, and the thin brick attachment mechanism must be carefully considered. Cutting a conventional brick down in size will create a minimum of one unfired brick surface, and the additional absorption of the cut face must be considered when constructing a wall system with cut units. By introducing a cut face into the assembly, the durability of the system can be reduced. Corner pieces for outside corners may be fabricated similarly by the manufacturer, field fabricated by cutting from solid brick, or fabricated by adhering multiple thin bricks together. We recommend using corners fabricated by the manufacturer whenever possible **(Fig. 1)**.

3. BRICK WALL SYSTEMS

Load-bearing mass masonry brick walls found in historic construction are built with multiple unreinforced brick wythes or rows of brick laid next to one another, transferring gravity loads from floors and roofs (load-bearing masonry) to the foundation below. Historically, unreinforced brick mass masonry walls were uninsulated and relied on the walls' mass to manage water through absorption and drying. This water management strategy depends on the brick mass, the masonry systems' continuity, and water storage capacity. These walls were not watertight, and there was always the possibility that some water could get through the walls during heavy rain. As buildings have become taller, labor and material costs have increased; insulation, air barriers, and water barriers have become mandated; and mass masonry brick walls have been replaced in contemporary construction. The industry moved to a brick veneer cavity wall system after phasing out mass brick masonry over the 20th century.

Brick cavity walls are non-load bearing, constructed using a single exterior wythe of traditional full-depth brick as a veneer in front of a backup wall structure, which provides lateral support of the veneer. The backup wall behind can be wood or steel framed, concrete block, or cast-in-place concrete, and it is typically covered with a dedicated air and water barrier (AWB). Veneer walls have a cavity between the brick veneer and the AWB; as energy efficiency has become an increasing consideration of the exterior wall assembly, more and more insulation has been introduced into the cavity at the face of the backup wall. The structural support for the veneer comes either from direct support at grade from the foundation or through relieving angles connected back to the building's superstructure. With increasing material and labor costs and increasingly stringent insulation requirements, the industry appears to be shifting toward thin brick systems, in part to help mitigate the increased material and labor costs, as well as costs that result from added structural brick support needed to accommodate increased insulation thickness and cavity depth.

4. THIN BRICK VENEER

The change from a brick veneer to a TBV system facilitates increasing insulation thicknesses without increasing the overall wall thickness and results in a reduction in facade weight with minimal aesthetic change. This helps design teams and owners meet current code requirements and find project cost savings. Although it is perceived as a simple change, selecting, detailing, and constructing the appropriate system are critical for project success. Considerations should include the facade geometry and system height, desired water management strategy, expected service life, insulation requirements, overall system height, and code requirements such as NFPA 285.

Facade Geometry and System Height

Thin brick systems are most beneficial on buildings with simple geometry and few plane changes other than corners. A uniform brick pattern is best; detailing and construction can become complex if multiple band courses and isolated brick course changes are required. Thin brick must never be used on skyward-facing surfaces, which must be protected with metal flashings. These flashings should also be integrated at windows, doors, and other fenestration within the design. Proprietary thin brick systems are typically designed for horizontal coursing and should not be used for vertical coursing unless specifically designed, tested, and manufactured for this application. As with brick veneer, expansion joints should be designed at regular intervals to manage the veneer's expected moisture expansion and thermal expansion and reduce the risk of cracking.

High-rise buildings require increased wind load demands on the veneer; this, in addition to code limitations, may dictate system selection. Adhered systems require tighter deflection requirements that may be challenging to achieve, and they also have height restrictions mandated by code. Designers need to be sure to review and confirm certified test results that can be obtained from the system manufacturers.

Water Management Strategy

Most thin brick systems installed today manage water in two different ways: via a drained system with a drainage plane behind the thin brick, or via a drained and vented system with a larger vented air cavity behind the tiles. Drained systems and drained and vented systems have similar outward appearances but manage water very differently. The system selection will also impact other materials found within the wall assembly, such as insulation. Drained systems are designed with a dedicated AWB with an understanding that water will enter behind the face of the wall. These systems function best when the drainage plane is designed with space or voids between the thin bricklayer and the materials behind it.

Drained and vented rainscreen systems also have a dedicated AWB within the wall system, which, tied with proper flashings, will direct water out of the wall. The major difference between drained systems and drained and vented rainscreen systems is the addition of steel or composite support members holding the thin brick away from the other materials and creating an air space.

Expected Service Life

The system's service life relies upon adhesives and/or mechanical attachment, and that service life will be dictated by the service life of the materials used. Since proprietary systems are relatively new, long-term maintenance methods have not been well established. It is currently unclear whether thin brick systems can be repointed without damaging the supporting elements when the mortar joints begin to erode or separate from the tile. Unlike brick cavity wall construction, proprietary TBV systems utilizing panels or rails rely entirely on light-gauge steel elements to provide gravity and lateral support. These steel elements may corrode over time. At the end of its service life, when mortar or steel supporting elements deteriorate, the system may need to be replaced in its entirety. Due to these factors, the thin brick repair projects we have been involved in have generally resulted in a recommendation for wholesale replacement of the TBV.

Insulation Requirements

Depending on a project's climate zone, insulation requirements will vary. In some zones, this could mean several inches of continuous exterior insulation. This creates an increased separation between the cladding and the structure behind it. Some systems require continuous support, and rigid insulation would be needed, while other drained and vented systems allow for mineral fiber or other insulation systems.

NFPA 285

The building's construction type classification may require exterior wall assemblies to meet the criteria of NFPA 285, Standard Fire Test Method for Evaluation of Fire Propagation Characteristics of Exterior Wall Assemblies Containing Combustible Components. When required due to building height and construction type, NFPA 285 applies to most exterior wall assemblies over 40 ft. This may limit the use of rigid insulation in some applications.

Differential Movement

TBV systems must be designed to handle the differential growth between the systems and the building's framing system. For example, cross-grain shrinkage of floor plates in woodframed construction versus brick growth can result in buckling and delamination between the substrate and the thin brick. Adding periodic control joints to the facade, following the guidelines of the Brick Industry Association's (BIA's)Technical Note 18A, Accommodating Expansion of Brickwork, as well as project-specific needs, can help mitigate this risk.

5. TYPES OF PROPRIETARY THIN BRICK SYSTEMS

Most of the available TBV systems are a variation of an adhered system, commonly used with the thin-set/adhesive method or thick-set/cementitious mortar method applied directly onto a substrate. Some proprietary systems available in the market fully support the thin brick tiles and provide mechanical attachment to the wall.

5.1 Adhered Systems

Some TBV systems utilize rigid insulation or perforated metal panels for tile support, with tiles bonded to these substrates using adhesives. These adhered thin brick systems rely on the adhesive bond between the thin brick and its substrate. Adhesives depend highly on the installer and the weather during installation and curing and should be monitored carefully. Designers should consider the specific properties of the substrate the thin brick is adhering to and its ability to manage water with the thin brick. The following summarizes common types of adhered systems seen today:

5.1.1 Thin-Set (Adhesive) Systems
Some TBV systems are adhered directly to rigid insulation, set in adhesive on ribbed, extruded polystyrene (XPS) insulation over a dedicated water barrier and wall sheathing. The insulation relies solely on light-gauge steel clips for its mechanical attachment to the framed backup wall structure. The clips securing the insulation are also embedded within the mortar joints of the thin brick. The compatibility of the adhesive and the insulation is critical to achieving proper adhesion to the insulation.

We observed a 15-year-old installation of this type of system on a building

FIGURE 2. Insulation clip in the wet zone of the wall, which has begun to corrode.

in the northeastern United States. We found that the insulation clips were exposed to water, and some had undergone severe corrosion with substantial section loss (**Fig 2**). Over time, continued corrosion will lead to loss of structural attachment of the TBV, including the XPS substrate. Light-gauge steel elements that this system relies on for structural attachment for unfortunately limited the service life of this wall system.

In more recent adhered or thin-set applications, the thin brick is installed over a cement board or exterior sheathing substrate. The substrate may be protected with multiple layers of a weather-resistant barrier (WRB), and the thin brick is adhered using adhesives or a modified thin-set adhesive. These systems are only appropriate where exterior insulation is not required, and they are often only used for limited installations on the first level of a building. Due to the lack of a reliable drainage plane, we do not recommend this installation as it is prone to failure and highly dependent on workmanship during installation.

During construction, it is essential to oversee the thin brick installation. Substrates should be thoroughly reviewed before the installation of the adhesives. Any dirt or construction material left on the substrates can drastically reduce the adhesive bond between the bricks and the substrate. We recommend that the substrate be thoroughly cleaned before any adhesive installation. Substrates should be within a tolerance of ¼ in. over 10 ft and generally smooth. Substrates should be installed to minimize gaps.

Manufacturers generally call for adhesives installed into quarter-sized spots of adhesive rather than continuous ribbons, which may collect and trap water, freeze, and damage the TBV.

5.1.2 Thick-Set (Adhesive) Systems Thin bricks can also adhere to a scratch coat of portland cement plaster and lath attached to the backup wall, installed per ASTM C926 and ASTM C1063. This system utilizes the bond of the cement plaster to the brick rather than adhesives. As with all thin brick systems, we recommend that these systems be designed with periodic control joints, a drainage mat, and a dedicated WRB behind the scratch coat to act as the drainage plane. Control joint spacing needs to be designed using the most limiting factor. Control joint spacing should be guided by both ASTM C1063 and BIA technical notes.

5.1.3 Panel-Backed System As the industry is looking for systems that increase the drainage within the system and allow for continuous insulation (CI), we are seeing increased use of proprietary systems utilizing manufactured, perforated metal backer panels (**Fig. 3**). These metal panels are typically very light-gauge galvanized or painted steel panels with perforations designed to allow water to pass through the metal panel and provide semicircular projections that engage the thin brick and mortar. The panel may also have ribs to hold the panel off the surface of a designated AWB to facilitate drainage. While these designs may provide some mechanical attachment from the backup wall to the thin brick via arching of the hardened mortar, the systems also rely on the quality and bond of the adhesive. Due to their light gauge of metal, the panels require a continuous solid substrate for attachment. They cannot be applied over furring or purlins to provide a vented cavity because the panels lack stiffness and can deflect without a continuous solid substrate, damaging the mortar and thin brick.

It is important to remember that the panels will experience wetting from moisture that passes through the thin brick tiles and mortar. For this reason, all cut edges of panels should be treated with corrosion-inhibiting coatings, and corrosion-resistant fasteners should be used to secure the panels.

The solid-substrate requirement of these systems can create complexity when trying to address CI requirements and create a thermally balanced wall assembly that limits internal condensation risks. To meet the CI requirement, the insulation is often located outside of the AWB used to protect the sheathing and structural system of the wall.

If rigid insulation is part of the wall system, the wall system must be designed to the requirements of NFPA 285, and the potential reduction of vapor diffusion must be considered. If the system cannot be designed to meet the requirements of NFPA 285, this may limit the allowable height of the system.

As backer panel-backed TBV systems age, weather, and move, the mortar joints can begin to separate from

FIGURE 3. Panel-backed TBV system.

the tile, allowing increased water entry and wetting of the galvanized panels. We expect this will accelerate the corrosion of metal components, limiting the service life of these systems, as observed above. While brick veneer or mass masonry can be repointed to address the weathering of mortar that comes over time, it is still unclear whether thin brick panels will be able to be maintained in this fashion or if they will require wholesale replacement. We expect mortar removal methods to damage the metal backer and/or dislodge the tiles.

5.2 Rail-Mounted Systems

Rail-style thin brick systems use a continuous galvanized C-like channel or rail, which fully engages slots cut into the top and bottom edges of the thin brick, providing mechanical, not adhesive, attachment. These rails are fastened back to vertical framing, typically to Z-girts or a rainscreen support system attached to the backup wall (**Figs 4** and **5**). These systems can be installed as a ventilated rainscreen, allowing drying of the cavity, which may prolong the life of the steel. These systems include a dedicated AWB at the backup wall and allow for cavity insulation. Insulation can be rigid or semi-rigid because the system does not rely on a continuous rigid backing as the panel-backed systems do. Cut ends of the rails should be protected with galvanizing compound to reduce corrosion potential, and we recommend using corrosion-resistant fasteners. Due to the mechanical attachment of the thin bricks and their ability to incorporate noncombustible semirigid insulation, these systems can be used in high-rise applications and to avoid the limitations of systems relying on adhesives.

Other Considerations

Traditional masonry uses cementitious mortar to bond masonry units into the wall assembly—the mortar weathers over time, which is accelerated in freeze-thaw conditions. During repointing, the outer portion of the mortar is typically replaced periodically; this is generally done on the outer ½ in. of mortar or more, depending on the level of deterioration. Where thin brick systems have only ½ in.

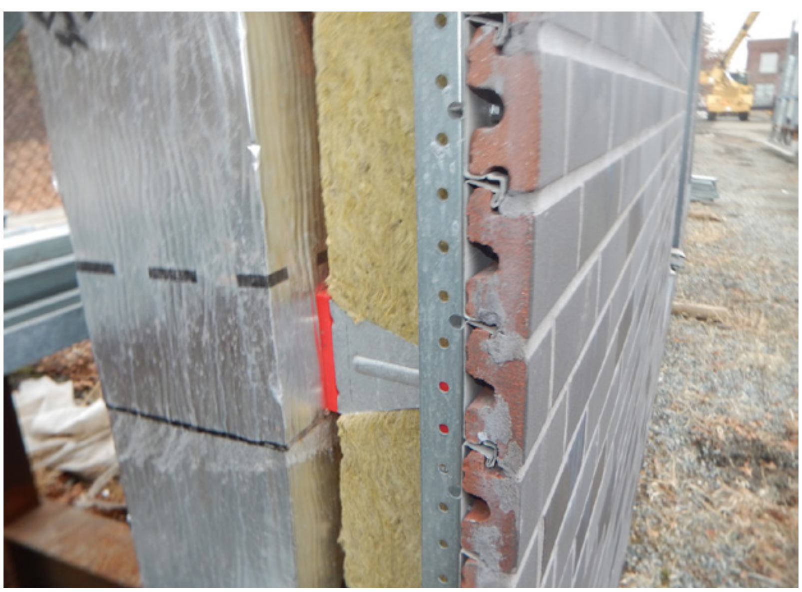

FIGURE 4. Thin brick rail system on a rainscreen support system.

FIGURE 5. Panelized thin brick rail system project during construction, rails installed, bricks set in place, mortar work not yet complete.

depth of mortar, the deterioration of the mortar will likely compromise the entire assembly. Manufacturers are using polymer-modified mortar formulations to help reduce moisture absorption and improve initial adhesion, which may enhance system longevity. However, while these mortar additives improve mortar's freeze-thaw performance and initial adhesion to edges, they do not eliminate freeze-thaw potential. Mortar joints should be a minimum of ¼ in. between bricks and a minimum of ½ in. deep.

6. SUMMARY

- Veneer sizes should not exceed 2⅝ in. thick or 36 in. on any side or 15 psf weight in adhered installations.
- Maintain a minimum ¼ in. joint between bricks. Install a separate pointing mortar full depth of the brick joints after the adhesion mortar is fully cured.
- Utilize polymer-modified mortar additives for increased resistance to moisture absorption and improved adhesion characteristics.
- Maintain the bottom of the brick a minimum of 8 in. above grade or as required by the manufacturer. Natural stone or full-depth brick can be installed at and below grade.
- Provide movement joints necessary to accommodate deflections and veneer movement. Refer to ASTM C926 and C1063 for adhered masonry veneer applications. Be particularly wary of cross-grain shrinkage at floor plates in wood-framed construction, and provide adequate means to accommodate shrinkage to avoid buckling.
- Provide a drainage layer, drainage mat, and flashing behind the veneer with weeps to allow the water to evacuate the system; this reduces efflorescence but does not eliminate it. Provide a dual-layer weather barrier consisting of SAM (self-adhered membrane) over the sheathing or WRB covered by 15-lb felt paper.
- Do not install thin brick in horizontal overhead applications or on skyward-exposed surfaces.

CHANGE IS IN THE AIR (BARRIER!)

ABSTRACT

Air barrier requirements for commercial buildings are undergoing substantial changes in the 2022 ASHRAE 90.1 Standard and the 2024 IECC. Continuous air barriers have been required in most buildings for a number of years. Recently the model commercial energy codes, ASHRAE 90.1 Standard and the 2024 IECC have been updated to provide more specific and stringent requirements for buildings. This presentation will provide expert insight from two individuals who have been engaged in the code development process for years and were specifically involved with the air leakage updates in both the ASHRAE 90.1 Standard and the International Energy Conservation Code. The air barrier updates include clarifications to the whole building performance testing methods and stringency, design phase requirements, material and assembly requirements, and on-site installation verification requirements. We will discuss the appropriateness of applications and the interaction between the building and energy code requirements. A discussion on new code development updates, design-based applications, and construction best practices will also take place.

LEARNING OBJECTIVES

- Discuss the impact of air barrier systems on energy efficiency.
- Recognize the changing building and energy codes and their interaction with building enclosure systems.
- Evaluate how to implement an air barrier strategy to comply with the code and owner performance requirements.
- Explain how specific examples and air barrier systems can be applied to current and future project designs.

SPEAKERS

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Benjamin Meyer, AIA, NCARB, LEED AP, is the building enclosure business director with Siplast. His previous experience includes enclosure consultant principal, technical management, research, and education for enclosure products, commercial design, real estate development and construction management on a range of projects that included residential, educational, offices, and DuPont industrial projects. Industry positions include voting member of the ASHRAE 90.1 Envelope and Project Committees, LEED Technical Committee member, past LEED Materials (MR) TAG, and director of the Air Barrier Association of America. Meyer has MBA, BS, and M. Arch degrees from the University of Cincinnati.

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Theresa Weston, PhD, is the president of the Holt Weston Consultancy, providing building science-focused expertise to increase sustainability and resiliency of the built environment. Weston is on the executive committee of the ASTM Committee on the Performance of Buildings and chairs the Subcommittee on Air Leakage and Ventilation. At ASHRAE, she is the immediate-past chair of the Residential Buildings Committee and immediate-past-Chair of the Standard for Energy Efficient Design of New Low-Rise Residential Buildings (90.2), and past chair of the Technical Committee on Building Materials and Building Envelope Performance. Weston is active in the International Code Council code development process.

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INTRODUCTION

Building energy codes are advancing toward providing greater levels of energy efficiency with the ultimate goal of obtaining net zero energy buildings. The *International Energy Conservation Code* (IECC) will, beginning in 2024, have the following intent for its commercial energy provisions:

> *The International Energy Conservation Code-Commercial provides market-driven, enforceable requirements for the design and construction of commercial buildings, providing minimum efficiency requirements for buildings that result in the maximum level of energy efficiency that is safe, technologically feasible, and life cycle cost-effective, considering economic feasibility, including potential costs and savings for consumers and building owners, and return on investment. Additionally, the code provides jurisdictions with supplemental requirements, including ASHRAE 90.1, and optional requirements that lead to the achievement of zero energy buildings, presently, and through glide paths that achieve zero energy buildings by 2030 and on additional timelines sought by governments, and achievement of additional policy goals as identified by the Energy and Carbon Advisory Council and approved by the Board of Directors. Requirements contained in the code will include, but not be limited to, prescriptive- and performance-based pathways. The code may include nonmandatory appendices incorporating additional energy efficiency and greenhouse gas reduction resources developed by the Code Council and others. The code will aim to simplify code requirements to facilitate the code's use and compliance rate. The code is updated on a three-year cycle with each subsequent edition providing increased energy savings over the prior edition. This code is intended to provide flexibility to permit the use of innovative approaches and techniques to achieve this intent. This code is not intended to abridge safety, health or environmental requirements contained in other applicable codes or ordinances." (ICC, 2021b)*

Required for obtaining these energy efficiency goals, air barriers and, more widely, air leakage requirements are increasing stringency in both required levels and in the verification of these proposals as the energy codes are developed. For non-low-rise residential buildings, the two pertinent documents to follow to understand the changes in air barrier requirements are ASHRAE Standard 90.1[3] and IECC. ASHRAE 90.1 is mandated as the federal minimum energy code in the Energy Policy Act of 2005. Both the IECC and ASHRAE 90.1 are on three-year development cycles. Local governments can choose to adopt one or both energy standards as their code requirements. The IECC is also written to reference ASHRAE 90.1 as an alternate compliance path, so both documents are relevant to energy compliance and air leakage.

THE CURRENT STATE

The current status of state code adoption is surveyed by the Department of Energy (DOE).[4] **Figure 1** presents the DOE's map of the status for commercial

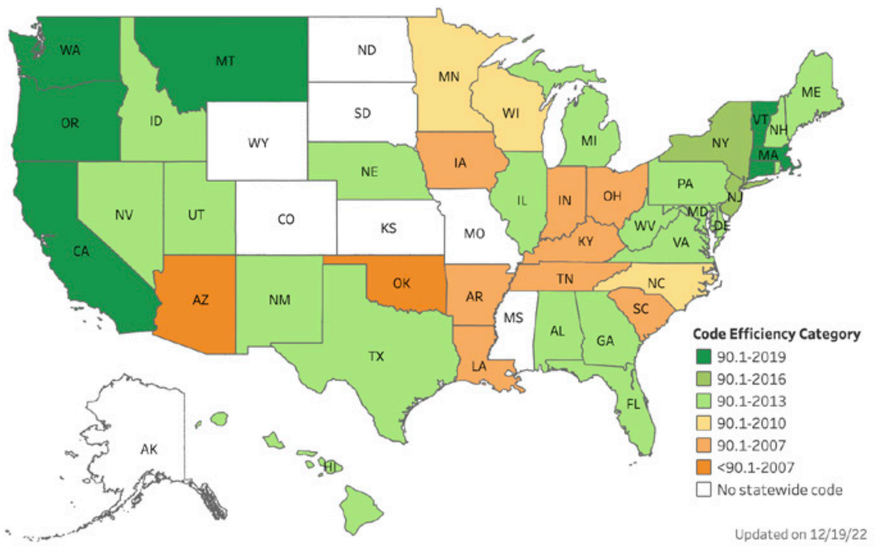

FIGURE 1. State commercial building energy code adoption as of June 2022. *Source: US Department of Energy Building Codes Assistance Project.* https://www.energycodes.gov/state-portal.

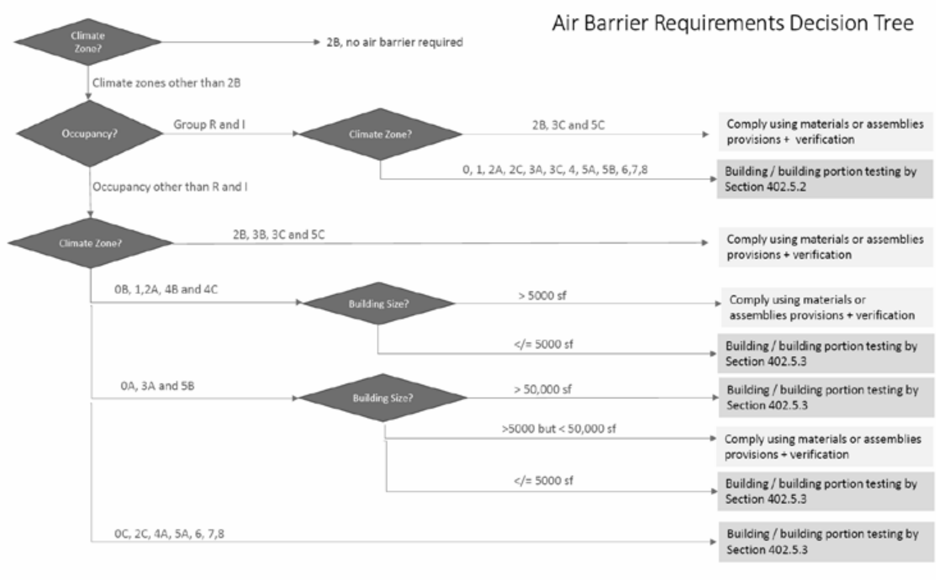

FIGURE 2. IECC-2021, Confusingair barrier requirements decision tree for the International Energy Conservation Code 2021.

energy code adoption as of June 22, 2022. While many states are lagging behind in adoption of the model commercial energy codes, we will consider ASHRAE 90.1-2016[5] and IECC 2018[6] as the baseline in this paper.

Both ASHRAE 90.1 and IECC have multiple compliance paths: prescriptive and performance. In both codes, air leakage provisions are mandatory across all the compliance paths, although a few minor exceptions are noted in **Tables 1**, **2**, and **3**.

In a prescriptive complying air barrier, the user must comply by meeting the air barrier requirements outlined in the text of the code or standard, but demonstrating on-site air leakage performance is not required. Starting with the prescriptive path, the code stringency can be described by examining the following requirements (see **Table 1**):

» Compliance specification options (material, assembly, and/or whole building)

» Design documentation requirements

» Installation verification through on-site inspection of the air barrier

TABLE 1. Comparison of baseline energy codes/standards

	ASHRAE 90.1-2016	IECC-2018
Compliance testing options	Materials, assembly, or whole building	Materials, assembly, or whole building
Maximum air leakage level	Materials ≤ 0.004 cfm/ft² @ 75 Pa	Materials ≤ 0.004 cfm/ft² @ 75 Pa
	Assemblies ≤ 0.04 cfm/ft² @ 75 Pa	Assemblies ≤ 0.04 cfm/ft² @ 75 Pa
	Whole buildings ≤ 0.40 cfm/ft² @ 75 Pa	Whole buildings ≤ 0.40 cfm/ft² @ 75 Pa
Whole building testing requirements	Option, not required	Option, Not required
Exceptions	Semiheated spaces in Climate Zones 0 through 6	Air barriers not required in Climate Zone 2B
	Single wythe concrete masonry buildings in Climate Zone 2B	
Installation verification	Design and installation verification *or* whole building air leakage testing	None specified
Performance modeling compliance	Design phase: Model-specified air leakage (energy savings credit for performance ≤ 0.40 cfm/ft²)	Follow mandatory requirements (0.40 cfm/ft² @75 Pa)
	Construction phase: Adjust model for actual tested air leakage, energy savings credit for performance better than 0.40 cfm/ft²)	
Simplified path compliance	COMcheck compliance: default 0.40 cfm/ft² air leakage input	Default 0.40 cfm/ft² air leakage input; reduced air leakage package for 0.25 cfm/ft² @ 75 Pa

TABLE 2. Comparison of current/active energy codes/standards

	ASHRAE 90.1-2019	IECC-2021
Compliance testing options	Materials, assembly, or whole building	Materials, assembly, dwelling unit (Group R and I) or whole building (required under certain conditions)
Maximum air leakage level	Materials ≤ 0.004 cfm/ft^2 @ 75 Pa	Materials ≤ 0.004 cfm/ft^2 @ 75 Pa
	Assemblies ≤ 0.04 cfm/ft^2 @ 75 Pa	Assemblies ≤ 0.04 cfm/ft^2 @75 Pa
	Whole buildings ≤ 0.40 cfm/ft^2 @ 75 Pa ("oops clause" allows up to 0.60 cfm/ft^2)	Whole buildings ≤ 0.40 cfm/ft^2 @75 Pa ("oops clause" allows up to 0.60 cfm/ft^2)
		Dwelling units ≤ 0.30 cfm/ft^2 @ 50 Pa, sampling allowed.
Whole building testing requirements	Required with exceptions for (1) buildings over 50,000 ft^2, which can be tested in parts; and (2) cases when verification of the design and installation of the continuous air barrier is conducted	Required for occupancies other than Groups R and I, with multiple exceptions for Climate Zone and building size
Exceptions	Semiheated spaces in Climate Zones 0 through 6	Air barriers not required in Climate Zone 2B;
	Single-wythe concrete masonry buildings in Climate Zone 2B	
Design and installation verification	Third-party verification required when whole building testing is not being performed	Verification required when testing by dwelling unit or whole building testing is not done
	Design to be detailed and identified in construction documents as continuous	Installation of continuous air barrier verified by the code official, a registered design professional, or approved agency. Includes review of construction documents, inspection during construction and a final commissioning report
Performance modeling compliance	Design phase: Model-specified air leakage (energy savings credit for performance ≤ 0.40 cfm/ft^2)	Follow mandatory requirements (0.40 cfm/ft^2 @75 Pa)
	Construction phase: Adjust model for actual tested air leakage, energy savings credit for performance better than 0.40 cfm/ft^2)	
Simplified path compliance	COMcheck compliance: Default 0.40 cfm/ft^2 @75 Pa air leakage input	Default 0.40 cfm/ft^2 air leakage input; reduced air leakage package allowed for 0.25 cfm/ft^2 @ 75 Pa

In a performance-based complying air barrier the user must comply by meeting the performance values in both energy modeling and on-site building performance. The code stringency for the performance path can be described by examining the following requirements (see **Table 1**):

» Modeling of the energy performance of the entire building, including specified air leakage performance

» Required whole building testing and exceptions for climate zone and building size to determine its actual level of air tightness

» Updating the energy model to reflect the actual building performance

THE NEXT STEP

ASHRAE 90.1-2019 and IECC 2021 took a step forward in energy efficiency requirements, with the IECC 2021 requiring whole building testing under certain conditions. The requirements are shown in **Table 2**.

ASHRAE 90.1-2019, undertook primarily a reorganization of the air barrier section (5.4.3) which establishes whole building testing performance as the overall performance metric in the standard. It also provides additional guidance for buildings that are tested and miss the requirement (informally known as the "oops clause"), clarifies testing options for large buildings, improves the design and detailing requirements, and moves the material and assembly requirements to the product information section (5.8).

FIGURE 3. ABAA Air barrier inspection and audit *Source: Air Barrier Association of America.*

In IECC 2021 for residential occupancies (multi-family buildings) testing by dwelling unit was introduced for residential occupancies (multifamily buildings). However, the requirements and exceptions, especially in the IECC-2021, were very complicated, and compliance may therefore be difficult to achieve. **Figure 2** shows the complicated and confusing air leakage testing criteria introduced in IECC 2021. Much of the complexity and confusing language in the code that is captured in **Figure 2** is resolved with the pending updates in IECC 2024.

The path to the enhanced requirements in ASHRAE 90.1-2019 and IECC-2021 was forged by early adopters at the state level. Early adopters of enhanced air barrier requirements including whole building testing include the Commonwealth of Massachusetts, Washington State, and the cities of Seattle, Washington, and Fort Collins, Colorado. Lessons learned from Seattle's experience have been shared by both regulatory officials and consulting engineers in the city. According to a consulting engineer.[7]

> The air barrier design and testing requirements in the greater Seattle area has allowed the industry to digest the realities of whole building air barrier design and testing. From a tightness standpoint, what we have found is that if the proper care is taken, buildings indeed can be designed to meet the requirements of 0.4 CFM/SF@1.57 psf and now, 0.3 CFM/SF @ 1.57 psf.

A Seattle regulatory official[8] offered the following conclusion:

> Seattle's principal message for jurisdictions that are considering a similar path is that air barrier testing initially is difficult for all of the industry players but soon becomes routine. As a result of Seattle's record-breaking construction boom over the past several years, hundreds of new buildings in the city have pressure-tested air barriers. Although the air leakage rates of buildings constructed in the decades before this code requirement took effect are not fully understood, it appears that the leakage in these new buildings has been reduced by more than half.

The experience in Seattle has shown that there is a fast learning curve on both the construction of airtight buildings and the testing of those buildings. This learning curve may be duplicated in other jurisdictions.

In addition to following the lessons of the early adopter jurisdictions, the industry has continued to improve the tools for building air leakage verification. In particular, the development of standard test methods for air barrier testing specifically designed for compliance with code or other specifications has aided the industry in being able to conduct building testing:

- ASTM E3158, *Standard Test Method for Measuring the Air Leakage Rate of a Large or Multizone Building*[9]: As stated in the scope of ASTM E3158, "This test method applies to an air leakage rate specification with a reference pressure greater than 10 Pa (0.04 in. WC) and not greater than 100 Pa (0.40 in. WC)." The method was specifically developed because air leakage testing was being increasingly proposed in building specifications and codes. More specifically, it was initially driven by the need to meet the U.S. Army Corps of Engineers (USACE) requirement of a maximum air leakage of 0.25 cfm/ft2 of building envelope @ 0.3 w. g. (75 Pa) (USACE 2009, USACE 2012.[10,11] In collaboration with USACE, the Air Barrier Association of America developed an air leakage test protocol to enable the testing required by the USACE directive.[12] Based on this test protocol, ASTM E3158 was developed through the consensus process. ASTM E3158 enables whole building air leakage testing because it was specifically aimed at specification/code compliance and development was led by testing practitioners.
- ANSI/RESNET/ICC 380, *Standard for Testing Airtightness of Building, Dwelling Unit, and Sleeping Unit Enclosures; Airtightness of Heating and Cooling Air Distribution Systems; and Airflow of Mechanical Ventilation Systems*[13]: As stated in the standard's purpose, "The provisions of this document are intended to establish national standards for testing the airtightness of enclosures and heating and cooling air distribution systems, and the airflow of mechanical ventilation systems. This Standard is intended for use by parties including home energy raters, energy auditors, or code officials who are evaluating the performance of Residential Buildings, or of Dwelling Units or Sleeping Units within Residential or Commercial Buildings." As with ASTM E3158, ANSI/RESNET/ICC 308 is specifically aimed to evaluate specification/code compliance by practitioners. This standard is especially important for multifamily residential dwelling unit compartmentalization air leakage testing.

Another important resource is the Air Barrier Association of America (ABAA) Air Barrier Quality Assurance Program (QAP),[14] which is a jobsite program that encompasses materials, installation, and inspection of the air and moisture barrier system. ABAA QAP program's frequency is determined by the size of the building. It is designed to utilize ABAA accredited contractors, ABAA-certified installers, ABAA-evaluated materials, and ABAA-trained third-party field quality control audits during the construction process with the goal of providing the complete range of services across the construction phases is to help minimize risk and liability within the building envelope (ABAA). An example of the field quality control audits is shown in **Figure 3**.

WHAT'S NEXT: THE WRITING IS ON THE WALL

The industry is currently developing the next generation of energy codes and standards, and our understanding of most of the air leakage requirements in those documents is somewhat speculative. **Table 3** shows the provisions expected to be included in the next editions.

Addendum *t*[15] to ASHRAE 90.1-2019, which updates air leakage requirements, is final and has been published and is included in the new ASHRAE 90.1-2022. This air leakage revision in 90.1-2022 improves the overall performance requirements for whole building testing, incorporates the new whole building airtightness testing standard ASTM E3158, establishes a set of smaller buildings that cannot be exempted from the testing requirements, improves the verification requirements in the construction documentation, and distinguishes modeled and simplified values for testing versus non-testing project compliance.

The first round of proposals related to air leakage requirements in the development of IECC 2024 has also been completed. Additionally, the air leakage section of IECC 2024 will be reorganized for clarity and the exceptions simplified to facilitate compliance. This reorganization is especially important with the very confusing air barrier requirements scattered throughout IECC 2021. In addition, the revisions in the current IECC 2024 draft closely mirror the changes published in ASHRAE 90.1-2019 Addendum *t*. IECC 2024 also will include additional options for residential (R) and institutional (I) occupancies for buildings that contain dwelling units.

THE FUTURE IS AIRTIGHT

As we look to the future, we anticipate that building enclosures will be increasingly airtight through the development of new technology, an increase in construction and installation verification, and, ultimately, the formalization of building enclosure commissioning (BECx) in specifications and codes.

As was the case with the USACE air leakage directive, proposed changes to air leakage requirements are likely to be piloted before they are incorporated into model building energy codes and standards. For example, in a sustainability/green construction standard from Phius, a nonprofit passive building organization, the maximum air leakage threshold for most projects is 0.060 cfm/ft^2 @ 50 Pa, which is significantly tighter than any current code values.[16] More than 15 states include Phius certification is included as a Qualified Allocation Plan for low-income housing tax credits. A passive building standard is also currently under development by ASHRAE.

Moving beyond voluntary and above-code programs into building energy codes will require technical and market research to demonstrate feasibility and cost-effectiveness. The following items are key next steps to continue to move the energy codes forward:

- Improvements in air leakage methodology. The energy savings attributed to air barriers is evaluated for code development using EnergyPlus building energy simulation software. However, recent research comparing building energy simulation models with an airflow and contaminant transport model found that the benefits of building air tightening were not fully captured in the current building energy simulation methodology and were underestimated by as much as 55%.[17]
- Accurate costs for whole building testing and on-site verification. Cost analysis is required for the inclusion of provisions in building energy codes. Few cost surveys or data summaries on the cost of air barrier verification or whole building air leakage testing are available. An example study on the economic effects of reducing building air leakage was published in support of code development for the state of California.[18] Specific data are needed to evaluate the following: (a) the reduction in test costs as the test process becomes more common; (b) the reduction in test costs as air leakage thresholds are reduced, and (c) the frequency of verification required for performance improvement and associated implementation costs.
- Improvements to the quality of whole building testing. There can be variability in the accuracy and experience of whole building testing providers. To provide guidance and establish a testing quality control process, industry training and certification programs are beginning to be initiated. One example is ABAA's Whole Building Airtightness Program, which was introduced in a Washington state pilot in 2022.[19]
- Understanding the relationship between energy efficiency benefits and other building benefits. Currently, air leakage is addressed primarily within energy efficiency codes and standards, whereas the benefits of air leakage control for indoor environmental quality (IEQ), moisture durability, and building resilience are covered in other documents or ignored. A more holistic understanding of how these different performance facets work together and are evaluated is necessary for a complete understanding of the value of reducing air leakage. For example, both IEQ and energy efficiency performance are improved by air leakage control, but in multifamily residences, IEQ is best characterized using compartmentalization testing and energy efficiency is best characterized using whole building testing. Testing protocols are under development that can provide both measurements instead of requiring separate testing or using one measurement as a surrogate for the other measurement.[20]

TABLE 3. Comparison of future energy codes/standards

	ASHRAE 90.1-2022*	IECC-2024†
Compliance testing options	Materials, assembly, or whole building	Materials, assembly, dwelling unit (Group R and I) or whole building (required under certain conditions)
Maximum air leakage level	Materials ≤ 0.004 cfm/ft² @ 75 Pa	Materials ≤ 0.004 cfm/ft² @ 75 Pa
	Assemblies ≤ 0.04 cfm/ft² @ 75 Pa	Assemblies ≤ 0.04 cfm/ft² @75 Pa
	Whole buildings ≤ 0.35 cfm/ft² ("oops clause" allows up to 0.45 cfm/ft²)	Whole buildings ≤ 0.35 cfm/ft² ("oops clause" allows up to 0.45 cfm/ft²)
		Dwelling units ≤ 0.27 cfm/ft² @ 50 Pa, sampling allowed.
Whole building testing requirements	Required for buildings less than 10,000 ft² and single-zone buildings	Required for occupancies other than Group R and I, except for buildings < 25,000 ft² in Climate Zones 0-4
	Required exceptions for when verification of the design and installation of the continuous air barrier are conducted	Groups R and I occupancies can be tested by dwelling unit
Exceptions	Semiheated spaces in Climate Zones 0-6	Air barriers not required in Climate Zone 2B
	Single-wythe concrete masonry buildings in Climate Zone 2B.	
Design and installation verification	Third-party verification required when whole building testing is not being performed	Verification required when testing by dwelling unit or whole building testing. Installation of continuous air barrier verified by the code official, a registered design professional or approved agency. Includes review of construction documents, inspection during construction and a final commissioning report
	Design to be detailed and identified in construction documents as continuous	Design to be detailed and identified in construction documents as continuous
	Construction documents to include inspection details, including (a) schedule/frequency, (b) scope of work, (c) critical observations, (d) document requirements, (e) corrective actions provisions	Construction documents to include inspection details, including (a) schedule/frequency, (b) scope of work, (c) critical observations, (d) document requirements, (e) corrective actions provisions
Performance modeling compliance	Design phase: Model-specified air leakage (energy savings credit for performance ≤ 0.35 cfm/ft2)	Follow mandatory requirements (0.35 cfm/ft2 @75 Pa)
	Construction phase: Adjust model for actual tested air leakage (energy savings credit for performance better than 0.35 cfm/ft2)	
Simplified path compliance	COMcheck compliance—whole building testing: Default 0.35 cfm/ft² @75 Pa air leakage input	COMcheck compliance: Default 0.35 cfm/ft² @75 Pa air leakage input
	COMcheck compliance—verification only: Default 0.45 cfm/ft² @75 Pa air leakage input	Allows reduced air leakage package on a sliding scale based on percent reduction of base mandatory level
		Reduced air leakage package extended to testing by dwelling unit

*Published late 2022, includes final 90.1-2019 Addendum *t*.

†Forecasted changes based on currently proposed *International Energy Conservation Code* revisions.

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CONCEPT OF DESIGN CONSIDERATIONS

ABSTRACT

Design reviews of plans and specifications are a critical step in the development of project manuals and "construction set" drawings. Design considerations, input from all stakeholders (owners, contractors, design professionals, and consultants) is added in the process as the project progresses from the original schematic design to a completed set of documents. There are several key resources (SpecsIntact, CSI, etc.) that provide key elements for the document development process. This presentation will address how to improve in-house standards for practice in design considerations and development and will focus on the uniformity of design documents over time and in different offices. Recommendations for minimum standards and compliance will be offered as a benchmark for design development.

LEARNING OBJECTIVES

- Identify strategies to improve in-house standards for practice in design considerations.
- Explain the importance of uniformity when multiple personnel or locations are involved in the process of design considerations.
- Discuss the minimum standards that follow industry guidelines and exceed the minimum requirements of codes.
- Provide minimum recommendations/comments when involved in design review and commissioning.

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The concept of "design considerations" started in the late 1980s for me, when as a roofing/waterproofing consultant for the Navy in the southeastern United States, one of my responsibilities was design reviews of new and renovation projects.

The majority of my comments were repetitive, and with word processing (computers) gaining in popularity, we created 07000 Reference Design Guidelines.

After I left the Navy in 1992 and started ADC Engineering, Inc. (ADC), the Navy's guide evolved into "Unified Facilities Guide Specifications (UFGS) - SpecsIntact" formerly 07 00 00 Roof Design Standards/Guides, which has now evolved into Whole Building Design Guide Roofing Systems https://www.wbdg.org/guides-specifications/building-envelope-design-guide/roofing-systems, which is a great guide/resource if you are not familiar with it.

At ADC, and now The Building Envelope Enclosures Group (The BEE Group), we developed an entire series of these "design considerations" for roofing issues (metal, photovoltaics, maintenance, etc.), a variety of building envelope/enclosure issues, such as exterior walls of various types (barrier, mass construction, redundant), and waterproofing; mechanical, electrical and Plumbing (MEP); roof maintenance, etc. These "design considerations" are used in house as a training tool and checklist, as well as an attachment to various design reviews, reports, and other correspondence.

ASTM E2813-18, Standard Practice for Building Enclosure Commissioning, includes both "Pre-Design" and "Design" (Schematic Design, Design Development, and Construction Documents) Phases. The Design Considerations can be used in all phases but are well suited during the Pre-Design and Design Phases.

As an example, below is our latest "Roof Replacement Design Considerations." These are "dynamic" documents, as they are constantly updated and revised with industry and code changes.

ROOF REPLACEMENT DESIGN CONSIDERATIONS

1. Roof design should meet current/applicable code requirements. Please note that the *International Building Code (IBC)* has made some significant changes in regard to minimum requirements of code based on replacement, re-cover, retrofit, or percentages of area included. If adopted in your area, the *International Existing Building Code IEBC* also has specific requirements related to re-roofing.
   - a. The *IBC* relies heavily on two criteria, (1) ASTM and the (2) manufacturer requirements.
     - 1) ASTM: Who meets ASTM standards for ethylene propylene diene monomer (EPDM), polyvinyl chloride (PVC), modified bitumen, etc.? All manufacturers do. You must define the classification, properties, and values that are applicable.
     - 2) Manufacturer requirements: When one is willing to rely on an unproven manufacturer for determining acceptable criteria, details, and/or products because of his allegiances or expertise, then he must truly read the fine print. Most guide specifications list or allow the majority (if not all) of manufacturers of a specific type of system. These guide specifications may be a starting point but should first be edited based on the specific project, geographic location, contractor base, project needs, and relevant performance issues.

b The latest (if applicable) *IEBC* and *IBC* makes specific changes in regard to several structural issues (deck diagram verification, parapet walls, drainage, etc.), and restrictions on the use of gravel/ ballasted roof systems in specific wind zones in the *IBC*.

2 The following industry standards contain the general construction and detail recommendations:

a National Roofing Contractors Association (NRCA), Asphalt Roofing Manufacturers Association (ARMA), Single Ply Roofing Industry (SPRI), and Sheet Metal & Air Conditioning Contractors' National Association Inc. (SMACNA).

3 As a minimum, the current NRCA *Roofing and Waterproofing Manual* and the Construction Details should be the basis for all design development. Specific, custom details will still be required for unique or special penetrations and project-specific terminations. Recommend adding a paragraph in the applicable Specification Section and note on the drawings stating any clarifications will be in accordance with this standard.

4 As a minimum, the most current SMACNA *Architectural Sheet Metal Manual*, 7th Edition, is also an excellent source of details and criteria for various sheet metal systems and components but requires specific needs to be identified for the specific job, since various configurations exist for different components. Recommend adding a paragraph in the applicable Specification Section and note on the drawings stating any clarification will be in accordance with this standard. Please note that several details/configurations often exist in this reference.

a SMACNA also has two additional standards: (1) Architectural Sheet Metal Inspection Guide and (2) Architectural Sheet Metal Quality Assurance Guide.

5 Coordinate with the owner any specific requirements that may exist from their insurance company (e.g., whether the building is FM insured, fire and wind resistance, builders' risk ,etc.).

6 Do not write specifications that rely on or allow any manufacturer to determine the criteria, acceptability of a condition, or the details to be used.

7 A proprietary specification does not have to list a specific product. Oftentimes, a specific property or test is specified that is unique to one manufacturer. If a specification is written with only one supplier, manufacturer, or contractor able to meet the requirements, the specification is proprietary. This could cost the owner due to a non-competitive bid environment and provide liability to the designer if the specified product fails.

8 Coordinate and edit the sections of the specifications that relate to but are not exclusive to the roof system (e.g., demolition, abatement, deck section, rough carpentry, roof insulation, sheet metal, roof accessories, sealants, etc.).

9 Provide an existing roof plan showing actual field dimensions, penetrations, and core sample summaries. Do not rely on as-built/record drawings! Show all penetrations on this roof plan and indicate (hatch) any abandoned equipment to be removed. Ninety percent of leaks are related to penetrations and terminations.

10 Clearly show sections of construction requirements for the deck, insulation, membrane, curb construction, base flashing, and counterflashing to ensure proper flashing heights and clearances for the subject system.

11 Provide a new roof plan; identify primary and secondary slopes (in structure or with tapered insulation) and all penetrations, terminations, and unique, custom requirements. Provide applicable details and detail references for the penetrations and terminations. Isometric details are preferred.

12 Manufacturer's warranty requirements should be based on owner's choice. Warranties do not protect owners; they limit the liabilities of manufacturers. Do not make decisions on the roofing system, its details, or variations based own warranties. All jobs should require a two-year contractor warranty as a minimum.

a Recommend a minimum two-year contractor warranty written by owner/consultant, and to be independent of manufacturer's warranty.

b If a manufacturer's warranty is required, recommend a 20-year no-dollarl-limit, (NDL) material and workmanship warranty (to include the insulation).

c The warranty is a requirement of the project, not the determining factor.

13 A sound substrate is critical to the success of any re-roofing system. A thorough investigation and core sample extractions are recommended for the buildings requiring roof replacement, re-cover, retrofit construction, or repair. Several existing deck types require special testing/verification to ensure adequate attachment can be accomplished. Cementitious wood fiber, lightweight insulating concrete fills, gypsum fills, and asphaltic perlites are examples.

14 A minimum of one roof core and one flashing core should be taken for each roof area to determine assembly and preliminary testing for asbestos containing roofing materials (may require three samples of each suspect homogeneous material). Additional cores or spikes shall be taken to determine if slopes are in deck, insulation, or a combination of both.

15 Request that the owner provide all environmental (asbestos, etc.) reports for roof and underside of deck. Preliminary core sample extractions of roof system should be tested for asbestos so that appropriate specifications can be written. As noted above, further sampling may be required to meet sampling requirements for non-friable asbestos-containing roofing materials (ACRM), considered a "miscellaneous"

or "other" material (Polarized light Microscopy [PLM] and possibly Transmission Electron Microscopy [TEM] testing may be needed.)

16 Various components within and/or associated with the roof assembly can become damaged or deteriorated and should be anticipated. For repair or partial replacement projects, consider the use of unit price items to be provided as part of the bid for items susceptible to damages (various deck types, wood nailers, etc.). The contract can include the anticipated quantity, and the unit price included will be provided as an add or deduct based on actual quantities used. Written notification should be provided when 80% of quantities are used.

17 Re-cover roof applications are common, but in my opinion, this is seldom a good, cost-effective long-term choice, unless a reduced service life is anticipated due to future plans. When a re-cover is being considered, significant fieldwork and a review of code requirements specific to this issue are necessary. Specific criteria exist for moisture surveys and removal in many situations.

18 Interior inspections should review the underside of the deck for MEP items, asbestos, fireproofing, general deck condition, and any other potential problems, if in the scope.

19 Mark (paint) all areas that require maintenance on roofs that are not being replaced. This will allow specifications to be developed based on identified areas/locations.

20 When completing maintenance and repairs to typical low slope roof types (built-up, modified bitumen, thermoplastic, and thermoset) use the *NRCA/ARMA/SPRI Repair Manual for Low Slope Maintenance Roof Systems*.

21 Nailers are always a controversial issue. Numerous benefits are gained by providing nailers around the perimeter of buildings and at various penetrations. In some instances, they are critical, or required. Do not eliminate nailers for a given location without proper research and coordination. Nailers should be a minimum 2 x 6 size, treated, and match insulation thickness. Sufficient attachment of nailers to the various substrates should be clearly specified. Ensure wood treatment type and fasteners used are compatible.

- **a** Code required ES-1 for metal copings and edge metals may affect this issue.
- **b** Use of copper azole (CA), alkaline copper quaternary (ACQ), and micronized copper quaternary (MCQ) treatments have caused issues with corrosion at some fasteners as well as aluminum and galvalume sheet metal that is in direct contact with wood.
- **c** With the new wood treatment types, aluminum and galvalume should not come in direct contact with rough carpentry. We recommend a waterproof underlayment under all sheet metal (as a separator and added protection/redundancy).
- **d** Stainless-steel fasteners, or "hard-coat" fasteners should be considered.

22 Ensure insulation is compatible with the membrane application subsequent layers of differing insulation types (including differing manufacturers), insulation, and the other materials to be incorporated into the roofing system. Verify limitations on the maximum and minimum specified insulation thickness, and sizes of insulation boards regarding specific types and applications.

23 Insulation should be installed in a minimum of two layers, with joints over bearing surfaces and staggered between layers for most roofing systems. Insulation thickness should be adequate to span rib opening on metal deck and fall within minimum and maximum thickness limitations of FM or UL rating as applicable.

24 All roof systems will have increased uplift forces at corners and perimeter. Ensure the design and the submittals identify the dimensions and how the increased attachment occurs at these locations. Also be aware of restrictions on wind speeds with warranties.

25 Roofs are to have primary and secondary drainage systems (outlets). This is recommended. Involve your MEP consultant to ensure an independent secondary drainage system is provided (i.e., overflow provisions). Indicate drainage points on the new roof plan.

26 All re-roofs are recommended to have positive drainage as a minimum. This is often defined as "no ponding water after 48 hours." Recommend a minimum finished slope should be one-quarter inch per foot for the primary and secondary slope whenever possible. Minimum slope for re-roof construction should be one-quarter inch per foot without further verification and review. Provide insulation/crickets/tapered sumps for secondary slope as necessary to assure drainage. Ensure you know the definition of "ponding" water versus "standing" water.

27 The heights of penetrations, parapet walls, locations of counterflashings and base flashings, insulation, and nailer thickness must be considered and coordinated, especially when using tapered insulations. If tapered insulation is to be used, determine how it will affect the penetrations, perimeter conditions, parapet walls, door thresholds, adjoining sections, roof-mounted equipment, overall drainage, etc.

28 If tapered insulation is to be used to improve primary and/or secondary slope, determine how it will affect the existing penetrations, perimeter conditions, parapet walls, adjoining sections, roof-mounted equipment, overall drainage, etc. This involves <u>thorough</u> coordination to ensure insulating values and clearances are maintained while improving slope and drainage.

29 When tapered insulation is used to accomplish the primary roof slope in re-roofing projects, as a minimum, compute the thermal resistance utilizing the "average thickness" method to ensure compliance with minimum required R-value per the energy code.

30 Use crickets, saddles, and edge strips (secondary slope) to direct water away from penetrations and parapet walls to ensure positive drainage to scuppers, drains, or gutters. Provide two times the primary slope to ensure resulting surfaces are sloped and drain properly. Maintain reasonably simple, "constructable" tapered insulation designs, and show the approximate design layout on roof plan. Require taper drawings to identify and coordinate all penetrations and terminations and show on new roof plan.

- **a** Crickets should have a 3:1 ratio to ensure positive drainage.
- **b** No solid curbs should exist within the valleys of the tapered crickets/saddles.

31 A minimum distance between all penetrations and terminations of 12 inches should be achieved for proper flashing.

32 A minimum height for all penetrations and terminations is 8 inches, but many owners are now requiring 10 or 12 inches. Without verification, maximum heights should not exceed 14 inches without special provisions and manufacturers input.

33 Minimize the use of penetrations to the greatest extent possible. Coordinate with the owner to remove/eliminate all abandoned equipment.

34 Recommend selecting roof types and manufacturers based on proven track record of specific system and not the warranty. The manufacturer's warranty should be a requirement for the project.

35 When considering a re-covered roof, ensure compliance with the specific requirements in *IBC* Chapter 15, Roof Assemblies and Rooftop Structures, and understand/define the responsibility and/or criteria for determining the extent of removal required. This is also required in the printed literature of the manufacturer.

36 Two types of metal roof systems are most commonly used commercially today: architectural standing-seam metal roofs and structural standing-seam metal roofs. Significant differences exist in the materials, profiles, insulation, installation, uplift testing criteria, and details of the systems. Hybrids of these two types also exist, which causes further confusion. Also, metal roof systems are often proprietary in some respect, and selecting and specifying systems takes effort and research.

37 Due to the large number of products, questionable track records for some, and the various characteristics and requirements for a specific system, research should be completed before specifying. All roofing materials are not created equally, nor do they have similar proven track records, even if they have the same acronym.

38 There are limited proper uses and life expectancies for the application of sprayed-in-place polyurethane foam (SPUF) in our region and the various coating systems often considered for existing roof systems. Once again, research should be completed before these types of system are used.

39 Vapor retarder design guidance can be found in the NRCA *Roofing and Waterproofing Manual*. When a "temporary roof" is used during a re-roofing application, and is left in place, it becomes a vapor retarder, and its effect on the roof system should be considered. Vapor retarders are not typically required in the southern US unless high interior humidity exist (e.g., paper mills, indoor pools, dry cleaners, etc.)

40 Sheet metal products incorporated into roof systems should have equivalent life expectancies (i.e., copper, stainless steel for slate and tile roofs, in wall assemblies; aluminum, galvalume, and when applicable aluminum and galvalume steel for built-up, modified bitumen, single ply and shingles). Exceptions should be justified and approved by owner.

41 Counterflashings should include prefabricated inside and outside corners, two-piece counter flashings, and hemmed edges. Metal coping and edge metals are required to meet ANSI/SPRI ES-1 per the *IBC*, as noted previously.

42 If applicable, coordinate lightning arrestor systems, photoelectric eyes, fire alarm transmitters, antennae, and other unique penetrations by showing on roof plan and providing appropriate details. Be aware of galvanic reaction problems over/on metal roofs and counterflashing. Oftentimes, these are added/completed after the roof construction and lack proper flashings/details.

43 If applicable, confirm that the existing lightning protection system does not require recertification. Also verify that the contractor properly disconnects, shifts/moves, and reconnects components on a daily basis to adhere to UL 96A.

44 Ensure appropriate sheet metal roof expansion joints are provided within the design for any deck separations, changes in deck directions where steel framing, structural steel, or decking change direction, changes in deck types, where differential movement between vertical walls and roof deck may occur or where independent structures exist (system expansion joint vs. building expansion joint).

45 Protective mats or walk pads should be strategically placed at entry/exit points, and where impact/traffic damages are expected. Please note, many manufacturers consider these "sacrificial" and are not commonly covered under the warranty and consider these items under the maintenance responsibility of the owner.

46 Use round shapes to construct equipment supports and avoid use of pitch pans, when possible. If used, include a hood/umbrella over the pitch pan. When existing pitch pans cannot be eliminated and preferred flashing methods utilized, ensure that the pitch pan is a performed pan with a minimum four-inch height and four-inch flange, with a minimum two-inch clearance on all sides of the penetration. The bottom one-third is to be filled with non-shrink grout with the remainder filled with pourable sealer, sloped to drain, and has a metal umbrella cap, when possible.

- **a** Be careful with liquid-applied flashing systems.

47 Avoid embedded metal details when possible. A common situation is the gravel stop (edge metal) flanges and stacks/curbs.

- **a** For gravel stops (edge metal), which allow drainage to the perimeter (gutter), use light gauge metals such as 16-ounce copper, 24-gauge galvalume steel or .04-inch aluminum. Nail with two rows, staggered at four inches on center maximum between each row of fasteners. Please note actual thicknesses vary between weights, dimensions, and gages.

48 Equipment supports should be a minimum of 14 inches but not less than shown in the tables below. Note that these minimums apply at the end of the equipment support on the high side of the slope. Round pipes are preferred for flashing performance/ drainage.

Width of Equipment	Height of Legs (round pipe preferred)
Up to 25″	14″
25″ to 37″	18″
37″ to 49″	24″
49″ to 61″	30″
61″ and wider	48″

49 Locate interior drains at mid-spans and low points of the roof deck. Do not locate drains at columns. When the flashing drains, taper the insulation from 24-inches out to around the drain. Extend the membrane, four-pound lead flashing, and strip flashing under the drain bowl clamping ring. Do not use exposed-lead sump pan details. Use bitumen/gravel stops around drains, if applicable.

50 Avoid interior or built-in gutters and built-in downspouts whenever possible. When unavoidable, plan and initiate a regular inspection and maintenance plan.

51 When a base bid and an alternate roof system are included in a project, include the most likely selected system as the base bid and provide adequate design, specification, and detailing for all systems for which a bid is to be provided, as the requirements may be significantly different (e.g., metal versus shingle systems).

52 If milestone or full-time quality assurance services are planned for a project, early involvement by the consulting firm can provide numerous benefits. Recommend having this firm conduct (or attend) the pre-roofing conference as the highest priority, and then have them perform milestone inspections during the early stages of the project, if the visits are to be limited. (Involvement during the design process is also encouraged.)

53 Verify with the owner whether a provision for fall protection or prevention (e.g., guard rail, banister, etc.) is to be required at locations of the roof where equipment is located with 10-feet or less of a roof edge condition where a parapet wall 42 inches or greater does not exist and per new 2018 OSHA requirements.

54 Take note of the 2018 International Fire Code, Section 316.4, Obstruction on Roofs

> Wires, cables, ropes, antennas, or other suspended obstructions installed on the roof of a building having a roof slope of less than 30 degrees (0.52 rad) shall not create an obstruction that is less than 7 feet (2133 mm) high above the surface of the roof.
>
> **Exceptions:**
>
> **1)** Such obstruction shall be permitted where the wire, cable, rope, antenna or suspended obstruction is encased in a white, 2-inch (51 mm) minimum diameter plastic pipe or an approved equivalent.
>
> **2)** Such obstruction shall be permitted where there is a solid obstruction below such that accidentally walking into the wire, cable, rope, antenna or suspended obstruction is not possible.

Hopefully, you have found these considerations helpful. There might be disagreement on some of them, and there is likely room for improvement; however, they have been beneficial in our business operations, and they may be worth your attention.

OH HAIL! METAL ROOFS, HAIL IMPACT, AND LONG-TERM PERFORMANCE

ABSTRACT

For several years, the insurance industry has been addressing an important question – what is considered cosmetic damage vs. functional damage when assessing hail impact on low-slope metal roofing? Cosmetic damage due to hail is often excluded from coverage. However, the distinction between the two types of damage has been challenged by forensic experts who base their opinions on unsubstantiated claims of reduced service life of a dimpled roof due to moisture retention and microfracture within a hail divot.

The Metal Building Manufacturers Association (MBMA) recently completed two research projects on the effects of hail on 55% aluminum-zinc (55%Al-Zn) alloy-coated steel roofing systems, which is the most common metal roofing material used in the US marketplace and typically sold under the trade name of GALVALUME®. These projects, which evaluated the coating damage due to rollforming of 55%Al-Zn alloy-coated steel roof panels and the water ponding drying rates for simulated hail impact divots in 55%Al-Zn alloy-coated steel roof panels, provide much-needed information to define the differentiators between "cosmetic" and "functional" damage

LEARNING OBJECTIVES

- » Identify the two common concerns regarding the long-term performance of hail-impacted 55% Al-Zn-coated steel roof panels.
- » Explain the empirical and scientific corrosion behavior of 55% Al-Zn coatings on steel roof panels.
- » Discuss the approach to apply results of drying rate and coating fracture studies to categorize hail impact damage as cosmetic or functional with dimensional delineations.
- » Define appropriate evaluation criteria for 55% Al-Zn coating damage due to hail impacts.
- » Defend your assessment of hail-damaged, 55% Al-Zn-coated steel roof panels

SPEAKERS

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Ron Dutton is the president of Ron Dutton Consulting Services. Dutton provides technical services for product development, failure analysis, and product training for the metal construction and appliance industries. With 40 years of experience in metallic-coated steel products and a strong emphasis on 55% Al-Zn alloy-coated sheets, Dutton has served in various organizations, such as the National Coil Coating Association (NCCA) Building Products and Residential Roofing Task Forces. He chaired the North American Zinc Aluminum Coaters Pre-Painted Building Inspection Committee and chaired the board for the Zinc Aluminum Coaters Association. Dutton holds BS and MS degrees in metallurgical engineering and materials science from Drexel University and Lehigh University.

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Robert Haddock is a metal roofing expert who has worked in the industry for five decades—first as a laborer, then contractor, forensic analyst, technical author, innovator, and founder of S-5!. He is a member of NRCA, ASHRAE, the American Society of Civil Engineers, Construction Specifiers Institute, and ASTM. He is also a lifetime honorary member of Metal Building Contractors and Erectors Association, and Metal Construction Association. Haddock innovated the concept of seam clamps for standing seam roof profiles. He has served as faculty for Roofing Industry Educational Institute, IIBEC, and the University of Wisconsin. He is a recipient of numerous awards including the IIBEC Richard M. Horowitz Award and was a charter inductee to the Metal Construction Hall of Fame.

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INTRODUCTION AND BACKGROUND

Prior to 2008, there were property losses to insurers arising from hail claims of US$8 to US$12 billion annually (adjusted for inflation). Since then, hail claims and the associated dollar losses have occurred at a rate of US$19 billion or above annually.[1] Several theories have been forwarded to explain this sharp increase: 1) global warming and weather pattern changes; 2) organized efforts of storm-chasing contractors and supporting opportunists; 3) escalation in costs of labor and material; 4) more lenient insurance adjusters; or 5) a possible increase in efforts by zealous plaintiffs' attorneys, "consumer advocates," public adjustors, and "forensic experts" with microscopy labs.

Hail severity and the geography thereof continued to increase from 2008. In late 2014, FM Global established a "Very Severe Hail" zone (hail 2 in. or larger) encompassing all of Kansas, Oklahoma, and North Texas. In 2018, FM Global redefined that zone, expanding east almost to the Mississippi River, west to the Continental Divide, south to the Gulf, and north almost to the Canadian border.[2] At that time, it encompassed all or part of 14 states. Circa 2019, the Insurance Institute for Business & Home Safety mapped "Hail Prone Counties" (hail 1 in. or larger) by county of a much larger geography extending east including Alabama, the Carolinas, and even parts of Florida and encompassing a total of 36 states—all but four east of the Continental Divide.

Whatever the debatable cause(s), insurers – like all businesses – segregate their income and expense and hope to drop some profit to the bottom line. When it came to hail claims, they were bleeding with loss and had to stop the hemorrhaging.[1] In 2013, the insurance industry began to raise premiums while reducing coverages on metal roofs, raising deductibles, and including the exclusion of "cosmetic damage" within policy language. Over the next several years, homeowner claims began to drop, but commercial losses continued to increase.

Metal roofs are expensive to replace. One proposed claim adjustment the authors investigated calculated at US$16.05/sq ft, based on removal and replacement of the metal roof and underlying insulation on a 40,000 square foot roof. So, a perfect storm brewed. Contractors were chasing this lucrative financial "opportunity" and utilizing labs and metallurgists to support their efforts with attempts to prove that even the slightest hail strike hits were "functional damage."

The result is inevitable: premiums for metal roofs rise, deductibles rise, exclusions are employed, thereby penalizing the most durable roof. Thus, the metal roofing industry found themselves in a severely compromised position, while some building owners and roofing contractors capitalized by replacing roofs that did not need replacement for any functional reason. Something had to be done.

In late 2017, the Metal Building Manufacturers Association (MBMA) embarked on a new research project to factually dissuade the two primary claims postulated by plaintiffs' experts and legal teams in their attempts to claim "functional" damage to low-slope metal roofs that had experienced a hailstorm.

The first of those claims was that coating microfractures observed under 50 to 1000x magnification in a hail divot constituted "functional" damage to the protective coating of steel sheet, abbreviating its life expectancy, therefore justifying roof replacement. The second claim was that hail divots retained water which would lead to premature corrosion at hail-strike locations thus justifying roof replacement. The reports of these two studies were published by MBMA to their membership in 2021. This paper summarizes the results of these studies and provides quantitative means to determine whether hail-strike divots on 55%Al-Zn alloy-coated steel roofing systems constitute functional damage.

I. COATING MICROFRACTURES AND SERVICE LIFE

Introduction and Background

Many metal roofing manufacturers offer coated steel roof products that achieve the highest hail-resistance (UL 2218 Class 4) designation[3], a determination based on functional damage observed during the test. The distinction between such functional damage and cosmetic damage has been somewhat elusive and, in many cases, determined by the "eye of the beholder." But such a distinction is critical in that a correct assessment of the degree of hail damage to a metal roof is essential for "doing the right thing" for the building owner, the insurance company and the roofing manufacturer.

As a first step in addressing this issue, the current study was undertaken to establish a clear, baseline assessment of damage incurred by bare (unpainted) 55%Al-Zn alloy-coated sheet steel due

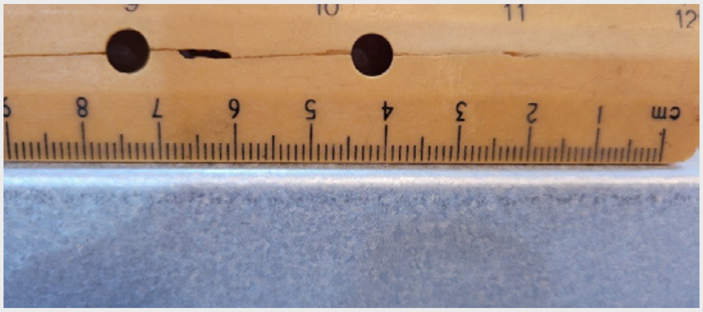

FIGURE 1. SSR panel shoulder radius showing no discernable coating crazing.

FIGURE 2. 55%Al-Zn alloy-coated steel SSR system erected in Denver, Colorado in 1977.

to normal rollforming processes used to fabricate roof panels. Using this baseline as a reference standard, damage incurred due to hail stone impacts can then be accurately characterized as either functional or cosmetic.

55%Al-Zn alloy-coated steel SSR panels were chosen due to their widespread marketplace acceptance as a premier roofing material, primarily for low-slope applications, with excellent long-term service life.[4-11] The product was invented by the Bethlehem Steel Corporation and has been commercially available in the United States since 1972. Licensing has made the product available from steel companies worldwide with more than 200 million tons produced. Building systems and component manufacturers use rollforming to produce these panels, either in the plant or at the job site.

Some roof panels rollformed in the 1970s were found to exhibit minor coating crazing along some profile rib radii. While this crazing was found to only result in cosmetic tension bend staining (TBS), it also led to improvements in rollforming practices by tooling manufacturers, as well as adjustments in 55%Al-Zn alloy-coated steel production practices to optimize coating microstructure, essentially eliminating such coating crazing in currently manufactured roofing panels. Such was the case when virtually no crazing was observed on freshly rollformed panels initially obtained for this project from two manufacturers, as shown in **Figure 1**.

Since freshly rollformed panels did not provide the degree of coating crazing deemed suitable for this study, attention was turned toward the use of existing roof panels. Numerous buildings in the United States have original 55%Al-Zn alloy-coated steel roofs which have been monitored since their construction in the 1970s. These roofs are inspected about every five years by engineering professionals, and roof performance is reported regularly to the licensed 55%Al-Zn alloy-coated steel producers which make up the Zinc Aluminum Coaters Association. About 40 of these roofs are currently available for these inspections, some being more than 45 years old. Thirteen of these roofs, ranging in age from 41 to 47 years, exhibit coating crazing and some degree of TBS on the shoulder radii of the panel profile. As such, they represented an ideal dataset from which to select samples for metallographic analysis to determine if corrosion resistance of the coating had been compromised. A review of these older roofs, located in various parts of the United States, identified a 43-year-old roof in Denver, Colorado, as being representative of TBS and available to provide samples for analysis.

Procedure

The following procedures were followed to obtain and analyze samples for

FIGURE 3. The 55%Al-Zn alloy-coated steel SSR with sample locations.

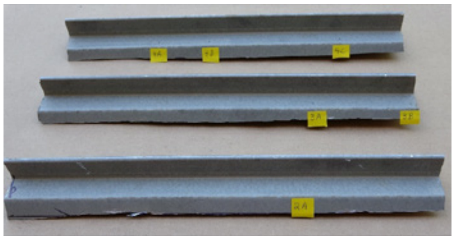

FIGURE 4. Three of the four panel profile samples.

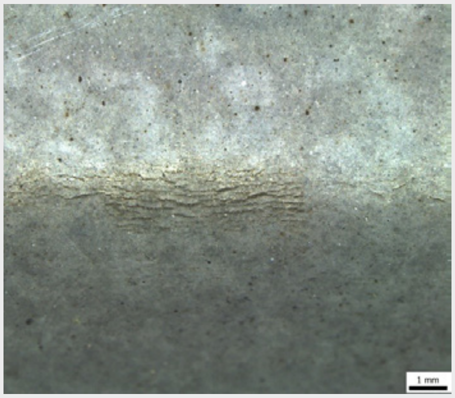

FIGURE 5. Stereoscope image of tension bend specimen 4C.

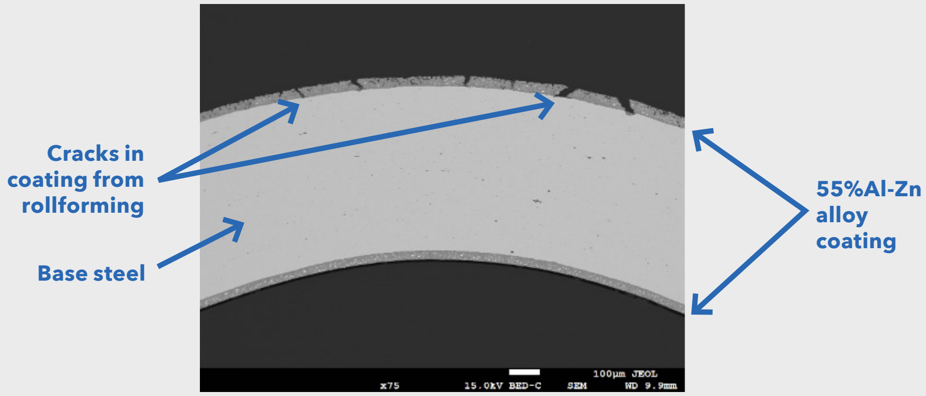

FIGURE 6. FESEM backscattered electron cross-section image of specimen.

evaluating coating damage done by rollforming of 55%Al-Zn alloy-coated steel roof panels.

Sample Procurement

The building selected for this study is in a commercial/industrial area north of downtown Denver, as shown in **Figure 2**. The building is used as an insulated, air-conditioned welding shop and repair facility. It was erected in 1977 and features a double lock seam standing seam roof (SSR) system. It is approximately 80 ft wide and 150 ft in length with a roof slope of ½:12. A wide-angle view of the roof, looking to the north, is seen in **Figure 3**.

Also shown in **Figure 3** are the locations of samples obtained for metallographic analysis from four locations on the east-facing slope of the roof. The samples, approximately 12 in. in length, were obtained by sawing through the seam and shoulder radius on each side of the seam and subsequently applying a weathertight repair patch (the bright patches noted in **Figure 3**).

Three of the four sample areas which were obtained are shown in **Figure 4**. The yellow tags identify specific areas of interest where coating crazing is noticeable, and which represent the areas subsequently used for metallographic analysis.

Metallographic Analysis

To characterize the tension bend samples, cross-section specimens were taken of locations 3A, 3B, 4A, 4B, and 4C, and analyzed via optical and field emission scanning electron microscopy (FESEM) with energy dispersive spectrometry (EDS). Backscatter electron (BED-C) imaging mode was used in the FESEM to visually illustrate differences in the elemental composition as compared to the steel matrix. The features observed during the initial analysis led to additional FESEM/EDS analysis where EDS elemental maps were generated for the selected areas. EDS elemental mapping allows identification of regions rich in specific elements by their intensity level that are then overlaid onto the back-scattered electron image.

Results and Discussion

The results of our study are discussed below based on macro- and micro-analysis of the procured specimens from a 55%Al-Zn alloy-coated SSR in service for 43 years in Denver, Colorado.

Preliminary Observations

Five tension bend specimens were selected for analysis. Each of these specimens exhibited some degree of crazing and a light stain, both visible with the naked eye. A stereoscope image of specimen 4C, which is representative of the five specimens selected for this analysis, is shown in **Figure 5**.

Field Emission Scanning Electron Microscopy

A representative FESEM cross-sectional photograph of these specimens at 75x is shown in **Figure 6**. The photograph shows the base steel in white, with the 55%Al-Zn alloy coating present on the top and bottom surface. 3A at 75x showing tension bend coating cracks due to rollforming.

The top, tension side of the coating exhibits cracks of varying sizes, some of which penetrate through the coating, exposing portions of the base steel. As expected, the bottom, compression side of the coating exhibits no cracks.

Specimen 4B was selected to illustrate the analysis performed for all five specimens from the roof. Specimen 4B also has the largest observed area of exposed base steel associated with a crack. Crack size and base steel exposure are discussed further in a following section of this paper.

The cross-section image of specimen 4B at 250x is shown in **Figure 7**. The morphology of the 55%Al-Zn alloy coating is just beginning to be discernable at this magnification. The largest crack area found on these specimens is seen in the upper left portion of the photograph. This area is explored in more detail in subsequent figures.

Energy Dispersive Spectrometry

In **Figure 8**, the coating structure becomes very clear at 700x magnification. The EDS elemental x-ray map overlaid onto this photograph is shown in **Figure 9** and gives insight to the unique 55%Al-Zn alloy coating microstructure and the corrosion mechanism by which it provides excellent, long-term corrosion resistance.

The microstructure consists of a corrosion-resistant overlay and an intermetallic alloy layer that bonds the

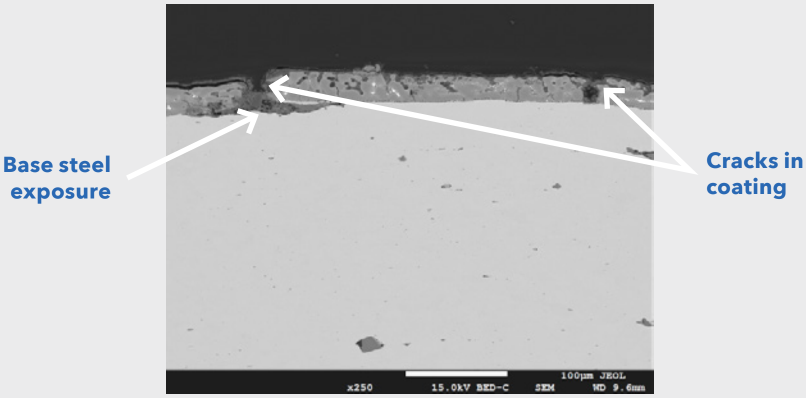

FIGURE 7. FESEM backscattered electron cross-section image of specimen 4B at 250x, showing coating cracks and a degree of base steel exposure.

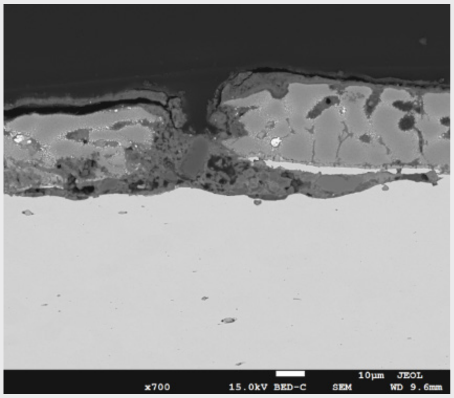

FIGURE 8. FESEM backscattered electron cross-section image of specimen 4B at 700x.

coating to the base steel. The overlay contains aluminum-rich dendrites and zinc-rich interdendritic areas interspersed with occasional acicular areas of silicon. By volume, the overlay contains about 80% aluminum.

This unique coating microstructure provides several benefits. First, during the early stages of atmospheric exposure, the zinc-rich interdendritic network is preferentially attacked. This circuitous network eventually fills up with corrosion products that become mechanically lodged in the coating and impede further penetration of the elements into the coating. Thus, excellent, long-term corrosion resistance is provided. [8, 11-14]

Second, the presence of a fine, circuitous interdendritic network helps to relieve strain in the coating during rollforming such that crack propagation through the coating is impeded, providing good coating ductility.[13]

Several features of the crack area can also be seen in **Figure 9**. The crack runs through the coating, exposing the base steel. The resulting iron oxide corrosion products are mostly adhered to the base steel, although some extend to the coating surface and are responsible for the light cosmetic stain observable to the naked eye in **Figure 5**. Some calcium can also be seen at the base of the crack and may be associated with the presence of airborne fly ash which is frequently used in structural fills or embankments on nearby construction sites.[14]

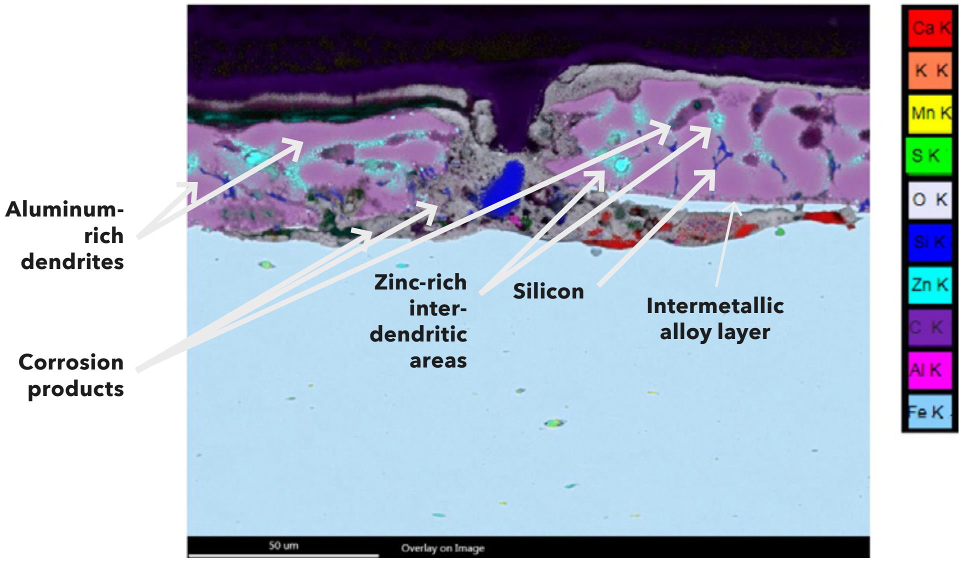

FIGURE 9. EDS elemental x-ray map overlaid onto Figure 8, showing key components of the 55%Al-Zn alloy coating and corrosion products after 43 years of atmospheric exposure on a roof in Denver, Colorado.

Crack Size

It is worth reiterating that not every coating crack was found to extend to the surface of the base steel and that the size of the one in **Figure 9** is a clear outlier in terms of crack sizes observed in these specimens. This can be seen in the data shown in **Table 1** where surface crack measurements and base steel exposure measurements are tabulated.

The presence of base steel exposure due to a crack has frequently become a critical argument by some who would say that this represents functional damage to the roof, and therefore, roof service life is compromised. The data in Table 1 show that for all but specimen 4B, in which a base steel exposure of 0.006 in (0.148 mm) represents a data outlier, the average base steel exposed due to a surface crack is either about the same size as the average surface crack or significantly smaller. So, while a minor degree of metallic coating crazing may occur and may occasionally penetrate through the coating to expose the base steel, on a 43-year-old roof in Denver, no detrimental corrosion has occurred.

To put the scale of these measurements in perspective, the data in Table 1 are plotted in **Figure 10**, along with a reference data point associated with a functional damage claim due to a hailstorm containing 1-to-2-in. diameter hail stones.[15] Note that the base steel exposures associated with fifty-two 55%Al-Zn alloy coating cracks on the Denver roof range from zero to 0.003 in. (0.071 mm) with one outlier at 0.006 in. (0.148 mm). The base steel exposure for

TABLE 1. Tension bend coating crack and base steel exposure data.

Specimen	Number of Cracks, n	Average Coating Surface Crack, mm	Range of Coating Surface Cracks, mm	Average Base Steel Exposed, mm	Range of Base Steel Exposed, mm
3A	7	0.026	0.013-0.047	0.027	0.0-0.063
3B	10	0.022	0.007-0.050	0.022	0.007-0.071
4A	14	0.023	0.007-0.047	0.010	0.0-0.033
4B	6	0.014	0.007-0.024	0.027*	0.0-0.148*
4C	15	0.018	0.006-0.038	0.004	0.0-0.031

* One base steel exposure of 0.148 mm also inflates the average for base steel exposure.

the claim is seen to be smaller by almost a factor of 50 compared to the outlier.

One final point of comparison can be made based on a study conducted by one of the licensed North American producers of 55%Al-Zn alloy-coated sheet. In that study, uncoated (bare) spots from early production runs in the 1980s were exposed in marine, industrial and rural atmospheres. Selected materials from non-standard operating conditions such as line start-ups were also exposed in an industrial atmosphere.

The results of this study showed that isolated bare spots up to 0.079 in. (2 mm), which is much larger than the base steel exposures on the Denver roof, showed no adverse effect on corrosion resistance after nine years of exposure in marine, industrial and rural environments. For clustered bare spots up to 0.020 in. (0.5 mm), the 55%Al-Zn alloy coating provided protection from corrosion for nine years in an industrial environment.[16]

Roof System Durability

Even when the Denver roof outlier of 0.006 in. (0.148 mm) is considered, the resulting size of base steel exposure is 3 to 13 times smaller than the range of bare spots in the licensee study that performed well after nine years. When the "functional" damage claim is considered, the size of that base steel exposure of 0.00008 to 0.00012 in. (0.002 to 0.003 mm) is about 50 times smaller than the outlier on the Denver roof which has performed well with no weathertightness issues for 43 years.

The sheared edges of these roof panels provide a more dramatic comparison: the base steel exposure on this typical 24-gage panel is approximately 0.022 in. (0.56 mm). Thus the "functional" damage claim base steel exposure is about 200 times smaller than these exposed base steel panel edges which have performed excellently for 43 years. Thus, a hail damage claim for functional roof damage due to cracks in the 55%Al-Zn alloy coating or base steel exposures of such minute dimensions is not supported by the results of this study.

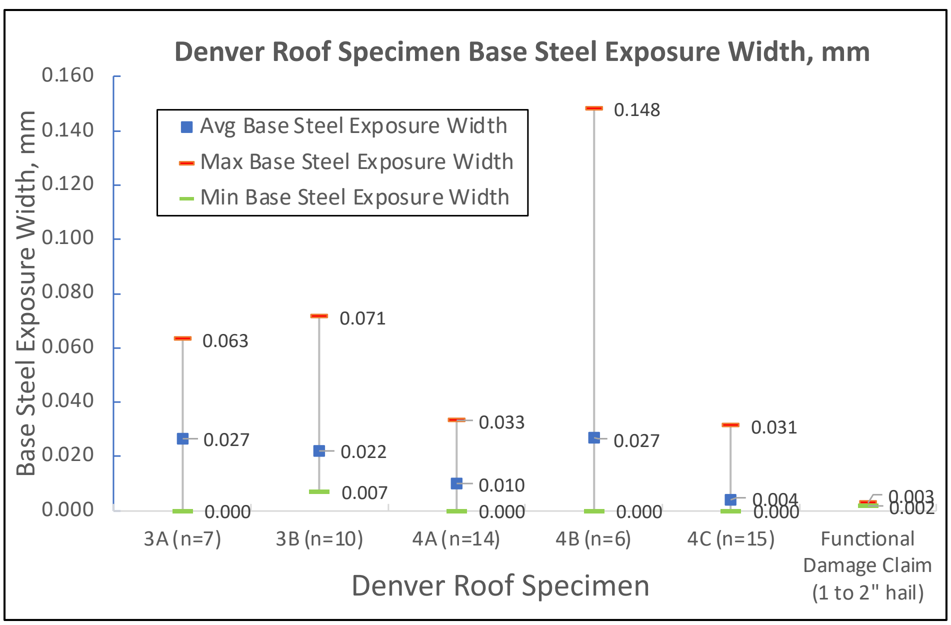

FIGURE 10. Range of base steel exposure widths associated with rollformed tension bend coating cracks compared to that of a functional damage claim due to hail.

Summary

A metallographic study was conducted to define a baseline for "damage" to 55%Al-Zn alloy-coated steel coatings which may occur during the normal forming processes used to manufacture a typical standing seam roof (SSR) panel. Results show that a minor degree of metallic coating crazing may occur and may occasionally penetrate through the coating to expose the base steel, but that on a 43-year-old roof in Denver, no detrimental corrosion has occurred. This conclusion is consistent with the unique and well-documented corrosion resistance mechanism which is characteristic of these coatings.

This minor degree of coating crazing is much smaller than 55%Al-Zn alloy-coated steel bare spots of up to 0.079 in (2 mm) in diameter which, upon exposure in an atmospheric corrosion study, were unnoticeable after 9 years.

This minor degree of coating crazing is also much larger than coating damage associated with a 55%Al-Zn alloy-coated steel roof which experienced hailstorms with reported hail diameters of 1 to 2 inches and which was claimed to result in "functional damage."

It is concluded that since these minor coating crazes associated with the rollforming of 55%Al-Zn alloy-coated

FIGURE 11. Representative example of panel flutes being last to dry after rain or morning dew4B at 250x, showing coating cracks and a degree of base steel exposure.

FIGURE 12. Representative panel eave location being last to dry after rain or morning dew.

steel SSR panels have proven to perform well in outdoor service for over 40 years, then hail divots, which might cause much smaller degrees of coating crazing, would not be expected to degrade the normal service life of a 55%Al-Zn alloy-coated steel roof.

II. WATER PONDING FROM HAIL DIVOTS AND SERVICE LIFE

Introduction and Background

Hail impacts on 55%Al-Zn alloy-coated steel roofs can impart divots which will catch rainwater or condensation (dew) for a time until it evaporates. This study was undertaken to determine whether the hail divots retain water long enough for functional damage to occur to the roof's service life due to accelerated corrosion in the "ponded" divots. This is clearly important information because the industry-accepted service life of a 55%Al-Zn alloy-coated steel SSR system is 60 years,[4] a determination supported by peer-reviewed work that evaluated corrosion on roof systems in place for up to 35 years[5-8] and by a general field observation study of roofs in service for up to 43 years.[17] Thus, any deterioration of such a service life would have significant financial implications.

This study measured the time of wetness associated with simulated hail divots compared to the time of wetness associated with 1) normal mechanical deformations imparted during the manufacturing process to provide panel rigidity and 2) panel eaves, both of which are known to be "last to dry" after rain occurs, as seen in **Figures 11** and **12**.

Procedure

The following procedures were followed to simulate hail strike divots of varying sizes on a 55%Al-Zn alloy-coated steel SSR panel and measure the resulting times of wetness associated with them.

Materials

A 55%Al-Zn alloy-coated steel SSR panel manufactured by Butler Manufacturing, designated MR-24, was selected for this study. The panel had been exposed at the Butler Research facility in Grandview, Missouri, for about 22 years at an approximate slope of 1 degree (¼:12) facing south.

Hail Divot Simulation

An apparatus normally used to conduct impact testing, shown in **Figure 13**, was used to produce simulated hail divots in the panel. A 0.625-in. (15.9 mm) diameter steel indenter weighing 4lb was dropped from various heights onto the panel to produce simulated hail divots that reflected impact energies of 1, 4, 8, and 13.3 ft-lbs (the maximum impact energy based on the height limitation of the apparatus). For each of the four impact energies, ten divots were produced in the panel, as shown in **Figure 14**.

Divot size was measured for depth with a digital depth gage and for width with a scale. The depth of divots varied as a function of impact energy and ranged from approximately 0.035 to 0.159 in. (0.89 to 4.04 mm). Based on work conducted by the National Bureau of Standards,[18] these impact energies correlate with the energies associated with hail stone diameters up to about 1¾ in. (44.45 mm) striking a surface at terminal velocity, as

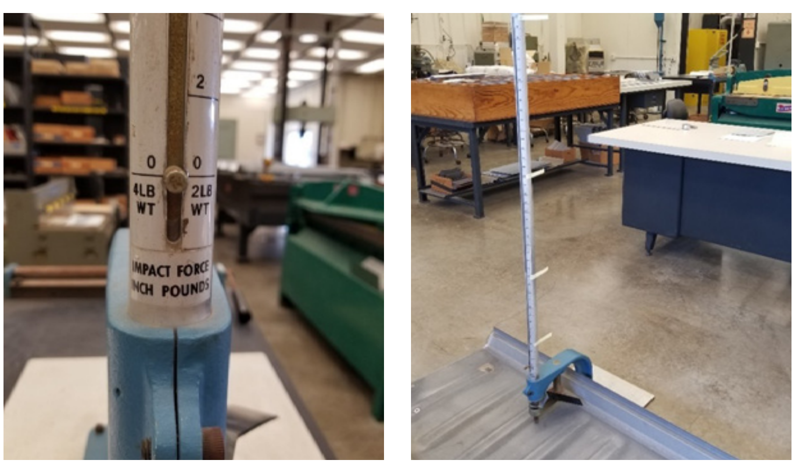

FIGURE 13. Impact test apparatus used to produce simulated hail divots in a Butler MR-24 panel.

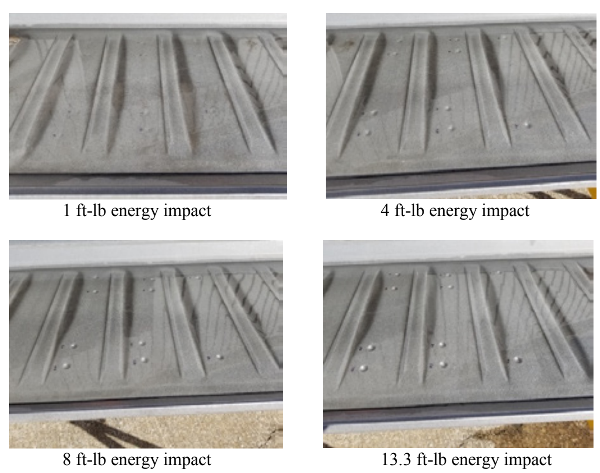

FIGURE 14. Simulated hail divots produced with four different impact energies.

TABLE 1. Impact energies associated with hail Stone diameters

Hail Diameter, in.	Impact Energy, ft-lb	Simulated Divot Depth, in.
1/2	0.1	
3/4	0.4	
1	1	0.035-0.042 (0.89-1.07 mm)
1-1/4	4	0.083-0.091 (2.11-2.31 mm)
1-1/2	8	0.118-0.125 (3.00-3.18 mm)
1-3/4	14	0.143-0.159 (3.63-4.04 mm)
2	22	
2-1/2	53	
2-3/4	81	
3	120	

TABLE 2. Comparison of industry standard classification for hail-resistant roofing products

Rating Classification	Impact Energy, ft-lb	
Class	UL 2218[3]	ANSI/FM 4473[24]
1	3.4	3.7
2	7.2	7.8
3	13.4	14
4	23	23.8

TABLE 3. Climatic and test conditions for water-drying runs

Variable	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7	Run 8	Run 9	Run 10
Air Temp, oF	73	74	74	74	74	74	74	74	74	74
Humidity, %	41	33	31	37	24	32	26	27	28	40
Panel Temp, °F	72.6	72.3	74	72.8	71.2	73	72.6	72.4	72.4	72.2
Water Temp, °F	68.4	69.2	67.1	68.2	65	64.4	63.8	66.8	66.2	69.8

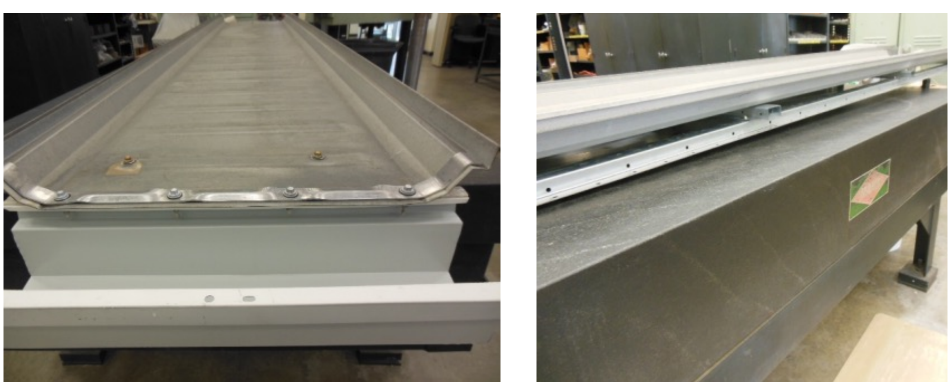

FIGURE 15. Laboratory set-up of panel to conduct water drying experiments

shown in **Table 1**. Hail stone diameters up to about 1¾ in. (44.45 mm) have been documented as representing about 75 to 95% of hail stone diameters associated with hailstorms in the United States. and Canada.[19-23]

Both UL 2218 and FM 4473 have Class 1 through 4 rating classifications for hail-resistant roofing products. In **Table 2**, these standard requirements for hail-resistant roofing products are shown. Thus, the scope of the current study essentially covers threshold damage resistance typically associated with roof systems that have obtained ratings of Class 1 through Class 3.

Panel Set-Up

The test panel was initially positioned outside but subsequently moved indoors to obtain more reproducible climate conditions for ten "runs." As shown in **Figure 15**, the panel was installed on a frame to maintain panel position and permit an easy means to adjust slope. The panel eave area included an aluminum cinch strap as is standard on this roofing panel for fixing the panel at the eave while also allowing proper water drainage through its three 3 individual channels. Panel slope was fixed at 1 degree, approximating a ¼:12 slope.

Testing Protocol

For each test run, the following variables were recorded:

- Air temperature
- Humidity
- Panel temperature
- Water temperature

Approximately 1400 ml of water were dispersed along the panel such that the surface was entirely wetted. Observers monitored the panel and recorded the times required for water to evaporate from each of the 40 divots as well as from the flutes and the eave.

Climatic conditions in the laboratory were maintained as constant as possible. The largest variation in climate was the relative humidity over which the laboratory had limited control. Test conditions of water and panel temperature were controlled to minimize their impact upon the resulting times for the divots to dry. **Table 3** shows the climatic and test conditions for the 10 runs.

FIGURE 16. Non-uniform water distribution on panel as it dries (L) and subsequent effect on volume of water flowing into divots on the opposing sides of the panel (R).

Results and Discussion

The results of this study are discussed below.

Preliminary Observations

Initial runs undertaken to develop the precise testing protocol revealed that very slight panel flatness deviations would impact the time for complete drying to occur on the left- vs. right-side of the panel. An example from an early outdoor run is shown in **Figure 16**.

This differential between left- and right-side drying times continued to be evident even when testing was moved indoors to provide more uniform climate conditions and to enable a more secure attachment of the panel to a frame. Our observations showed that even slight deviations from total panel flatness unevenly disrupted the water flow such that one side of the panel had more of a water "reservoir" feeding into the divots on that side, thus extending the times for total water evaporation to occur. Accordingly, left-side divots and right-side divots were treated as separate populations for statistical analyses.

The summary statistical data are shown in Table 4. In general, there was good correlation (coefficient of determination, R^2) between divot drying time and divot depth when all 40 divots were considered. The correlation improved when the left- and right-side divots were considered separately. Drying time correlated even better with divot volume with R^2 values above 0.90 for left- and right-side divot populations.

Panel Flute Drying

Although humidity did vary the most during the runs, its impact on the time for all flutes to dry was "well-behaved" from a statistical point of view as seen by the strong correlation in **Figure 17** between higher humidity levels and longer times for complete drying of the flutes to occur. This result is consistent with the fact that the atmosphere was more saturated with moisture at elevated humidity levels than at lower humidity levels, thereby hindering water evaporation from the panel into the atmosphere. Drying times for individual divots followed the same pattern as the flutes, in that higher humidity levels required more time for total evaporation to occur from each divot.

Effect of Divot Depth

Divot depth ranged from 0.035 to 0.159 in. (0.89 to 4.04 mm). The time required for the divots to dry was strongly correlated with divot depth as shown in **Figure 18**. The data in this and following charts are taken from Run 6 which was found to be most representative of all ten runs in terms of the coefficient of determination, R^2. That is, the value of R^2 for Run 6 was closest to the average value of R^2 for all ten runs.

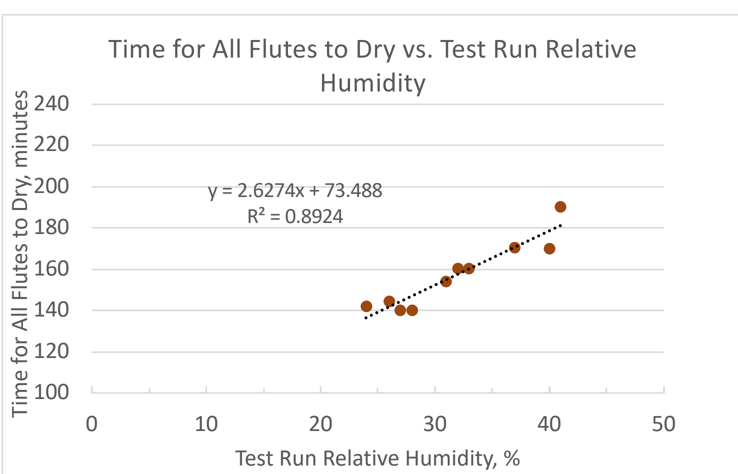

FIGURE 17. Effect of test run relative humidity on the time for all flutes to dry.

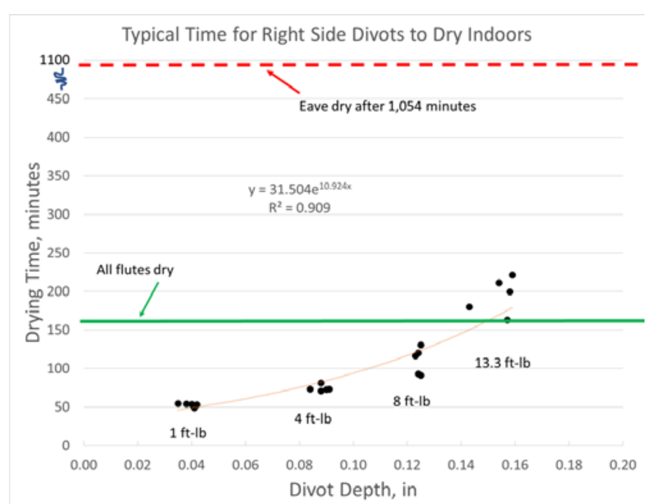

FIGURE 18. Effect of divot depth on drying time of the right panel side divots.

TABLE 4. Coefficients of determination (R^2) Between divot depth and divot volume and time for divots to dry

Statistical Treatment		Panel Location	Number of Divots	Avg R^2, Ten Runs
Divot **Depth** vs. Drying Time		Full Width	40	0.7327
Divot **Depth** vs. Drying Time		Left Side	20	0.8392
Divot **Depth** vs. Drying Time		Right Side	20	0.8980
Divot **Volume** vs. Drying Time		Full Width	40	0.7775
Divot **Volume** vs. Drying Time		Left Side	20	0.9101
Divot **Volume** vs. Drying Time		Right Side	20	0.9445

As described earlier, the right- and left-side of panel divots represent two different statistical populations due to the difference in water volume flowing into the divots because of slight panel flatness deviations. Right-side divot results are presented in this report to maintain a constant dataset for comparison. Left-side divot results were very similar, however, as shown in **Table 4**.

Figure 18 shows the relationship between drying time and the depth of the divot. Also shown for comparison are the drying times for all the panel flutes and for the panel eave. Divots with depths of about 0.150 in. (3.81 mm) or less consistently dried before all the flutes dried. Divots above about 0.150 in. (3.81 mm) dried a short time after all the flutes dried, but well before the panel eave dried. In fact, the eave typically took more than three times longer to dry than did the largest group of divots in this study.

Effect of Divot Volume

Again, utilizing the data from Run 6, the theoretical volume of water held by each divot was calculated from depth and width measurements. These values were then used to plot **Figure 19**, which shows the relationship between drying time and the divot volume. Again, drying times for all the panel flutes and for the panel eave are shown for comparison. The observations described on the previous page for **Figure 18** are similarly confirmed by the data plotted in **Figure 19**.

Correlation with Field Data

Divots on actual hail-impacted roofs differ somewhat in size and shape from the lab divots at the higher impact energy levels in this study. This difference is due to the inherent limitations of the lab impact apparatus which used a constant indenter diameter of 0.625 in. (15.88 mm) to produce the lab divots vs. the larger sizes of hail that produce larger divot widths when they impact roofs during hailstorms. However, enough data has been generated in the current study to demonstrate the applicability of the findings to actual hail impacts on roofs.

In **Figure 20**, hail divot width is plotted against hail divot depth. Data from the field show that divot width continues

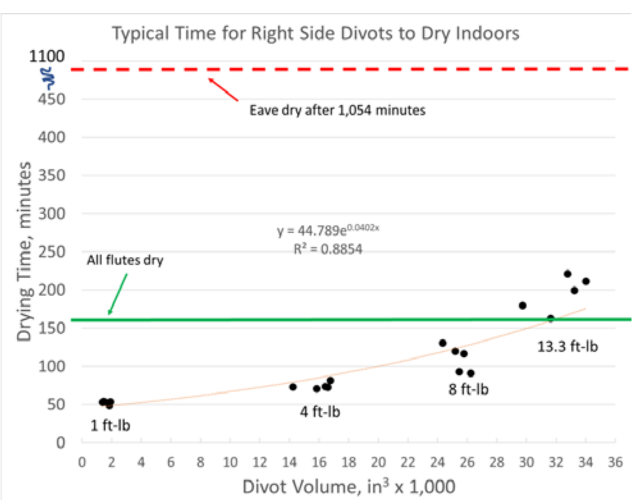

FIGURE 19. Effect of divot volume on drying time of the right panel side divots.

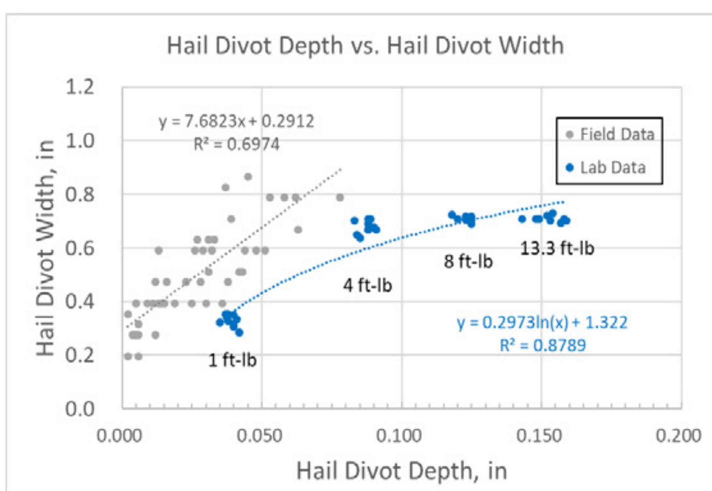

FIGURE 20. Divot depth vs. divot width for lab and field data.

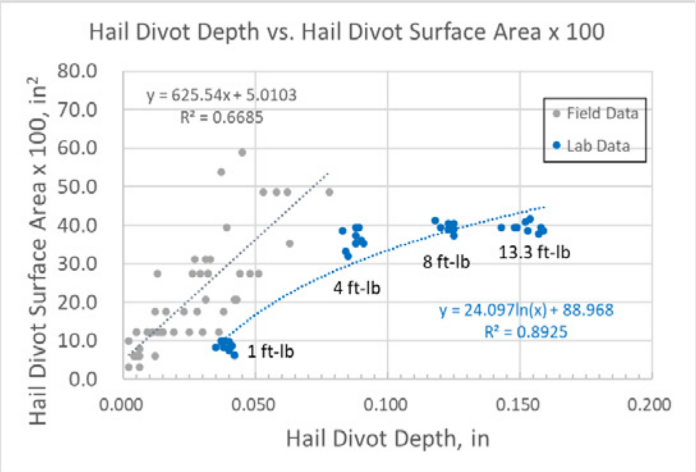

FIGURE 21. Divot depth vs. divot surface area for lab and field data.

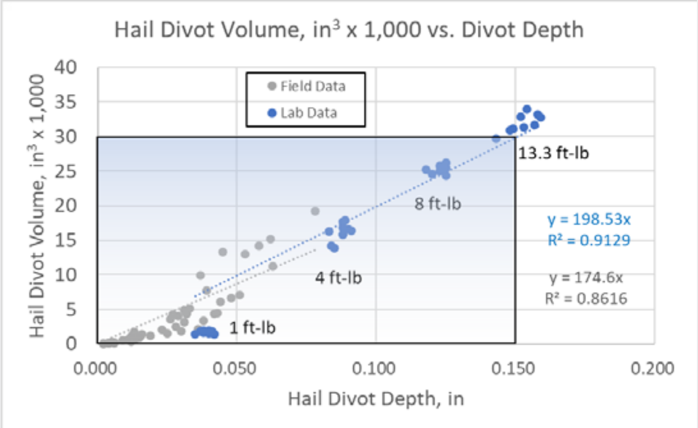

FIGURE 22. Divot depth vs. divot volume for lab and field data.

to increase as divot depth increases, the result of larger hail stones striking roofs with higher energy impacts which produce deeper divots. By contrast, lab data show divot width increases until about 0.070-in. (1.78 mm) depth, beyond which it plateaus.

Using width and depth measurements, two other variables can be calculated. One is the surface area associated with the width of the divot, assuming a circular divot. This is an important variable to consider since larger surface areas allow for more evaporation to occur, thereby shortening drying times for a given volume of water. **Figure 21** shows divot surface area plotted against divot depth for lab and field data. Comparing the two datasets, the larger surface areas of field divots would indeed result in higher rates of evaporation. However, the total time for complete drying to occur in any divot will also depend on the volume of water in the divot, all other climate conditions being equal. Thus, water volume is the second important variable to consider.

Divot volume associated with lab and field divots is shown plotted against divot depth in **Figure 22**. As expected, the larger surface area of field divots shown in **Figure 21** translates to larger volumes of water in the field divots for a given divot depth compared to lab divots.

Recalling the drying data plotted against divot volume in **Figure 19**, an area can be defined in **Figure 22** where total divot drying would be expected to occur in the field divots before the flutes dried. Based on the actual data points in **Figure 19**, lab divots dried before flutes if their volume was below about 0.026 cu in. (26 on the x-axis). Using the data curve in **Figure 19**, a case can be made for extending that to about 0.032 cu in. where the curve intersects the "All flutes dry" line. Employing a more conservative approach, 0.030 cu in. was used in **Figure 22** to show the range of divot volumes (shaded area) where drying would occur in the divots before the flutes completely dry. Thus, divots up to about 0.150-in. (3.81 mm) depth can be expected to dry before the panel flutes. When the drying time for the panel eave is considered, as shown in **Figure 19**, divots of even larger dimensions can be expected to dry significantly faster.

Roof System Durability

Building design and construction professionals know that using premium materials and getting water off a roof are necessary ingredients to providing a long service life. That is why 55%Al-Zn alloy-coated steel roofing systems are so prevalent and why these roofing systems feature a positive slope to drain water. In this study, it has been shown that even when hail divots up to about 0.150-in. (3.81 mm) depth are present on a 55%Al-Zn alloy-coated SSR panel, they will dry before a typical panel-strengthening flute does.

When the panel eave area is considered, all divots of up to about 0.160-in. (4.06 mm) depth dried significantly faster. The eave area required more than 17 hours to fully dry compared to an average of about 5½ hours for the largest group of divots in this study. This is a significant result when the real-world performance demands of a sheared-edge eave are considered.

Based on these results, hail stone divots up to about 1¾-in. diameter, where impact energies up to 14 ft-lb and divot depths up to about 0.160 in. (4.06 mm) are typical, would not be expected to accelerate corrosion on a 55%Al-Zn alloy-coated steel roofing panel due to "ponding" water in the divots. Such roofing systems featuring flutes and sheared-edge conditions, characteristics considered standard in the industry, have performed excellently in service for over 40 years. Since the divots in this study have been shown to dry before these standard roof features, the claims of diminished roof service life for such hail-impacted roofs hold no water and thus do not constitute "functional" roof damage.

Summary

A controlled study was conducted to evaluate time of wetness of simulated hail divots on a 55%Al-Zn alloy-coated steel standing-seam roof (SSR) panel. Results show that lab-produced divots of up to about 0.150-in. (3.81 mm) depth dried faster than areas of the panel where normal mechanical deformations provide panel rigidity. When drying of the panel eave area is considered, all divots of up to about 0.160-in. (4.06 mm) depth dried significantly faster.

Field survey data of divots produced by hail on 55%Al-Zn alloy-coated steel SSR panels were compared with the results from lab-produced divots. Despite differences in shape characteristics between lab and field divots, there was good correlation between divot depth and the resulting volume of water which would reside in the divots after a rain, with all but very small field divots exhibiting larger surface areas than the lab divots. Since larger surface areas promote faster evaporation rates, actual hailstorm divots would be expected to dry more quickly than lab divots of the same depth, assuming equal volumes of water in the divots.

It is concluded that since these normal panel mechanical deformations and eave areas have proven to perform well on 55%Al-Zn alloy-coated steel SSR panels in outdoor service for more than 40 years, then hail divots of up to about 0.160-in. (4.06 mm) depth, which dry in shorter periods of time, would not be expected to diminish the normal service life of such a roof.

Findings and Conclusions

Based on the results of a metallographic analysis of 55%Al-Zn alloy-coated SSR panel profile specimens from a 43-year-old roof in Denver, Colorado, the following findings and conclusions are presented:

1. Coating crazing on rollformed profile radii of this 43-year-old roof is minor, generally ranging from 0.0002-0.0020 in (0.006-0.050 mm).
2. In some cases, the coating crazes expose the base steel, allowing the intrusion of elements and creating conditions leading to the light TBS observed on the roof panels.
3. Base steel exposure size is minor, generally ranging from zero to 0.003 in (0.071 mm) with one outlier at 0.148 mm (0.006 in). Even when considering this outlier, it is still about 3 to 13 times smaller than the range of bare spots reported to not impede corrosion resistance after nine years in a range of environments.
4. Compared to a functional damage claim associated with 1-to-2-in. diameter hail impacts, base steel exposure from standard panel rollforming is about 50 times larger and has not impaired corrosion resistance on the Denver roof for 43 years. Thus, hail-strike divots on such roofs, which might cause much smaller degrees of coating crazing, are not expected to degrade service life or be considered "functional" damage.
5. Currently manufactured 55%Al-Zn alloy-coated steel panels from two manufacturers, originally procured for this study, exhibit negligible coating crazing along rollformed profile radii, thereby confirming the improvements made in panel manufacturing processes over the last 40 years.

Based on the results of a laboratory study of drying times associated with simulated hail divots on a 55%Al-Zn alloy-coated steel SSR panel, together with analysis of field data, the following findings and conclusions are presented:

1. Hail divots up to about 0.150-in. (3.81 mm) depth dry faster than normal mechanical deformations associated with panel flutes typically employed to strengthen roof panels.
2. Hail divots up to about 0.160-in. (4.06 mm) depth, the largest divot in this study, dry in a small fraction of the time required for the panel eave to dry.
3. Actual divots observed on hail-damaged roofs, while slightly different in shape than the laboratory-produced divots, correlate well with an analysis using divot volume, with divot depths up to 0.078 in. (1.98 mm) (the largest divot in the field dataset) expected to dry before panel flutes.
4. "Ponded" water in hail-strike divots up to about 0.160-in. (4.06 mm) depth do not represent functional damage to the service life of 55%Al-Zn alloy-coated steel SSR systems which have performed well in the metal building industry for over 40 years.

Acknowledgments

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SIX FREQUENTLY MISUNDERSTOOD TOPICS RELATED TO COMMERCIAL BUILDING ENCLOSURES

ABSTRACT

Discussions between architects, building enclosure consultants, product manufacturers, and contractors frequently center around common misunderstandings regarding commercial building enclosures. This presentation will address five common topics using case studies, specific project examples, and citations from previous literature. These topics are: 1) The importance of relative humidity, dew point, and how they are managed in a building enclosure and the difference between a static analysis and dynamic analysis (WUFI). 2) How the permeability of individual layers of the building enclosure versus moisture flow through an entire assembly. 3) The importance of aligning the control layers at penetrations such as windows and how to ensure continuity at important interfaces such as the roof and foundation. 4) An analysis of when structural sheathing is needed in commercial construction compared with wood-framed construction and when structural sheathing can be used to improve the efficiency of a building schedule. 5) An overview of NFPA 285 testing, the information it provides, and its importance for the safety of buildings. Finally, the presenters will discuss a bonus topic citing specific examples of how results found via lab test methods can set unrealistic expectations for in-field conditions.

SPEAKER

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Andrea Wagner-Watts is the commercial application leader for DuPont Performance Building Solutions. Wagner-Watts has worked in the construction industry for over 15 years, where she has successfully helped develop multiple sealants and air/water barrier system solutions. Currently, she focuses on improving the overall performance of the building enclosure through application innovation, and new product development. She has published on building science, interfaces, durability, and resiliency. Wagner-Watts holds two patents, is a LEED Green Associate, and is the Air Barrier Association of America Technical Committee Chair

LEARNING OBJECTIVES

- Define the terms "relative humidity," "dew point," and "vapor permeability," and describe how each can impact a project.
- Explain NFPA 285 and how fire-rated assemblies are critical components for life safety.
- Discuss how and when to incorporate structural sheathing in a project.
- Review how to properly interpret and apply lab testing of products and assemblies to field conditions

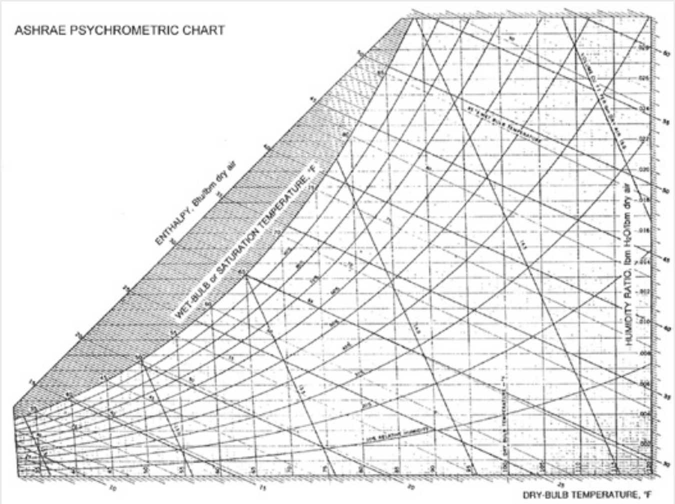

FIGURE 1. ASHRAE Psychrometric Chart

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The building enclosure industry is filled with jargon and technical terms that are assumed to be known and understood by everyone involved. Although these terms are frequently related to topics that are not necessarily taught in school, they are embedded in codes, standards, and other documents from industry organizations. Unless an experienced colleague or mentor has taken the time to teach these terms, new professionals are left to assume their full meanings based on context. Additionally, the definitions and understanding of this information is continually evolving as building science progresses. As a result, people working in the industry frequently make assumptions and have misconceptions about terms and topics important to the construction of commercial building enclosures.

This paper covers several of the biggest areas of misunderstanding related to commercial building enclosures:

- The importance of relative humidity (RH) and dew point, how they are managed in a building enclosure, and the difference between a static analysis and dynamic analysis
- How the permeability of individual layers within the building enclosure affect moisture flow through an entire assembly
- The importance of aligning the control layers at penetrations such as windows, and how to ensure continuity at important interfaces such as the roof and foundation
- The purpose of structural sheathing in commercial construction and load considerations when structural sheathing is used
- The scope of the National Fire Protection Association's *Standard Fire Test Method for Evaluation of Fire Propagation Characteristics of Exterior Wall Assemblies Containing Combustible Components*[1] (NFPA 285), the information NFPA 285 testing provides, and the importance of such tests for the safety of buildings
- The distinctions between laboratory testing of materials and in-field evaluations of quality during construction, including differences in the objectives, methods, and measures

While multiple papers have been previously written on each of these topics individually, the goal of this paper is to provide a simplified explanation of each of these topics as well as resources for in-depth study for when a more thorough understanding of the topic is desired.

RELATIVE HUMIDITY AND DEW POINT ANALYSIS

The introduction of code requirements for continuous insulation, air barriers, and water-resistive barriers (WRBs) has increased the amount of attention paid to the movement of moisture through the building enclosure. These requirements, requirements for increased airtightness, known issues with portions of the current building stock such as sick building syndrome,[2] and increased use of user-friendly modeling software have made questions of moisture movement and dew point within the building enclosure important during the design and material-substitution phases of the building construction process. These are complex topics that cannot always be described in one sentence or one data point.

It is important to start with clear definitions of the key words at the center of this discussion. Relative humidity is the ratio of the amount of water vapor in the air to the amount of water vapor that the air can hold at saturation at a given temperature and pressure.[3] This ratio, which is typically stated as a percentage, is calculated as the amount of moisture in the air in vapor form divided by the total amount of moisture the air can hold. In warmer temperatures, air holds more moisture, increasing the absolute amount of water in the air. Conversely, in colder temperatures, air holds less moisture at saturation than it does at warmer temperatures at the same relative humidity. The change in absolute moisture content in the air is also referred to as the change in vapor pressure. Dew point is the temperature at which the RH is 100%, the air is completely saturated, and moisture will begin to condense.[4] Psychrometric charts, such as Chart 1 in the *ASHRAE Handbook: Fundamentals*,[3] are the simplest way to determine the dew point at a given temperature without a detailed calculation (**Fig. 1**).

Dew point can also be calculated using one of several formulas, some more complicated than others. For example, Lawrence's simple description of the relationship between RH and dew point can be summarized by the following equation for RH greater than 50%:[5]

$$T_d = T - \frac{100 - RH}{5}$$

where

T_d = dew point temperature, °C

T = observed temperature, °C

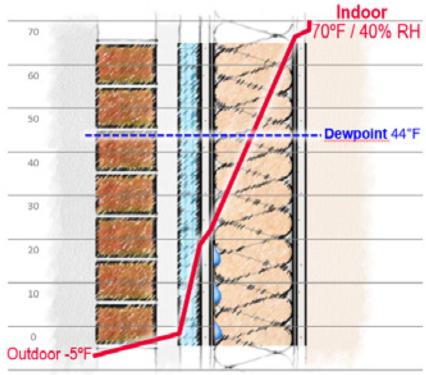

FIGURE 2. Dew point calculation through a wall assembly.

Laurence also presents a simplified general rule that the dew point temperature decreases by about 1°C for every 5% of RH. This rule is a good reference point when one is working to quickly understand the change in dew point temperature and other tools are not available.

The dew point temperature location can be calculated using the Glaser method or a heat, air, and moisture (HAM) model. **Figure 2** shows an example of how the Glaser method can be used to define the dew point temperature across a wall assembly. It is critical to understand that while the dewpoint can occur within any space of the wall assembly, moisture can only condense on a surface. The moisture will condense on the next cold surface after the dewpoint is reached. **Figure 2** shows an assembly where the dew point was reached within the batt insulation of the stud cavity. Because it could not condense within the cavity itself, moisture in the air condensed on the next coldest surface, which, in this case, was the interior side of the exterior sheathing. If the condensing surface is within a portion of the wall assembly where liquid water can be managed (for example, drained or wept), the condensed moisture is not an issue. However, the condensation can become a problem if it forms in a sensitive location, such as the back of exterior sheathing as shown in **Fig. 2**. One way to prevent this issue is to move more of the insulation out of the stud cavity onto the exterior. That moves the dew point and the potential condensation point outside of the exterior sheathing and into a space where the condensed moisture cannot cause damage to the structure.

For many years, it was assumed that calculating the location of a dew point temperature at one or several temperatures was sufficient to provide confidence in the design of a building enclosure assembly. However, these steady-state calculations say nothing about what happens over the life of the structure. The Glaser method does not account for moisture storage within materials or capillary flow of water through a material.[3] Questions such as "Does moisture accumulate within the wall assembly over time?" and "What is the moisture content of the interior gypsum within this building enclosure after 10 years?" can now be relatively easily answered using hygrothermal analysis and software such as WUFI (Fraunhofer Institute for Building Physics, Stuttgart, Germany), which incorporates heat and moisture transfer into the calculations. Straube and Schumacher have published several case studies showing how this type of modeling can prove beneficial in the building design process.[6]

VAPOR PERMEABILITY AND PERMEANCE

The next frequently misunderstood building science and material physics topic is that of vapor permeability and vapor permeance. The terms "vapor permeability" and "vapor permeance" are often used interchangeably in requests for material properties data from testing according to ASTM E96, *Standard Test Methods for Methods for Gravimetric Determination of Water Vapor Transmission Rate of Materials*,[7] or another similar method. ASTM E631, *Standard Terminology of Building Constructions*,[8] defines water vapor permeance as

> the time rate of water-vapor transmission through a unit area of a flat material or construction induced by unit vapor-pressure difference between the two specified surfaces, under specified temperature and humidity conditions.

ASTM E96 goes on to clarify that permeance is a performance evaluation of a material, not a property. Permeability, on the other hand, is the product of the thickness of a material and the tested permeance at a given humidity and temperature differential. The permeability calculation turns the data point into a material property, which is usually reported in perms.

Because of the humidity and temperature aspects of the datum, neither permeance nor permeability are steady-state numbers that apply to all conditions that a material will experience during its lifetime. Several manufactures are marketing "smart" or variable vapor retarders that are intended to work effectively in varying conditions. The variable permeance of materials, if known and characterized, can be used to help better understand how moisture moves through a wall assembly; the objective is to design materials that allow moisture to move through the enclosure when necessary and prevent the movement when it is not. This varying material characteristic is important to understand because it has a substantial impact on the total moisture content of the wall assembly. Recall that the total amount of water vapor that is available to move through a material is greater at higher temperatures than at lower temperatures. If the permeability of the material changes as the temperature and relative humidity change, the amount of absolute moisture moving through the assembly will also change. Wagner et al.[9] present an example of this behavior in which a silica-modified organic air and water-resistive barrier had a permeance ranging from 0.02 perms at 5°C/5% RH to 11.63 perms at 5°C/100% RH. Because this membrane has a permeance of 0.81 perms at the standard reporting conditions of 23°C/50% differential RH, the International Building Code[10] would classify the membrane as a vapor barrier or Class I vapor retarder. However, the membrane performs as a Class III vapor retarder under low-temperature, high-humidity conditions. This information is not typically found on a data sheet nor disclosed by many manufacturers.

The vapor permeance of materials is most often referenced for air and water barriers; however, every material in a

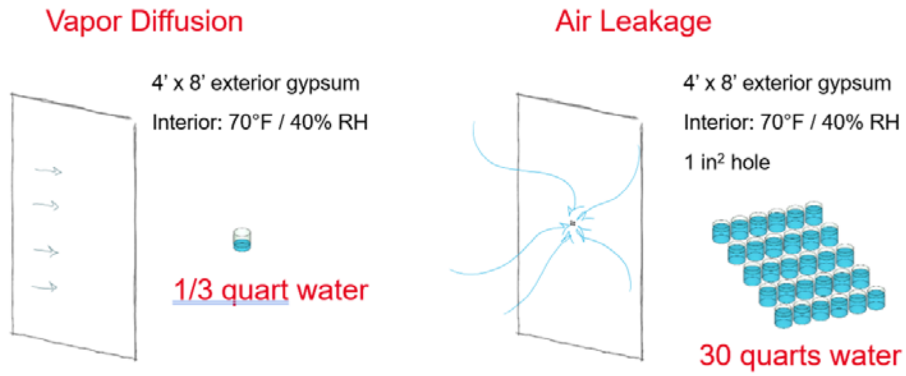

FIGURE 3. Difference in water-vapor movement through vapor diffusion verses air leakage.

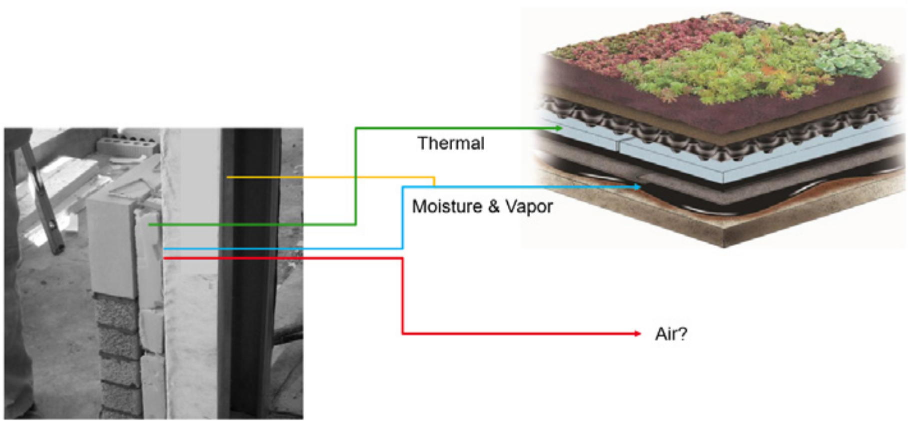

FIGURE 4. Control layers within a wall assembly verses a roof assembly.

wall assembly can be characterized in the same way. It is important to understand how water vapor will travel through the insulation, the sheathing, and even the aesthetic surfaces such as paint. One of these materials will end up being the limiting layer for how moisture moves through the assembly. Furthermore, water vapor is not usually an issue if it is able to move easily through the assembly, but it can become an issue if it is stopped within a moisture-sensitive material and that material cannot release it back as vapor.

There are two final points to keep in mind when working with vapor permeability of a wall assembly. First, the second law of thermodynamics still applies: moisture vapor will always move from an area of high pressure to an area of low pressure. The direction through a wall assembly is likely to change during at least an annual cycle of the building's life. Also, the vapor moving through the assembly only becomes a problem if it is able to condense in a moisture-sensitive location—which leads us back to the importance of hygrothermal analysis of the assemblies.

Second, as Lstiburek has highlighted, the amount of water vapor that will move through a hole in the air barrier assembly is more than 100 times the amount moved via vapor diffusion of a material.[11] His point is highlighted graphically in **Fig. 3**, which shows that only one-third quart of water will move through an entire 4 × 8 ft exterior gypsum board in a year whereas 30 quarts of water will move through a 1 × 1 in. hole in that same gypsum board under the same conditions over the same time frame.

CONTINUITY OF CONTROL LAYERS

There are four primary control layers within the building enclosure: air, water, thermal, and moisture vapor. High-performance, energy-efficient buildings (and, really, all buildings) can only perform well when each of these control layers is continuous on all six sides of the building enclosure. It is relatively easy to accomplish continuity of layers within the bulk, opaque portion of the wall assembly. However, continuity is much more challenging where different assemblies interface, such as where a window assembly meets a wall or where the wall meets the roof. Traditional wall assemblies will frequently put the control layers in a different order than a traditional roof assembly does; as a result, the layers must somehow cross each other at the transition points (**Fig. 4**).

Fortunately, if careful thought is put into the required transition, continuity becomes not only possible but also simple for the tradespeople installing the systems to achieve. **Figure 5** shows an example in which the continuity of all four control layers at the roof-to-wall interface is provided. To ensure continuity to the thermal layer on the roof, the insulation goes around the entire parapet and additional insulating blocking is installed within the cavity. The primary air and water barrier is then taken from the face of the insulation (which is serving as the air and water barrier in this example) around the top of the parapet onto the top of the insulation on the roof. Finally, the vapor barrier can be applied to the roof deck and tied into the vapor-tight spray polyurethane foam on the wall.

Unfortunately, the control layers within a window assembly are not always as

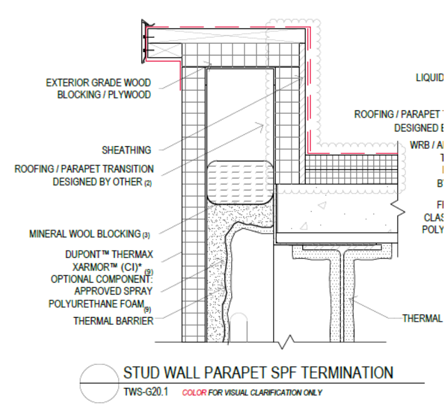

FIGURE 5. Continuity of control layers from the wall onto the roof.

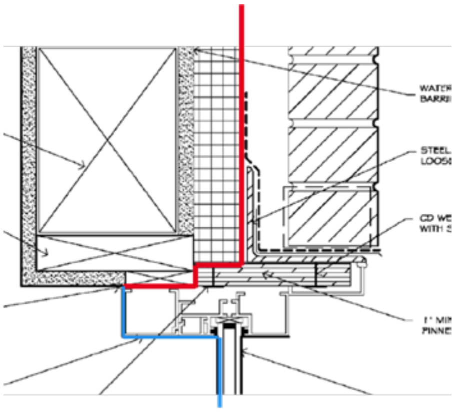

FIGURE 6. WRB continuity between a window head and a wall assembly.

easy to clearly identify as they are in a roof assembly. **Figure 6** shows the head of a window assembly with the WRB in the wall highlighted in red and the WRB in the window highlighted in blue. The point at which the two lines in the figure meet is the most critical point for preventing water infiltration through the interface. The window rough opening must be flashed and tied into the wall WRB, and a sealant bead or other transition that is compatible and adheres to both substrates must be used to seal between the window frame and the WRB flashing. The window itself is in line with the continuous insulation in the example wall. This allows for a minimal thermal break between the assemblies. If the window were recessed or protruded, as in **Fig. 7**, there would be a disconnect between thermal layers in the window and the wall while the WRB is continuous. That type of design requires a different solution to tie the two thermal layers together, or it needs a more robust thermal break in the assembly to prevent the window frame and rough opening from getting cold and being at risk of condensation.

In general, the best time to solve any potential issues with continuity at building assembly interfaces is during the design process. If possible, involve the contractors who will be installing the systems to ensure that they can correctly install whatever detail is designed. Manufacturers are continuing to work to develop materials to simplify interfaces between assemblies for all four control layers in different configurations. Improved materials will make the process of both designing and installing the interface transitions less complicated while still allowing for the design freedom desired.

STRUCTURAL SHEATHING IN COMMERCIAL CONSTRUCTION

The need for structural sheathing (if any) in commercial construction can be a complicated subject, which typically gets delegated to the structural engineer. However, it is important for building enclosure consultants and contractors to understand the basics. The first thing to understand for both commercial and wood-framed construction is what the structural sheathing is expected to do. Structural sheathing is used to help support the loads acting on the structure, including wind loads, shear loads, live loads, and more. For commercial construction, when structural sheathing is required, it typically supports wind loads and transfers them back to the structure. Wind loads are both positive and negative, pushing and pulling on the structure.

The studs behind the sheathing provide bracing for the pushing of the positive

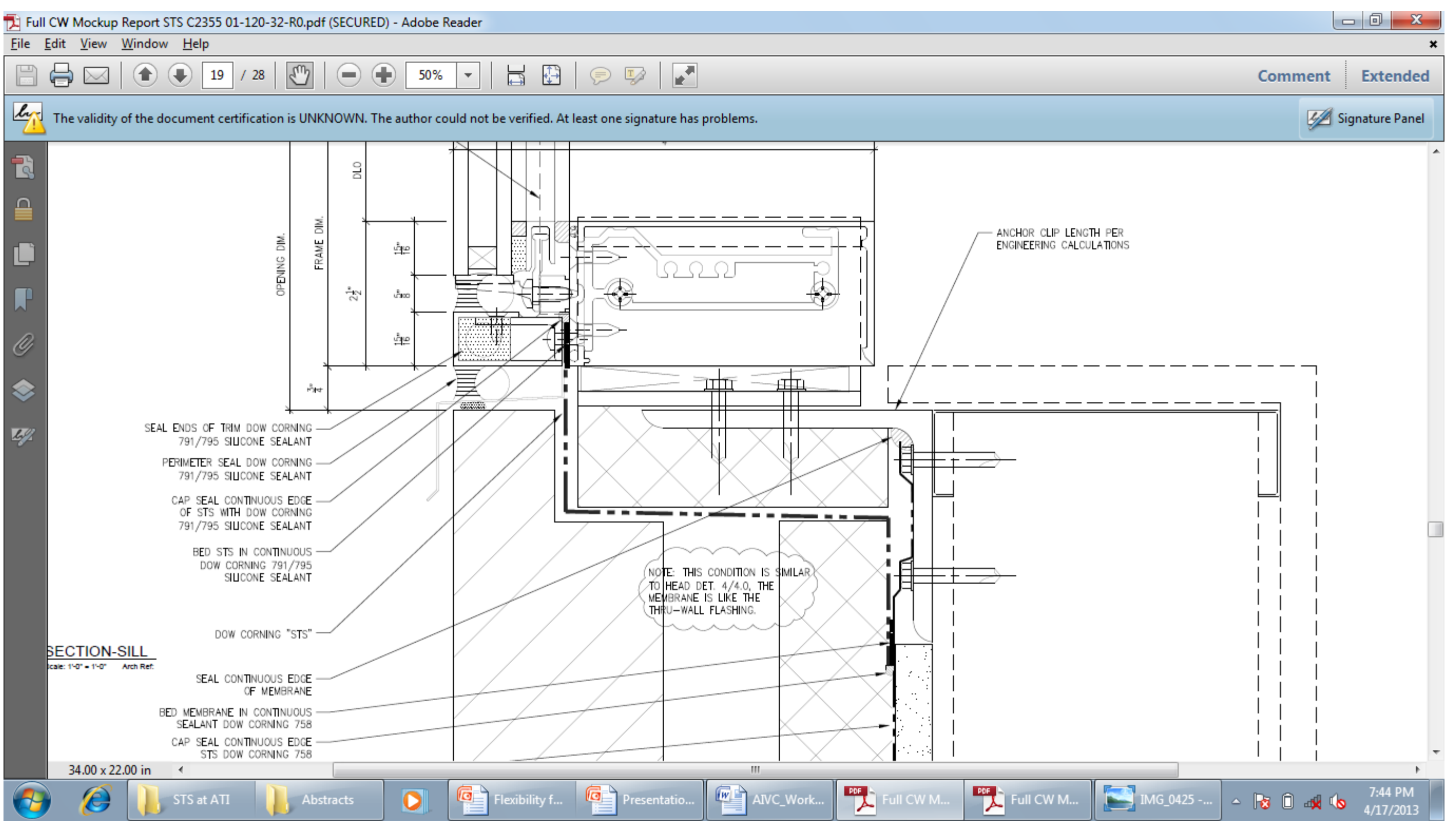

FIGURE 7. Misaligned thermal barrier at the window to wall interface.

FIGURE 8. NFPA 285 wall assembly while burning from both the exterior and interior.

wind loads, whereas the negative wind loads are typically the limiting factor of the design. The same can be stated for other structural requirements such as the requirements for structural glazing joints in curtainwalls, where wind loads can suck the insulating glass units off the building. Furthermore, wind loads are typically highest at the corners of the building. These corner wind load zones become the limiting loads used for calculations to provide consistency of design and construction throughout the building whenever possible. The maximum load is then used in combination with the deflection properties of the material to design the stud spacing of the structure. This material property is defined as a ratio of the maximum deflection to the length between the span being braced, with the most common ratiosbeing *L*/240 and *L*/360. The more brittle the material is, the less wind load it can take at wider spans; therefore, stud spacing of 12 in. on center or less is used for weaker, more-brittle materials whereas stud spacing of 16 in. or more on center is used for stronger, less-brittle sheathing and claddings.

Another important aspect of structural sheathing to consider is the ability of a sheathing to take on the loads of the structure and also carry and transfer the loads from the cladding back to the structure. In residential construction, for example, plywood is often referred to as a "nail base." Cladding (often siding) nails can go directly into the sheathing without being required to also penetrate the stud. In contrast, when exterior gypsum is used, all cladding attachments must be secured to the stud to achieve the amount of support required. Fastening back to the stud can become more complicated when clips or girts are needed to attach cladding (such as metal panels) over exterior continuous insulation; in these situations, the project may require a structural engineer to calculate the fastening pattern and screw strength if the insulation is thicker than the prescribed thicknesses in Table 2603.12.1 of Chapter 26 of the *International Building Code*.[10]

Recently, several products have been introduced to the commercial construction market that can take on these cladding loads and perform as a fastener base without requiring the fasteners to continue through to the stud. The use of these products allows for flexibility regarding where to attach girts and fasteners since it is not necessary to rely on the structural stud spacing or workmanship to hit the stud during installation. Eliminating the need to find a structural stud can then help prevent additional holes through other layers of the wall assembly, specifically the air barrier and WRB.

ASSEMBLY FIRE TESTING

The appropriate use of NFPA 285[1] is a complex subject that relies on project-specific discussions about the application of the testing method and the materials involved. It is so complex it is frequently a topic of daylong learning sessions. Very simply put, NFPA 285 is an assembly testing method that is designed to evaluate how a wall assembly will burn when (a) a fire is on the interior of a building burning out through an opening, and (b) when the fire is on the exterior of the building burning upward at the opening. Chapters 14 and 26 of the *International Building Code* require this test for all buildings that contain combustible components such as foam plastic insulation.[10] The base assembly used in the test (**Fig. 8**) consists of a two-story wall with a floor-to-wall intersection and an opening designed to emulate a window opening (window openings are among the weakest points within the building assembly during a fire). The intent of the test is to ensure that a fire cannot easily spread up the exterior face of the building or through the interior along the wall from floor to floor. The test results are used to quantify fire propagation up a building facade.[1]

NFPA 285 is not used to determine whether a wall has a 1-hour (or more) fire rating; these fire resistance ratings are determined by ASTM E119, *Standard Test Methods for Fire Tests of Building Construction and Materials*,[12] or ANSI/UL 263, *Standard for Safety of Fire Tests of Building Construction Materials*.[13] For example, if a Type I building has a wall assembly that contains foam plastic insulation along with a wall that must achieve a 2-hour fire rating due to proximity to a lot line, that specific exterior wall assembly must comply with both NFPA 285 and have a 2-hour rating by way of ASTM E119.

Both NFPA 285 and ASTM E119 tests are completed on full wall assemblies. No individual material can be tested

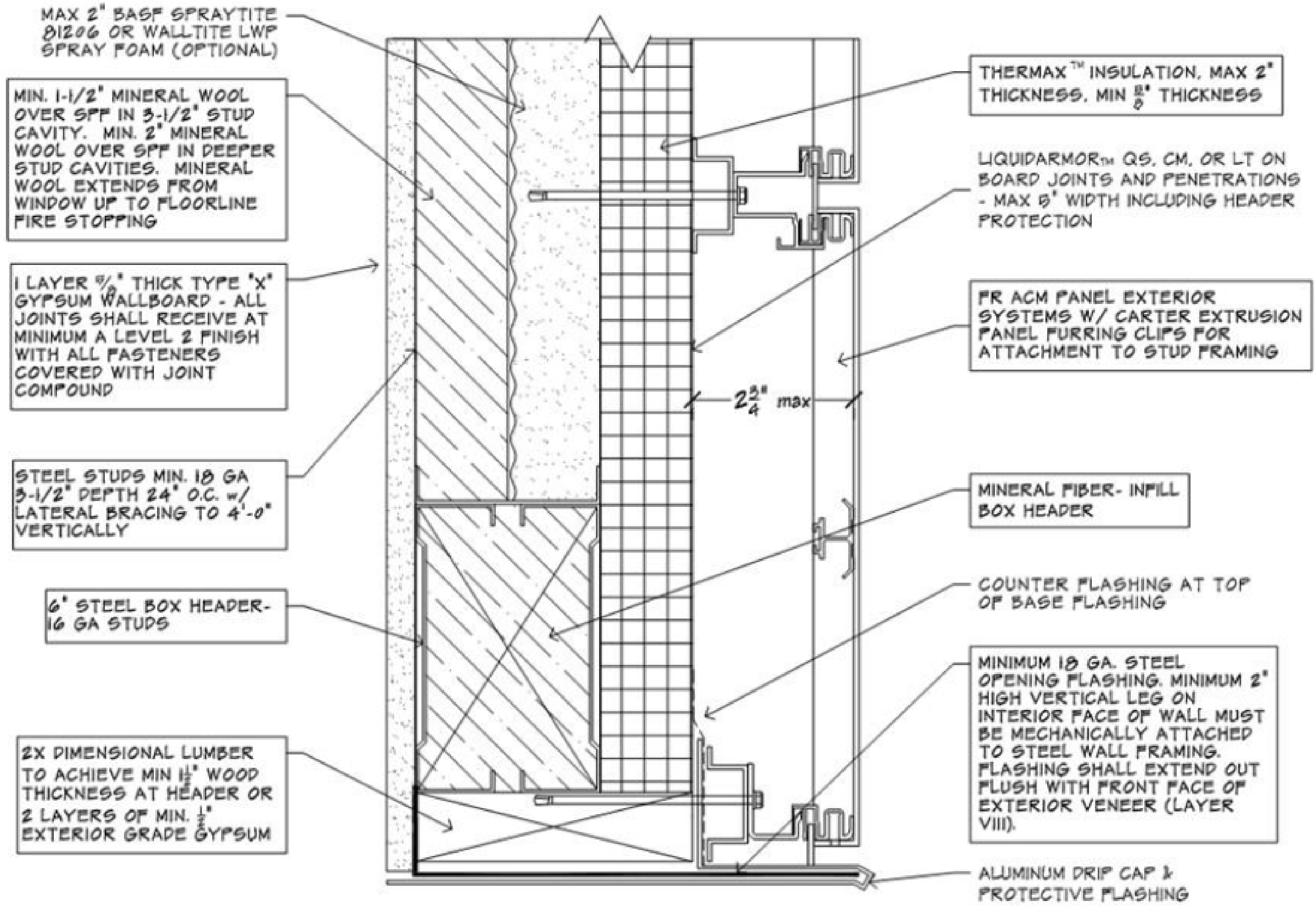

FIGURE 9. NFPA 285 window header detail with MCM cladding panels.

or pass these tests on their own; they must be tested as part of an assembly. It is common for product manufacturers to have multiple wall assemblies tested to these standards. Additionally, engineering evaluations are used to extend the testing to additional assemblies that will also meet the requirements of these standards based on tests performed and the fire-performance characteristics of materials not tested in the specific assembly. Fire engineers perform these engineering evaluations based on a set of criteria that will soon be officially added to the NFPA 285 standard.

The specific details of the wall is constructed for the test are as important as the materials used. For example, the treatment of the window header detail may determine whether a wall assembly passes NFPA 285. The details shown in **Fig. 9** and **10** show two different assemblies that can pass NFPA 285 as long as the thickness of the foam plastic insulation and the cladding type used are correctly specified and installed. If the cladding changes from a non-combustible cladding such as fiber-cement siding to a metal composite material panel, the treatment of the window-header detail may also need to change to ensure compliance with NFPA 285. These changes in material choices can affect whether an assembly will meet the fire requirements for a project and should therefore not be overlooked.

LABORATORY TESTING VERSUS FIELD TESTING

Manufacturer data sheets and material code testing requirements report results from testing done according to methods published by ASTM International, the Fenestration and Glazing Industry Alliance, and other organizations. These test methods are developed to be repeatable tests that are used to determine the performance and physical properties of a given material or assembly. Most often, they are designed to be performed within the controlled environment of a laboratory. These test methods are not necessarily designed to be completed on a project jobsite, where it may not be feasible to use the type of equipment required, prepare sample material, or otherwise conduct the test according to the standard requirements.

Adhesion testing of materials is one area where test methods and test data for quality control can differ from the test methods and test data used to create data sheets. Both sealants and air and water resistive barriers are subject to adhesion testing in a laboratory and in the field for quality control, but the test methods are not the same. For sealants, ASTM C794, *Standard Test Method for Adhesion-in-Peel of*

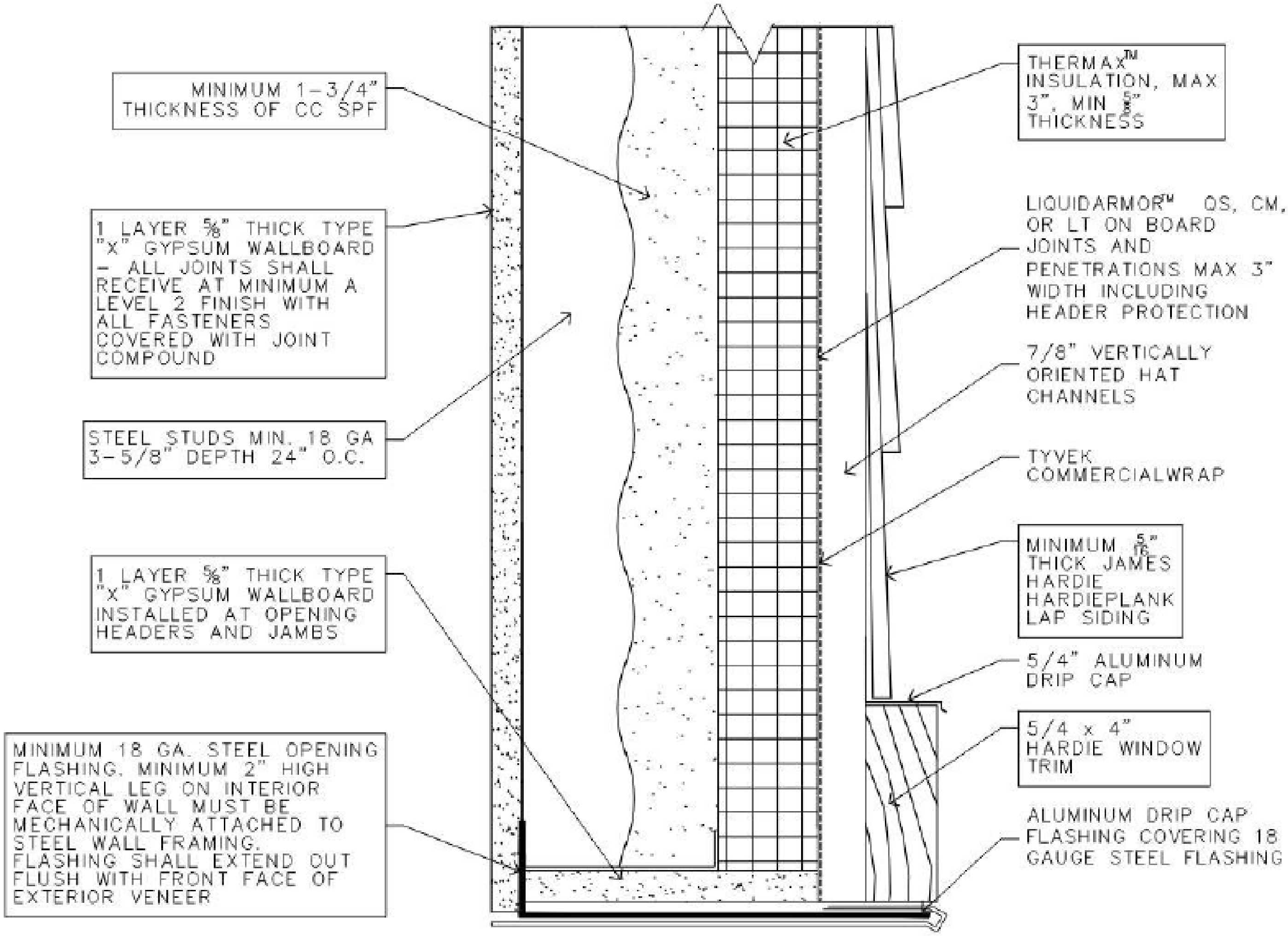

FIGURE 10. NFPA 285 window head detail with fiber-cement siding.

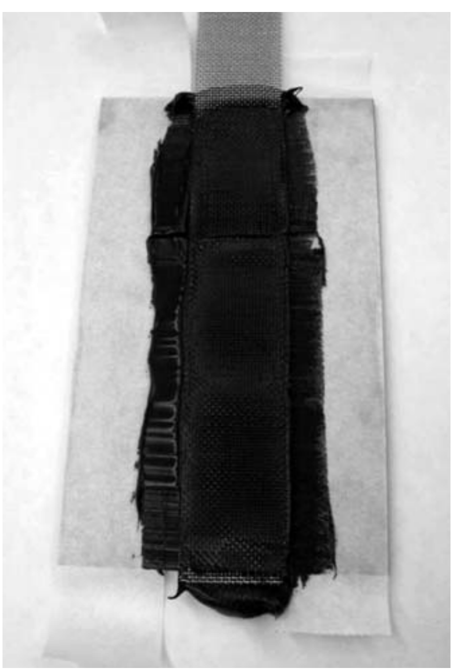

FIGURE 11. Adhesion-in-peel test specimen after imbedding wire mesh screen.[10]

Elastomeric Joint Sealants,[14] is the most-used test method for laboratory adhesion testing. This test method involves imbedding a wire mesh within a bead of sealant (**Fig. 11**). The mesh screen is installed into an tensile testing machine and pulled at 180 degrees from the surface of the substrate. The tensile strength required to pull the sealant is measured in pounds per linear inch, and observations of the failure mode (cohesive or adhesive) are taken. This is the test method that is cited by ASTM standards and specifications such as AAMA 714-20, *Voluntary Specification for Liquid Applied Flashing Used to Create a Water-Resistive Seal Around Exterior Wall Openings in Buildings*.[15] The specifications define a minimum peel value per this method that is required for the material in the intended application.

Laboratory adhesion testing is important because it is quantifiable and repeatable for the purposes of product development and adhesion comparison between specimens. It is not practical for in-field quality control of adhesion. ASTM C1521, *Standard Practice for Evaluating Adhesion of Installed Weatherproofing Sealant Joints*,[16] is the recommended standard for field adhesion testing of sealants. Method A of this test method describes how to test an in situ sealant joint by evaluating certain properties such as elongation of the sealant prior to adhesion loss and type of adhesion loss. Most sealant manufacturers will then set pass/fail criteria for acceptable performance based on this test method, such as 100% cohesive failure or a 100% elongation before any adhesion loss occurs. Other methods within the standard practice will have different types of pass/fail criteria.

In addition to differences in the tests being performed, other factors that affect the performance of a material in a laboratory setting versus in the field include the following:

- The cure conditions of materials are controlled in the laboratory (often set at approximately 23°C and 50% RH), whereas field conditions such as temperature and RH can vary considerably.
- There is minimal cross contamination with other materials, dirt, grime, and so on in a laboratory, whereas materials in the field are at risk for contamination, particularly when they are used in adverse conditions or when they are left exposed for extended periods of time.
- Laboratory instrumentation is controlled and frequently calibrated, whereas field testing is often completed only by hand, without the assist of specific equipment.

The differences in testing and differences in results do not necessarily mean that long-term performance will be compromised in the field relative to the laboratory. The best manufacturers set performance targets for laboratory testing that compensate for less-than-ideal field conditions. Laboratory testing requirements are intentionally set higher than what will be expected in the field so that crews can easily meet the required field quality control testing minimums through quality workmanship while still producing a completed assembly that is expected to perform per the life of the products. To prevent frustration, all parties (the installer, the designer and the manufacturer) must communicate clearly regarding what test method and pass/fail criteria will be used; these parties must all understand field quality control requirements expectations before installation begins.

CONCLUSION

Commercial construction involves many multifaceted systems and assemblies that are meant to perform for many decades. The successful design and installation of these systems and assemblies depend on expertise from a wide range of fields, including the building trades, architects, engineers from multiple engineering disciplines, building scientists, and materials scientists. To ensure smooth communication among all of these actors, it is important that all speak the same language and share a common baseline of knowledge on different topics that frequently come up during project design and construction.

ACKNOWLEDGMENTS

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WIND TUNNEL TESTING OF EDGE METAL

ABSTRACT

Wind resistance of edge metal continues to be a concern during high-wind events. Edge metal at perimeters and corners is often determined to be the initial point of failure of roofing systems during wind events. The loss of edge-metal functionality can lead to progressive failure of a larger portion of the roof system, potentially allowing water infiltration and damage to or loss of assets in the interior.

As part of the Wind Hazard and Infrastructure Performance (WHIP) Center's research initiatives, GAF and Florida International University (FIU) performed full-scale wind-tunnel testing of edge metal at FIU's Wall of Wind. Four (4) full-scale wind-tunnel tests were performed using one (1) contractor-fabricated, 24-gauge L-shaped edge metal system with an 8-inch face, 4-inch horizontal flange, and a ¾-inch drip edge. Two (2) different 22-gauge cleat shapes were used—a standard 6-inch cleat and an 8-inch cleat with a 1-inch horizontal return. Four (4) different cleat-fastener locations were used—one low, one in the middle, and one high on the vertical surface, as well as one on the horizontal surface.

A discussion on the test parameters and outcomes of the different cleats and associated attachment locations will be provided. Best-practice design and installation recommendations will be given.

LEARNING OBJECTIVES

- Discuss and review the current code-mandated test methods (i.e., ANSI/SPRI/FM 4435 ES-1) for determining wind resistance of edge metal shapes.
- Demonstrate the failure modes of L-shaped edge metal relative to cleat engagement, cleat shapes, and fastener locations when subjected to wind tunnel testing.
- Compare test results of full-scale wind tunnel testing with an equivalent ES-1-tested L-shaped edge metal assembly.
- Evaluate test methods, loading methodologies, and wind directions related to the determination of edge-metal wind-resistance capacity.

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Wind resistance of a low-slope roof system's edge metal has improved over the past couple of decades.[1] However, there continues to be a concern during high-wind events. Edge metal at perimeters and corners is often determined to be the initial point of failure of roofing systems during wind events.[2] The loss of edge-metal functionality from high winds generally creates a breach in the building enclosure in regard to weatherproofing and water intrusion. A breached roof-to-wall interface can lead to localized failure of the roof, or a progressive failure of a larger portion of the roof system, potentially allowing water infiltration that may cause damage to or loss of assets in the interior.

As part of the Wind Hazard and Infrastructure Performance Center (WHIP-C), GAF and Florida International University (FIU) performed full-scale wind tunnel testing at FIU's Wall of Wind in February 2022. Four full-scale wind tunnel tests were performed using a contractor-fabricated, 24-gauge L-shaped edge metal system with an 8 in. (20.32 cm) face, 4 in. (10.16 cm) horizontal flange, and ¾ in. (1.9 cm) drip edge. Two different 22-gauge cleat shapes were used—a 6 in. (15 cm) cleat and an 8 in. cleat with a 1 in. (25.4cm) horizontal return. Four different cleat-fastener locations were used—one low, one in the middle, and one high on the vertical surface, and one on the horizontal surface.

This paper will discuss the test parameters and outcomes of testing that used different cleat types and attachment locations. Observations made during testing are discussed, as well as how those observations may be put into practice. Aerodynamic tests were performed to determine pressure coefficients, and failure assessment testing was performed to assess failure modes of various installations.

EXPERIMENTAL APPROACH

The overall research approach was to build multiple full-scale edge metal systems and test them in a wind tunnel on a turntable to learn how wind speed and wind direction affect edge metal's wind performance based on varying cleat-fastener locations. Wind pressures acting on the edge metal and roof system and the vibration of the edge metal were recorded and analyzed to assess performance. Testing to failure was also performed to understand failure mode and performance variations during testing.

Two test decks were constructed on site by J Quintero Roofing of Miami, Florida. Each test deck included a wood structure, a thermoplastic polyolefin (TPO) roof system, and two of four different edge metal systems. FIU personnel instrumented each test deck to record pressures related to the fascia system and roof system and to record wind-induced vibration of the fascia system. GAF provided guidance about roofing installation practices and assisted with the installation of certain components of the roof system installation and instrumentation setup.

Each test deck included two variations of the edge metal system, with two contiguous sides installed per configuration. The corner was central to each configuration (**Fig. 1**). Specifically, one test deck included edge metal configurations 1 and 2, and the other test deck included edge metal configurations 3 and 4. In total, four configurations were tested for this research. Additional testing specifics

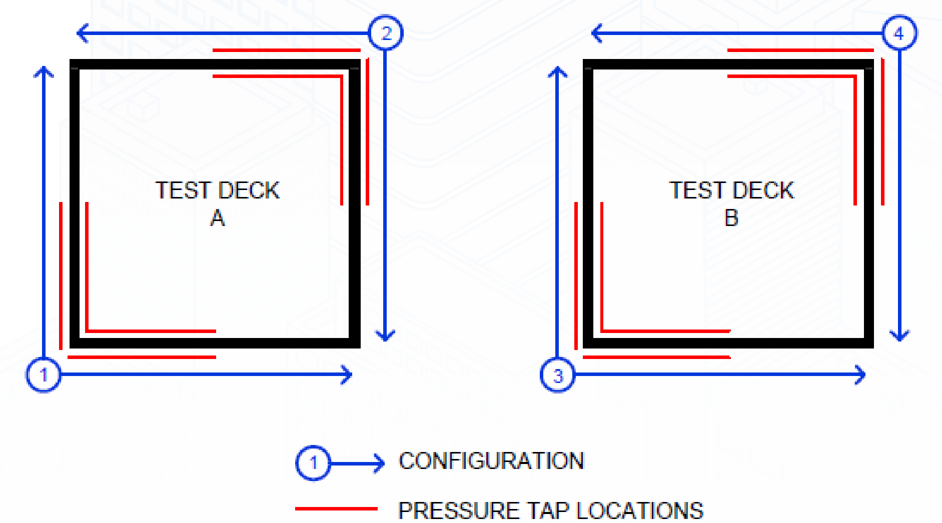

FIGURE 1: The overall layout of the 4 Configurations and pressure taps locations on the 2 test decks.

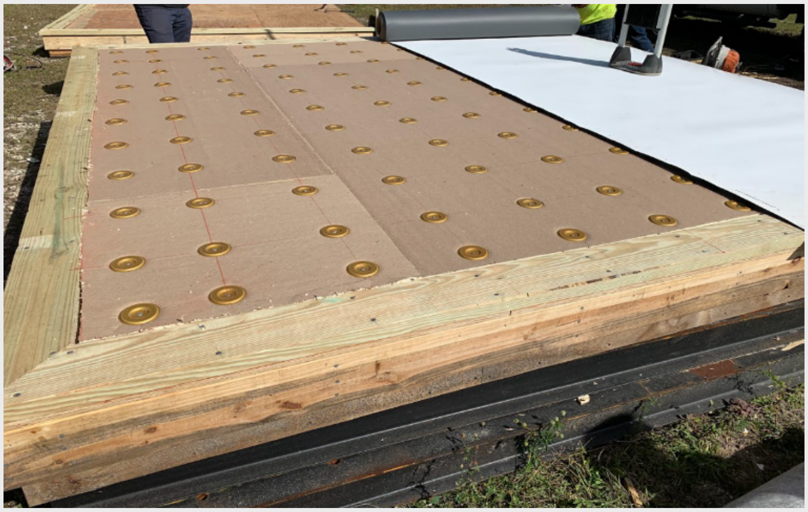

FIGURE 2. Photo showing the nailer, insulation, IW plates and fasteners, and a TPO sheet being installed on one of the 11ft x 11ft test decks.

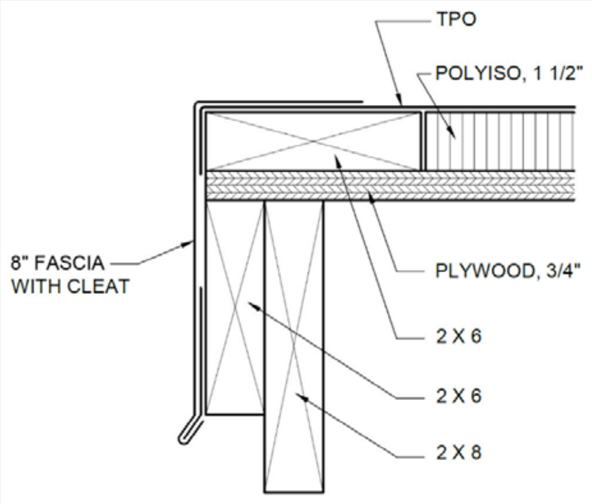

FIGURE 3. Graphic showing a section view of the construction of the two roof decks.

are explained in the "Physical Testing" section of this paper.

Importantly, the wind tunnel base that supports and secures the test decks is able to rotate 360 degrees. The ability to rotate allows for data collection across a 360-degree rotation. Wind tunnel testing that utilizes a rotatable turntable allows a fuller set of data collection, which, in turn, provides a fuller perspective on how edge metal systems may perform in the field during high winds.

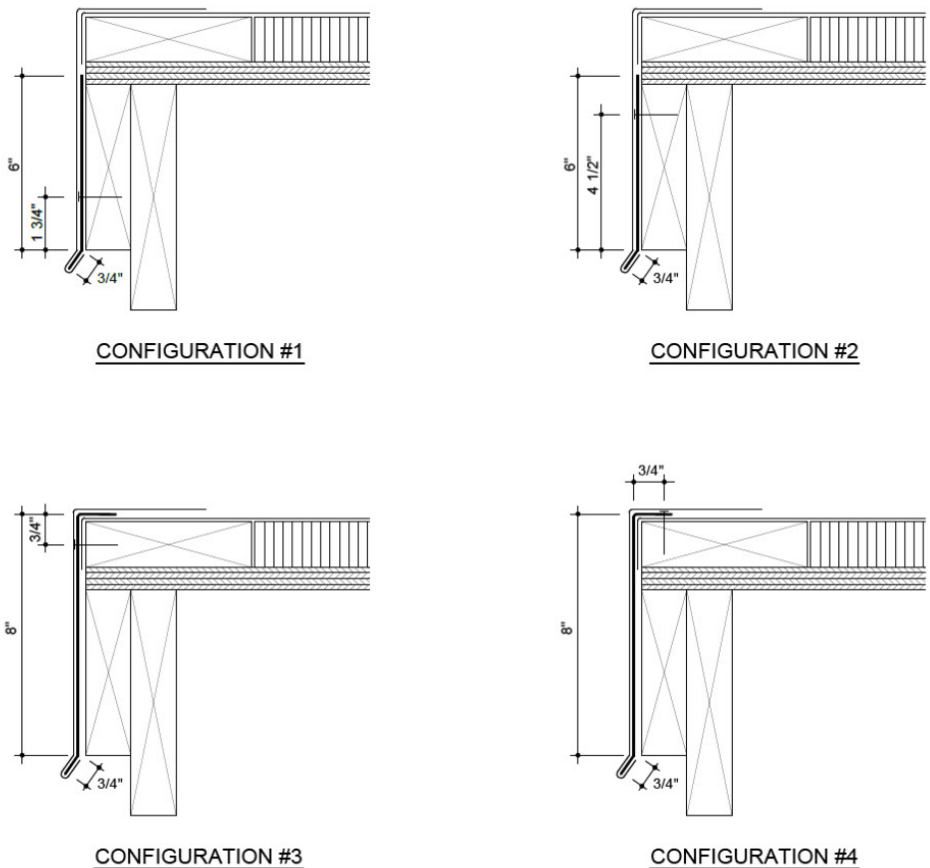

FIGURE 4. Graphics showing cleat types and cleat fastener locations for Configurations 1, 2, 3, and 4.

TEST APPARATUS AND TEST ROOFS

Two 11 × 11 ft (3.3 × 3.3 m) wood roof decks were constructed. Each consisted of ¾ in. (2 cm) plywood over traditional "2x" construction. The roof system consisted of an underlayment minimally fastened to the wood deck, a single layer of 1.5 in. (3.81 mm) thick polyisocyanurate foam insulation (polyiso), and a 60 mil TPO induction welded (IW) to "IW" plates. The fasteners and IW plates were installed at 1 ft (0.305 m) on center (o.c.) in both directions. A dense fastening pattern was used to help ensure the roof system itself would not fail during wind tunnel testing of the edge metal system. A 2x6 wood nailer was fastened along the perimeter edge for securement of the edge metal system and to create a perimeter for the polyiso (**Fig. 2**).

Full-scale wind tunnel tests were performed on four edge metal systems. All four configurations used a galvanized, 24 gauge, L-shaped edge metal fascia with a galvanized, 22-gauge cleat. The fascia had an 8 in. (20.32 cm) vertical face, a 4 in. (10 cm) horizontal flange, and a ¾ in. drip edge at the bottom that engaged the cleat. The cleat also had a ¾ in. drip edge (**Fig. 3**).

The four configurations varied based on the location of the fasteners for the cleat as well as cleat type (**Fig. 4**). It is important to note that the location of the fastener for the fascia metal was the same for all four configurations.

For configurations 1 and 2, a 6 in. (15 cm) cleat was used. In general, it is understood that when cleats are "nailed low," a short cleat is most commonly used, meaning the cleat does not have a horizontal flange at the upper edge. For configurations 3 and 4, an 8 in. (20cm) cleat with a 1 in. (25.4 mm) horizontal return was used.

For configuration 1, the cleat-fastener location was selected based on current industry approval listings for contractor-fabricated edge metal.

It should be noted that most building codes require edge metal systems to be tested to determine wind resistance using the appropriate test method(s) in ANSI/SPRI/FM 4435/ES-1.[3] Manufacturers and contractors (through the National Roofing Contractors Association [NRCA] ES-1 Program) provide many edge metal systems—both prefabricated and contractor-fabricated—that have been tested to determine their wind resistance.

The edge metal system used for this research (using an 8 in. [20 cm] face) intends to imitate a currently available, ES-1-tested edge metal system. The edge metal profile, ITS-30, is available from NRCA.[4] This is one of many contractor-fabricated edge metal and coping profiles tested according to ES-1. ITS-30, titled Embedded Edge (L-Type), has a tested resistance of 210 lb/ft2 (95kg) in the outward direction and the cleat fastener is 1¾ in. (4.4 cm) above the break line at the drip edge. Configuration 1 of this research used the same cleat-fastener location. For all four configurations, fastening of the horizontal flange of the fascia also imitated the ITS-30 nailing pattern.

Configurations 3 and 4 are installation locations where only a single 2x wood blocking is provided on top of a wall system. The cleat fastener location for configurations 3 and 4 means the system reacts to the wind differently than configurations 1 and 2 (which are "pinned" at each end). The roofing industry has long recommended that cleats be fastened as low/as close as possible to the drip edge, and to avoid fastening "high" on the cleat. In this research, intentionally placing cleat fasteners high on the cleat provides information about these types of installations.

Of the four configurations installed, three had fasteners in the vertical face and one in the horizontal. More specifically:

- **Configuration 1:** The cleat was nailed 1¾ in. (4.4 cm) above the drip edge at 6 in. (15 cm) o.c. into the wood substrate (to imitate/mimic ITS-30).
- **Configuration 2:** The cleat was nailed 4½ in. (11 cm) above the drip edge at 6 in. o.c. into the wood substrate (3½ in. [8.8 cm] down from the top of the wood blocking).
- **Configuration 3:** The cleat was nailed ¾ in. (1.9 cm) down from the top edge at 6 in. o.c. into the face of the wood nailer.
- **Configuration 4:** The cleat was nailed ¾ in. back from the outer edge of the cleat at 6 in. o.c. into the wood nailer.

PHYSICAL TESTING

Test Decks

Each 11 ft (3.3 m) square test deck was secured to the top of an 11 × 11 ft base building. The same base building was used for each test deck. The base building was secured to the turntable in the wind tunnel. The interior of the base building was accessible via a small door. Data acquisition systems, tubing, and wiring were contained within this interior space. With the roof test deck installed onto the base building, the roof membrane was approximately 6 ft (1.8 m) above the floor of the wind tunnel.

Wind Tunnel

The wind tunnel can generate a maximum wind speed of 157 mph. However, maximum wind speed occurs approximately 10 ft (3.0 m) above the floor of the wind tunnel; the wind speed was lower at the height of the roof deck used in our testing. The reported wind speeds were measured at the roof height, with a maximum of approximately 134 mph (215 km/h) being achieved.

Two sets of tests were performed on each of the four edge metal configurations: aerodynamic and failure assessment. This paper provides an overview of the aerodynamic testing; however, the primary focus, perhaps having a more immediate effect on the roofing industry, is the failure assessment portion of the research. **Figure 5** shows the test deck in the wind tunnel.

FIGURE 5: Photo of a test deck in the wind tunnel (inactive).

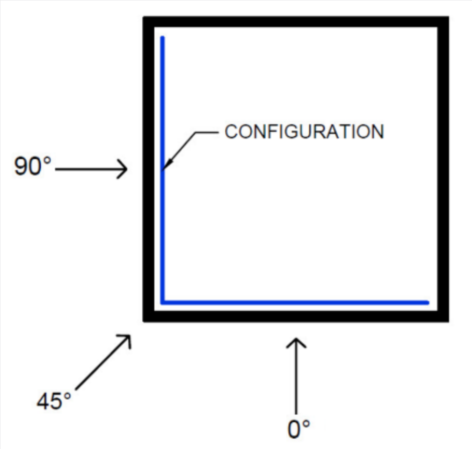

FIGURE 6: Wind directions used for Failure Assessment testing.

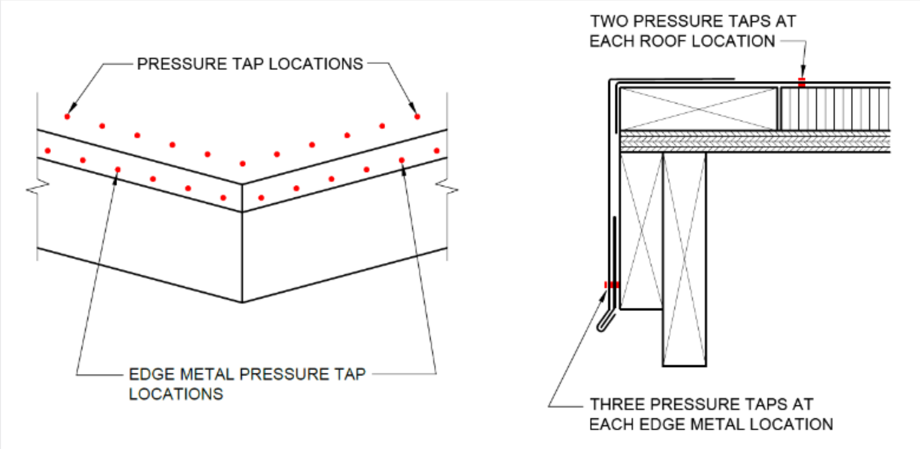

FIGURE 7: Graphic showing pressure tap locations on the roof and edge metal, with a section view showing the number of pressure taps at each tap location.

Aerodynamic and Failure Assessment Testing

Aerodynamic tests were performed to determine pressure coefficients, and failure assessment testing was performed to assess failure modes of various installations.

Aerodynamic experiments were conducted at mean wind speeds ranging from 26 to 87 mph (41 to 140 km/h) at the mean roof height of 5 ft 9 in. (1.75 m) using simulated Exposure Category C (Open Terrain). At each wind speed used, pressures were recorded (using 58 pressure taps) at 15-degree rotations from 0 degrees to 360 degrees for the aerodynamic data collection testing. This work was done to incorporate a wide range of wind directions to determine pressure coefficients at many wind directions. This helps determine the most vulnerable wind direction acting on edge metal systems. This information, ultimately, could be implemented into standards and test methods that might be used for code reference.

In addition to aerodynamic tests, high-speed failure assessment tests were also performed. Wind speeds started at approximately 80 mph (128 km/h) and peaked at approximately 134 mph (215 km/h). Seven wind speed levels were used, increasing by roughly 8 to 9 mph (12 to 14 km/h) each level. The same terrain exposures were used. Each level (**Fig. 6**) included three wind directions: 0 degrees, 45 degrees, and 90 degrees. Wind testing was halted immediately after a system failure was observed so that the team could further investigate and document the system performance and determine the mode and extent of the failure.

DATA COLLECTION

The deck systems were instrumented with pressure sensors and accelerometers to collect data throughout the wind tunnel testing. Pressure taps were installed across the roof membrane system and the edge metal systems to quantify the wind-induced pressure differential. Accelerometers were installed on the edge metal to investigate wind-induced dynamic effects in the system.

Each configuration had 58 pressure taps (**Fig. 7**). Pressure taps were located on the outer vertical surfaces of the fascia, cleat, and substrate, respectively, as well as on the upper (or top) horizontal surfaces of the insulation and membrane. Data were recorded for one configuration at a time due to the symmetry of the test deck. For the edge metal, pressure taps were placed at 6 in. (15 cm) from the corner, then spaced every 12 in. (305 mm) (**Fig. 8**). Each side of the configuration has a total of six pressure tap locations.

The pressure taps on the roof surface were placed to match the spacing of the pressure taps on the vertical faces of the edge metal system. At each rooftop pressure tap location, a pressure tap was installed on the exterior surface of the insulation and on the exterior surface of the TPO.

FIGURE 8. Photos showing pressure taps. The left photo shows outermost pressure taps on the fascia. The right photo shows the pressure taps for the membrane. *Note: the pressure taps were trimmed flush to the fascia and membrane prior to testing.*

RESULTS AND OBSERVATIONS

Aerodynamic Testing

The results of the aerodynamic testing showed the following:

» Wind direction affects peak outward and upward pressures

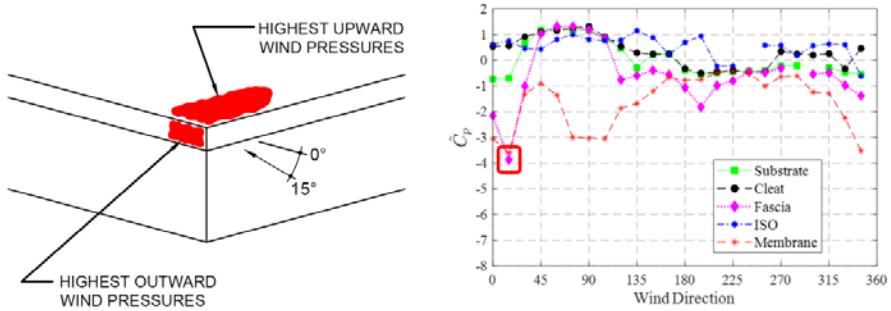

FIGURE 9. Graphic showing a 15 degree wind direction ("near parallel") creates the highest wind pressures on the vertical face of the fascia and the horizontal roof surface.

- Area averaging influences pressure coefficients

Analysis of the data obtained during the aerodynamic testing was done by FIU students and the primary investigators who were part of this research. Using the data from the aerodynamic testing, pressure coefficients were determined for each pressure tap location. One result of the aerodynamic testing showed that near-parallel wind flows (i.e., 15 degrees from parallel) created the highest outward and upward wind pressures on the fascia and roof surface[5] (**Fig. 9**).

It is important to note that when wind hits a building, a negative pressure is exerted on the fascia (due to suction) and positive pressure is exerted on the inner face of the fascia (due to wind getting behind the edge metal). In effect, the fascia is simultaneously being *pulled off* from the outside and *pushed off* from the inside.

Failure Assessment Testing

Failure occurred when the fascia (with or without the cleat) flipped upward and back onto the roof. Wind-induced actions such as bending, oscillation, fastener pull-out, and cleat disengagement as well as the location of these actions were also observed and recorded by video. The wind speeds at which these actions and failures occurred were recorded. Importantly, field observations of poststorm damage have documented edge metal failure when the wind angle was presumed to be perpendicular to the edge metal. It is likely that the overall context of the building and its specific environment might affect the most damaging wind angle. Additionally, while there are straight-line wind storms, many high-wind storms (e.g., hurricanes) have a spiral effect, effectively covering all wind angles.

Configuration 1

Configuration 1 used cleat 1, the 6 in. (15 cm) cleat. The cleat fastener was located 1¾ in. (4.4 cm) above the break line for the drip edge. Configuration 1 was the only configuration to have a failure before reaching the wind tunnel's maximum wind speed. During the "low-speed" aerodynamic testing, the fascia released from the cleat nearly the entire length of the test deck, but did not flip up and onto the roof. It was observed that the cleat was likely set too high, and therefore, the engagement of the cleat and the fascia was greatly reduced. The failure assessment testing was initiated without adjustment of the fascia or cleat. During the next wind-speed level, the fascia completely folded back onto the roof its entire length. At that point, the system was considered to have failed. Observations made of the failure found the cleat was indeed set too high (approximately ¼ in. [.06 cm]) relative to the location of the drip edge portion of the fascia. This reduced the amount of engagement between the cleat and the drip edge (**Fig. 10**).

Interestingly, configuration 1 was believed to be the most robust of the four installation methods; building-code-required tests for edge metal systems generally confirm this assumption. During this testing, however, this configuration failed at the lowest wind speed due to the misalignment of the cleat fastener. This emphasizes the importance of a well-engaged cleat–drip edge interface.

Configuration 2

Configuration 2 used cleat 1, the 6 in. (15 cm) cleat. The cleat fastener was located 4½ in. 11 cm) above the break line for the drip edge. A small amount of cleat/drip edge separation with some minimal fluttering of the fascia was seen in the higher wind-speed levels. The small amount of fluttering was located adjacent to the corner where the drip edge receiver was disengaged from the cleat (approx. 6 to 10 in. [15 cm] in length). There was little outward permanent deformation of the fascia and cleat system; the edge metal system appeared to remain able to perform until the point it failed. The failure occurred at approximately 134 mph. The failure was immediate; there was a small amount of flutter at the

FIGURE 10: Photo of failure mode of Configuration #1.

FIGURE 11: Photos of failure mode of Configuration #2.

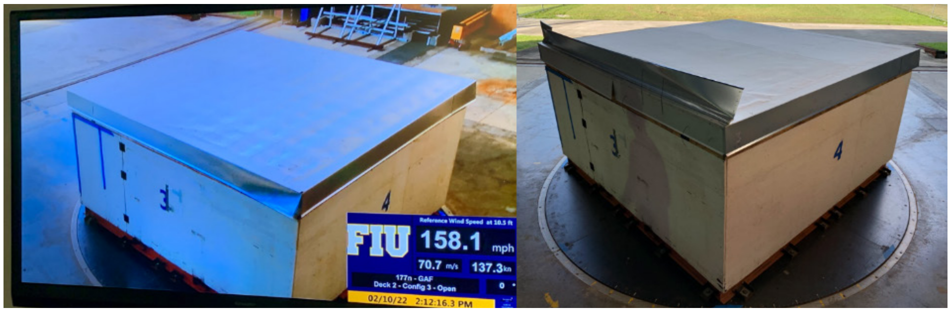

FIGURE 12: Photo of failure mode of Configuration #3.

FIGURE 13. Photo of failure mode of Configuration #3 up close.

FIGURE 14: Photo of failure mode of Configuration #4.

corner, then it was folded up and on top of the roof (**Fig. 11**).

Configuration 3

Configuration 3 used cleat 2, the L-shaped cleat. The cleat fastener was located on the vertical portion of the cleat approximately ¾ in. (1.9 cm) from the top. Like the other configurations, there was some separation at the corner seam, some fluttering where the cleat became unattached from the receiver (approximately 18 in. [45 cm] from the corner), but overall the fascia stayed in place and was observed to be able to perform until failure occurred.

At approximately 134 mph (215 km/h), the fascia folded up and over the horizontal surface (**Fig. 12**). The drip edge separated from the cleat. The cleat did not have any permanent deformation and remained in place (**Fig. 13**).

Configuration 4

Configuration 4 used cleat 2, the L-shaped cleat. The cleat fastener was located on the horizontal leg approximately ¾ to 1 in. (1.9 to 2.5 cm) from the face. This configuration began fluttering at a lower wind speed relative to configurations 2 and 3, which was not unexpected considering the location of the cleat fastener. There was some separation of the cleat and drip edge (approximately 12 to 18 in. [30 to 45 cm] from the corner) as fluttering increased with the increase in wind-speed levels.

The portion of the cleat closest to the corner stayed in place while the portion of the cleat farthest from the corner folded upward (**Fig. 14**). Some of the nails pulled out at the far end of the fascia. This seemed to imply that there was a greater pressure at the far end of the "left side" of configuration 4 relative to the other configurations.

Overall, the edge metal system appeared to remain able to perform up to the point of failure, albeit there was larger outward permanent deformation with increased wind-speed levels. Permanent outward deformation, even at a small scale, creates vulnerability (i.e., reduced weatherproofing performance) at the roof-to-wall interface.

It is noteworthy that when the cleat and fascia lifted and were folded back, the edge of the roof was exposed, which is more likely to compromise the weathertightness of the roof-to-wall interface. This type of failure only occurred with the L-shaped cleat when it was nailed in the horizontal flange.

Summary of Test Results

Pressure coefficients

- Pressure coefficients (i.e., GCp values) for specific pressure tap locations were found to be higher than GCp values used in code-referenced standards.
- Historically, the most conservative wind direction has been presumed to be "near perpendicular" relative to edge metal, and as such, is reflected in the code-required test methods. In contrast, this research showed "near-parallel" winds (15 degrees from

parallel) were most conservative when determining wind pressures acting on the edge metal.

Performance

- For all cases (except the misinstallation previously noted), the edge metal system did not fail until the wind speeds reached approximately 134 mph (215 km/h) at the test deck.
- The structural dynamics of an edge metal system change based on the location of the cleat fastener. With a low-fastened cleat (i.e., near the drip edge), the fascia is constrained at both ends. Conversely, with a high-fastened cleat (i.e., near the top of the fascia or into the horizontal), the edge metal system can flutter more easily because there is no substrate attachment on the lower portion of the cleat or fascia. The stiffness of the metal (i.e., gauge and yield strength) becomes an important factor.

Failure assessment

- During the failure where the cleat was set too high, a small (approximately ¼ in. [0.6 cm]) misalignment reduced the amount of engagement between the cleat and the drip edge and significantly reduced the wind speed at failure of the edge metal system.
- Failure occurred when the fascia (with or without the cleat) flipped upward and back onto the roof, resulting in a roof edge condition that was considered to have been immediately vulnerable to high winds as well as water entry into the building.
  - As noted in the "Failure Assessment Testing" section, there was some disengagement of the fascia from the cleat at wind speeds less than 134 mph (215 km/h). This could compromise long-term performance and would likely need to be repaired if this were to occur on an existing building.
- For configurations 1, 2, and 3, the fascia became detached from the cleat at the corner for a short length. The fascia remained nearly in place (with small [1 to 2 in. {2.5 to 5.0 cm}] permanent deformation), and appeared to be largely functional. The extent of functionality reduction due to permanent deformation was not attempted to be quantified during this research.
- For configurations 1, 2, and 3, the failure (i.e., complete displacement of the fascia piece) was initiated because of the disengagement/release of the drip edge "receiver" from the cleat. Once disengaged, the fascia was more easily folded up and over onto the horizontal rooftop by high winds.
- For configuration 4, the cleat did not entirely disengage from the fascia. Both pieces of metal failed—the metal folded up and over along the length—while still engaged at the drip edge. Failure occurred because the metal yielded; only a few nails pulled out, and only at the far corner.
- Only configuration 4 had nails pull out of the substrate. A small number of nails fastening the fascia on the horizontal at the furthest end from the corner pulled completely out of the 2x6 wood nailer. No specific reason was determined.
- None of the nails fastening any of the cleats in any configuration pulled out of their respective 2x6 wood blocking.

CONCLUSION AND RECOMMENDATIONS

Testing of edge metal roof systems completed as part of WHIP-C provided the opportunity to investigate the aerodynamic and failure performance of four different cleat and fascia systems. The following is worth noting:

- Installation practices
  - Three configurations (2, 3, and 4) failed at the wind speed of 134 mph (215 km/h).
  - Considering the low wind speed that was needed to prematurely fail the edge metal with a misaligned cleat, and that all four configurations failed at the same wind speed, this suggests that the drip edge/cleat engagement is as critical to long-term wind performance as the location of the nail.
  - High-nailed L-shaped cleats performed well—in fact, better than expected. Edge metal, with an L-shaped cleat, fastened high (either face) performed equivalently (to failure) to the low-fastened cleat installations that have been presumed to have higher wind resistance. This finding does not align with previous field investigations that concluded that high-nailing reduces wind uplift resistance. However, it is still recommended to locate cleat fasteners as low on the cleat as possible.
    - The L-shaped cleat is a very simple, cost-effective way to increase accuracy during installation. An L-shaped cleat is considered to be "self-locating." This provides an effective quality control advantage for installers and inspectors, which helps to ensure proper cleat/drip edge engagement. It is a very reasonable approach to help protect against blow-offs due to cleat/drip edge misalignment. Additionally, using an L-shaped cleat does not preclude nailing low on the cleat.
    - Oftentimes, there is only a single wood blocking on the top of a wall, which can mean that fastening low on the horizontal surface becomes difficult. This research shows that this does not necessarily reduce the wind resistance of edge metal systems. However, an ES-1-tested edge metal system should be installed as tested to meet building code requirements.
    - High-nailing would be found to be very weak (low resistance) when using test methods that are in the building code as requirements.
  - Cleat/drip edge engagement is critical to long-term wind resistance. Using a cleat that "self-locates" is prudent.

Note that the attachment of the substrate for the edge metal (e.g., a 2x wood nailer) is of critical importance.[1] The edge metal is only as good as the substrate it is attached to. Designers should ensure there is a properly designed and executed load path that has appropriate capacity to resist the anticipated design loads.

» Performance evaluation
  – Failure assessment testing (i.e., testing to failure) helps uncover issues or expand knowledge so that performance can be assessed more confidently.
  – The highest wind pressures acting on the face of the edge metal came from a "near-parallel" direction.
  – The highest wind pressures were within 1 to 2 ft (30 to 60 m) of the corner. Averaging pressure coefficients across a large area may underestimate the wind pressures that should be used for design of edge metal systems at the immediate corner.
  – Load sharing happens between the L-shaped cleat and the fascia when the cleat is nailed on the horizontal top flange (e.g., configuration 4). This is to be expected, given that both pieces of the edge metal system—the fascia and the cleat—are nailed in the horizontal portion of the substrate, allowing the two individual pieces to move simultaneously. Remember, the cleat is one gauge heavier than the fascia in these studies and often in the field.

» Codes and industry practices
  – Cleat engagement is critical; the margin of error is small. Current designs, listings, and specifications typically use a ¾ in. (1.9 cm) cleat. Is this adequate? Perhaps drip edges and cleats should be longer, as was recommended decades ago.[6] Any improvement in the strength of the cleat/drip edge engagement (e.g., stiffer cleat, larger engagement) may prove beneficial to the overall performance of the edge metal system.

Future Work

Additional research on this topic would be beneficial to better understand if "high-nailed" L-shaped cleats really do perform as well as was indicated by this full-scale wind tunnel research program. Also, it is important to consider the development of new test methods that might better replicate outcomes discovered during full-scale wind tunnel testing. Current test methods use static testing; dynamic testing methods may better replicate field conditions as well as potentially address long-term fatigue of metal components.

Other questions that may need to be addressed include the following:

» What is the most conservative wind direction to test edge metal?

» Are more stringent requirements at corners appropriate based on what was learned about area-averaging of pressure coefficients?

ACKNOWLEDGMENTS

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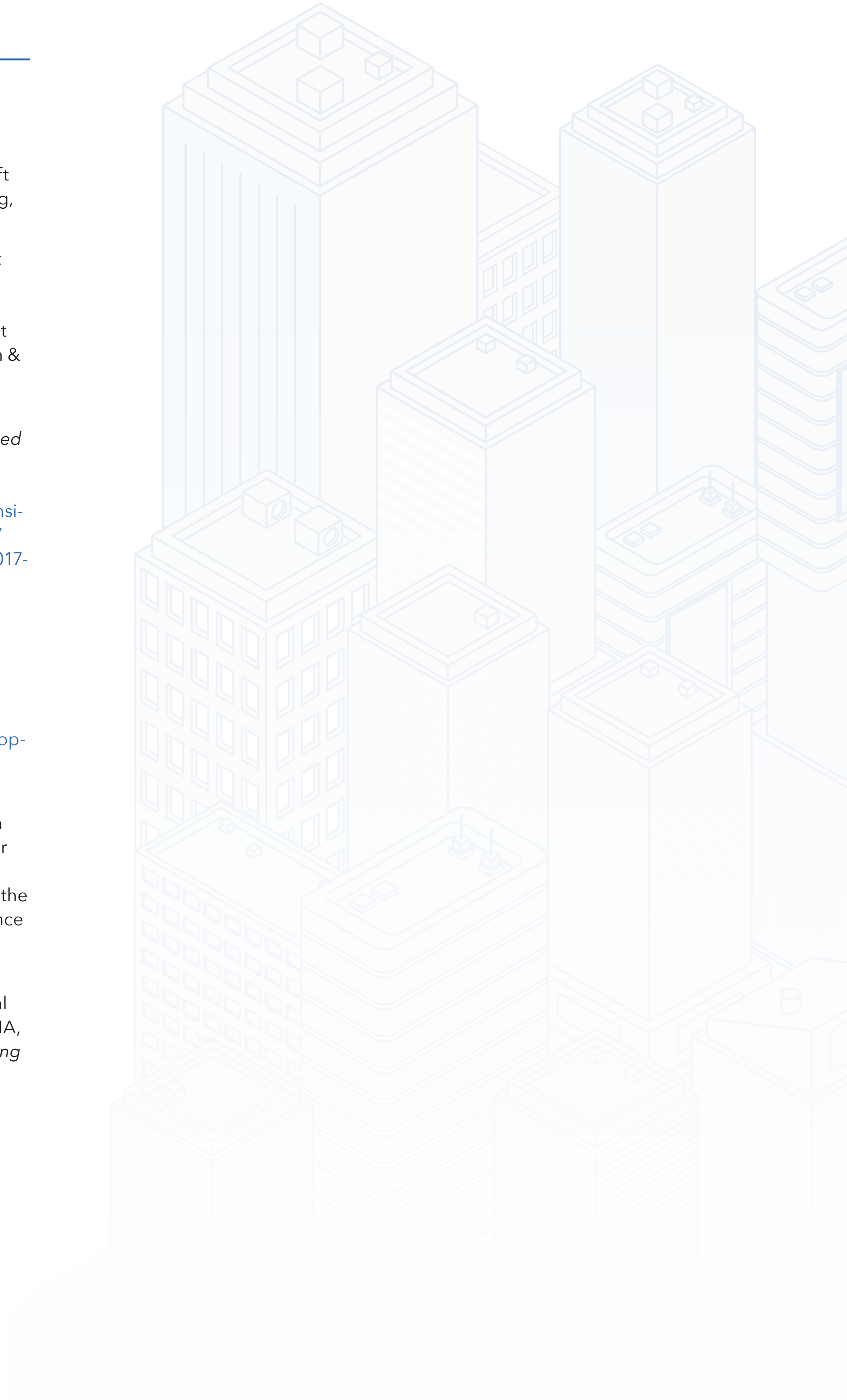

IMPLICATIONS OF BUILDING CODES FOR THE REPAIR OF BUILDINGS AND BUILDING ENCLOSURES

ABSTRACT

Building codes can be complicated and often subject to the interpretation of design and public officials alike. These interpretations can have significant effects on the scope and cost of repairs to existing buildings and building enclsoure components, such as roofs, windows, and cladding. In this presentation, learners gain an understanding of the basic requirements of building codes and how those requirements relate to existing buildings and building enclosures during the investigation and/or repair process. In many jurisdictions, the International Existing Building Code governs the remediation of damage to a structure or enclosure components, which are classified as "repairs." Repairs can often be completed in like kind and quality of materials without triggering code upgrades for egress or thermal efficiency. The case studies highlight real-world examples of code analysis pertaining to egress interpretations for repair/ replacement of window units in Florida hurricane zones and unsafe/dangerous roof deck conditions that existed prior to wind damage at a warehouse in the Midwest. In both cases, a combination of adopted codes governed and detailed code analyses will be presented to the learners to understand the differences and when each applies.

LEARNING OBJECTIVES

- Review the building code history and the adoption process.
- Evaluate how building codes relate to existing buildings during the repair process.
- Discuss when repair work triggers code upgrades and/or compliance with new building code standards.
- Identify the definitions and ramifications of unsafe and dangerous conditions and substantial structural damage.

SPEAKERS

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Alan Mullenix, PE, is a senior project engineer with Envista Forensics with over 15 years of experience in the practice of civil and structural engineering. He received his bachelor of science in civil engineering from Georgia Tech and a master of science in civil engineering from the University of Illinois at Urbana-Champaign. Mullenix has experience in commercial design and in the nuclear and petrochemical industries. At Envista, he conducts forensic investigations involving structural failures, building enclosure deficiencies, construction defect claims, catastrophe responses, and fire damage to residential, commercial, and marine buildings and structures.

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Erik Wetzler has nearly 20 years of experience as a structural engineering expert where he has directed and executed a wide range of forensic investigations, structural analyses, and litigation support cases. His practice is concentrated in tall towers, office and retail, education, warehouses, and historic buildings throughout the US, UK, Dubai, New Zealand, and globally. His experience includes the evaluation and assessment of earthquakes, hurricanes, structural and vessel collapses, building enclosure failures, and a variety of common and complex property loss scenarios.

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As the number and severity of storms and damaging forces facing our built environment increase, there is an ever-increasing risk of damage to structural and building enclosure elements of existing buildings. The significant technical hurdles associated with the repair of an existing building can be made all the more challenging with the array of building codes in use today. These codes cover all aspects of building systems from low-rise residential structures to tall towers and commercial buildings and include requirements for fire protection, structural integrity, plumbing, electrical, and energy efficiency systems. Each code may have multiple versions which are further subject to local amendments. To determine a reinstatement methodology, owners, design professionals, and contractors are required to determine the applicable building code in force and decipher the requirements governing the work. Misunderstanding these requirements can cause unanticipated and unnecessary schedule delays, project costs, or complications in the permitting process and execution of the work. In addition, determining the applicable building code requirements can be complex and the path to clarity varies within each jurisdiction.

It has been our experience as structural experts that local building code officials are typically well versed in the requirements and provisions that govern new construction but often lack the same level of experience in the application of code specifications for existing buildings. This paper seeks to provide general guidance for the design professional engaged in repairs of structural and building enclosure elements and provides two case studies for consideration. However, this paper is not a substitute for engaging local qualified design professionals and building code officials to determine the appropriate reinstatement methodologies for a specific project.

MODEL BUILDING CODES AND THE ADOPTION PROCESS

If there were just one code that applied to repairs of all existing buildings, that would be convenient for those involved. In fact, the *International Existing Building Code*[1] (IEBC) was developed to be such a resource; its purpose is to simplify the task of navigating code requirements for design professionals, building owners, and local code officials (**Fig. 1**). The IEBC, which was born out of earlier editions of the *International Building Code*[2] (IBC), seeks to "encourage the use and reuse of existing buildings" and applies to the "repair, alteration, change of occupancy, addition, and relocation of existing buildings."[1] To meet these goals, the IEBC guides stakeholders through common situations experienced during the life cycle of a building and provides flexibility and alternative approaches to achieve compliance with minimum requirements to safeguard the public health, safety and welfare. In many scenarios, these goals can be accomplished without requiring costly upgrades to meet the standards established for new buildings contained in codes such as the IBC.

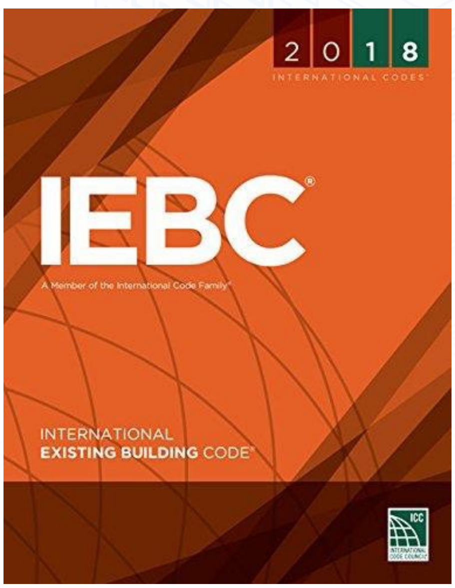

FIGURE 1. Cover of the 2018 edition of the *International Existing Building Code*.

New and existing buildings are subject to requirements specific to local municipalities or jurisdictions. As such, the IEBC, like other codes in the series developed by the International Code Council (ICC), serves only as a model code. A model code is a template developed by committees of subject matter experts that can be adopted, amended, or altogether replaced depending on the needs determined by the authorities having jurisdiction. Each model code developed by the ICC is updated every three years, and then the model code updates may or may not be adopted by local jurisdictions. It is important to note that a model code is not law until it is adopted by a jurisdiction. In addition, it is up to each individual jurisdiction to determine which model codes and versions they adopt and how they may amend code provisions. A jurisdiction is not required to adopt all model codes from the offerings of a model code family such as the ICC. Therefore, a jurisdiction may have adopted codes from the ICC, the older Uniform Building

Code series, a set of municipal-specific codes, or a combination of some or all of these code sets. For a given project location, it is important for experts and design professionals to understand the requirements of the local jurisdiction and educate project stakeholders throughout the repair determination and implementation process.

Thus, the shortest path to understanding the code requirements is to start within the local jurisdiction. Within metropolitan areas where building development is common, the code requirements for new and existing buildings can often be found by searching for the code of ordinances for the governing municipality, such as the local city or county. Within the code of ordinances, one can typically find the adopted model codes (including which edition has been adopted), along with applicable amendments. Local jurisdictions frequently reference state code adoptions and additional state amendments that must be considered. Within rural municipalities and jurisdictions, the applicable building code requirements may not be as easily located, and the expert or design professional will often require input f rom a local building code official either through a phone call or a more formal request.

INTERNATIONAL EXISTING BUILDING CODE REQUIREMENTS

Once the specific requirements of a local jurisdiction have been established, the expert or design professional will usually find that they are operating within the general guidance provided by the IEBC or earlier editions of the IBC. (Note: The current edition of the IEBC[1] was published in 2021. However, this paper references the 2018 IEBC[3] because, at the time of the publication of this paper, it is the edition currently being enforced in most jurisdictions.)

When it comes to the evaluation of damage and determination of a repair methodology for structural and building enclosure elements, there are three main takeaways that can be gleaned from an analysis of the IEBC. First, when work is done on a damaged structure to remediate the damage and return the structure to its former condition, that work is formally classified as a repair. Classifications in the IEBC define whether the work completed on existing structures should be governed as a repair, an alteration, an addition, or something else. IEBC defines a repair as "the reconstruction, replacement, or renewal of any part of an existing building for the purpose of its maintenance or to correct damage," whereas an addition is "an extension or increase in floor area, number of stories, or height of a building or structure."[3] An alteration, by contrast, is simply work that does not qualify as either a repair or an addition. Confusion can arise when remediation of damaged structural and building enclosure elements is classified as an alteration instead of a repair. Work classified as a repair is typically not subject to upgrade provisions (which can be expensive), whereas alterations can trigger additional upgrade requirements and have a higher standard for construction. These higher standards are often the provisions of the code standard for new buildings, such as the IBC.

The second noteworthy takeaway from an analysis of the IEBC is that if a structure has sustained limited damage and no *dangerous* or *unsafe* conditions exist, the structural or building enclosure elements can be repaired to their condition before the damage by using materials of like kind and quality. Several sections of the 2018 IEBC[3] provide the basis for permitting the remediation of damage on a like-for-like basis.

» Section 401.2 states that "the work shall not make the building less complying than it was before the repair was undertaken."

» Section 302.5 states that "except as otherwise required or permitted by this code, materials permitted by the applicable code for new construction shall be used. Like materials shall be permitted for repairs and alterations, provided that *unsafe* conditions are not created."

» Section 302.4 states that "materials already in use in a building in compliance with requirements or approvals at the time of their erection or installation shall be permitted to remain in use unless determined by the building official to be *unsafe*."

When damage to existing buildings occurs, questions are invariably raised about whether the repairs themselves and the building as a whole are now required to meet all provisions of the new building standard, particularly with regard to egress and energy efficiency. Sections 302.4, 302.5, and 401.2, among others, essentially indicate that (a) the damaged part of the building can be repaired with like materials, (b) the rest of the undamaged building and building materials can remain in place, (c) upgrades to egress and energy requirements are not required simply because the building sustained damage, and (d) the repair work cannot make the building worse in those areas than it already was. This is particularly relevant to older and historic buildings that have been safely occupied for years and would be subject to prohibitively costly and potentially infeasible upgrades in the event of limited or moderate damage. The main caveat is the presence of an *unsafe* or *dangerous* condition.

IEBC defines an *unsafe* condition as follows:

> Buildings, structures or equipment that are unsanitary, or that are deficient due to inadequate means of egress facilities, inadequate light and ventilation, or that constitute a fire hazard, or in which the structure or individual structural members meet the definition of Dangerous, or that are otherwise dangerous to human life or the public welfare, or that involve illegal or improper occupancy or inadequate maintenance shall be deemed unsafe. A vacant structure that is not secured against entry shall be deemed unsafe.[3]

While the definition of *unsafe* includes "inadequate means of egress," the IEBC sections noted previously indicate that as long as the building was legally occupied before the damage and the repairs will not make the building less complying, specifically with regard to egress, the repairs may be completed without having to meet current egress standards.

It should be noted that the definition of *unsafe* specifically refers to a *dangerous* condition. A *dangerous* condition is determined in the IEBC by a two-part test: either (a) the building or structure has collapsed, moved off its foundation, or lacks necessary support or (b) a significant risk of collapse, detachment or dislodgement of the building, or portion thereof, exists. The structure or element only needs to meet one of the above conditions to qualify as *dangerous*, and it is up to the professional judgment of the professional engineer (or code official) to determine whether the condition is deemed *dangerous*. **Figure 2** presents an example of a roof in *dangerous* condition.

In previous editions of the IEBC, if a *dangerous* condition were discovered during the course of an investigation, it did not matter whether an element was damaged or not; in all cases, the IEBC required that the *dangerous* condition be eliminated. For example, the 2009 IEBC[4] stated in Section 506.1,

> Structural repairs shall be in compliance with this section and Section 501.2. Regardless of the extent of structural or nonstructural damage, *dangerous* conditions shall be eliminated. Regardless of the scope of repair, new structural members and connections used for repair or rehabilitation shall comply with the detailing provisions of the *International Building Code* for new buildings of similar structure, purpose and location.

However, this language has been revised in the Section 302.2 of the 2018 edition of the IEBC as simply "the code official shall have the authority to require the elimination of conditions deemed dangerous."[3] In our experience, building officials will require the elimination of dangerous or unsafe conditions that are made known to them, so the outcome remains essentially the same as the previous, more-prescriptive text would indicate.

The third key takeaway of our analysis of the 2018 IEBC is that a structure requires upgrading if it has sustained *substantial structural damage* or has a dangerous or unsafe condition. The IEBC definition of *substantial structural damage* is complex, but, in general, substantial structural damage is determined by comparing the calculated structural capacities of the vertical structural elements such as

FIGURE 2. Dangerous roof condition with broken and dangling roof members and cladding.

walls and columns before and after the damage occurred. Building enclosure features in- or composing the exterior wall such as punched windows or curtainwalls are not considered part of the main structural system and would not be considered in the determination of substantial structural damage. Therefore, it is unlikely that damage sustained only by the building enclosure elements would exceed the threshold for determination of substantial structural damage.

In structural analysis and design, there are two primary categories of loads that are resisted: gravity and lateral. The gravity load–carrying systems carry their own self-weight and the loads of occupants, partitions, contents, and external environmental loads such as snow, ponding rain, floodwaters, and the vertical components of wind and earthquake loads. The gravity system typically includes most of the columns and structural walls in the structure. The lateral system is designed to resist primarily horizontal or shaking-type environmental loads such as the horizontal components of wind and earthquake loads. The structural elements composing the lateral system are typically a subset of the gravity system's elements—that is, not all gravity system elements participate in the resistance of lateral loads; some simply "go along for the ride." With that distinction in mind, the IEBC[3] generally defines *substantial structural damage* as the condition where any one of the following applies:

- The structural capacity of the vertical lateral system elements in any single story have been reduced more than 33%.
- Where approximately 30% of the vertical elements of the gravity or lateral system have suffered a reduction of 20% or more in their structural capacity.
- Where the structural components carrying snow loads of more than 30% of the roof area have suffered a reduction of 20% or more in their structural capacity.

If the damage sustained is greater than *substantial structural damage*, the building must be evaluated by a licensed engineer to determine whether the structural systems are compliant with current new building standards. Depending on the results of that evaluation, the building may require upgrades to meet the current new construction building code standards (typically the IBC).

With these three main takeaways in mind, consider the following case

FIGURE 3. Warehouse roof

FIGURE 4. Wind damage in the southwest corner of the roof.

studies, which demonstrate how building code requirements can be properly understood and applied to the repair of building enclosure elements.

CASE STUDY 1: LOW-SLOPE WAREHOUSE ROOF

In November 2021, we were engaged to determine the extent of damage and develop a conceptual repair methodology for reported wind damage to a low-slope roof of a warehouse in an industrial complex. A portion (approximately 20% to 25%) of the roof covering and complete roof assembly (which included the metal deck) was missing in the southwest corner the warehouse roof (**Fig. 3** and **4**). Where the full roof assembly was missing, the interior of the building was exposed to the elements and only the supporting open web steel joists remained. The condition of the roof was consistent with damage caused by elevated wind speeds. The roof system consisted of metal deck supporting approximately 1½ in. (38 mm) of mechanically attached extruded fiber insulation, which was overlaid by a layer of fully adhered modified bitumen membrane. On top of the modified bitumen layer was a mechanically attached thin layer of foil-faced insulation and an outer covering of a fully adhered thermoset rubber membrane. The roof system was supported by open web steel joists spanning between wide-flanged steel beams on wide-flanged steel columns.

The extruded-fiber insulation boards were saturated with water throughout the exposed area and at the perimeter extents of the missing roof system. The metal deck, visible from the interior, exhibited discoloration and staining consistent with long-term corrosion throughout the roof (**Fig. 5**). In many locations, the corrosion had advanced through the full metal section, resulting in holes and discontinuities in the deck substrate. In addition, there were multiple unpainted sections of the relatively brightly colored metal roof deck, indicating a history of metal roof deck replacement to mitigate the ongoing corrosion (**Fig. 6**).

Our detailed analysis of the state building code and state existing

FIGURE 5. Typical metal deck corrosion.

FIGURE 6. Replacement deck at right center.

building code, with provisions like those quoted in this paper above, concluded the following:

- The state existing building code governs the repairs to the damaged roof covering and metal roof deck.
- The work to reinstate the damaged portion (approximately 20% to 25%) of the roof is considered a repair ("the reconstruction, replacement, or renewal of any part of an existing building for the purpose of its maintenance or to correct damage").
- Repairs can be completed with like kind and quality of materials, provided that an *unsafe* or *dangerous* condition was not created.

This analysis indicated that the metal deck, roof covering, insulation, and attachments currently in use at the roof could be replaced (throughout the damaged area only) in like kind without being subject to upgrades. In addition, it was unlikely that the work to repair the damaged area of the roof would create an unsafe or dangerous condition. However, the corroded and deteriorated condition of the remaining metal roof deck constituted an unsafe and dangerous condition. It was our opinion that the metal roof deck in its observed condition met the definition of a dangerous condition that warranted full metal roof deck replacement.

To perform a complete repair of the damaged roof area, a portion of the work would have been considered a roof recover because the roof membrane would have been installed over the existing substrates at the perimeter extents of the damaged area. The state building code states that roof recovering is not permitted where the "roof or roof covering is water soaked or has deteriorated to the point that the existing roof or roof covering is not adequate as a base for additional roofing." The roof inspection revealed that the insulation materials within the roof assembly were water saturated at the perimeter of the damaged area. These water-damaged materials would have to be removed and replaced before a potential roof recover repair could take place. The work would include removal of all overlaying roofing materials, essentially leaving only the original metal roof deck in place. However, the original metal roof deck had deteriorated to the point that it was not adequate to serve as a base for additional roofing and would therefore be subject to replacement as well. Based on the observed water-saturated materials at the perimeter of the existing damage and the potential for other puncture damage in the thermoset rubber membrane from windborne debris resulting from the original wind damage, we anticipated that water-saturated roofing materials would be found beyond the extents of the original damage and potentially throughout the subject roof. Where water-saturated materials are encountered, they must be removed. Where those water-saturated materials are removed, there must be an adequate base to attach replacement materials. The existing metal roof deck was not an adequate substrate and would also require removal and replacement. Therefore, the entire subject roof, including the metal roof deck, was at risk for full replacement. A full roof replacement that included the metal deck would have been subject to the new building standards of the current state building code requirements, including upgrades to thermal insulation.

CASE STUDY 2: CONDOMINIUM WINDOWS

In the fall of 2018, a hurricane caused damage to several buildings of a two-story condominium complex located along the Gulf of Mexico's northern coast (**Fig. 7** and **8**). It was reported that repairs to the damaged window assemblies throughout the property would trigger code upgrades of that assembly to meet current thermal/energy, egress, and material provisions. Specifically, upgrades were required because the minimum clear opening required for egress in sleeping rooms had increased since the original windows were installed. A code analysis was performed of the repair and replacement methodology for the various windows and doors throughout the condominium complex to determine which code would govern their repair and/or replacement.

FIGURE 7. Typical condominium building elevation.

FIGURE 8. Typical window configuration.

FIGURE 9. Typical window damage (broken lite and displaced glazing bead).

FIGURE 10. Typical window damage (fogged lite).

When buildings or structures are located in a high-velocity hurricane zone (HVHZ), the state building code requires that building enclosure products meet stringent standards for impacts and wind speed resistance. Because the condominium property was located outside of the specifically designated counties, the special provisions required for a HVHZ did not apply. However, the state building code did require safety glazing and laminated or tempered glass in hazardous locations such as glass in doors, glass immediately adjacent to doors, glass in windows close to the floor that people can walk next to, glass in railings, glass in wet areas such as bathrooms, and glass adjacent to stairways, ramps, and landings.

The local building department specifically stipulated that the state existing building code governed the appropriate repair and/or replacement methodologies for direct physical hurricane damage to the individual glazing lites and window assemblies at the property. The definition of repair in the state existing building code allowed for the renewal or restoration of any part of an existing building to correct damage, including direct physical damage attributable to the hurricane that resulted in failed (fogged), broken, and/or scratched individual glazing lites throughout the property.

Sections of the state building code indicated that replacement glass shall meet the requirements for new construction, which would include safety glazing in hazardous locations, impact resistance in HVHZ, thermal/energy efficiency, and/or egress requirements. However, sections of the state existing building code allow for materials in use to remain in use and for like materials to be used in repairs.

Further, the state existing building code indicate that when conflicts between the codes arise, the provisions of the state existing building code would control over the state building code. It should be noted that the use of like kind and like quality of repair materials was appropriate only where no unsafe or dangerous conditions existed or would be created by the repair. Inspection of the buildings and window assemblies throughout the property did not identify evidence that dangerous or unsafe conditions existed at that time (**Fig. 9** and **10**). In addition, the repair and/or replacement of the windows would not result in the creation of dangerous or unsafe conditions. Therefore, repairs of damaged individual glazing lites would be permitted on a like-for-like basis, subject to the provisions for safety glazing noted previously.

It was also reported that repairs to the window units in sleeping rooms would be required to meet the egress provisions of the Emergency Escape and Rescue chapter in the state building code. However, provisions of the state existing building code indicated that repairs that maintain the current level of egress are permissible. In addition, the state existing building code stated that its provisions control in the event of a conflict between the codes. Further, the stated intent of the state existing building code was to provide flexibility to use alternative approaches to reach compliance with minimum requirements. Finally, provisions in the state existing building code allow repairs to be done in a manner that does not make a building less conforming than it was before the damage occurred. It was not expected that the repair or replacement of individual glazing lites in like kind would make a building less conforming. Therefore, the replacement of failed (fogged), fractured, or scratched glazing lites, whether in a sleeping room or not, would be allowed on a like-for-like basis without being subject to the emergency escape and rescue requirements of the state building code. It should also be noted that upgrades to meet current thermal/energy requirements would also be preempted by sections of the state existing building code.

If it were determined that a window or door assembly warranted replacement in its entirety, that work would be considered to go beyond the definition of repair set forth in the state existing building code. Therefore, that replacement of a window or door assembly in its entirety would be subject to the requirements for new window assemblies in accordance with the state building code, including those concerning safety glazing in hazardous areas, energy efficiency, and emergency egress.

CONCLUSION

In this paper, the overall discussion and the case studies demonstrate the need for design professionals to discern and communicate building code requirements associated with repair efforts to existing buildings. Code requirements for existing buildings can be complex and can vary across different jurisdictions, and project stakeholders often operate on unsubstantiated assumptions that present significant risks to project schedules and budgets. The requirements governing repairs must be established early in the repair design process, with buy-in from all project stakeholders, and they must be revisited periodically as the implementation progresses to completion.

REFERENCES

1 International Code Council (ICC). 2021. *International Existing Building Code*. Country Club Hills, IL: ICC.

2 ICC. 2021. *International Building Code*. Country Club Hills, IL: ICC.

3 ICC. 2018. *International Existing Building Code*. Country Club Hills, IL: ICC.

4 ICC. 2009. *International Existing Building Code*. Country Club Hills, IL: ICC.

RESILIENCE AND THE IMPACT ON ROOFING

ABSTRACT

The industry has a greater focus over the recent years from the industry is resilience, where several industry organizations are taking their personal view on the issue from their specific industry concerns. Because of this, there are several different definitions of what resilience might be. In addition, there is a push to make resilience part of the building code. Put in the simplest terms, a natural disaster occurs potentially damaging the building and to be resilient, the building needs to be operational as quickly as possible. Just like a boxer taking a punch, getting knocked down, but gets right backup.

So how do we develop, specify, and install a resilient roofing assembly?

There are several factors that need to be considered, such as the geographic severe weather, materials, testing, assemblies, specifications, and installation practices all of which should be well-thought-out.

This paper will offer a guide on how the assembly must be durable, sustainable, and resilient, based on geography, training, materials and developing a plan for redundancy within the roofing assembly. With this information, it can be applied into an architectural specification to assist in the resilience of the installation.

LEARNING OBJECTIVES:

- Learn the definitions of Resilience and how these definitions from organizations differ.
- Review of standards and how they can be used to go beyond the building code
- Case examples of real-world projects that incorporate a redundancy with consideration of worse case weather events
- Offer a check list on what to consider based on specific concerns by the building owner and/or the use of the building

SPEAKER

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Brian Chamberlain has been with Carlisle Construction Materials since 1987. He graduated from the University of Wisconsin at Milwaukee, with a bachelor's degree in the science of architectural design. Since joining Carlisle, Chamberlain has been assisting architects, consultants, and specifiers in developing special engineered roofing, focusing on performance and sustainability assemblies. He is part of a team that is responsible for system configurations, details development, and code testing. He has been involved in numerous technological presentations throughout the US, Canada, and overseas. Chamberlain is a member of IIBEC, the Construction Specifications Institute, and the Single Ply Roofing Institute. He has 35 years of experience in the roofing industry.

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Oxford Languages[1] defines resilience as…

1. *the capacity to recover quickly from difficulties, toughness.*
2. *the ability of a substance or object to spring back into shape; elasticity.*

Though these are the definitions of resiliency, they are not specific enough to be applied to roofing assemblies. There are several organizations that define resiliency, such as the US Department of Homeland Security (DHS), who analyze the need for the country to be resilient from natural disasters, as well as manmade disasters, such as war, terrorism, and even unnatural viruses. The National Institute of Building Sciences (NIBS) has the viewpoint of breaking it down to the city infrastructure and preparing that infrastructure to return to operation as quickly as possible. If an unexpected weather event should happen—such as the snowstorm that hit Texas in 2021, causing massive power failure of the electrical grid, freezing all the turbines in the windmill generators—they decide what is required to return to full operational readiness. These and other organizations (**Fig. 1**) continue to work and break down the viewpoint to even finer detail and want to develop a plan that deals with possible structural concerns, such as roads, bridges, and buildings with a grand holistic scale. As these organizations continue to break the view down on resiliency in construction, they begin to look at individual building industries, such as foundations, walls, and roofing.

Thomas Lee Smith's definition for a "Wind-resilient building: A building that is capable of resisting damage from wind and wind-driven rain; furthermore, if damaged, the building can be readily repaired so that important functions are maintained during and/or after a windstorm."

An easy way to understand resiliency is to consider the children's toy Weebles (**Fig. 2**). The selling point of this toy was "Weebles wobble, but they don't fall down." A child can tip the toy over, and can right itself instantaneously. And that is the true focus of the discussion: if a natural disaster weather event should occur, how can we mitigate the damage caused to make the building interior operational and a secure healthy environment for the occupants?

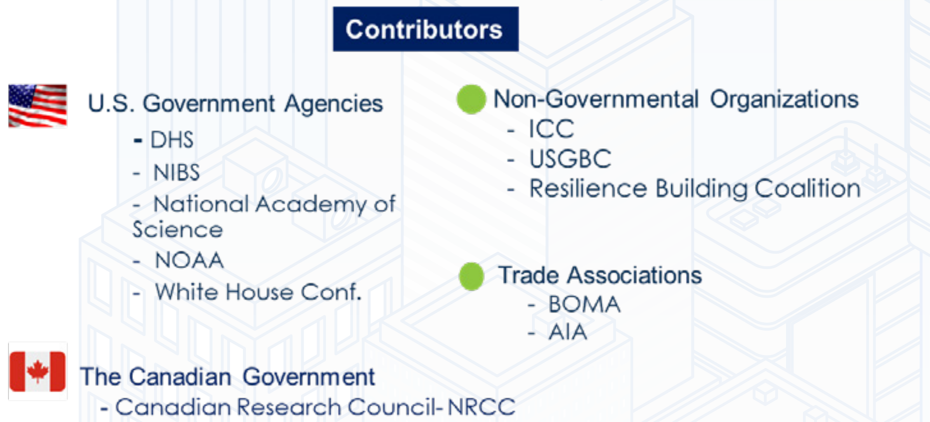


FIGURE 1. Contributors. *(Source: Carlisle Construction Materials)*

FIGURE 2. Weebles. *(Source: Jazwares)*

FIGURE 3. Warehouse failure. *(Source: Carlisle Construction Materials)*

Even with all these definitions, there are a number of ideas on how to best address resiliency in roof design. But well-known speaker Laverne Dalgleish, executive director of the Air Barrier Association of America, once said about the industry in general, "This is not rocket science…it's worse." So not only do we need to consider how we might be (able to design and install a resilient roof, but designers also need to understand some of the issues that impact a roof beyond the actual materials and construction of the roofing system.

For a holistic consideration, the building envelope needs to be designed to withstand the pressures generated by a weather event that can be calculated by following the American Society of Civil Engineers (ASCE) Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7) standard. The first step is to decide what guides we use and what impact they will have.

Do we use the most current adopted ASCE 7 within the state? Do we use the latest published ASCE 7, even though it is not adopted by the state? Do we use the ASCE 7-2022, Chapter 32 (Tornado Loads) calculations, for EF1 and EF2 tornadoes, following the flowchart criteria? Do we use the guides on "Storm Shelters" or "Safe Rooms" from FEMA P-361 or ICC 500, which are focused on protecting the occupants? In all these calculations, a design professional could incorporate greater conservatism by including a safety factor, be it a multiplier creating higher wind resistance results, or reduce the lab-tested uplift ratings, typically by a half? Though the ASCE 7 lists the minimum mean return incident (MRI) based on the risk category of the building, a design professional could choose a longer MRI to increase conservative results. So, do we design for a 10,000-year MRI, a 100,000-year MRI, or a 1,000,000-year MRI, etc.?

Once the external pressures are finalized, is there a way to reduce internal pressures that may contribute to potential failures? Internal pressures can be increased by large openings in buildings and HVAC clean air systems adding pressures. **Figure 3** shows a building whose back wall was penetrated during a tornado; this allowed additional pressure inside the building, which led to further damage.

Once the external uplift pressures and possible internal pressures are determined, the design professional compares those pressures with tested roof assemblies' uplift ratings, which are required to meet or exceed the determined pressures, to confirm the correct roof assembly for the building.

Why are we considering the roofing assembly last? Because addressing the building structure is far more important than the roof assembly that is attached to that structure. If any of the structural systems fails, then the potential failure of the roof assembly increases exponentially. Keeping those concerns in mind, and assuming that they will be taken care of and addressed correctly, we can then focus on roofing assemblies specifically which would need to be resilient to wind, hail, and wind-borne debris.

When looking at resilient roofing, we need to consider five major factors. The first is durability of the products, the second is training, the third is sustainability of design and materials to protect the environment, the fourth would be resiliency by using design redundancy, and lastly the previous four must be balanced with the cost. Cost is easy: it falls under the simple categories of good, better, and best. Where good is meeting the building code, better is going beyond the code, and best is exceeding the building code with consideration of the local geography, local weather, and experience.

Though durability, sustainability, and resiliency are all individual concepts, they overlap each other, and the intersection where all the concepts converge would logically offer the best of the best design considerations (**Fig. 4**). Durability with roofing would be the materials and the science associated with the products such as the single-ply membrane, multi-ply assemblies, cover board, insulation, thermal barriers, adhesives, securements, uplift testing, fire testing, special considerations, etc. Sustainable factors associated with roofing materials such as recyclable products, reusable products, and renewable products are beneficial to the environment. In addition, with goal to reduce impact on the environment from these materials following a cradle-to-grave viewpoint. This would also include factors for the design to have a cradle-to-cradle approach with the roofing assembly itself, in which the material reenters the manufacturing pipeline (be it the original manufacturer or directed to be repurposed and avoid the landfill). The roof assemblies must be designed with materials that are durable in order to make them last longer which will minimize the impact on waste. Adding resiliency, the roof assembly's design must be robust and redundant so the roofing assembly can withstand the impact of an extreme weather event and offer a quick way to have the building return to operation.

DURABILITY

Manufacturers are constantly considering manufacturing techniques to improve products, enhance ease of installation, and upgrade the training of installers. This can be seen in how roofing has changed over the last 35-plus years. In the single-ply industry, as an example, the most commonly used material in the past was ethylene, propylene, diene terpolymer (EPDM), and typically there was a choice of two thicknesses; either 45 or 60 mils thick non-reinforced membrane offered two types of assemblies: a ballasted loose-laid assembly, or an adhered-membrane assembly. Over the years, the industry has now adapted to utilizing reinforced membranes, such as thermoplastic polyolefin (TPO), polyvinyl chloride (PVC), and others. Along with reinforced membranes, there are now exterior non-reinforced membranes, better known as fleece-backed or felt-backed membranes. In addition, the membranes have become thicker. The industry currently has over 250 different options in membranes that can be installed on various roofs. Innovation of materials and the science that accompanies that innovation continues to be developed with a focus on durability, sustainability, and resiliency.

TRAINING

One of the key components in installation is the training of installers. This has become a focus of membrane manufacturers and other organizations such as the National Roofing Contractors Association NRCA which offers a program called ProCertification for contractor training. A poorly installed roof assembly, whether its uplifting rating is 60 psf (pounds per square foot) 300 psf could potentially fail with uplift pressures less than 30 psf. Installation is a critical component of durability, sustainability, and resiliency.

These training programs offer the contractor a way to have their new applicators learn the basics and become certified on different membranes, materials, and installation, with classroom-style classes intermixed with hands-on installation training.

Experienced contractors can also use the basic training as a refresher but then move on to advanced application methods and products, in addition to learning the latest in product technology and labor-saving options.

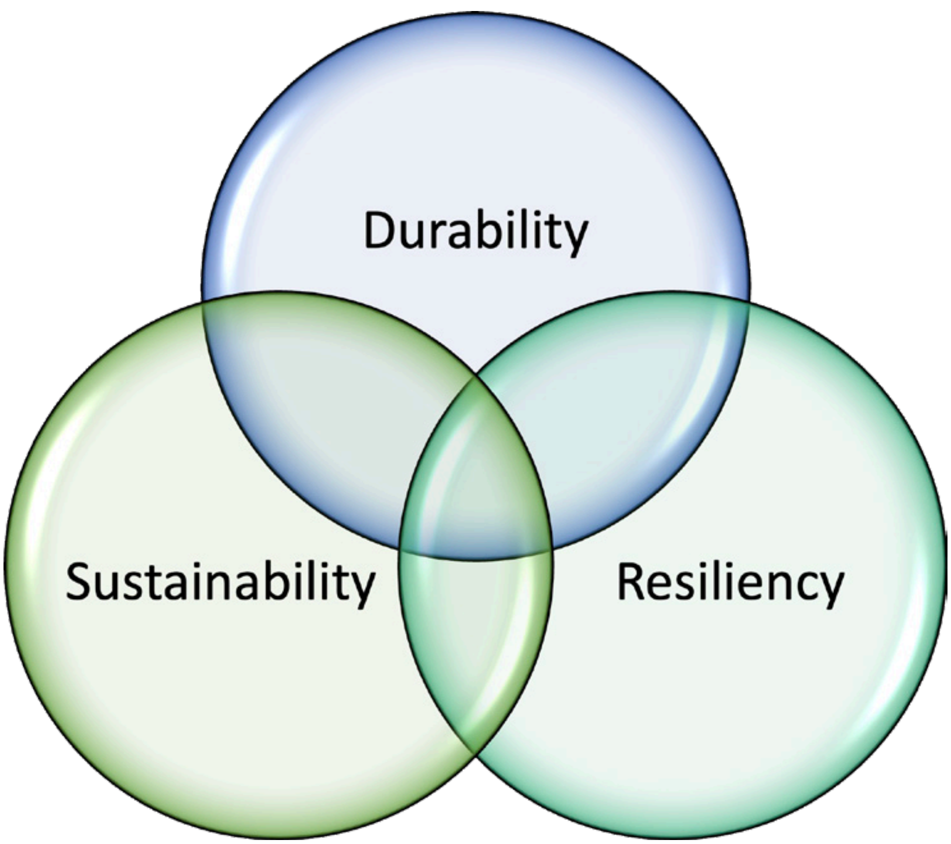


FIGURE 4. Intersection of concepts. *(Source: Construction Materials)*

ROOFING SYSTEM COMPONENTS

After training, the next step in moving to a more resilient roofing system is consideration of the available components and options are associated with durability. It is logical to evaluate an assembly from the deck up with more durable substrate boards. These boards provide a platform for adhering components of the assembly such as air barriers, vapor retarders, or products that do both. These boards are typically secured with fasteners, although adhesives have been used. Fasteners can range from #12 type fastener to a #15 type fastener, with the latter offering a better engagement with the decking material, such as plywood, wood plank, and steel. Concrete decks typically do not need these types of boards.

The next step is the selection of the appropriate insulation, such as the polyisocyanurate with higher compressive strength and coated glass facers that are moisture, mold, and heat resistant. With the proper assembly components, expanded polystyrene (EPS) or extruded polystyrene (XPS), which are also highly moisture resistant. Many options for cover boards and composite boards are available as the top layer of the assembly to add durability and to increase the adhesion strength for the membrane. In addition, a durable cover board will assist in protecting the insulation from normal wear and tear, as well as hail. All these options could be adhered with two-part urethane adhesive installed in ribbons. The Midwest Roofing Contractors Association (MRCA) Technical and Research Committee has shown in their laboratory study that the closer the ribbons' spacing, the greater the uplift performance and adhesion of these boards.

In this same respect, the adhesive used to adhere the membrane has shown to be very strong, whether it is water-

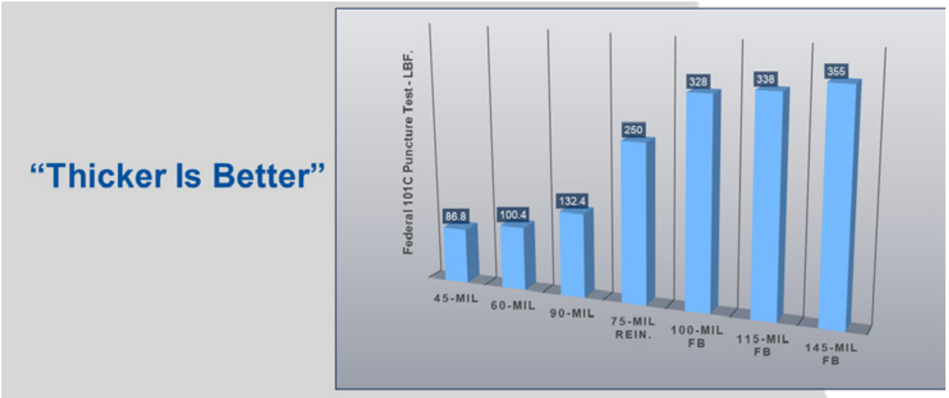


FIGURE 5. Puncture testing. *(Source: Carlisle Construction Materials)*

based, low VOC, or a two-part urethane adhesive with ribbons spaced closely together, keeping in mind that specific adhesive must be used with compatible membrane types and proper training.

Studies have shown that the thicker the membrane is the more durable it will be from puncture, tearing, and weathering. Thirty-five years ago, the industry was flush with loose-laid ballasted systems as an inexpensive alternative to roofing in general. At that time, the hot-commodity material was only 45-mils thick. It did its job, but because of the low puncture resistance, it was susceptible to unintentional damage resulting from human activity. After installation, if rooftop traffic was kept to a minimum, the roof would perform far beyond typical expectation. If it had frequent maintenance on rooftop units, the odds increased that a service person who was not familiar with the roof's limitations could easily damage the roof unknowingly. This would take time and money to find the leak and repair it, so that the roof would still function as it should. Though 45-mil membrane is still used to some extent, 60-mil is preferred, and 80-mil or 90-mil are sometimes considered. Currently there are products that have fleece or felt backing on them that add an exterior reinforcement to its physical properties (**Fig. 5**).

FIGURE 6. Gutter securement failure. *(Source: Carlisle Construction Materials)*

One of the major components that offer the best resistance to wind is the metal edging. Properly designed metal edging and gutters, be it with a continuous cleat, thicker-gauge metal, or even premanufactured metal, all must be tested following the required standards listed in the *International Building Code* (IBC); ANSI/SPRI FM 4435 ES-1 and ANSI/SPRI GT-1 so they meet the expected pressures with proper installation. The photo below (**Fig. 6**) demonstrates what failure might occur when a non-tested gutter is installed.

Wood nailers should be attached around the perimeter edge solidly and include an airtight configuration. After installation of the wood nailer, the membrane should be secured with a robust metal edging that has been tested.

When thinking of the watertight integrity of the roofing assembly for long-term performance, one should consider premanufactured accessories such as pressure-sensitive adhesive applied to flashing, premolded pipe boots, corners, membrane, etc. All these types of products are manufactured in a controlled environment and are installed easily and economically.

Even with a focus on resiliency, we cannot disregard basic design considerations that need to be followed, such as designing the roof to have positive drainage. The building design should include considerations of potential internal moisture and vapor drive concerns with a vapor retarder. Along with controlling the air within the building to avoid having air escape and causing a loss of energy, it is also important to incorporate air barriers and air seal tie-ins. In addition, consider trends within the market that involve enhanced uses of roof surfaces, such as photovoltaics and other overburdens, such as roof gardens, hardscapes, and decorative materials, such as planters and loose-laid stone.

Included in this focus is choosing a basic roofing assembly, be it an adhered membrane assembly, mechanically attached membrane assembly, or a loose-laid ballasted membrane assembly. For all the assembly types, we should consider thicker membranes and redundancy within the seams. For the assembly types that need attachment of the components, a greater securement should be considered, such as reducing the ribbon spacing of adhesives, increasing the fastening density within the boards, and reducing the sheet width on a traditional mechanically attached membrane. To be sufficiently durable to resist hail, the assembly should incorporate an adhered cover board, such as a gypsum-based or high compressive strength polyisocyanurate, to mitigate the impact of the hail. Storm Strips or Peel Stops could be incorporated around the perimeter as a second line of defense if the perimeter edge should fail. These items are not addressed within the building code but offer guidance on how to enhance the wind resistance of a roof assembly to exceed the IBC and standards.

Even though these are the more popular assembly options, we cannot forget the advancement of other types of assemblies that could be durable, sustainable, and resilient, such as air pressure-equalized or vented systems, metal roof panels and sprayed polyurethane foam roofing. With some adhered membranes, the adhesive is already applied to the back of the membrane to offer a consistent layer of adhesive. Fleece- or felt-backed membranes can be adhered with urethane adhesives, water-based

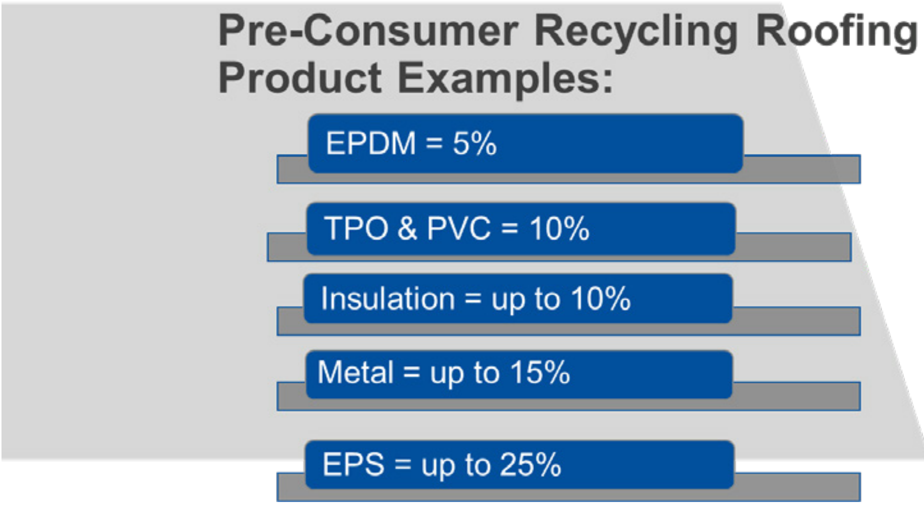


FIGURE 7. Recycled content. *(Source: Carlisle Construction Materials)*

adhesives, or cold-applied adhesives. Hook and Loop membrane, which does not use any adhesives, has shown that after installation, through expansion and contraction, the locking ability within the loops increases in strength.

Though the limited list of options with material and assemblies include durability, they also indirectly incorporate components of sustainability.

SUSTAINABILITY

This concept has also matured and become multifaceted over the years, not only from the aspect of the materials and roofing assemblies, but also from the manufacturing processes, all with a conservation focus and minimizing the impact on the environment.

In addition to utilizing recycled or repurposed material, such as the membranes being reincorporated into the materials, but also makes them last longer and usable and easily repairable in order to delay the eventual removal and replacement of the roofing assembly. Most membranes, insulation, metalwork, etc., have some recycled content to them now, to meet the criteria set forth by sustainable organizations (**Fig. 7**). Even so, the material science including recycled products must not impact the durability factor necessary for long-term performance. The balance between these two concepts is imperative.

Combining life cycle assessment of the roof assembly with cradle-to-cradle manufacturing, when the material or assembly reaches its end of the serviceable life, it can then be reincorporated into other products to again avoid landfills, etc.

We now recycle membranes and insulations (such as polyisocyanurate, expanded polystyrene (EPS), etc.), but manufacturers are not stopping there — they are also developing other recycling methods for all products. There are now cover boards that are manufactured with 100% recycled materials, removing a potential source from post-industrial and post-consumer waste streams.

Even though life cycle assessment and the cradle-to-cradle manufacturing are important, a large portion of sustainability is focused on building energy consumption and savings (**Fig. 8**).

Energy savings is a focus from several organizations, such as Leadership in Energy and Environmental Design (LEED), The American Society of Heating, Refrigeration and Air-Conditioning Engineers (ASHRAE), International Energy Conservation Code (IECC), and the International Green Construction Code (IgCC) to name a few. All have increased R-value within the roof assembly over the years and plan to continue that trend in the near future (**Fig. 9**) based on geographical areas. Basic requirements to make sure that the R-value stays the highest level, is to incorporate multiple layers of insulation with staggered joints. To improve thermal efficiency, the insulation fastener plates are removed from the top layer component and the insulation layer, or the cover board is adhered with adhesives. Blowing agents for the insulation in the manufacturing of the products limited to zero ozone depletion is a sustainable goal.

Certain overburdens on the membrane have proven to offer energy efficiency, such as 14-psf of roofing ballast, which has the equivalent performance as a reflective membrane based on the Oakridge Laboratory study “Modeling the Thermal Performance of Ballasted Roof Systems” and assists the natural process of the earth by absorbing the sun’s heat and releasing it naturally back into the atmosphere. Roof gardens with

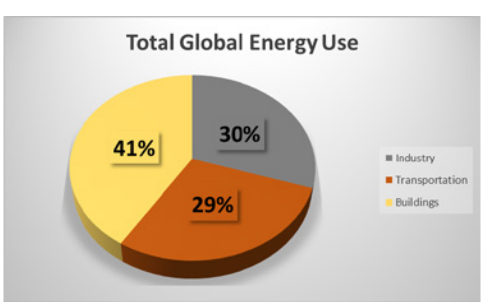


FIGURE 8. Energy use. *(Source: U.S. Department of Energy)*

Minimum R-Value (Non-Residential, Above Roof Deck)

Zones	ASHRAE 90.1				ASHRAE 189.1					IBC – IECC				IGCC			
	2010*	2013*	2016*	2019*	2009	2011*	2014*	2017*	2020*	2012*	2015*	2018*	2021*	2012*	2015*	2018*	2021*
0	-	-	25	25	-	-	-	27.02	27.02	-	-	-	20	-	-	27.02	27.02
1	15	20	20	20	20	20	20	21.9	21.9	20	20	20	20	22.3	21.1	21.9	21.9
2	20	25	25	25	25	25	25	27.02	27.02	20	25	25	25	22.3	26.3	27.02	27.02
3	20	25	25	25	25	25	25	27.02	27.02	20	25	25	25	22.3	26.3	27.02	27.02
4	20	30	30	30	25	25	35	35.08	35.08	25	30	30	30	27.8	31.6	35.08	35.08
5	20	30	30	30	25	25	35	35.08	35.08	25	30	30	30	27.8	31.6	35.08	35.08
6	20	30	30	30	30	30	35	35.08	35.08	30	30	30	30	33.5	31.6	35.08	35.08
7	20	35	35	35	35	35	40	38.98	38.98	35	35	35	35	39	36.9	38.98	38.98
8	20	35	35	35	35	35	40	38.98	38.98	35	35	35	35	39	36.9	38.98	38.98

FIGURE 9. R-values. *(Source: Carlisle Construction Materials)*

the vegetation controlling the energy through a natural plant growing process and the earth media also offer natural R-value.

Photovoltaic (PV) systems installed over a roof assembly are heavily promoted and, in some locations mandated, but the interface of the PV should be with a solar-ready roof. A "solar ready" roof should incorporate items that could offer additional durability to the roof assembly, such as adhered cover boards, redundancy within the seams, thicker membrane, and coordinating roofing installation with the solar installation, making sure that both installer trades work together.

Even though the building code appears to be focused on white surfaces to reflect heat away to reduce the heat island effect, this cannot be a universal solution. I was visiting my primary-care doctor and asked him a simple question. Is a white-colored roofing material more energy efficient than a black-colored roofing material? He responded, "Though this might be a trick question, white-colored roofing material." I said that he was not wrong, but then I asked if could offer me a single pill that will cure all my ills. He said that he could not, for he had to look at my age, my heath, my health history, my family health history, etc., to develop or design the correct medication for me. That is what roof designers need to do: design the roof assembly for the geographic area and the surrounding environment without relying completely on prescriptive thinking.

Reports have been published showing the reflection of sunlight could cause unexpected consequences to the surrounding environment. The Vdara in Las Vegas, Nevada; the 20 Fenchurch Street "Walkie-Talkie" in London, England; and the Museum Tower in Dallas, Texas; are all examples of good intentions by reflecting sunlight, but the results were damaging to the surrounding buildings, cars, artwork, and people. In other situations, reflective membranes have been reported to cause concerns with airline pilots as they try to land planes, and they have also caused residential buildings to overheat, thus requiring additional energy usage to cool the buildings. This type of prescriptive thinking, without considering the consequences of the surrounding environment, is not proper designing.

The best way to address energy efficiency within a building is to consider increasing the R-value of the insulation and incorporating air barriers and details. Incorporating white roofing material, when considered, should be reviewed to make sure sustainability addresses the surrounding environment to avoid needless impacts on others.

RESILIENCY

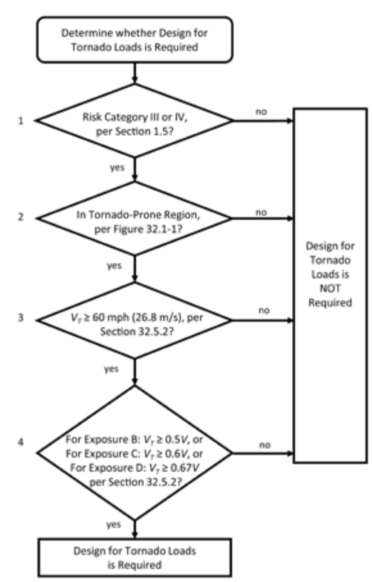
By reviewing and understanding durability and sustainability we have moved even closer to a resilient assembly, but now we need to overlay resiliency with those focuses. Unlike the prior discussions, resiliency in roofing is a mitigation of Mother Nature, so the roof assembly, which protects 90% the worth of a building, can recover after a severe natural event as quickly as possible. The goal is to mitigate the damage by enhancing the roof assembly, so remediation and any repairs of the interior can be quickly handled.

Organizations such as ASCE, Pacific Earthquake Engineering Research Center (PEER), NRCA, ERA, IIBEC, etc. are developing language to be incorporated into the building code to advance resiliency. Though not adopted at this time, ASCE has a *"Prestandard for Performance-Based Wind Design"* as an option. The ASCE has recently published ASCE 7-2022, has a chapter on tornadoes and how to calculate uplift pressures, which is limited to EF1 and EF2 Category type tornadoes (which are more common than EF3, EF4, and EF5 tornadoes) for Risk Category III/IV buildings. FEMA P-361 and ICC 500 have been moving forward for documentation on storm shelters and safe rooms, keeping in mind that their documents are for worst-case event by having a direct hit. But do we need to have this type of construction everywhere? What adopted standard should we use as guidance when we should be focused on a resilient roof?

ASCE 7-2022 offers an idea of how this might be utilized. Within the Tornado Loads chapter, Figure 32.1-2 is a flow chart that goes through four steps in determining whether one should design for tornado loads (**Fig. 10**).

The answer to each step is yes, tornado loads are required.

In addition, ASCE 7-2022 suggests that Risk Category IV tornado hazard maps should be used; although they have a 3000-year MRI associated with them, they do offer extended greater MRIs up to 10,000,000 years (**Fig.11**).



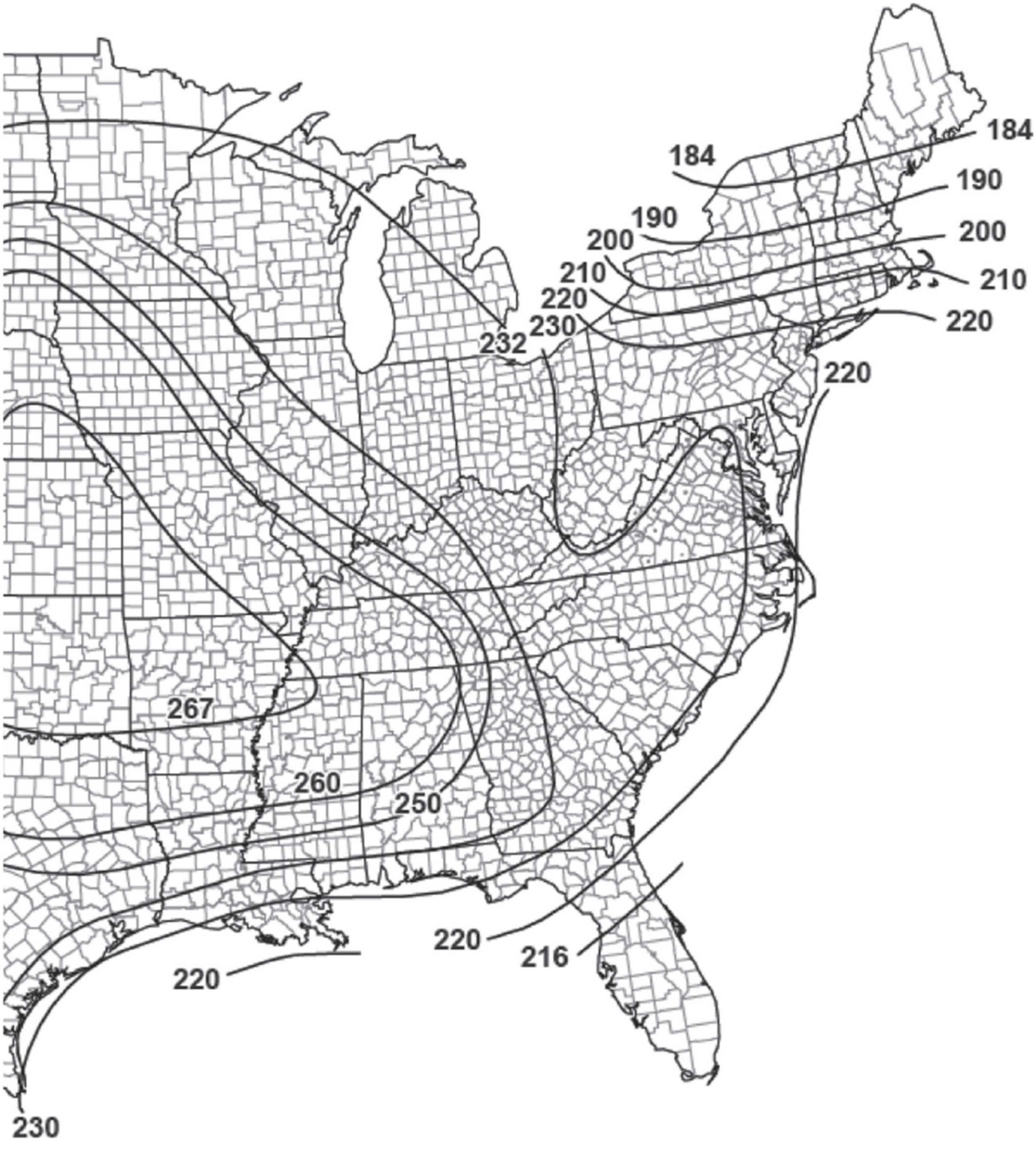
STEP 1:
Is the building Risk Category III or IV defined usage?

STEP 2:
Is the building in a tornado-prone region?

STEP 3:
Are the tornado speeds as shown on their tornado hazard maps, greater than or equal to 60 mph?

STEP 4:
Are the tornado speeds as shown on their maps, greater than or equal to the ultimate winds Risk Category III or IV based on exposure (B [0.5V], C [0.6V], or D [0.67V])?

FIGURE 10. Figure 32.1-2. *(Source: ASCE 7-2022)*



4. Islands, coastal areas, and land boundaries outside the last contour shall use the last tornado speed contour.
5. Tornado speeds correspond to approximately a 0.0005% probability of exceedance in 50 years (annual exceedance probability = 0.0000001, MRI = 10,000,000 years).
6. Location-specific tornado speed is permitted to be determined using the ASCE Tornado Design Geodatabase, available at the ASCE 7 Hazard Tool (https://asce7hazardtool.online), or approved equivalent.

Figure G.2-4E (*Continued*). Tornado speeds for 10,000,000-year MRI for effective plan area of 100,000 ft^2 (9,290 m^2).

FIGURE 11. ASCE 7-2022.

Similar guidance would need to be developed to assist the designer when a resilient assembly should be used with helping to keep people and the surrounding environment safe and healthy.

I had an open discussion with a designer and his engineer on a project in Kansas, where the state had adopted the requirement of a storm shelter to be incorporated in the schools. As such, they had designed the tornado shelter within the general school structure with easy access for all expected occupants. The engineer completed his calculation and the structural components had been determined necessary to make the occupants safe. She did have a concern about the roofing and wanted to know if that also had to meet the same criteria. Her specific concern was, should the roof assembly be installed to meet this pressure, and if so, would it throw off her calculations by incorporating unexpected stress on the structure potentially causing unplanned failure, or should she consider just meeting the building code uplift pressures for the roof assembly? To avoid this potential failure, her thought was to allow the roof assembly to perform normally, and if a weather event were to a direct impact on the school, the roof assembly would be expendable. To mitigate storm waters from entering into the shelter, it was designed to incorporate a drainage system under the decking material to keep the occupants relatively dry. This type of discussion of redundancy and understanding the dynamics helps the resiliency discussion even further.

Even though the building code and standards are the minimum requirements acceptable for roofing performance, designing for severe weather following guides for storm shelters and safe rooms, could be exceeding code requirements, depending on state adoption. This type of design is geographical and needs to be clearly understood in the early design phase. Along with wind, other weather such as heavy snow, torrential rain, and excessive hail needs to be considered based on geographic location.

One of the steps moving forward is that a coalition of the American Institute of Architects (AIA) and NIBS plus 19 other members and code agencies are developing language to be included within the building code and policies to advance resiliency to exceed the baseline building codes. The base of performance would set goals that are more stringent than the minimum standards in the current building code.

A resilient roof will require a system approach rather than just looking at the components of the assembly. The focus must be on a robust design with durable components that addresses the sustainable needs. The designer must incorporate stricter quality control at the jobsite, as well as consideration beyond the roof, such as strengthening roof-mounted components.

Though the design and installation of the roof needs to be robust, consideration of redundancy within the assembly might be necessary. Another example was a project I worked on in the Midwest, where the building was a data center, and the designer wanted to ensure that the building's interior was protected as much as possible. Within the design he had a secondary roof membrane under the primary roof assembly. In the designer's mind the needed to have a stronger attachment to the structural concrete deck versus the primary roof assembly attachment to the secondary. The attachment of the secondary roof was based on following ASCE 7 calculations, but for a wind speed of 250 mph. The primary roof attachment was based on the local wind from the ASCE 7, which was a much lower wind speed. He believed this would offer the most competitive assembly without reducing the resiliency nor jeopardizing the interior.

The primary roofing membrane in a severe weather event might be breached, such as by flying debris, but the secondary would avoid water infiltration into the structure.

There are a number of ways to introduce redundancy within the roof assembly, such as an impermeable roofing membrane, which could be used as an air barrier or even a temporary roof. This could be accomplished over steel and wood decks by installing a thermal barrier secured to the deck to support the air barrier, which then could have subsequent layers of materials to adhere with adhesives. In addition, there could be a secondary drainage system between the base layer of insulation and the vapor retarder and/or air barrier to limit the damage if a section of the roofing assembly were to fail Today there are options of also incorporating "leak detection systems" for early warning or for remediation after a severe weather event.

The insulation and top layer could be adhered to each other with the ribbon-type application of adhesives, removing insulation fasteners and plates being placed directly under the membrane.

The goal with resiliency and redundancy is a way that the building owner would be assured of continuous building operations after an extreme weather event. So, when considering resilience within the roofing design, redundancy of the system offers greater resilience, with the inclusion of the durability of the products and designing for anticipated interruptions but it should also offer simplicity and ease for repairing and recovering quickly.

Resiliency should not be component-based, but rather a collaboration with the design community and ongoing research with sound guidelines to enhance the performance of new and existing roofing assemblies. All the above, in combination with sound contractor training, periodic roof inspections during installation, and then follow up with a regular maintenance program, will assure that the roofing assembly will bounce back quicker than a Weeble.

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RECERTIFICATION RESURGENCE: MITIGATING RISK IN OUR NATION'S OLDER STRUCTURES

ABSTRACT

On June 24, 2021, tragedy struck South Florida as the Champlain Towers South condominium partially collapsed in Surfside, Florida. From the rubble of this collapse, a strong resurgence in demand sparked for building condition assessments and repairs. Most notably, the 40-year recertification process that started in Miami-Dade County has come to the forefront of the building diagnostics industry. The frequency and detail of these assessments have been the subject of much debate within the community; government; and the engineering, condominium, property management, legal, and insurance industries.

The collapse in Surfside triggered changes in the inspection requirements at the county and state levels. Understanding all the recent changes in the assessment and recertification processes will help property managers and condominium owners prevent further catastrophic events and plan for repair and maintenance to prolong buildings' useful service life.

LEARNING OBJECTIVES

- Describe the components of 40-year recertification and changes at the county and state level after the Surfside collapse.
- Identify assessment methods for determining the condition of existing enclosures.
- Outline best practices for the design of new buildings to prevent premature distress.
- Discuss potential code changes and the future of building assessment technology.

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Tricia Fitzgerald, PE, LEED AP, is an engineer and senior project manager in our Diagnostics Group with more than 20 years of experience in structural engineering, building enclosure consulting, and forensic consulting. Fitzgerald's expertise includes evaluating, assessing, and designing repairs for structural systems and building enclosure components. She develops work scopes, repair details, repair procedures, and technical specifications for concrete restoration, waterproofing, stucco and EIFS cladding, and window rehabilitation projects.

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Tarcisio Noguera, PE, LEED AP, WMI, THLV2, is an engineer and senior project manager in Walter P Moore Diagnostics who focuses on existing structures. He has more than 15 years of domestic and international experience in the field of building enclosures and forensic engineering. Noguera expertise includes assessing and designing repairs for distress related to moisture management, roofing systems, and below-grade waterproofing on concrete substrates. He has also developed work scopes, repair details, repair procedures, and technical specifications for roofing, waterproofing, restoration, and rehabilitation projects. Noguera has performed multiple 40-year recertifications to buildings in Florida's Miami Dade and Broward County.

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A building's structure and enclosure are related to its long-term performance. Assessing structural and building-enclosure components with recognized industry evaluation methods is paramount to determine the current condition and identify areas of concern for future performance, and is the core of the 40-year recertification process. Many buildings need significant repairs after being in service for over 30 years. Designers of new buildings can prevent premature distress by thorough planning and detailing of building enclosure systems and by incorporating corrosion protection methods during initial design and construction.

In light of the Champlain Towers South collapse in Surfside, Florida, structural engineers, architects, and enclosure consultants should anticipate related changes to building codes and recertification requirements. These changes will no doubt necessitate more frequent building assessments. Advancements in building assessment technology, including aerial drone surveys and robotic systems capable of automating visual reviews, will aid in the increased demand and help engineers maintain an efficient process while mitigating risk in aged buildings.

EXISTING STRUCTURAL RECERTIFICATION PROGRAMS AND ANTICIPATED CHANGES

What Is a 40-Year Recertification?

The 40-Year (and older) Building Safety Inspection Program was created in 2005 by Miami-Dade County, based on a program established in the mid-1970s and replicated in Broward County, Florida, shortly after. The recertification program calls for safety inspections for buildings that are 40 years old and again every 10 years thereafter. It is intended to ensure that buildings are structurally, mechanically, and electrically safe as they age. Currently, all commercial buildings over 2,000 square feet in Miami-Dade County and all commercial buildings over 3,500 square feet in Broward County are required to comply with the inspection program. **Figures 1 through 4** show typical overview photos for the roof, balcony, façade, and parking garage from a 40 year recertification reports.

What Is Reviewed During 40-Year Recertifications?

Structural Assessment:

- Foundation
- Overall structure
- Masonry bearing walls
- Floor and roof system
- Steel framing system
- Concrete framing system
- Windows, storefront, curtainwalls
- Wood framing
- Facades

  Parking illumination: compliance with Section 8C-3 of the Miami-Dade County Code

  Parking lot guardrail: compliance with Section 8C-6 of the Miami-Dade County Code

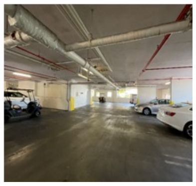
Electrical Assessment:

- Electrical service
- Metering equipment
- Electrical room
- Gutters
- Electrical panels
- Branch circuits
- Grounding of service
- Grounding of equipment
- Service conduit
- Wires and cables
- Thermography inspection
- Emergency lighting
- Building egress lighting
- Fire alarm system
- Smoke detectors
- Exit lights
- Exterior lighting
- Emergency generator

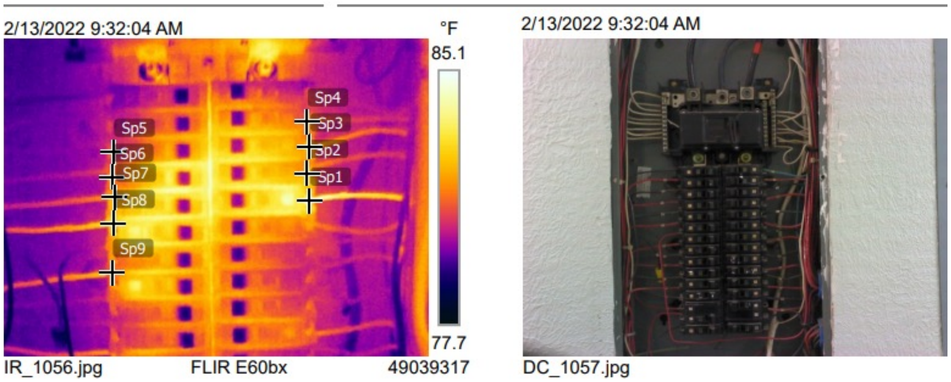
What Are the Benefits?

The benefit of recertification is to reduce the risk of structural, electrical, and mechanical failure and mitigate exorbitant maintenance costs in the future. Obtaining a 40-, 50- or 60-year recertification from a qualified engineering firm helps owners and property managers understand and prevent structural, electrical, and mechanical failures, and create a



FIGURES 1-4. Typical general overview pictures from a 40yr recertification report: Roof; Balcony; Façade; Parking garage.



FIGURES 5 & 6. Typical Thermography and regular image of an electrical panel.

capital asset management plan, which allows property managers a high level of predictability of expenses over the useful life of the asset and ensures they can adequately fund future expenses that prolong the useful service life of the building.

The Collapse of Surfside Tower Triggered Many Changes

In February 2022, Miami-Dade County commissioners unanimously approved an ordinance that revises procedures and strengthens the existing building recertification code, providing increased safety to condominium residents. This motion includes the following:

1. A 30-year inspection (rather than the county's current 40-year mark), and every 10 years after
2. A two-year notice before the structure is to be inspected
3. Provision for an electrical disconnect if the building is declared unsafe

In November 2021, the Miami-Dade County Board of Rules and Appeals approved the revision to this program with the following changes:

1. Adding an infrared thermography inspection for electrical systems operating at 400 amps or greater, performed by a Level II or higher certified infrared thermographer who is qualified and trained to recognize and document thermal anomalies in electrical systems and possesses over seven years of experience in commercial inspections. A typical thermographic image included in a recertification report is shown in **Figure 5**, with the corresponding photograph of the electrical panel shown in **Figure 6**.
2. Adding a new category for building facade assessment; examining the entire exterior facade of a building ensures that various adhered or mechanically attached components do not detach and spall. This new category considers many miscellaneous building components that the previous program did not include, such as exterior walls and appurtenances, chimneys, seawalls, and large sculptures.
3. Adding inspection for structural glazing of curtain wall systems at six-month intervals for the first year after installation. Subsequent inspections shall be performed at least once every five years at regular intervals for structurally glazed curtain wall systems installed on threshold buildings.

What Statewide Changes Are Anticipated?

On May 25, 2022, Florida lawmakers passed the condominium reform package:

1. Statewide recertification inspection requirement applies to all condominium buildings three stories or taller. Those located within three miles of the coast must undergo safety inspections when the building reaches 25 years old, and again every 10 years afterward.
2. Buildings located more than three miles inland will have the initial and follow-up inspection schedule beginning at age 30.

Inspections on all eligible buildings in all Florida counties must be completed by the end of 2024 (**Figure 7**).

What Is Coming Nationwide?

Many cities are reviewing their codes or ordinances and adding or reducing the time to inspect building structures and building enclosures. One example is

FIGURE 7. Typical view of a condominium on the west coast of Florida.

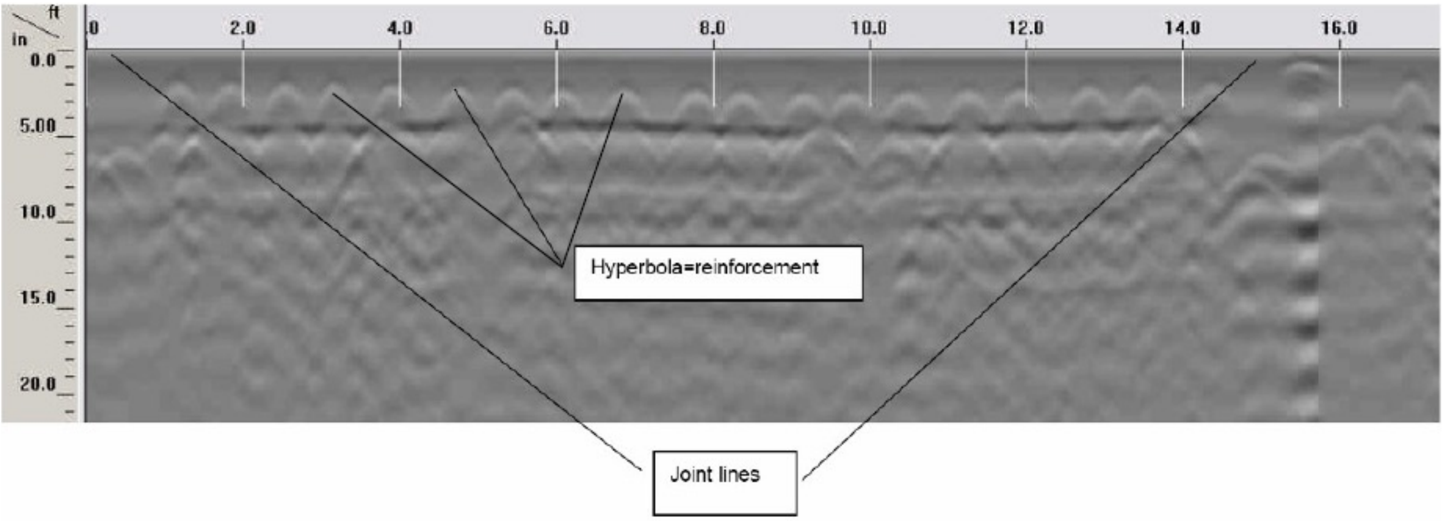


FIGURE 8. Typical output from GPR equipment. *Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.*

Jersey City, New Jersey, ordinance 21-054, Ordinance Amending Chapter 254 (Property Maintenance):

> "As result of the tragic event in Florida involving the collapse of the Champlain Towers South Condominium Association, the City of Jersey City believes it is important to adopt regulations requiring certain buildings to conduct, through a licensed professional, an inspection of the building's structure every ten (10) years and the building's façade every five (5) years."

ASSESSMENT METHODS TO DETERMINE THE CONDITION OF EXISTING STRUCTURES

Dozens of nondestructive testing (NDT) methods are currently available. Some are tried and true; others are still in the early phases of research and development. This paper will discuss those that fall under the tried-and-true category. Even with scientifically tested and proven methods, each method is good for some applications but may be less practical or feasible for a huge number of applications. The effectiveness of the technique used s hall be aligned with the investigation needs. The technique must be scalable to the assessment needs—think about cost, time, risk, quality, and environmental impacts as they are related to the assessment.

As a point of clarification, two similar terms are commonly used in the industry: NDT and NDE (nondestructive evaluation). NDT is the process of gathering data; NDE is the process of interpreting the data to develop a course of action. NDT is often done by a testing lab that may or may not have a licensed design professional (LDP) to interpret the results. On the other hand, NDE should be performed by an LDP to ensure subsequent investigative work, including any recommended repair or renewal management strategies, conforms to local building codes and related design standards.

This paper will discuss several of the most utilized techniques but will not be an exhaustive list of all assessment methodologies used to evaluate the condition of existing structures.

Visual Condition Assessment

A good visual assessment starts with a document review. The more you know about the system/structure before arriving on site, the better you can plan how to assess and what needs assessment.

After familiarizing yourself with the documents, the site assessment is performed. Try to observe as much of the site as possible.

Talking with the maintenance personnel or building occupants will help to identify hidden issues. Determine how many of these hidden conditions need exploratory openings. Are there conditions that need further testing (destructive or nondestructive)? If so, start developing the plan for such testing.

Issue your report of findings. What is the overall condition? What was the scope of your assessment? What was the general condition? Are there any hazardous conditions that need immediate attention?

Nondestructive Testing

Ground-Penetrating Radar

Ground-penetrating radar (GPR), also known as short-pulse radar, generates electromagnetic waves that penetrate solid objects and reflect from interfaces. GPR uses an antenna to transmit short pulses of electromagnetic energy that penetrate the structure. When an interface between different materials is encountered, a portion of the energy is reflected to the antenna. The signal received by the antenna is processed and can be used to determine information such as element thickness, the location of reinforcing steel, and the concrete cover over the reinforcing steel. **Figure 8** shows a typical output from a GPR survey, where the hyperbolas (wavy lines at the top of the image) indicate the location of reinforcing steel in a concrete slab.

Percussive Sounding

Sounding involves striking the concrete surface and interpreting the sound produced.

During a sounding inspection, the inspector typically strikes a hammer or drags a chain on the deck surface

FIGURE 9. Typical view of carbonation results.

to excite an acoustic response from the concrete. This acoustic response, or "echo," is used to differentiate between intact and delaminated areas. Solid concrete will produce a ringing sound, while concrete that is spalled, is delaminated, or contains voids will produce a flat or hollow sound.

Thermography (for Electrical Systems Too)

Infrared thermography is one of the most commonly used NDT techniques in the construction industry. Infrared (IR) radiation is very similar to visible light but lies outside the range perceptible to the human eye. IR thermography is a tool to extend the range of perception to identify things that would normally be invisible.

IR radiation is emitted by an object in proportion to its temperature. Variations in temperature signal variations in conditions that help identify problems. It is important to keep in mind that when an IR camera detects radiation from a source, it is detecting not only emitted radiation but also reflected and transmitted radiation. Because the camera does not differentiate between emitted, reflected, and transmitted radiation amounts, its resulting temperature reading is therefore skewed due to the reflected and transmitted radiation components. Thus, those objects that register a temperature on an IR camera closest to their true temperature are materials with a higher emissivity. Most common building materials are suitable for IR thermography, but special attention is needed when dealing with metals due to surface reflectivity. Following are a few examples of IR thermography in the construction industry.

When looking at a block wall, it can be hard to tell where the bond beams and grouted cells are … but not with IR. The bond beams and grouted cells heat and cool more slowly than the empty cells and are visible in IR.

With IR thermographic scanning, anomalies associated with missing, inadequate, or uneven insulation as well as sources of air and water leakage can be seen and/or detected.

IR scanning can detect

- anomalies that may cause long-term degradation of the building structure;
- "hot" electrical panels, breakers, switches, and wire connections;
- heating and cooling duct placement;
- pipe location; and
- pest infestation.

Destructive Testing

Carbonation Testing

Carbonation is the reaction of carbon dioxide in the environment with the calcium hydroxide in the cement paste. This reaction produces calcium carbonate and lowers the pH to around 9. At this value, the protective oxide layer surrounding the reinforcing steel breaks down and corrosion become possible. The reaction of carbon dioxide and calcium hydroxide only occurs in a moist environment, *and it will be slow in very dry conditions*. In saturated concrete, the moisture presents a barrier to the penetration of carbon dioxide and again carbonation will be slow. The most favorable condition for the carbonation reaction is when there is sufficient moisture for the reaction but not enough to act as a barrier. In most structures made with good-quality concrete, carbonation will take several (or many) years or even decades to reach the level of reinforcement.

The revealer commonly used for carbonation penetration is phenolphthalein. When phenolphthalein comes into contact with high-pH concrete (greater than 10), the solution turns bright pink. When the solution comes into contact with low-pH concrete (<10), the solution shows no color change and the concrete can be considered carbonated. **Figure 9** is an image of a cored sample taken from the soffit of an underground garage roof slab. The depth of carbonated concrete shown with a pink dashed line is approximately ¼ to 7/16 in. (6 to 8 mm) from the topside of the specimen, which means the depth of carbonation has not reached the top bars in concrete.

Carbonation depth was obtained using a phenolphthalein pH indicator solution applied on freshly saw-cut and lapped surfaces of the concrete sample. This testing was part of the petrographic analysis and was performed in a laboratory. Carbonation begins at the surface of the concrete as carbon dioxide penetrates the concrete. The penetration of carbon dioxide results in a low-pH front that advances into the concrete with time.

Chloride Testing

Concrete normally provides reinforcing steel with excellent corrosion protection. The highly alkaline environment in the concrete (pH of 12.5 to 13.5) forms a protective film around the steel called the passive layer. Chloride ions, the major cause of premature corrosion of reinforcing steel, break down the highly alkaline protective film. Chloride ions can be found in the mixed ingredients or can diffuse into concrete from external salting or a marine environment. In the presence of moisture and oxygen, chloride-induced steel corrosion causes concrete cracking, spalling, and staining. In some cases, the corrosion may be severe enough to cause the failure of the structure or facade system.

The risk of corrosion increases as chloride content increases. When the chloride content in the concrete at the level of the steel exceeds a certain limit, called the threshold value, the steel begins to corrode. Current concrete design standards' requirements for reinforced concrete limit the water-soluble chlorides in reinforced and prestressed concrete. Most chloride corrosion problems are the result of external chlorides. In parking garages, deicing salts tracked in by tires and chloride-contaminated drip water from vehicles expose the concrete to chlorides.

Chlorides diffusing into concrete from an external source form a chloride profile. The redline in **Figure 10** shows the chloride profile developed from the chloride testing. The profile typically shows more chlorides near the top, indicating the need to sample the chlorides at the level of the reinforcement, either at the top or bottom layer of steel or both. If the concrete chloride content at the level of steel is at or above the threshold value, which is generally taken as 0.15% chloride ions by weight of cement or approximately 1lb (5 kg) of chloride ions per cubic yard of concrete for a typical concrete mixture design having 600 lb (272 kg) of cement per cubic yard, then steel corrosion can start. How fast corrosion occurs depends on available levels of oxygen and moisture. Keep in mind, changes in chloride ion content imply external chlorides are penetrating at cracks.

The threshold given in the previous paragraph is based on water-soluble chloride ion content. Extra attention is needed when testing core samples taken from existing structures. In concrete structures where calcium chloride was used in the concrete mixture design to accelerate cement hydration and to reduce set time, the acid-soluble test method will dissolve the internal chloride ions, and hence

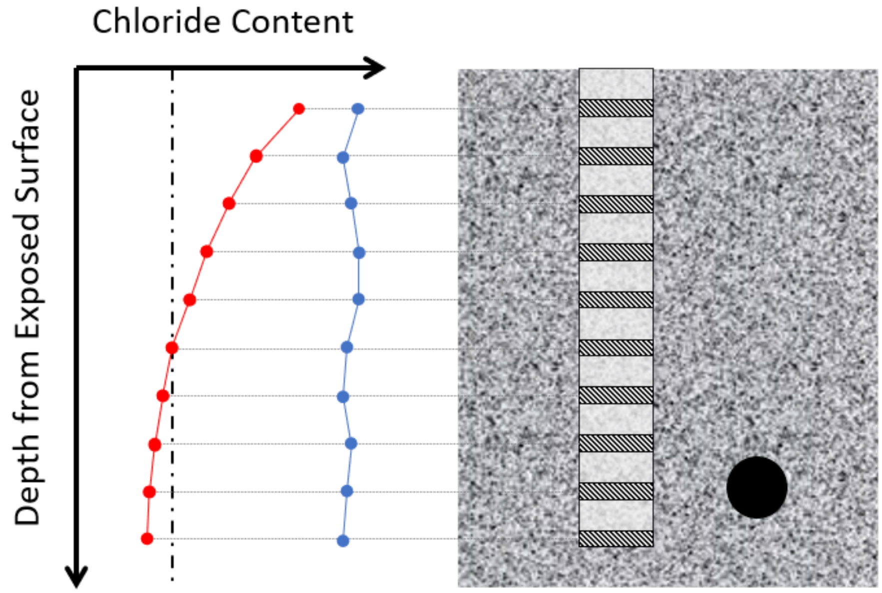


FIGURE 10. Typical results of chloride testing.

the threshold value for steel corrosion will be increased to 0.2% by weight of cement or, for the same cement content of 600 lb (272 kg), 1.2 lb (5 kg) of chloride ions per cubic yard.

To obtain concrete samples for testing, several concrete cores were required to be extracted from the precast concrete slabs or the structural element to a specific depth between 2 and 3 in. (50 and 76 mm). The concrete core samples were placed in a sealed plastic bag for further testing and sent to the laboratory. Testing depths were designated for each inch on the concrete cores. The core samples were cut at the designated depths, crushed, dried in an oven, and processed to pass the U.S.A. Standard Sieve No. 20. Testing was performed following ASTM C1152-20, *Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete*.[1]

Performing visual surveys and conducting destructive and nondestructive testing are useful in assessing the current condition of a building. The results from these assessments guide consultants' and design professionals' recommendations for maintenance and repair plans to extend a building's useful service life. Conversely, when a new building is being designed, consultants, architects, engineers, and owners can improve its performance and longevity by incorporating best design practices.

BEST PRACTICES FOR THE DESIGN OF NEW BUILDINGS TO PREVENT PREMATURE DISTRESS

Licensed design professionals such as architects, engineers, and building enclosure consultants influence the long-term performance of the structure during the design process. The selection of materials and systems to withstand predictable distress during design should be considered for both structural elements and the building enclosure. Most buildings constructed in South Florida and multistory condominiums throughout Florida are reinforced concrete structures. The distress of concrete structures is typically caused by corrosion of the embedded reinforcing steel, which can affect the structural integrity of the building.

Corrosion of reinforcing steel in concrete structures typically occurs due to elevated chloride content; carbonation; or other chemical attacks, such as alkali-aggregate reaction, acid rain, or sulfate attack. For buildings in Florida, elevated chloride content is typically the primary cause of corrosion. This paper will focus on design practices to improve reinforced concrete durability by mitigating reinforcing steel corrosion caused by elevated levels of oxygen, water, and chloride salts in concrete.

Increasing the durability of reinforced concrete or improving its ability to withstand deterioration over its service life is greatly affected by decisions made before and during construction. Concrete mixture design, concrete cover, and corrosion protection of reinforcement affect the long-term durability of a reinforced concrete system and must be considered and specified by licensed design professionals. Concrete mixture design and cover are typically specified by the structural engineer; however, the architect, engineers, building enclosure consultants, and landscape architect all play key roles in the protection of the reinforcement during the design of the building.

ACI 318-14, *Building Code Requirements for Structural Concrete*,[2] published by the American Concrete Institute (ACI), specifies concrete durability requirements in Section 19.3 for most buildings. However, many commercial and multifamily residential structures include parking and plaza or recreation decks, which are often large and provide more exposure to rain, chloride deposits, and temperature variations. ACI also published ACI 362.1R-12, *Guide for the Design and Construction of Durable Concrete Parking Structures*,[3] which includes additional recommendations for parking structures due to their "reduced roofing, cladding, and climate control that result in a more severe direct or indirect exposure to rain, snow, sunlight temperature variations, and airborne chlorides" and "large plan size that magnifies the potential for damage caused by restraint of movements and forces associated with volumetric changes." The licensed design professional should evaluate the design of the structure and implement additional concrete durability requirements from ACI 362.1R-12 for large plaza areas, parking structures, and balconies.

Concrete Mixture Design

Concrete is made up of cement, water, aggregate, and admixtures. Water, aggregate, and admixtures have the greatest effect on chloride concentration in concrete. Reducing the permeability of concrete by adjusting the water–cementitious materials ratio *w/cm* using supplementary cementitious materials, the addition of corrosion inhibitors, and appropriate use of air entrainment to improve corrosion resistance concrete should be considered and specified by the licensed design professional during the design of the concrete mixture.

W/cm is the ratio of the weight of water to the weight of cementitious materials and is key to increasing the durability of concrete. Reducing the water used for the hydration of cement increases concrete compressive strength f'_c and decreases permeability. Reduced permeability slows the rate of flow of water, oxygen, and chlorides through the concrete, thus slowing corrosion initiation.[4] The licensed design professional should evaluate water-reducing admixtures that increase workability but do not increase the chloride content of the concrete mixture or contribute to increased cracking of the concrete surface.

Supplementary cementitious materials such as silica fume, fly ash, and slag cement can improve pore distribution, reduce permeability, and increase the electrical resistivity of the concrete. Silica fume has a reduced permeability and, when used, reduces the permeability of the concrete. The lower permeance increases the electrical resistance of the concrete mixture, which can lower the rate of corrosion once initiated. Fly ash can be used to reduce the heat of hydration, which is important during hot-weather placement, but it reduces early compressive strength gains. Concrete containing moderate to high amounts of fly ash has been shown to improve resistance to chloride penetration. Slag cement has properties similar to fly ash and chloride resistance is improved with its use.[5]

Corrosion inhibitors can be added to the concrete mixture to improve the durability of the concrete. The structure type and exposure to chlorides should be considered when determining whether to include a corrosion inhibitor in the concrete mixture. For example, parking garages, elevated walkways, balconies, and sea walls are typically exposed to elevated amounts of chlorides in structures located close to large bodies of salt water. Licensed design professionals should consider the use of corrosion inhibitors to slow the onset of initial corrosion of reinforcing steel when designing the concrete mixture for these elements.

The most commonly used corrosion inhibitor is calcium nitrite, which delays the onset of corrosion or slows the rate of corrosion that has been initiated. Calcium nitrite also accelerates the setting time of the concrete and increases early compressive strength.[5] If setting time acceleration is not desired, ACI 201.2R recommends using an air-entraining admixture. However, while air entrainment is recommended for concrete placed with exposure to freeze-thaw cycles, it increases permeability and may not be suitable for concrete exposed to airborne chlorides but not freeze-thaw cycling.

Water used for concrete mixing should not contain elevated levels of chlorides. ACI 222.3R-11[4] recommends limiting the chloride content of mixing water per ASTM C1602, *Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete*.[6] This standard limits the chloride content of mixing water to 1000 parts per million for concrete in moist environments or with the embedment of dissimilar metals.

Construction in South Florida in the 1960s often utilized beach sand for small aggregate, which contributes to elevated chloride content and corrosion of reinforcing steel. This aggregate is no longer used, but licensed design professionals should still consider the chloride content of aggregates in their mixtures and aggregates should not contain water-extractable chlorides. Table 19.3.2.1 of ACI 318-14 limits the maximum water-soluble chloride ion content in concrete for prestressed and nonprestressed concrete for Exposure Category C, corrosion protection of reinforcement. The chloride ion content for prestressed members does not vary depending on protection from moisture.[2]

Concrete Cover

Concrete cover over reinforcing steel is critical to reducing corrosion. Corrosion damage observed in new structures is often caused by insufficient concrete cover.

For nonprestressed members exposed to weather or in contact with the ground, ACI 318-14, Table 20.6.1.3.1, specifies a 2 in. (50 mm) cover for reinforcement bars No. 6 through 18 (19M through 57M), and 1½ in. (38 mm) cover for bars No. 5 (16M) and smaller, including W31 or D31 wire and smaller. ACI 117-10, *Specification for Tolerances for Concrete Construction and Materials*,[7] includes a reinforcement tolerance of ±3/8 in. (10 mm) for members ranging in thickness from 4 to 12 in. (101 to 305 mm) and ±½ in. (13 mm) for members with a depth greater than 4 in.

For prestressed, cast-in-place concrete members exposed to weather or in contact with the ground, ACI 318-14, Table 20.6.1.3.2, requires 1 in. (25.4 mm) of cover over reinforcement for slabs, joists, and walls and 1½ in. (38 mm) cover for all other structural elements. ACI 117-10 provides horizontal and vertical tolerances for the placement of prestressing reinforcement but does not separately address nonprestressed reinforcement in a prestressed concrete member. Therefore, reinforcement tolerances between 3/8 and ½ in. (10 and 13 mm) would be used.

The reduction of clear cover for reinforcement in prestressed members specified in ACI 318-14 combined with the allowable tolerances of ACI 117-10 allows the reinforcement to be placed ½ to 5/8 in. (13 to 16 mm) from the top of the concrete surface. The highest chloride content is typically located in the top ½ in. of concrete and increases in depth over time. Table 6.3.1.6b in ACI 362.1R-12[3] recommends 1½ in. (38 mm) clear cover for the top of slabs, beams, columns, and walls, regardless of chloride exposure. Licensed design professionals should consider increasing clear cover as recommended by ACI 362.1R for elements such as balconies and plaza decks that are exposed to weather, even if additional weather protection is planned.

Designing a concrete mixture to reduce permeability and specifying concrete cover to provide protection are the two most important factors in concrete structure durability, which is usually specified by a structural engineer. Additional corrosion protection of concrete structures includes drainage, waterproofing membranes and sealers, protection of embedded elements, and exterior wall design.

Corrosion Protection of Reinforcement

Drainage

For horizontal concrete surfaces exposed to weather, drainage to prevent ponding of water should be designed and specified on the construction documents by the design team. Water on slabs of parking, plaza, and roof decks should provide a positive slope to floor drains without water ponding. High and low points should be clearly identified on plans and crickets to redirect water around columns or other obstructions. Water should also be directed away from slab edges, stairs, and elevators. Additionally, construction and expansion joints should be located at high points of the slab to prevent water ponding at the joint. Other locations that require positive drainage of the concrete surface include balcony slabs, concrete windowsills, tops of exposed walls, and parapet beams.

Design of Balconies and Walkways Exposed to Chlorides

Unprotected balconies and exterior walkways commonly require concrete restoration after around 30–40 years when located in a corrosive environment. To slow corrosion initiation and progression, the licensed design professional should incorporate recommendations for the concrete mixture design and concrete cover discussed earlier. Additional protection for the concrete can be provided by providing a positive slope to prevent ponding water. For post-tensioned concrete structures, the curling of the slab should be considered when specifying the minimum slope. Installation of waterproofing membranes should be considered when evaluating the life-cycle costs for the building. These membranes reduce chloride, oxygen, and water contact with the concrete surface, thus reducing exposure to corrosion.

Moisture Protection

Structures must be protected from water and air intrusion for building occupants' comfort and to protect the structural integrity of the building. Roofing assemblies should be designed for wind pressure, but also with enough slope to quickly drain water from the surface. The licensed design professional should coordinate the termination of the roofing assembly with the air and weather barrier for the exterior wall. Gaps between these systems are guaranteed leak locations.

Expansive elevated plaza decks with landscaping, parking, and recreation elements are common in multifamily construction. A waterproofing membrane applied to the top of the structural slab is typically specified to protect the concrete. These systems deteriorate over time and the replacement of a buried waterproofing membrane is extremely disruptive and costly. The licensed design professional should discuss the long-term performance goals to guide the selection of waterproofing materials to meet those requirements.

When designing plaza decks, the licensed design professional must coordinate the thickness of finishes, waterproofing assembly depths, and the slope of the structural slab and topping slab, if applicable. The deck should slope to drains and not to doors, elevators, expansion joints, or slab edges. A minimum slope of 1% to 1.5% is recommended. While many waterproofing manufacturers claim that their product can withstand constant ponding water, this is not good practice, and failures are often observed when waterproofing is constantly submerged.

Exterior Walls

Exterior walls must protect the structure from the air, water, and wind. Air and weather barriers installed behind an exterior veneer should be designed to drain water that bypasses the veneer from the wall assembly so that water does not bypass the weather barrier. Section 1403.2 of the *Florida Building Code*[8] provides an exception to this requirement: "A weather-resistant exterior wall envelope shall not be required over concrete or masonry walls designed in accordance with Chapters 19 and 21, respectively." Most exterior wall construction in Broward and Miami-Dade counties and for structures located near the coastline consists of reinforced concrete masonry. These are typically infilled walls placed between concrete slabs and beams when used in a reinforced concrete structure. Detailing of the joints between dissimilar substrates, such as concrete and concrete unit masonry, should be coordinated and designed by licensed design professionals to provide a continuous weather barrier that accommodates shrinkage of concrete materials, temperature variations, and differential structural movements. Stucco is usually continuously applied to exterior walls and cracks develop. If a continuous weather barrier is not present behind the stucco, water will migrate through the cracks in the stucco and into the exterior wall and structure.

Windows and Doors

Windows and doors create vulnerabilities in exterior wall assemblies that can introduce water into the building and create corrosion-related damage. Windows and doors must be anchored to structural components to withstand wind pressure and windborne debris in hurricane-prone regions. Code changes in 1994 to the *South Florida Building Code*[9] due to Hurricane Andrew in 1992 caused the market to redesign windows and doors to meet these structural load requirements. Windows and doors are typically anchored through the jamb, head, and sill rather than using fin anchorage. The anchorage condition requires the licensed design professional to properly coordinate flashing, shim spaces, perimeter sealant, and exterior cladding.

Windows and doors in Florida typically utilize aluminum frames with either a powder coat or Kynar finish. Concrete screw anchors are the most common fastener used to connect the windows and doors to the structure. These screw anchors are made of carbon steel and typically have a corrosion protection finish. Damage to the corrosion protection can occur during installation of the screw and deterioration of these screws is commonly observed within five years of installation. The carbon steel screw and aluminum frame are dissimilar metals and galvanic corrosion occurs when they are in contact. Utilizing marine-grade stainless steel fasteners to anchor windows and doors is recommended to reduce the galvanic corrosion potential. Further sealing the fasteners to the frame to reduce oxygen and water vapor exposure can provide additional corrosion protection to the screws.

Flashing of openings is required by the code to "prevent moisture from entering the wall or to redirect that moisture to the exterior."[8] Because windows and doors are often recessed into openings, flashing design typically consists of fluid-applied or self-adhered membranes applied to the perimeter of the opening. This does not prevent water that bypasses the perimeter seal from entering the interior space, causing damage to interior finishes, ponding water on concrete surfaces that may not be designed for corrosion resistance, and causing corrosion or deterioration of exterior framed walls and sheathing. The licensed design professional should consider adding pan flashing at the sill to redirect water to the exterior in addition to the flashing provided at the opening.

Properly designed and installed perimeter sealant joints are critical to preventing water intrusion to the interior of the building and the protection of the structure. Shim spaces for many windows are limited to a maximum of ¼ in. (6 mm) per their product approval drawings. Additionally, aluminum extrusions often do not have adequate extrusion in the joint for proper bond width. ASTM C1193, *Standard Guide for Use of Joint Sealants*,[10] provides guidelines for joint configuration. The bond line for butt and fillet sealant joints is typically equal to the joint width. However, ASTM C1193 does not recommend the installation of a liquid sealant in joints less than ¼ in. wide. The licensed design professional must reconcile the joint width, minimum bond line, and interface with other exterior finishes to ensure that properly configured sealant can be installed.

The licensed design professional must also coordinate the termination of exterior finishes with the windows and doors. Portland cement stucco is commonly used throughout Florida for the exterior wall finish. The termination of the cementitious stucco must not contact the aluminum frame because this can cause aluminum corrosion. When aluminum corrodes, it does not expand like steel and this condition can be concealed for a long time, resulting in severe loss of aluminum cross section and loss of structural integrity of the window or door frame.

Embedded Elements

Embedded elements cast into or connected to reinforced concrete members should be designed to resist corrosion. Aluminum railing posts are commonly used throughout Florida and can be grouted into recessed pockets or anchored to the top of the concrete surface. When embedded, the post pocket locations should be carefully coordinated with the reinforcement placement to avoid a reduction in concrete cover. Aluminum should also be protected from direct contact with concrete to prevent corrosion.

POTENTIAL CODE CHANGES AND THE FUTURE OF BUILDING ASSESSMENT TECHNOLOGY

Code Changes

Building codes are generally written to address new building construction. While corrosion protection mitigation methods are included in ACI 318-14, licensed design professionals should consider additional protection if the building exposure warrants more protection for long service life. Building enclosure components and systems also protect structural elements and proper installation of these elements is critical to preventing water ingress that causes deterioration to the structure. Coordination of all the systems is key to success and should be performed by the design team; however, many owners also utilize building enclosure commissioning to implement a plan to coordinate enclosure systems during design and review the quality control and quality assurance of the design team and contractor during construction.

Proper installation, maintenance, and periodic replacement of building enclosure components, including waterproofing, roofing, sealants, paint coating, exterior cladding, windows, and doors, are necessary to protect the structural integrity of buildings. Current building codes do not require design professionals to specify periodic maintenance or assessment of buildings. Warranty requirements generally drive the owner's maintenance and replacement plan. These maintenance requirements should be provided to the owner by the contractor after construction. However, periodic assessments of the building enclosure by licensed design professionals can help owners plan and budget for the repair and maintenance of these systems to extend their service life.

Future Condition Assessments

A discussion among engineers, owners, property managers, local and state officials, and others is ongoing in Florida. While new legislation has been passed requiring earlier condition assessments for condominiums located in more corrosive environments, other commercial buildings and multifamily residential properties located in the same environment were not included. Some condominium owners are selling all building units to developers to terminate their condominiums and avoid compliance with the new state assessment requirements. There are many additional questions that deserve exploration, including the following:

- Should property owners continue to be trusted with the responsibility of assessing their structures and performing timely repairs?
- Should local or state governments require condition assessments? Who defines the evaluation criteria?
- Will all assessments be of the same quality and detail?
- If repairs are recommended, is the owner required to perform the repairs? Can owners delay implementing repairs?
- Can owners obtain multiple assessments if they disagree with the original assessment or desire a second opinion? If opinions differ, who determines what repairs are required?
- What experience and qualifications are required by the engineer or architect performing the inspection?
- Will architects and engineers assume liability for existing structures by performing condition assessments?
- Are there enough qualified licensed design professionals in Florida to perform these assessments?
- Should all commercial buildings be assessed as required by Miami-Dade and Broward counties?
- Should the severity of airborne chloride exposure affect how often structures are evaluated?
- Should construction type affect how often structures are evaluated?

Licensed design professionals experienced in performing condition assessments and repairs should be actively involved in discussions with all interested parties to educate and advocate for changes to improve the maintenance of existing buildings and continue to protect the health, safety, and welfare of the occupants.

Conclusion

As our building inventory continues to age, extending the service life of existing structures is sustainable and can be economically advantageous to building owners. Evaluation of concealed structural elements continues to challenge licensed design professionals when assessing the building. Owners would benefit from early leak and corrosion detection. When found early, repairs will likely be isolated and smaller, reducing building life-cycle costs to the owner. Current technology exists for continuous leak detection for roofs and waterproofing assemblies. Could additional technology be developed and installed at the perimeters of windows and doors to detect leaks early, before biological growth initiates on interior surfaces? Continuous corrosion monitoring is being studied, and if the technology is developed and brought to market, it will provide an excellent tool to assist owners and engineers in better monitoring reinforced concrete structures.

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DURABILITY, THE FORGOTTEN PILLAR OF SUSTAINABILITY

ABSTRACT

Building envelope durability is essential for limiting critical structural and envelope failures that could have significant operational, financial, or comfort- or health-related implications on the building and its occupants. Building envelope durability is also important to the overall resilience/sustainability of buildings — less durable systems need to be replaced or repaired more often, leading to additional operational energy use, greenhouse gas (GHG) emissions, and environmental impact to perform those repairs. Moreover, with the increasing frequency and severity of major weather events due to climate change, it is crucial that engineers stay up to date on new requirements being adopted in building codes and technical standards to address these changes. This paper is a review of recent literature exploring the relationship between the durability and sustainability of building envelope systems. Life cycle assessment (LCA) research exploring the relationship between product durability/service life and life cycle impact is summarized and, where possible, implications and considerations for building envelope designers are highlighted. Relevant standards and best practices are discussed for the benefit of designers looking to design for durability.

LEARNING OBJECTIVES

» Describe the literature exploring the effect of building envelope durability and service life on building LCA results, including life cycle GHG emissions.

» Examine the trade-off between reducing embodied impact and improving building envelope durability.

» Evaluate the effects of climate change and worsening weather events on building envelope durability.

» Explain recommended design practices to improve the durability and resilience of building envelope systems.

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INTRODUCTION

The term *durability* can often be vague in the context of buildings. Early researchers studying durability in buildings identified that durability of a whole building cannot be measured or assessed, and that it is much more important to consider the service life (SL) of the building's components (Legget and Hutcheon,1959). Thankfully, the topic of building element durability and service life (SL) has since been further researched and is better understood. Durability is defined in *CSA S478:2019,* – Durability in buildings, as: "the ability of a building or building element to perform its functions to the required level of performance for its design SL in its structure environment under the influence of environmental actions." (CSA, 2019). In this context, a building refers to the entirety of a building; whereas a building element is defined as "a material, component, or assembly used in or forming part of a building," and is the unit of analysis considered when designing for durability. This is a more useful definition because it introduces the concepts of design SL, structure environment (outside and inside influences to which building elements are subjected and that can act as agents causing environmental actions), and environmental actions (chemical, biological, and/or physical action causing degradation of a building element) (CSA 2019).

One aspect that is not immediately evident in this definition, however, is the importance of durability or SL to the overall sustainability of a building or building element. This relationship seems intuitive upon reflection—durable building components will last longer before requiring replacement, and require less maintenance throughout their SL, reducing overall impact over the life cycle of the product.

Several researchers have previously investigated the effect that building element durability and SL have on the overall environmental impact of a building and its components. Although research has generally found that increased building SL (Aktas and Bilec 2012; Marsh 2017; Palacios-Muñoz et al. 2019b) and building component SL (Grant et al. 2013; Grant et al. 2014; Morales et al. 2022) favorably impact important metrics including life cycle cost, energy use, and global warming potential (GWP), there are many nuances and uncertainties related to this research and described further in this paper, which are important for designers to understand when making decisions related to building envelope durability. To highlight this important finding and further discuss these nuances, this paper will serve as a review of the prior research investigating the relationship between durability and environmental impact. To empower designers looking to implement best practices in design for durability, this paper will also provide an overview of important concepts, standards, and best practices related to building envelope durability, as well as the implications of climate change on the durability of buildings.

LIFE CYCLE ASSESSMENT

Life cycle assessment (LCA) is widely recognized as the most complete tool for quantifying the environmental impact of the production, use, and end of life of a material or product (Asdrubali et al. 2013; Junnila and Horvarth 2003). The theory and procedures for completing an LCA of a product are specified in ISO 14040—Environmental Management—Life Cycle Assessment—Principles and Framework (ISO 2016a) and ISO 14044—Environmental Management—Life Cycle Assessment—Requirements and Guidelines (ISO 2006b).

In brief, LCAs assess the potential environmental impacts associated with the entire life cycle of a product (cradle-to-grave). In general, the LCA process involves compiling an inventory of relevant inputs and outputs associated with a product and evaluating the potential environmental impact associated with those inputs and outputs. Impacts are typically categorized into three kinds: resource use, human health, and ecological health. The methodology for conducting an LCA typically includes the following steps (ISO 2016a):

- Definition of goal and scope
- Life cycle inventory analysis
- Life cycle impact assessment
- Life cycle interpretation
- Critical review (optional)

Defining the goal and scope of the LCA is intended to clarify the need for, intended application, and audience of the study. Life cycle inventory analysis involves collecting data and calculating the relevant inputs and outputs of the product. Life cycle impact assessment involves evaluating the significance of potential environmental impacts related to those inputs and outputs. Life cycle interpretation combines the findings of the two previous steps to make conclusions or recommendations related to the study goal. Critical review can be done to ensure that the LCA has been done in accordance with the requirements of ISO 14040.

It is worth noting that ISO 14040 states some limitations of the LCA methodology. These include the subjective nature of the some of the choices made; statistical modelling limitations; potential inaccuracy compared to local conditions; limited access to relevant data; and uncertainty introduced in impact assessment due to lack of spatial and temporal dimensions (ISO 2006a).

BUILDING LIFE CYCLE ASSESSMENT AND DURABILITY

Thankfully, building LCA research has started to address some of the limitations inherent to the generic LCA methodology. For example, many researchers have proposed methodologies to account for the temporal nature of buildings and other long-lived products (Grant et al. 2013; Palacios-Muñoz et al. 2019b; Pan et al. 2018; Toufeili et al 2022). And according to a recent literature review paper by Feng et al. (2022), a search for "life cycle assessment" returns 427 results, indicating that there has been significant research within this field.

Additionally, several standards have been developed for assessing the sustainability of entire buildings including *EN 15978,* – Sustainability of construction works – Assessment of environmental performance of buildings – Calculation method (BSI 2011), and EN 15643, – Sustainability of construction works – Framework for assessment of buildings and civil engineering works (BSI 2012). These documents provide standardization to the process of building LCA, improving the reliability of comparative LCA studies (Feng et al. 2022). For example, EN 15978 specifies the standardized stages for a building LCA:

- Product Stage
  - A1 – Raw Material Supply
  - A2 – Transport
  - A3 – Manufacturing
- Construction Stage
  - A4 – Transport
  - A5 – Construction
- Use Stage
  - B1 – Use
  - B2 – Maintenance
  - B3 – Repair
  - B4 – Replacement
  - B5 – Refurbishment
  - B6 – Operation Energy Use
  - B7 – Operation Water Use
- End-of-Life Stage
  - C1 – Deconstruction Demolition
  - C2 – Transport
  - C3 – Water Processing
  - C4 – Disposal

Despite the recent advances in building LCA research, the field is still relatively young and faces many challenges that lower the reliability of LCA results (Feng et al. 2022; Hoxha et al. 2016; Morales et al. 2022). A recent study by Feng et al. (2022) critically reviewing the building LCA literature identified three primary challenges affecting the reliability of LCAs:

- Variances on goal and scope definition
- Building structure complexity
- Use of different LCA databases and methods

Their respective recommendations for addressing these challenges included: applying more recent building standards (e.g., EN 15978 and EN 15643) and whole-building LCA approaches (i.e., International EPS, Athena Sustainable Material Institute); integrating Building Information Modelling (BIM) into LCA procedures; and promoting environmental product declaration to reduce uncertainties. Regarding the use of different databases, Ramesh et al. (2010) provide a summary of commonly used data sources for building LCA.

Of significance to the discussion of building durability and SL, Grant et al. (2013) identified that many LCAs make a variety of simplifications with regard to building SL. For example, many researchers have noted that building LCA studies assume a wide variety of building lifespans or reference study periods (RSPs), usually ranging between 50 to 100 years, but that the typical practice is to use a 50 or 60-year SL (Grant et al. 2013; Morales et al. 2022). Authors have argued that this standard practice may not appropriately assess the impact of relatively long-lived building elements, as it may not adequately account for the reduced use stage impacts of building elements that surpass the study period (Grant et al. 2014; Morales et al. 2022; Palacios-Muñoz et al. 2019a).

Research has also shown that LCA results are influenced by building lifespan, with many studies showing that longer building lifespans result in lower environmental impacts (Aktas and Bilec 2012; Marsh 2017; Palacios-Muñoz et al. 2019b). Marsh et al. (2017) found that increasing the lifespan of a Danish building from 50 to 100 years could result in a 38% reduction in environmental impact. These findings highlight the importance of encouraging the design of buildings with long-design SLs as a sustainable design strategy (Azari 2014) and considering the impact of SL assumptions in LCA studies.

On the other hand, many buildings are demolished before they reach the end of their design SL. A new owner may decide to demolish an existing building before it reaches the end of its SL. Marteinsson (2005) found that only 17% of building demolitions were due to deterioration. This highlights the importance of considering human factors in LCA research and building design (Li et al. 2020), considering the intended design SL of a building during its design (CSA 2019), and also designing for adaptability, so that buildings are able to meet the ever-changing requirements that we subject them to. Current research has not settled on the optimal building lifespan from a life cycle perspective, but it is generally agreed that building SL modelling should be done methodologically (Grant et al. 2013).

Furthermore, many LCAs have greatly simplified the maintenance, repair, and replacement requirements of building material and systems over time, otherwise known as SL estimation. Others have completely excluded the impact of material maintenance and only considered energy use related to heating, cooling, lighting, and equipment during the life cycle of the building. This is relevant because LCA studies have shown that maintenance can account for 4–25% of the life cycle impact, depending on impact category (Grant et al. 2013). Several studies have shown that increased building envelope SL and durability can significantly reduce the overall impact during the building use stage (Grant et al. 2013; Grant et al. 2014; Carlisle and Friedlander 2016). This demonstrates that building LCAs that do not adequately account for the effects of SL and maintenance may not be fully capturing the benefit of more durable systems with longer SLs and reduced maintenance requirements.

In order to improve the reliability of LCAs, a recent study by Morales et al. (2022) analyzed the influence of uncertainties related to the product stage and uncertainties related to SL definition on the GWP impact of four building elements: exterior cement plaster exterior clay brick wall, exterior painting, and interior painting. These materials were selected to compare elements with relatively long SLs and high SL variability to elements with relatively short SL. The study controlled for RSP (50, 100, and 500 years) and distribution choice regarding the SL uncertainty analysis. This study found the following:

- Distribution choice had a significant impact on predictive SL, and thus a significant impact on GWP.
- Components with a longer SL introduce greater uncertainty to LCAs and (Morales et al., 2022); uncertainties from SL were greater than product stage uncertainties for the four building elements analyzed.
- Uncertainty increases as RSP duration is extended.

These findings highlight the importance of further studying the impact that design SL has towards reducing life cycle environmental impact.

"GREEN" MATERIALS AND DURABILITY

One question that may come to a designer's mind who is looking to minimize the environmental impact over a building's life cycle is the trade-off between embodied and operational environmental impact (e.g., energy use or carbon emissions). Many researchers have shown that the building use phase is the most significant part of its life cycle (Azari 2014; Asdrubali, 2013). Ramesh et al.'s (2010) review of building LCA literature found that buildings generally use more energy during their operating phase (80-90%), which includes maintenance, in comparison to their embodied energy (10- 20%). This same paper found that the reduction in life cycle energy use was correlated with the number of energy reduction strategies that were undertaken for the building, even if it meant a small increase in embodied energy. This has led some researchers to suggest that it is more important to reduce the operational energy over a building's life cycle, even at the expense of higher embodied energy.

However, reliance on operational energy reductions alone may not be optimal to reduce overall life cycle impact. For example, Ramesh et al. (2010) found that net-zero-energy buildings (nZEB) had *higher* cradle-to-grave life cycle energy use than low-energy buildings because of the high amount of embodied energy needed to create the nZEB. This is because, as buildings become more energy efficient, material-embodied energy becomes even more important (Li et al. 2020). Asdrubali and Grazieschi (2020) found that embodied impact can raise from 20 – 25% for conventional buildings up to 60% for nZEB. Röck et al. (2020) found that embodied energy can exceed operational energy when a 50-year period is considered. Li et al. (2020) noted that embodied energy of building contributes greatly to carbon emissions. They also noted that system degradation can significantly impact the life cycle energy use of a building, as it becomes a more significant portion of the overall energy use. These factors will become even more important as energy production moves towards greater use of renewable and low-carbon-emitting sources. As such, embodied energy should not be ignored when trying to design low-impact, durable buildings, and the use of so-called "green" materials should be considered by design and construction professionals.

One low-embodied-energy building material that is often cited the available LCA literature is natural earthen and bio-based building materials, such as cob walls, rammed earth, light straw clay (Arrigoni et al. 2017; Ben-Alon et al. 2021; Melià et al. 2014). It was found that these products generally perform better from an LCA perspective than conventional building materials, mainly due to lower impact and embodied energy during the product stage (Ben-Alon et al. 2021). Other commonly used construction materials with low embodied energy are timber and cellulose. One challenge, however, is that few studies have explored in a systematic manner the effect that the durability of these materials has on life cycle impact. Arrigoni et al. (2017) explored this relationship for stabilized rammed earth. This suggests that further research related to the likely SL, and methods of improving the SL, of these low-embodied-energy technologies would be of great benefit to the field.

Another important influence on the life cycle impact of a building elements is related to its end of life. Carlisle et al. (2016) found in a comparative LCA of various types of window frames that aluminum window frames were the best of the options considered from a life cycle perspective. They identified recycling rates and long SL as being very important towards reducing the impact of aluminum products. Iordanis et al. (2016) found that reuse of an existing steel-framed building was the optimal option from a life cycle perspective. Morales et al. (2022) identified recycling and reuse of building elements as an area requiring further research.

Given the research noted above, it can be argued that there exists a "sweet spot" between reducing embodied energy use and environmental impact, improving durability of a building element, and reducing operational energy. Iraldo et al. (2017) have studied this trade-off in the context of energy-intensive products, but less research has been done to study this for building envelope components. In that study, they concluded that high-durability products are preferred from an environmental and economic perspective when the product impact is highest during the production and end-of-use phases. It is the opinion of this author that this question has not been sufficiently addressed in the current LCA literature, but that the use of highly durable products with long SLs, which require minimal energy to produce, is likely critical for the overall sustainability of buildings.

ADJUSTING FOR CLIMATE CHANGE

Not only does building envelope durability reduce the life cycle environmental impacts of buildings, thereby reducing human contribution to climate change, but it will also be important for dealing with the effects of climate change. It is expected that the changing climate will affect building envelope systems through gradual changes in weather patterns, increasing variability, and potential increases in extremes (CSA 2019). The primary weathering mechanisms that will impact building envelopes are expected to be to wind-driven rain, freeze-thaw cycles, frost penetration, wetting and drying, wind-driven abrasion, solar and ultraviolet (UV) radiation, and atmospheric chemical deposition on materials (CSA 2019).

These changes are anticipated to affect materials and systems differently depending on their properties. Although not the main focus of this paper, some examples of the anticipated effects on various envelope systems are discussed below:

- Porous building stones (i.e., masonry) in certain geographical locations may have a reduction in risk of frost damage by up to 70% due to temperature increases, and an increase risk elsewhere due to an increased number of freeze-thaw cycles (CSA 2019).
- Increased atmospheric carbon dioxide levels are anticipated to increase the carbonation depth (Peng & Stewart 2016) and rate of steel corrosion in concrete structures (Saha and Eckelman 2014). Saha and Eckelman (2014) found that carbonation and chlorination depths may exceed beyond current code-recommended cover depths by 2077 and 2055, respectively.
- Vulnerability of wood frame structures is likely to increase in some environments due to increased risk of wood decay (CSA 2019).
- Corrosion of metals may decrease slightly at inland locations but is expected to increase significantly in coastal regions (CSA 2019).
- Accelerated deterioration of plastics, rubber, and wood materials due to increased UV radiation (CSA 2019).

It is very important to understand that the relative impact of these mechanisms on building envelope systems will be geographically dependent. Peng and Stewart (2016) found that mean carbonation depth could increase by as much as 45% in studied Chinese cities, and 7–20% in temperate or cold-climate locations in China. Due to this variation regionally, it can be difficult to make generic recommendations to address the potential impacts of climate change. CSA S478 makes the following general recommendations regarding product selection for increased durability:

- Use dimensionally stable products where they are exposed to temperature changes.
- Products with enhanced elasticity are recommended at locations subjected to multiple movement cycles (e.g., joints).
- Products with increased resistance to heat aging and UV radiation are recommended for areas directly exposed to the sun.
- Cladding systems with improved drainage design (e.g., rainscreen) will minimize water retention.
- Use dimensionally stable materials where they may be exposed to water regularly.
- Use metal products with enhanced resistance to corrosion.

If durability and the effects of climate change on durability are not accounted for in the design of building envelope systems, then these systems may be subjected to premature deterioration and reduction in SL. This is why it is critical for designers to consider durability in their design in a methodological way, otherwise known as "design for durability." The next section of this paper will focus on providing general recommendations for designers towards that end.

DESIGN FOR DURABILITY RECOMMENDATIONS

Failed envelope systems can also reduce the SL of other elements of the building, such as the building structure. These failures often lead to significant downstream impacts and costs in excess of those directly associated with the building envelope failure itself. Some classic examples of failures that come to mind might include a leak in the moisture control layer leading to accelerated deterioration of the interior structural elements and finishes, or frequent condensation in a wall assembly leading to mold growth over time. There are some examples that may not be immediately evident. For example, a continuous exterior insulation layer, with a high R-value and minimal thermal bridging, will reduce the exposure of the interior structural elements to the effects of thermal expansion and contraction due to temperature changes, reducing wear and increasing SL. Similarly, a poor drainage design can lead to accelerated deterioration of other envelope systems such as roofs and walls.

The evident link between efficient building envelope design and building durability, as well as the relationship between durability and the overall environmental impact explored in earlier sections, highlights the importance of good building envelope design towards improving the overall sustainability of buildings. This goes beyond simply reducing the energy use of buildings by increasing R-value. It also requires that envelope systems be durable throughout their life cycle in order to reduce additional impact related to the building's use phase. Since it is not feasible to discuss every design problem that affects building envelope durability within the context of this paper, this paper and associated presentation will focus on presenting a general framework, which designers ought to consider and adopt, in order to achieve adequate durability in their building envelope design.

There are many standards related to durability that can be referenced for designers looking to design for durability. Although intended for a

TABLE 1. CSA S478:19 Failure categories and minimum design service life

Failure Category	Consequences of Failure	Minimum Design SL of Building Element
1	Minor	Owner defined
2	Reduction in serviceability state	
3	Reduction in resistance capacity OR Moderate repair cost	20% of building design SL
4	Loss of resistance capacity OR High repair cost	50% of building design SL
5	Risk to health and safety of building users	100% of building design SL
6	Injury, loss of site, or loss of asset	
7	Prohibitive repair cost	

Canadian context and not currently mandated by the National Building Code of Canada, CSA S478:19, Durability in buildings, is a particularly useful resource for designers looking to design for durability. It sets the minimum requirements to assist designers looking to create durable buildings. Generally speaking, the requirements of the standard are enforced through the production of a building durability plan.

In general, CSA S478 covers the following with regard to the design for durability:

- Provides definitions for key concepts related to durability including performance, failure, SL, etc.
- Fundamental durability requirements for a new building or the repair/renovation of existing buildings or building elements
- Compliance criteria for the design of buildings and building elements (e.g., design SL, predicted SL)
- Compliance criteria for construction processes
- Compliance criteria for quality management, operation, maintenance, and inspection of buildings and building elements
- Requirements for investigation, assessment, and repair of existing buildings and building elements

Of particular use to designers, CSA S478 specifies a framework for determining minimum building design SL for various building types; categorizes building element failures based on the consequence of the failure; and specifies minimum design SL of building elements, as a percentage of minimum building design SL, based on failure category. These are summarized in **Table 1**.

Other useful features of the standard include Clause H.6, which provides a list of relevant standards related to the design for durability of various envelope systems. Annex C provides resources for determining and classifying the environmental conditions of buildings and building elements, including reinforced concrete, as well as corrosion of metals. Annex D provides mitigation strategies for a variety of degradation mechanisms. Annex E provides a summary of the effect of climate change on the durability of buildings. Annex F provides a list of principles for designing functional building envelope systems.

CONCLUSION

Due to the sources of uncertainty related to building LCAs, caution is recommended to designers looking to use LCA research to inform their design. It is important to carefully read and understand the goal, scope, and data sources of an LCA. For example, many LCA studies use an RSP of 50 or 60 years, which may not be appropriate for a given building design. These factors influence the overall quality and reliability of the LCA. Greater uncertainty during the building's use phase relative to its production phase (Morales et al. 2022) highlights the importance of selecting durable materials and implementing required maintenance processes, so as to minimize the potential additional impact resulting from maintenance and replacement, therefore reducing uncertainty.

More research comparing the life cycle impact of various envelope systems, which also account for the influence of durability, is needed in order to make sound recommendations to inform design decisions (Grant et al. 2010; Iraldo et al. 2017). It has been shown by multiple researchers that the use of products with longer SLs can reduce life cycle impact (Aktas and Bilec 2012; Marsh 2017; Palacios-Muñoz et al. 2019b; Grant et al. 2013; Grant et al. 2014; Morales et al. 2022). Ultimately, more research is needed to better understand the effect that component SL has on a building's overall environmental impact (Hoxha et al. 2016; Morales 2022), and how this can be balanced with building adaptability, but current research suggests that durability is an important factor that designers should consider. Such research would allow designers to better understand the conditions under which it is desirable to maximize durability or minimize energy use during the product stage.

Use of highly durable products with low embodied energy use/environmental impact become critical in energy efficient buildings, because these factors become relatively more important compared to traditional buildings (Li et al. 2020; Asdrubali and Grazieschi 2020). These products also become more important in the context of climate change, where changing weather patterns are expected to increase the rate of deterioration for many building envelope systems. This is why it is important that designers consciously design for durability, using standards such as CSA S478, which has been briefly discussed in this paper.

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MID-CENTURY MODERN MASONRY MISHAPS

ISSUES AND CHALLENGES IN THE EVALUATION AND REPAIR OF MID-CENTURY MASONRY CLADDING SYSTEMS

ABSTRACT

Many of the buildings constructed following World War II up to around 1975 can generally be categorized as mid-century modern. During that time, new technologies allowed masonry to be used to achieve minimalist expressions with an emphasis on functionality, organic and geometric forms, and mixing of materials, all characteristics common to the mid-century modern style. During this period, the masonry palette used by designers often included thin stone cladding systems, stack-bonded brick, flat terra cotta panels, and concrete and glass block. As these facades have aged, however, issues related to durability, serviceability, and water management developed, which were uncommon within traditional masonry facades. The response to these issues includes evaluation, maintenance and restoration; including evaluation, maintenance, repair, and restoration of these architectural gems; this is generally governed by the Secretary of the Interior Standards for Rehabilitation. Considering many of these buildings are more than 50 years old, they are categorized as "historic," often qualifying for the National Register of Historic Places. This paper will break down the typical materials and systems used in masonry construction during the mid-century modern era, explore commonly occurring issues with these structures, and review repair approaches, all supplemented with short case studies.

LEARNING OBJECTIVES

- Describe materials and cladding systems that were typically used during the period between 1940 and 1970.
- Discuss examples of the mechanism of deterioration and distress in midcentury cladding systems.
- Examine the repair and approaches that could be considered for various conditions that occur in the midcentury cladding systems.
- Evaluate potential new issues that could develop in midcentury cladding systems as they continue to age.

SPEAKERS

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Edward Gerns is a project manager and project architect/engineer experienced in the investigation and repair of deteriorated conditions in existing buildings. He performs evaluations of brick, terra cotta, and stone masonry; assesses causes of collapse or distress in existing cladding systems; and has inspected numerous structures damaged by wind, ice, snow, and fire. Gerns has overseen the preparation of repair documents for contemporary and historic buildings and structures.

Leah Ruther
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Leah Ruther has been involved in numerous projects related to both structural engineering and architecture. Her typical responsibilities have included the investigation and analysis of existing and damaged structures, the development of technical repair and rehabilitation documents, and construction observations. Ruther has performed a variety of structural analyses on steel, concrete, masonry, and wood structures. She has assisted with several non-destructive investigations and has completed multiple condition assessments.

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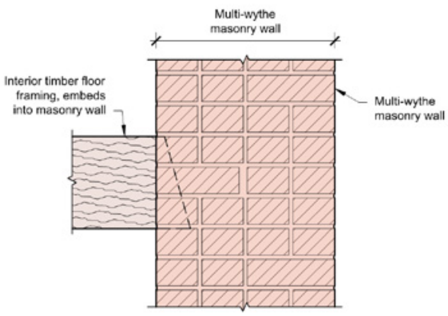

FIGURE 1. Four-wythe mass masonry wall detail.

INTRODUCTION

Prior to the 1870s, exterior walls of buildings functioned as both the building's structural system and the enclosure for the interior space. During this era, exterior walls of most significant buildings consisted of monolithic multi wythe masonry, which also supported the floor structure (**Figure 1**). The building height was typically proportional to the wall thickness while also limited by the load bearing capacity of the underlying soil. Wall systems transitioned from the 1870s to the 1890s with the introduction of cast iron and wrought iron structural systems, which were integrated into the traditional wall systems. By the 1890s, the skeleton frame structural system had evolved with the development of rolled steel sections as well as reinforced concrete frames.

Between 1890 and 1940, hybrid or transitional wall systems were widely used throughout the United States (**Figure 2**). While numerous variations of the system exist, generally they can be characterized as three- to five-wythe-thick masonry exterior walls with integral steel or concrete members serving as the main structural system supporting floor loads. The exterior wythe consisted of some combination of brick, terra-cotta, and stone, typically supported by rolled steel shapes attached to the main structural system. The interior wythes are header bonded to each other and, to some extent, to the outer wythe, consisting of brick masonry and/or extruded terra-cotta blocks. These wall systems are intended to function as barrier systems to manage water infiltration and exfiltration.

In 1929, Architectural Record produced an extensive article on new construction methods, which included the following remark, "It is the opinion of some leading architects of Europe and America [that] it is entirely practical to eliminate masonry by using metal mullions [i.e. curtainwalls], as in the Bauhaus. Or, if one desires to have solid walls for architectural effect, one may use metal panels between the mullions."[1]

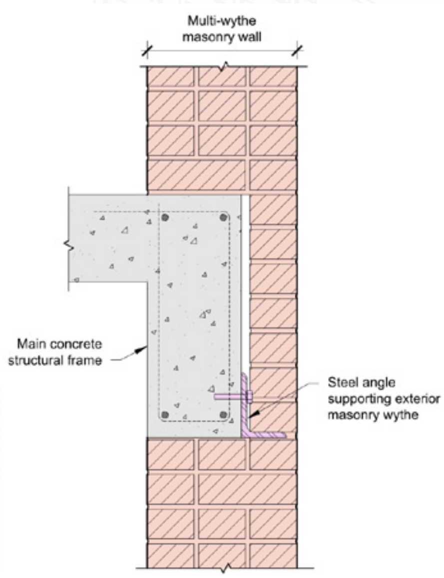

FIGURE 2. Transitional wall detail.

As the push for taller buildings rose, the need and desire for lighter walls became more and more of a necessity. As stated in a special issue of *Engineering News Record* in February 1931, the author discussed a need to move to lighter external walls.[2] He regarded the use of masonry walls in tall buildings as an "illogical heirloom". The idea was so deep rooted that practically all building codes continued to require masonry wall thicknesses be graduated from top to bottom. However, reducing the thickness of masonry walls led to infiltration, and so attention was directed to improving waterproofing measures, rather than to developing a new technology, the task being to make the design independent of the standard of workmanship in the masonry because cost cutting had resulted in poor standards of waterproofing application.[3]

By the early 1930s, almost all construction halted for the next 15 years in response to effects from World War II. Following the war, the construction industry shifted yet again, with an emphasis on the architectural aesthetic. While masonry-clad buildings were still being constructed, the next 20 years were defined by the rise of the glass andmetal curtainwall.

By the 1950s, the start of the mid-century modern period, masonry wall systems were beginning to evolve toward conventional cavity wall systems

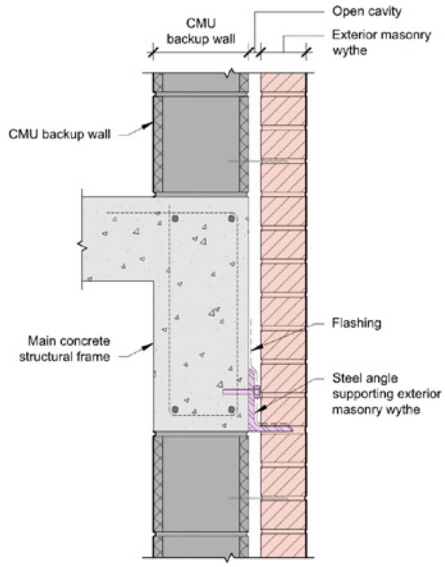

FIGURE 3. Mid-century wall detail.

(**Figure 3**). Various combinations of brick, terra-cotta, stone, cast stone, and precast concrete were being used in combination with clay masonry and concrete block backup walls. These materials were used in increasingly thinner applications. Technologies developed between the 1930s and 1950 resulted in improvements in waterproofing systems, including sealants and flashing materials. This enabled the water management systems for the walls to transition from mass walls to something more closely resembling today's drainage walls. Most notably was the transition from a filled collar joint (1 in. or less) to a true open cavity.

As mid-century building wall systems became thinner, lighter, and less redundant, the effect of the environment and the interrelationship of wall system components became more pronounced. Mid-century masonry wall systems were also more dynamic than the older systems because they were lighter and thinner. The lack of understanding, inexperience, or recognition of actual wall system behavior often resulted in physical damage, water leakage, increased maintenance costs, and newly emerging failures. In response to these concerns, the building industry shifted focus to the behavior and physical properties of masonry materials in the new wall systems to better predict and accommodate the interaction between wall components. The introduction of expansion joints, flashings, and alternative anchorage methods was the direct result of this desire to control and accommodate movement resulting from intrinsic material properties, exposure to the environment, and lateral and gravity loads.

MID-CENTURY MASONRY MATERIALS

Brick

Brick masonry used during the mid-century period was generally used in similar applications to those prior to the 1940s. Stack bond patterns were a popular aesthetic complementing the cleaner look of mid-century architecture. The exterior brick wythe was supported at each floor with shelf angles anchored back to the building frame. Lateral support was provided by various types of straps and wires engaged into the backup masonry wall assembly, spanning the collar joint to engage the cladding and resist outward load while inward load was generally resolved by the partial or complete filling of the collar joint. The primary difference between brick systems of this vintage and earlier versions was that flashings were incorporated into the system and the collar joints were beginning to be treated as drainage cavities. Concrete block was commonly being used as a backup system, but unlike modern systems, weather-resistive barriers were typically not incorporated. These systems showed traits of modern drainage walls, but they rarely functioned as intended.

Terra-Cotta

Another clay-based product of the mid-century period was terra-cotta. Unlike previous applications of architectural terra-cotta, the units tended to be very simplified and were fabricated using an extrusion process. While support systems varied, the units tended to be supported in a similar fashion to brick masonry (i.e., shelf angles); however, thinner applications were often supported by a series of clips and ties integral to or anchored to the structural frame. The terra cotta industry struggled during this period with the rise in popularity of the glass and metal curtainwall and the decline of classical aesthetics. The industry attempted to adjust by marketing flat terra-cotta panels that could be used to "modernize" traditional older buildings.

Stone

Unlike manufactured materials used in construction, the physical characteristics of stone vary greatly between geologically different stones as well as between stones within the same classification. These variations contribute to the inherent beauty of stone as well as its physical characteristics. While stone has been used in building construction for thousands of years, by the 1950s, stone applications as thin as ¾ in. had become increasingly popular. Some stone types were well suited for thinner applications; however, the physical properties of certain stone types, as described in the table below, weren't always conducive to thinner applications.Stone, used in building construction, is categorized as one of the types shown in the table.

Stone within each of the geologic categories has distinct physical characteristics, which become more relevant when used in thinner applications. Historically, stone was used as very compact shapes that were subjected primarily to compressive forces. The building was physically massive enough that lateral loading on individual components was inconsequential and was naturally resisted by the geometry of the structure rather than individual components. The transition from stone cladding systems designed as load-bearing masonry structures to individually supported stone panels beginning in the 1940s was not smooth and uniform.

MID-CENTURY CLADDING STRUCTURAL SYSTEMS

Gravity Load

The early anchorage systems typically used carbon steel or galvanized steel shelf angles anchored at each floor to support the masonry cladding systems. With regard to stone panels, the panel directly above the shelf angle was notched to accommodate the thickness of the angle or a supplemental piece

Sedimentary	Limestone, sandstone, brownstone, and shale. Sedimentary stone is the product of deposits of sediment materials in prehistoric river and lake beds. Distinct bedding planes between individual layers of material and grain size characterize sedimentary stone, which thus make it less suitable for applications less than 4 in. thick. However, some architectural details found in industry publications during this time period showed limestone as thin as 2 ½ in. supported by systems which had traditionally been a minimum of 4 in. thick.
Igneous	Granite and schist. This type of stone is the result of volcanic activity and the consolidation of molten magma. Igneous stone generally is much harder, and less absorptive and has higher strength than sedimentary stone and as such, there are many examples of successful fabrication and installation of panels as thin as ⅞ in.
Metamorphic	Marble, serpentine, and slate. Metamorphic stones are the result of sedimentary, igneous, or other metamorphic stone being subjected to millions of years of heat and pressure, leading to a recrystallization of pre-existing rock. Marble was particularly popular in thin facade application beginning in the 1950s with panels also being fabricated as thin as ⅞ in. But unlike igneous stone, thin applications of marble, depending on the support detailing, were found to be problematic within the first few decades of service. To understand the issues with some metamorphic rock in thin applications one has to understand the two metamorphic processes that occur to change rock. The first process is thermal, where rock is subjected to prolonged exposure to heat in a confined environment. This is the process by which limestone is converted to marble. The second process is regional metamorphism, which is associated with the creation of mountains where rock is subject to extended periods of stress or pressure. During this process, the recrystallization of the stone results in new rock particles forming parallel to the pressure. Because of the grain structure of metamorphic rock, it is susceptible to a phenomenon known as hysteresis, which will be discussed in more detail in a case study presented later in this paper.

of stone was added to the back of the panel to provide a bearing ledge, often referred to as a liner block. Panels above the bearing unit were stacked on shims up to the next support. Sealant or mortar was installed in the joints between panels, concealing and protecting the shims. Typically, the shims were lead; however, carbon steel or wood shims were sometimes substituted. As the joint material between panels failed in these early systems, the carbon steel angles and steel shims would readily corrode. Since corrosive scale occupies a greater volume than uncorroded steel, the joints were not adequate to accommodate the scale, resulting in spalling and cracking of the panels.

Lateral Load

Lateral load was previously managed naturally by way of header brick units being incorporated into the composition of the mass masonry wall, locking the outer wythe to the inner wythes and thus permitting the wall to act monolithically to resist lateral loads. During the mid-century era, however, outer wythe of masonry did not always include headers and thus relied on alternate mechanisms to transfer lateral load to the backup wall, typically via ties or straps.

Early thin stone cladding resisted lateral loads by brass pins set into holes drilled in the edges of the panels. The pins were secured with wire anchored to structural members or embedded into grout- or plaster-filled pockets in concrete systems. This method of attachment was common for interior applications of stone but was also occasionally used in exterior applications. Alternate systems utilized a bent plate with the outstanding leg nested into the joint between adjacent units. A pin inserted through a hole in the plate extended into holes drilled into the edge of the stone panels.

Inward loads, with regard to stone cladding systems, if not accommodated by a rigid lateral anchor, were frequently resolved by placing mortar or plaster spots in the cavity between the substrate and the back of the panel. Masons frequently used a gypsum-based plaster, or they added gypsum to the mortar to speed up the setting time. Although this technique was successfully employed in many interior applications, deterioration would occur when both interior and exterior applications were exposed to moisture. Essentially, when gypsum becomes wet, a chemical reaction occurs between the cement, gypsum, and water, resulting in ettringite formation. The crystal structure of ettringite occupies a volume larger than the original materials. Failures of the stone claddings frequently occurred when the mortar was originally confined and the expansion could not be accommodated. Calcium chloride and other salts were also occasionally added to mortar to act as an accelerator or retarder, or to improve bond strength. The presence of chlorides significantly increases the corrosion of the carbon steel structural components.

Tolerances/Constructability

Although not as significant in older load-bearing structures, tolerances are a major factor affecting mid-century cladding systems. Tolerances and detailing for tolerances must include both fabrication and construction variations. These variations must be accommodated within the system, specifically the anchor and support detailing, to ensure a proper installation. Frequently, inadequate adjustability within a mid-century cladding system resulted in field modifications that deviated from the original design intent and compromised the performance of the system.

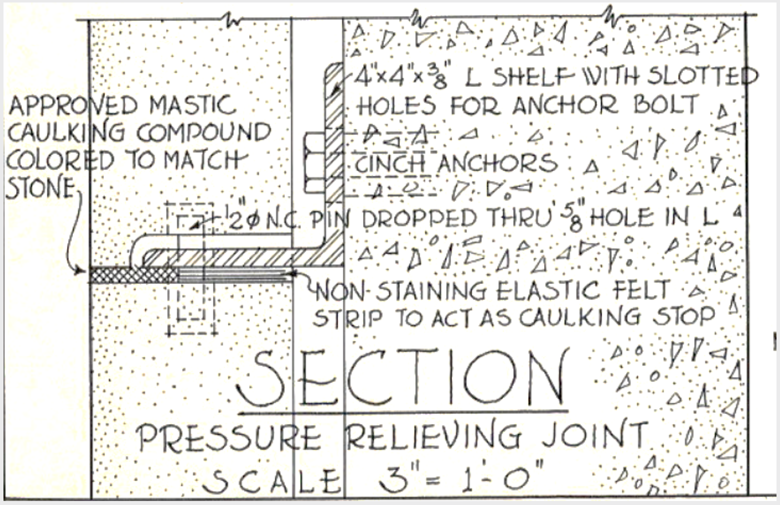

FIGURE 4. Detail excerpt from Indiana Limestone Institute, 1950s.

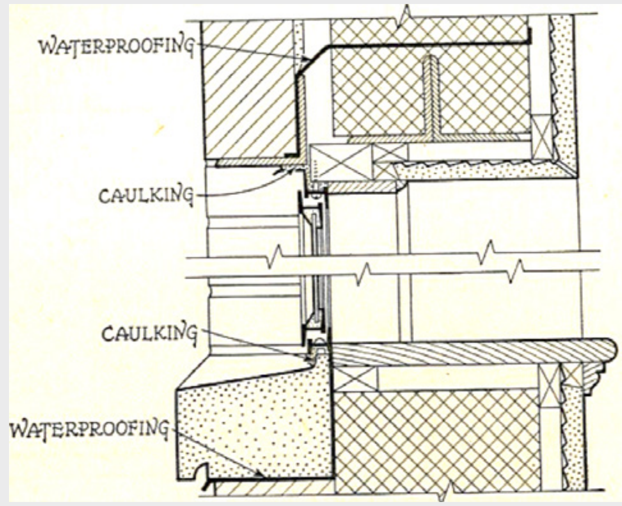

FIGURE 5. Representative detail incorporating "waterproofing" membrane. Note waterproofing membrane does not extend to exterior face of masonry.

Tolerances became more significant as the thickness of the panel decreased.

Installation tolerances, or the relationship between the cladding and supporting structure, can vary dramatically. Industry standards for steel may require as much as 5 in. (3 in. outward and 2 in. inward) of potential in/out adjustment for cladding systems on structures greater than 36 stories. If this was not properly designed, excessive shimming or field modifications to the masonry units may have occurred during installation.

Also related to constructability are techniques for installation of the last panel in a system or in a course of stone. These panels are typically located at corners or at the top of a building, where they are subjected to the highest wind loads and have the greatest potential to compromise public safety. Frequently, the responsibility for an installation and attachment scheme was considered "means and methods" for the contractor to dictate. The critical pieces may have been anchored with a "blind" system or by some other improvised technique. Careful attention was necessary to provide adequate anchorage for all panels within the cladding system and not simply the "typical" detail.

Movement

Proper consideration and accommodation of all potential movement within the cladding system as well as within the structural system was necessary to prevent both local failures and system failures. Thermal, seismic, wind, creep, and shrinkage movements must be considered for individual panels as well as the entire system. In addition, initial moisture expansion of clay-based masonry can also result in movement that should be accommodated. During this period, vertical and horizontal expansion joints began to more regularly be incorporated into cladding systems. These joints, when properly designed and installed, reduced the potential for failures due to these movements.

Horizontal expansion joints may or may not have been included below the shelf angle supports. The lack of incorporating an expansion joint – particularly in concrete frame buildings, which are subject to shrinkage and frame shortening – could result in cracking, spalling, and displacement of brick cladding due to accumulation of compressive stresses (**Figure 4**).

MID-CENTURY MASONRY ISSUES

Water Infiltration

One of the most fundamental issues affecting almost all exterior building components is water infiltration. Mid-century cladding systems rely on the relatively thin cladding panels and the joint material as the primary line of defense against water infiltration to the interior of the building. Obviously, these systems may be somewhat watertight initially, but as the joint materials deteriorate, water will reach the underlying substrate and anchorage. A second line of defense against water infiltration, specifically flashing and some other weather resistant material were beginning to be incorporated into the design of these wall systems. In this era detailing and implementation of flashings lacked foresight of water movement, and thus details including flashing laps, end dams, etc., were not considered (**Figure 5**).

Corrosion

Another ramification of water infiltration was corrosion of the support system for the cladding. Early systems frequently did not consider the effects of water penetrating the cladding system. Galvanized steel may have been used for connection components including shelf angles, lateral straps, and bolts. The rate of corrosion was greatly reduced depending on the thickness of the zinc coating. Frequently, some but not all of these components may not have been galvanized. As the system aged, galvanized and unprotected steel would eventually corrode and result in the failure of components, adjacent components, or of the entire system.

The rate of corrosion when the pH of a material is between four and ten is essentially constant and relatively low. When the pH falls below four, the rate of corrosion accelerates dramatically. In masonry wall systems, mortar and cement materials initially create an alkaline environment with a pH of approximately ten. As carbon dioxide from the environment penetrates the mortar and causes carbonation, the pH is reduced, resulting in increased corrosion. By far the greatest cause of corrosion, however, is water infiltration.

Corrosion of embedded steel components can manifest itself in many different distress mechanisms:

- Loss of lateral anchorage. The deterioration of lateral anchorage within a wall system may be difficult to detect, particularly if the anchors are light-gauge straps that can disintegrate without causing externally visible distress, such as cracking or displacement. The most reliable method for evaluating the condition of these lateral anchors is by direct observation of representative anchors in areas with and without apparent external distress.
- Cracking due to corrosion of gravity support. Localized cracking is usually the result of corroding shelf angles and window lintels. This corrosion typically results in characteristic step-cracking at window openings or spalling of the face of the adjacent masonry (**Figure 6**).
- Cracking due to confinement of anchors**.** Very minimal corrosion of the embedded anchorage is necessary for cracking to occur related to anchors for stone or terra -cotta, particularly if mortar was packed into the anchorage holes. The resulting crack could propagate to either the external or internal face of the unit. Cracking or spalling toward the back face of the piece is much more difficult to detect with non-destructive techniques.
- Displacement of masonry. Although compressive stresses within the facade caused by moisture and thermal expansion of the cladding as well as shrinkage and creep of the structural frame can also contribute to both localized and overall displacements, in many cases, corrosion of underlying steel contributes to the displacement of masonry cladding materials (**Figure 7**).

FIGURE 6. Step cracking due to corroded lintel.

FIGURE 7. Displaced masonry due to corroded lintel.

Other Considerations

Mortar additives: Modifications to mortar during the mid-century period often created unintended consequences. Pre-bagged mortar was gaining popularity and theoretically improving quality and consistency of the mortar. In some instances, however, the additives had detrimental effect on the long-term performance of the system, most specifically accelerated deterioration of the embedded steel components.

Hazardous materials: Construction materials of this period also frequently incorporated materials which improved the performance and durability of the material but have subsequently been found to have harmful effects on people. Lead, asbestos, Polychlorinated biphenyls (PCBs) and many other materials may exist in mid-century buildings. It is prudent to retain an industrial hygienist to determine the extent of these materials that may exist and thus determine appropriate remedial actions.

CASE STUDIES

Brick Masonry Failure

Building Description:
The first case study relates to an 11-story building, constructed in 1972 in Michigan. The facade features large bays of brick cladding with cast-in-place concrete backup walls. Based on the original drawings, as well as the onsite investigation, the brick masonry is supported by shelf angles at each floor, which were shown in the original drawing to be anchored to the slab with an embedded hooked anchor. Flashing was depicted to be installed on the shelf angle and held back from the outside face of the brick. Lateral ties consisted of galvanized dovetail ties spaced approximately 24 in. apart, nested within vertically oriented galvanized dovetail slots that were cast into the cast-in-place backup wall.

Failures:
Following a failure of the cladding system where an entire bay of brick (approximately 23 ft x 9 ft) at the eleventh floor fell to the ground below (**Figure 8**), close-range observations revealed remnants of galvanized ties within the dovetail slots. The exposed and readily visible portions of the angle were also severely corroded, inclusive of accumulation of corrosion scale (up to ½ in. in some areas), resulting in section loss over the majority of its length. While flashing was present atop the shelf angle, it consisted of a very thin sheet

FIGURE 8. Overview of failure. Note large area on facade with missing masonry cladding.

FIGURE 9. Corroded galvanized brick tie.

FIGURE 10. View of shelf angle detailing.

of copper, which was loose, unadhered, and unsecured at the top edge.

The failure was the result of severely corroded brick ties (**Figure 9**). Concurrently, the shelf angle assembly, which previously supported the bay of brick that failed, was corroded, resulting in significant accumulation of corrosion scale that induced stresses on the brickwork above and below the angle. This condition is further exacerbated where the angles are not anchored to the structure, resulting in load accumulation potentially over the full height of the wall. Further, the lack of expansion joints between the horizontal leg of the angle and the brick below also results in load accumulating over the height of the wall.

Improper detailing of the copper flashing – including lack of a seal along the leading edge (i.e., top horizontal edge of the flashing), lack of proper daylighting, etc., and lack of properly sealed side laps and incorporation of end dams – permitted water to flow around the ends of the flashing and back onto the horizontal leg of the shelf angle, leading to corrosion of the angle (**Figure 10**). Additionally, sealant applied at the toe of the angle restricted water from exiting the system, accelerating the rate of shelf angle corrosion.

FIGURE 11. Overall view of marble cladding system on building.

Repair Recommendations:
To repair the facade, the authors recommended that the conditions observed, which were determined to be systemic, should include installing supplemental lateral support anchors such as stainless-steel helical anchors within all areas of the masonry cladding. In-situ testing of the anchor system prior to installation was recommended to determine the extent of required anchors (i.e., quantity/spacing) and verify efficacy of the repair approach. In addition, recommendations included removing and replacing the shelf angle assemblies at all locations including installation of a new flashing system. Such a repair requires localized brick removal above (three courses) and below (one course) the existing shelf angles, removal of the existing angle, installation of a new angle with a new anchorage system, installation of a new flashing with appropriate detailing, and installation of new brick masonry including a horizontal expansion joint below the new shelf angle.

Marble Cladding Failure

Building Description:
The second case study relates to a 17-story building constructed in 1971 in Texas (**Figure 11**). The majority of the building exterior is clad in thin marble panels. Marble facade panels are installed at columns, soffits, window sills, and floor lines at all facades. Typical panel thicknesses range from ¾ in. to 1 in. thick. Anchorage of the stone panels to the concrete building structure consisted of bronze wires set into drilled holes at panel edges and spanning between the building structure and the back face of the panel. A mortar spot was typically installed to encapsulate the wire tie (**Figure 12**).

Mortar was generally noted at the wire anchors, spanning between the building structure and the back face of the panels, which remains generally well adhered at locations of minor bowing.

Failures:
Cracking of the marble facade panels was noted throughout the facade areas, with the majority propagating from panel edges and extending horizontally toward the middle of the panel (**Figure 13**). Typical horizontal cracking occurred at approximately the same locations at panel edges. Vertical

FIGURE 12. View of copper wire ties originally installed as lateral anchors in combination with mortar spots. Note that the wire tie is installed very close to the inside edge of the panel rather than centered.

cracking was also noted propagating from the top and bottom panel edges. Diagonal cracks were observed in the field of a large number of panels.

Areas of rougher textural appearance, referred to as "sugaring," of the marble cladding were observed at many of the panels at all facades. Deep graining, inherent in the composition of marble, was observed at several discrete panels. Outward bowing of the panels was noted in both the vertical and horizontal directions. Where measured, the outward deflections ranged from 1/8 in. to 1 in. In several locations, sealant was visibly pulled away from the substrate, presumably due to panel movement.

The bowing of the thin marble cladding panels is a result of thermal hysteresis, particular to fine- to medium-grained marbles of this morphology (**Figure 14**). Anisotropic expansion of the marble's calcite crystals with thermal changes, along with prolonged exposure to atmospheric moisture, creates permanent expansion of the stone. This expansion is generally more pronounced on one face of the stone panels, leading to the observed bowing. In addition to bowing, these types of marbles can lose significant strength over time, which is generally attributed to separations at grain boundaries. This loss of strength over time can compromise the panel system, leading to failure of the panels as the stone strength can no longer accommodate imposed loads, primarily from wind.

At locations of more prominent bowing, the mortar spots at the wire ties are separated from either the building structure or the back face of the stone. At panels with less bowing, the mortar generally remained bonded to the stone and concrete. Wire anchors of this nature rely on the mortar for stiffness under positive wind pressures, when the wire is in compression. Without mortar engagement with the stone and building, wire anchors can buckle, causing rattling of the panels and fatigue in the wires, which could eventually lead to failure.

Cracking noted at anchor locations, at the exterior face of the stone units, is a result of bowing and strength loss. Cracks propagated at the anchor locations because of the restraint of the anchor as the panel bowed. Cracking noted at the interior face of the panels, also at anchor locations, is a result of wire anchors bearing against the stone as the panels bow outward. This cracking and spalling of the stone was not readily visible without removal of the sealant between the panels. Cracks and spalls at these anchor locations significantly reduce the capacity of the anchorage system under negative wind pressure. Additionally, this distress is likely to continue as the bowing of the panels increases.

Cracking of the panels away from anchor locations, most typically observed at or near the centerline of the long axis of the panels, is a result

FIGURE 13. View of typical cracking of panels.

FIGURE 14. Representative view of bowed facade panel.

FIGURE 15. Gravity support of panels consisting of a liner block adhered to the back of the panels and bearing on a shelf angle.

of panel bowing along with loss of strength. As the panels bow, the face of the panel in the direction of bowing is in tension, while the opposite face of the panel is in compression. Stone is typically stronger in compression than tension, and as stated above, marble tends to lose strength over time. As the bowing increases, the tension force at the face of the stone can surpass the tension strength of the stone, resulting in a crack. As the bowing progresses, the crack propagates, eventually extending through the thickness of the panel. This type of cracking will typically progress over time as the panel bowing increases and the stone continues to lose strength.

Gravity load support of the stone panel system consists of steel bearing angles attached to the concrete building structure, which engage liner blocks attached to the back face of selected stone panels. Subsequent panels are stacked onto these panels until the next bearing angle. Where observed, the bearing angles were in good condition, with little to no noted corrosion. The bowing of the panels, however, had progressed to a point where the liner blocks no longer engaged the bearing angles. Continued bowing of the panels will increase the likelihood that additional liner blocks will become disengaged from the bearing angles, which may cause the panels to lose support and potentially fall from the building.

Repair Recommendations:
Without repair, the marble panels will continue to bow and lose strength, creating a potential hazard over time. Limited analysis was performed regarding panel stress and anchor loads. Subsequently, supplemental anchorage was recommended to be installed in the panel system to increase its load carrying capacity. Repair of the existing panel system would generally include installation of anchors through the face of the stone panels into the building structure. The structural concrete frame of the building allows for significant latitude regarding the placement and type of anchors. Given the condition of the gravity support system, discussed above, the design of the supplemental anchorage system should take into account gravity and lateral loads.

CONCLUSION

Mid-century modern buildings represent a pivotal shift in the history of cladding assemblies. The materials, detailing, and overall cladding performance exemplify intentional modifications to transition from the pre-World War II mass masonry structures to a more economical cladding assembly. The outcome, however, highlighted several shortcomings in these well-intentioned efforts including cracking, displacements, bowing, and loss of gravity and lateral support, which in some extreme instances has resulted in catastrophic failures. Still, without these subsequent failures, we may not have arrived at the more refined conventional wall system commonly used today.

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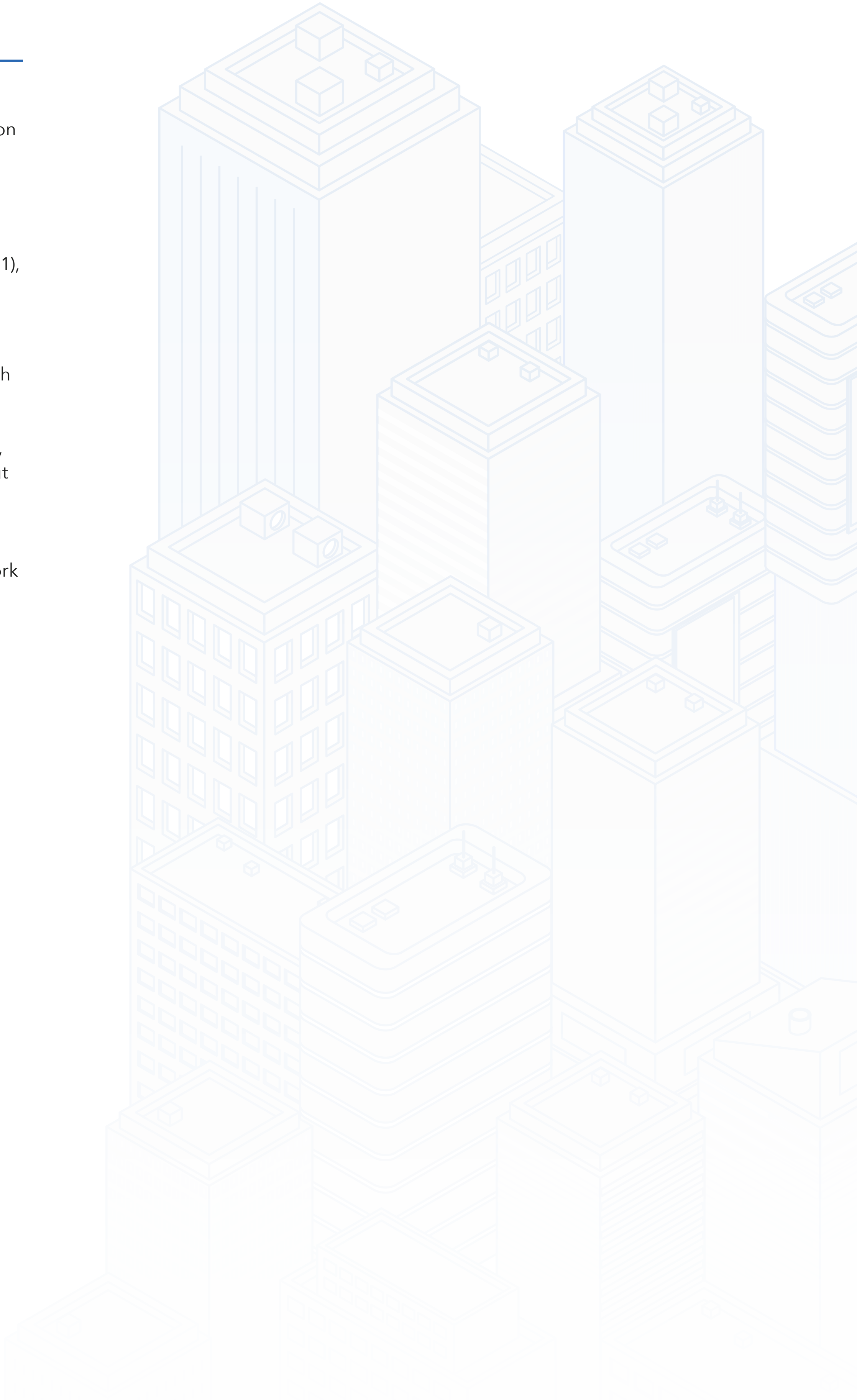

PREFABRICATED WALL PANELS: LESSONS LEARNED

ABSTRACT

Over the last 10 years, the use of prefabricated wall panels has become a common option for exterior wall systems. These systems range from backup panels (e.g., metal studs, with pre-applied exterior sheathing and the air/water barrier) to completed wall panels complete with cladding (e.g., finished exterior insulation finishing systems) or masonry veneer exterior system) to unitized curtain wall assemblies. Prefabricated wall panels allow for increased quality control of the prefabricated components and increased construction speed, but prefabrication is not a solution for all buildings or exterior wall types.

Many prefabricated assemblies require substantial work and care in the field to address panel-to-panel joints or field conditions, thus limiting the benefits of prefabrication. This presentation will discuss what types of buildings and structures are best suited for prefabricated wall panels, particularly opaque cladding systems. A discussion on the principles of prefabrication and the typical prefabrication process, including coordination and development during the design phase, typical shop drawing and submittal process, prefabrication, and erection/assembly will be provided. The presenters will also discuss project-specific lessons learned, providing both a contractor and consultant perspective.

LEARNING OBJECTIVES

- Define prefabricated wall panels.
- Compare varying types of prefabricated wall panels.
- Summarize the principals of prefabricated wall panels.
- Discuss strategies to determine if prefabricated wall panels are appropriate for a given project.

SPEAKERS

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Nicholas Floyd is a licensed professional engineer who joined Simpson Gumpertz & Heger Inc. (SGH) in 2003 and specializes in the design, investigation, and remediation of building enclosures. His past and current projects include enclosure investigation, design, and construction administration of several large public structures, educational facilities, and commercial properties. Floyd has experience designing and investigating contemporary cladding systems, as well as fenestration, roofing, and waterproofing.

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FIGURE 1. Prefabricated backup wall panels with framing, sheathing, and AWB. Some panels are installed into the wall, and panels on the floor await erection.

INTRODUCTION

Over the last 10 years, the use of prefabricated wall panels has become a common option for exterior wall systems. These systems range from backup panels (e.g., metal studs with pre-applied exterior sheathing and the air/water barrier (**Fig. 1**) to finished wall panels complete with cladding (e.g., finished exterior insulation and finish system (EIFS) or masonry veneer exterior system [MVES] (**Fig. 2**). This paper focuses on opaque wall assemblies, but unitized curtainwall assemblies (**Fig. 3**) and precast concrete facade panels are other "prefabricated wall" assemblies that are also commonly used, and which follow similar principles. These wall panels can be manufactured and installed in a slab-to-slab application, a bypass (curtainwall) application hung from slab edges or load bearing.

In a slab-to-slab application (**Fig. 4**), a prefabricated wall panel is placed between the floor slabs, extending from the top of a slab to the underside of the slab above. In this application, gravity and wind loads are resolved at each floor level. Once the panels are secured in place (typically fastening at the bottom, with an oversized deflection track at the top to resist wind loads but allow floor-to-floor deflection), the area between the panels (outside face of the slab edge) is constructed in the field to match the adjacent prefabricated wall panel.

FIGURE 2. Completed building with prefabricated wall panels with a combination of EIFS and MVES.

FIGURE 3. Unitized curtainwall in bypass application. Note unitized stack joint (circle) and curtainwall anchored to face of slab below (arrow).

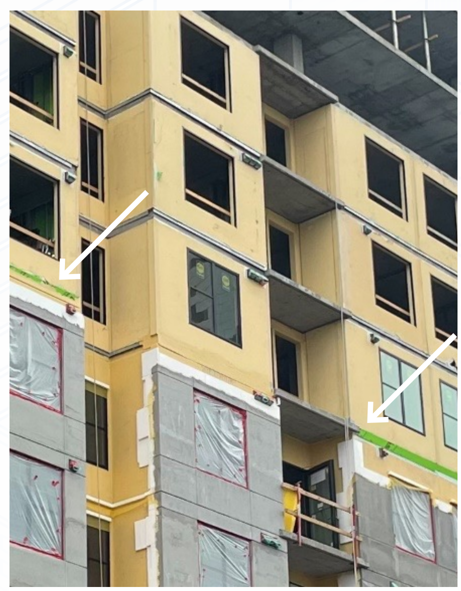

FIGURE 4. Slab-to-slab panel application. Note exposed floor slab between panels at upper floors and the in-progress panel infill below (exposed green sheathing at arrows).

In a bypass application (**Fig. 3**), a prefabricated wall panel bears on a structural connection (varies based on the design) that is anchored to the slab edge to resolve the panel gravity load; the panel can span multiple floors. The panel is also attached to each slab and to the prefabricated panel above; these connections are typically slotted connections that limit lateral movement (i.e., wind loads) but allow vertical movement. AWBs are integrated along all sides of each panel to the abutting panels.

In a load-bearing application, a prefabricated wall panel is structural and supports the weight of the building above and potentially shear loads, depending on the design and attachment. The two main types of load-bearing panels are exterior structural panels with a finished veneer and interior structural panels. Load-bearing panel systems can incorporate miscellaneous steel and other attachments, such as an edge angle installed on the interior side of the exterior panels for metal decking support. Since these panels often form or support the primary structure, they typically require design coordination with the structural engineer of record early in the design process.

Prefabrication allows for increased quality control of the prefabricated components and increased construction speed, but the authors of this paper (a contractor that has constructed and placed prefabricated wall panels on over 100 buildings and an engineering consultant that has peer-reviewed and performed quality control services on large, prefabricated wall projects) have learned that prefabrication is not the best solution for all buildings or exterior wall types. The authors have also learned that many prefabricated assemblies require substantial work and care in the field to address panel-to-panel joints or field conditions, thus limiting the benefits of prefabrication. Therefore, the purpose of this paper is to discuss the following:

- The types of buildings and structures that are best suited for prefabricated wall panels.
- The typical prefabrication process required for a successful project, including coordination and development during the design phase, typical shop drawing and submittal process, prefabrication, and erection/assembly.
- Project-specific lessons learned from a contractor and consultant perspective.

TYPES OF BUILDINGS BEST SUITED FOR PREFABRICATED WALL PANELS

Cost and schedule are usually the primary reasons for developers/owners to consider prefabricated wall panels versus a traditional field-constructed assembly. Buildings with a repetitive layout or uniform design conditions can provide the greatest cost and schedule savings, and thus are usually the best suited for prefabricated wall panels. Examples include multi-story buildings with repetitive floor plans or large buildings with relatively simple or repetitive exterior wall layout, particularly when these buildings have a consistent window layout (both horizontally and vertically). These types of projects maximize the use of a limited number of "typical" panels that can be constructed over and over to complete most of the exterior assembly. Repetitive panel construction maximizes the cost-effectiveness of the prefabricated approach because it requires a relatively limited wall panel design package, simplifies quality control measures during fabrication, and helps increase the speed of erection and any post-erection field work required at the panels.

Buildings with varying or inconsistent layouts (either in plan, in floor height, or in fenestration size or locations) increase the number of panel configurations. Multiple cladding configurations, combined with these layout/size differences further compounds the quantity of panel types needed. As the number of panel types and varying conditions increase, so does the complexity of the design and fabrication process. Higher complexity makes quality control more difficult and often increases the amount of field coordination and field work required to complete the assembly. These factors increase the overall cost and typically reduce the benefits of prefabrication.

Also ideal for prefabricated wall panels are projects in which turnaround time for interior build-out is critical. For these types of projects, schedule is the primary driver and cost may be of less importance. In these projects, prefabrication can be beneficial to preconstruct much of the enclosure concurrent with other on-site construction activities, regardless of the potential added complexity and related costs. Examples of these projects include hospital expansions or other buildings/structures that can benefit from generating revenue due to a compressed construction schedule and early opening.

PREFABRICATION PROCESS

Prefabrication is the practice of assembling components of a structure in a factory or other controlled manufacturing site, and then transporting these completed assemblies or sub-assemblies to the construction site where they can be erected or installed directly onto the structure. This approach differs from the more conventional construction practice of transporting the basic materials or components to the site and constructing much of the assembly in the field. Although the prefabrication concept sounds simple, there are considerable processes that need to be completed for a successful application and prior to even starting to assemble the components.

Design Considerations

Coordination for a prefabrication approach needs to begin early, preferably during the design phase and must be continuous through procurement and construction. Prefabrication has limitations and understanding these limitations during the design process can save time or may fundamentally change the design approach. If prefabrication is not the original project design intent, changes will likely be required during the design and construction phases to fit in the constraints of prefabrication – e.g., adjusting wall layouts or cladding and window configurations to make the best use of "typical" panel shapes/sizes, accommodating transportation and erection limitations at the site, modifications to the structural design and/or partially delegating the wall or cladding design responsibility to the prefabrication subcontractor.

Once the prefabricated design concept is agreed upon, the design of the individual panels can begin. This process typically starts with a relatively extensive shop drawing process to design and lay out the panels to coordinate with field-constructed components and to provide detailed direction for the eventual fabrication work. As the panels are being designed, it is imperative that they are brought into the building information modeling (BIM) system typically used on large construction projects. This process can be overlooked as the BIM process often focuses on the mechanical (M), electrical (E), plumbing (P), and fire protection (F) trades and not the exterior wall contractors. If the prefabricated wall panels are not included in the model, conflicts/clashes can occur between exterior wall studs, primary structure, and the MEPF trades. In traditional construction, this coordination is often done in the field without any prior planning, as the exterior wall studs can be field measured to accommodate the primary structure and placement adjusted as needed to avoid the MEPF systems. In prefabrication, the panels arrive in a specific size (allowing limited accommodation of field tolerances) and the studs are designed and spaced in a specific manner and may not be able to be relocated (or such relocation would cause additional rework to the attached sheathing, AWB, cladding attachments, and/or possible structural redesign or evaluation).

While the design of the panels is in progress, the structural components and incorporation of cladding/veneer in the manufacturing approach needs to be considered. These considerations will determine how much of the overall wall system will be assembled in the prefabrication facility versus what will be built at the site. Typically, bypass panels can more readily accommodate a complete panel that includes all the exterior cladding components installed in the manufacturing facility. Slab-to-slab panels, by definition, require some tie-in to field-constructed slabs, and thus the amount of cladding prefabrication that is feasible can vary depending on the slab edge detailing condition. Whether to include cladding on the prefabricated panels requires consideration of what cladding/finish material is desired, construction access needed after panels are installed, installation and access requirements for other field-installed facade components (e.g., windows, screens, canopies, etc.), and the timing of the panel installation. These processes have a direct impact on the cladding approach and scheduling for fabrication, delivery to the site, and erection on-site.

In the authors' own experience, we have observed the following common issues with including cladding in the prefabrication process. These issues can be difficult to remedy without additional field work or preplanning.

- Many common contemporary cladding systems (e.g., EIFS/stucco, thin masonry veneer, siding, etc.) rely on a backup membrane as the primary AWB. The AWB typically requires field-constructed tie-in between panels, and AWB continuity cannot be reliably provided if the panel edges are already covered with cladding material, preventing access to the AWB plane. As such, pre-clad panels typically require portions of cladding to be field constructed after the AWB tie-in is completed. This work can reduce cost and time savings, and even if carefully executed these patch/tie-in areas may be visible in some cladding assemblies.
- Some cladding systems (e.g., EIFS) can be easily damaged by swing staging or similar access methods, and thus may require rework or repair if there is subsequent construction access after panel installation.
- If all the finish material is not installed at the manufacturing facility, or if the color or material lots are not fabricated at the same time, there is a possibility of slight color variations between panels.

Schedule and logistics

The prefabrication approach is often selected to address schedule uncertainty, so early and decisive actions on the above-mentioned processes should be made with the project schedule in mind. Delays in the selection of the approach, cladding conditions, and of the subsequent panel engineering and design processes will reduce the overall turnover speed for the prefabrication process. Once the manufacturing of wall panels commences, schedule certainty can be locked in and coordinated with on-site construction scheduling. Fabricating these assemblies in an offsite location typically results in a more concise manufacturing process and yields more reliable and specific completion dates for the delivery schedule and erection times. Manufacturing can be done in a controlled environment off-site, so weather may not be a factor for weather-sensitive activities and overall quality may be improved. Off-site fabrication also allows for consistent work hours, so the overall time from start to finish is more predictable and typically reduced versus similar conditions built on-site. Off-site fabrication also results in labor reductions due to the ability to control the environment and the use of jigs and other manufacturing processes that are not available on a construction site.

These conditions also allow for increased quality control processes since the work is performed in a controlled environment and the completed assemblies are readily accessible (versus installation on-site and via swing stage or other construction access equipment), allowing for a closer and more detailed review process to be established. The quality control process must remain intentional because in these controlled settings and with repetitive work, complacency can set in. Owners, general contractors, and designers not intimately familiar with the fabrication work may assume that because the labor is in a better, more controlled environment, the workers will produce better work. While this is generally true, the quality of the work is still dependent on the tasks of each worker, and even brief lapses in worker performance

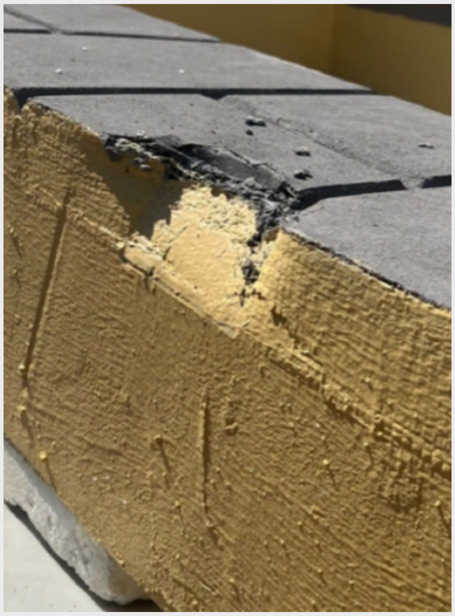

FIGURE 5. Strap damage on panel edge.

FIGURE 6. Panels with damaged AWB membrane from stacking during transport.

coupled with complacent quality control measures can lead to significant issues on-site once the panels are erected. The cost for repairs after erection exponentially increases due to access, the potential for additional investigation to understand the extent or frequency of such defects, and potential additional rework (e.g., more widespread coating or repair work to blend repairs into adjacent finishes).

As the manufacturing of panels continues, the logistics of deliveries can be finalized. In addition to delivery dates, the construction team must confirm delivery requirements and costs, as well as setting the standards for installation crew sizes and speed (e.g., on-site storage locations, labor and equipment handling needs, etc.). Understanding the speed at which the building structure is being constructed will directly affect the number of panels that can be installed, and thus the quantity and frequency of deliveries. Panels cannot be installed at a faster rate than the structure is being built, and there should be a steady cadence between the structural erection and installation of panels to optimize on-site panel erection crew size, the quantity of panels on-site at any given time, and a consistent installation of panels. Constant attention to these processes by the general contractor and panel subcontractor helps to provide the schedule certainty that prefabrication was designed to achieve. Properly coordinated, these factors will significantly reduce the crew size needed on the jobsite, reducing the construction site risks that an exterior wall contractor typically experiences. Reductions in crew size for installing panels and for any field-related work also increases schedule certainty by reducing chances for installation mistakes and rework. Generally, prefabrication results in the scope of field-related activities being reduced substantially, and thus the caliber of the skilled labor required naturally increases while the need for a higher quantity of laborers decreases.

Last, but certainly not least, prefabrication will directly impact the safety of each employee and the overall project. Controlled environments for manufacturing typically lead to more controllable safety measures. The manufacturing facility is typically a safer place compared to a construction site. On-site, those same employees would be subject to the hazards of the site and access methods, along with all the other trades that are currently working there. Work occurring in a manufacturing facility also reduces the number of workers on-site during the wall/cladding fabrication process, reducing potential hazards to both those workers and others on-site. Once the manufacturing process transitions to the site, the reduction in on-site scope and workforce continues to reduce these potential on-site hazards. The workload in particular areas and the time it takes those areas to be completed are also greatly reduced. The more environments and processes can be controlled, the more comprehensive and controllable those safety measures can be.

LESSONS LEARNED

From manufacturing, installing, and observing the construction of prefabricated panel construction, the authors of the paper have learned some valuable lessons over the last several years. These lessons can result in major repairs to the panels, which can be costly and time consuming, in addition to minimizing the advantages of prefabrication compared to traditional field-applied construction. These lessons include the following.

Shipping Damage

When transporting panels from the manufacturing facility to the project site, the panels are typically stacked on top of one another with dunnage between each panel. Once stacked, the panels are shrink-wrapped (for protection) and then placed on the bed of a tractor trailer. Straps are utilized to secure the panels for transportation. When strapping, care should be taken not to overtighten, because overtightening can result in strap marks (**Fig. 5**). Strap marks that damage framing or finished exterior conditions of the panels require repair and can result in refinishing of entire panels.

Membrane/AWB Cure Time Issues

Since the panels are stacked on top of one another during the transportation process, it is critical for the AWB membrane on backup panels to be properly cured prior to shrink-wrapping and shrink wrap and for transportation. If not properly cured, the membrane may stick to the dunnage; and when the dunnage is removed, the membrane may be damaged or in some instances ripped off the exterior sheathing (**Fig. 6**). Panels may require sheathing replacement or additional field work to address damaged membrane areas.

Proper Cure Time between Cladding Components

Since prefabrication is a manufacturing process, it is important for the fabricator to have constant production runs and not have panels sitting idle in the manufacturing facility. However, with any product installation, care must be taken to follow manufacturer recommendations related to cure times. For example, for some MVES systems, the slurry coat over the cement board sheathing must cure for a minimum of 24 hours prior to placing the brick. Once the masonry veneer is adhesively attached, this cladding layer also must cure for a minimum of 24 hours. Additionally, once the joints of the masonry veneer are grouted, the masonry (with placed mortar) must cure for approximately 10 days before the masonry cladding may be cleaned. If these cure times are not achieved due to production schedules, efflorescence or other issues may occur in situ.

Access to the Site

During the design phase, the team must determine whether or not there is sufficient access to the proposed work areas on the project site to transport and erect panels. At sites in crowded urban environments, access to the site may be particularly complex (**Fig 7**). Sites with one-way streets (especially with street parking allowed) may not provide adequate access to the site. In addition to street access, tree clearance and electrical line/pole clearance should also be reviewed to confirm adequate clearance is available for erecting the panels. Understanding the site access and potential limitations to this access around the building are critical to the panel design and to ensure that the panels can be delivered and installed.

Coordination (Crane Time and Sequencing)

During the preconstruction phase, coordination and project sequencing should be discussed and planned. Panels are set with cranes, either in place or loaded into the building. Therefore, shipping panels to the site must be coordinated with the project team (i.e., the general contractor, the concrete subcontractor, etc.) to minimize downtime for crane usage. If this is not properly coordinated, erection of the panels can be delayed, and stored panels will congest the site and be prone to damage.

Panels must also be fabricated and erected prior to subsequent trades (interior framers, plumbers, mechanical, etc.). If plumbers and mechanical contractors install their work prior to panels being placed, panels must either be reworked/reframed or the plumbing/mechanical work must be removed and replaced/reworked. Coordination and scheduling of these trades are critical to maximize the cost and schedule benefits typically expected when using prefabricated wall panels.

Dimensional Consistency

To help ensure there is dimensional consistency between the prefabricated panels and the structure, the engineering/layout of the panels must adhere to the design drawings. This effort requires an experienced engineering team that understands the structure, including slab and exterior beam thicknesses, which can vary around the building. Coordination with the structural design, as well as any structural modifications during construction, is critical in order for the panels to be planned and fabricated to the correct dimensions.

FIGURE 7. Example of tight site access around street poles and trees.

Prior to panels arriving on-site, the erector should check the dimensions within the structure and check the flatness and tolerances of the concrete placement. These dimensions should be referenced versus the final layout plan/engineered panel dimensions to verify the as-built structural dimensions.

In load-bearing panel applications, if the slab is incorrect or exceeds allowed tolerances, the slab must be prepared and corrected prior to the panels arriving. Slab/structure modifications can also be performed in slab-to-slab and bypass panels, but these modifications are less common. In slab-to-slab panels, a deep leg deflection track in the stud framing typically allows the erector to adjust panel connections and accommodate vertical dimensional variance within the structure. There is typically very limited tolerance allowance in bypass panels, but since the panels bypass the structure, the erector can generally adjust the connections to limit the impact of dimensional variations in the structure. If there are significant dimensional issues with the concrete placement, it is important to catch them early so that the concrete can either be repaired or the panels can be reworked. Reworking of the panels is often performed in the field, which may require the addition of studs, replacement of sheathing, and repair, replacement, or refinishing of panel cladding.

Once panels arrive on-site, the fabricator/erector typically will use a jig to measure the panels to help ensure that the panels match the ship tickets and are the correct dimensions. The erector will often select the tallest panel and use it as the benchmark. The remaining panels will be adjusted/shimmed as needed to help align the panels.

If panels are the wrong size due to insufficient coordination with the team, or engineering or fabrication issues by the panel fabricator team, then panels must be refabricated or modified in the field. This could result in large costs and delays to erecting the panels and drying in the building.

Panel Size vs. Setting Efficiency

Cranes are the most efficient way for setting bypass panels. However, on most projects, crane time is at a premium. For slab-to-slab panels, the crane may only be utilized to load panels onto the floor, and then panels can be erected by hand if the panel size can be maintained at about 15 ft. or less. If the panels are larger than 15 ft. for slab-to-slab panels, mechanical means are typically required due to the overall weight and magnitude of each panel. When additional mechanical means are needed for panel setting, the time in which it takes to set panels may be tripled.

Building Is Not Watertight Until Sealants Are Installed and Slab-to-Slab Infills Are Complete

One of the reasons why prefabrication is preferred over traditional field-applied construction is the speed of installing the exterior cladding. However, the exterior walls and cladding are not watertight until AWB tie-ins and/or cladding sealants are installed and until the slab edge conditions are completed on slab-to-slab panels. Installing sealants is typically one of the last components installed on the prefabricated wall panel assembly. The sequencing of this sealant activity should be confirmed with the construction team and understood in relation to other trades (e.g., interior work) prior to choosing prefabrication.

Damaging Finishes on Completed Panels During Slab Infill and Sealing of Joints

Access to the panel exterior when installing slab infills or sealant joints is typically provided by swing stages. Stages often travel multiple trips up and down the facade during these activities, and the swing stage rollers can leave marks or damage the new panel finish. This access-related damage can require refinishing of large areas of the panels. As discussed above, for some cladding systems the applicator may consider prefabrication of a backup panel or a panel without finish, and then installing the cladding system/finish in the field to limit this access damage.

Matching Finish of Slab Infills to Adjacent Completed Panels

The panels are made in a prefabrication facility, in some cases several months prior to installing the panel on the structure. Depending on the color chosen for the finish or possible changes in the manufacturing process during this time, the finish on panels produced later in the process or on field-installed slab infills between slab-to-slab panels may not match. These color match issues can result in refinishing/recoating large areas of the panel construction. Color matching over a long schedule duration is another reason why the applicator may consider prefabrication of a backup panel or a panel without finish, and then installing the cladding system/finish in the field.

Staining of Finish Related to Temporary Waterproofing Efforts

As mentioned above, a prefabricated wall system is not watertight until AWB transitions are made and sealants are installed. Attempts to temporarily waterproof terminations, joints, etc., often occur during the setting process until the AWB, cladding seals, and roof copings or other closure-type flashing is installed. These temporary terminations may still allow some water entry and can inadvertently cause water to pond on floor slabs behind the panels; this water can then flow over the face of the panels and stain finished surfaces (**Fig. 8**). To minimize this damage, temporary waterproofing should be installed to redirect water off of or out of the building to limit ponding on slabs or other surfaces where it can collect construction debris and then flow over the face of prefinished panels.

Damaged Finish Related to Welding Connections

Weld plates are often used to attach prefabricated panels or other adjacent structural features, requiring field welding operations near the panels. Welds that occur on or near the backside of finished panels can result in burn marks or stains in the exposed exterior finish (**Fig. 9**). Repairing these burn marks or stains often requires repair of the panel cladding or refinishing of the exterior surface.

CONCLUSION

Prefabricated wall systems can save time and cost and have become a more common construction approach on

FIGURE 8. Stains from ponded water running off temporary waterproofing and over panel finished surface

FIGURE 9. Weld damage on the exposed exterior face of a prefabricated panel.

many projects. However, prefabricated wall systems are not suitable for all buildings or exterior wall types. Therefore, the design and construction team should:

- Understand the types of buildings and structures that are best suited for prefabricated wall panels.
- Understand the prefabrication process and construction practices that are necessary for a successful prefabrication project.
- Learn from mistakes and lessons learned by others to avoid pitfalls that reduce the advantages of prefabrication versus traditional field-applied construction.