Dimension Stone Use in Building Construction

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Editors

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Peer Review Policy

Each paper published in this volume was evaluated by two peer reviewers and at least one editor. The authors addressed all of the reviewers' comments to the satisfaction of both the technical editor(s) and the ASTM International Committee on Publications.

The quality of the papers in this publication reflects not only the obvious efforts of the authors and the technical editor(s), but also the work of the peer reviewers. In keeping with long-standing publication practices, ASTM International maintains the anonymity of the peer reviewers. The ASTM International Committee on Publications acknowledges with appreciation their dedication and contribution of time and effort on behalf of ASTM International.

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Foreword

This publication, *Dimension Stone Use in Building Construction*, contains twelve peer reviewed papers presented at the symposium of the same name held in Tampa, Florida on October 31, 2007, and published in the Journal of ASTM International. The symposium was sponsored by ASTM Committee C18 on Dimension Stone and co-chaired by Kurt R. Hoigard, P.E. of Raths, Raths & Johnson, Inc., Willowbrook, Illinois, USA, and Michael J. Scheffler, P.E. of Wiss, Janney, Elstner Associates, Inc., Northbrook, Illinois, USA.

Front and Rear Cover Images

Constructed as part of a cooperative study by ASTM Committee C18 on Dimension Stone and the National Bureau of Standards (NBS, the predecessor to the National Institute of Standards and Technology, NIST) to study weathering effects on dimension stone, the NIST Stone Exposure Test Wall is approximately 38 feet long, 13 feet high, and incorporates 2,352 individual dimension stone samples. The stone samples represent the merging of several collections including: the Centennial Collection of U.S. Building Stones displayed at the 1876 centennial exhibition in Philadelphia, PA; commercial building stones assembled for the 1880 United States census; and building stones from other countries originally displayed in the Smithsonian Institution. Erected in 1948 at the old NBS facility in Washington, D.C., the wall was moved to its current site on the NIST campus in Gaithersburg, MD in 1977 where it remains under the care of the Building Materials Division of the NIST Building and Fire Research Laboratory and is periodically visited by ASTM Committee C18. Additional information can be found at www.stonewall.nist.gov.

Contents

Overview

This book represents the work of 19 authors that prepared papers for presentation at the Symposium on Dimension Stone Use in Building Construction held in Tampa, Florida, USA on October 31, 2007, and publication in the Journal of ASTM International. Prior to publication, each paper underwent two reviews by peers knowledgeable of the subject matter. Sincere thanks are offered to the writers, presenters, and reviewers who donated countless hours of their time in order to share their knowledge and without whom neither the symposium nor this book would have been possible.

The symposium was held in conjunction with a regularly scheduled meeting of the symposium sponsor, ASTM Committee C18 on Dimension Stone. Its objective was to promote information exchange regarding the state of the art in the use of dimension stone in building and pavement construction. In the eight years since the 1999 Symposium on Dimension Stone Cladding, and the subsequent publication of *ASTM STP 1394, Dimension Stone Cladding: Design, Construction, Evaluation, and Repair*, a substantial amount of work has been done in the fields of new dimension stone design and the assessment and rehabilitation of existing stone installations. Twelve presentations were grouped into four sessions: strength testing, design, evaluation and investigation, and durability. Written versions of all twelve of these presentations are assembled in this book.

Strength Testing

Both of the papers in this section address testing to determine strength characteristics of dimension stone cladding panels. Authors Naggatz and Gerns discuss differences in test results obtained using the relatively small test specimens prescribed by ASTM C 880 and the larger ASTM C 1201 specimens which include entire cladding panels and their connections. Scheffler and Kneezel address strength, durability and performance characteristics of composite stone-faced aluminum honeycomb cladding panels, the performance of which is highly dominated by the aluminum portion.

Design

The four papers in this section cover a wide range of topics. Hensel presents the advantages and disadvantages of three common dimension stone paving installation techniques, including pertinent stone material properties and detailing issues. Authors Lammert and Hoigard discuss relationships between stone material strength, anchorage strength, and induced stress states for four common dimension stone cladding anchorage configurations. Conroy and Hoigard address the interaction of stone, anchorage, and back-up structure relative stiffness on dimension stone cladding anchor loads and panel stresses under lateral loads. Lewis provides an update on ASTM Committee C18's progress toward developing building code requirements for exterior stone cladding installations.

Evaluation and Investigation

The three papers in this section offer author observations regarding investigations into the causes of dimension stone cladding deterioration and failure. Authors Erdly and Valentino describe their experience investigating granite and limestone failures on a 100+ year old church facade. Lewis discusses various issues that can affect the integrity of thin dimension stone cladding. Chin addresses the mineralogy, structure, strengths, and weaknesses of travertine as a cladding material and some common causes of travertine cladding failures.

Durability

The three papers in this section address the complex issue of dimension stone durability using three different approaches. Authors Grelk, Christiansen, Schouenborg, and Malaga summarize findings from a review of over 140 papers on this topic published between 1897 and 2006. Schouenborg, Grelk, and Malaga describe a large-scale European research project to investigate the causes of marble and limestone cladding panel bowing, develop preconstruction testing parameters to assess bowing potential, and assess proposed remedial efforts to reduce or inhibit ongoing bowing. Authors Bortz, Powers, and Wonneberger describe a proposed laboratory test to estimate weathering-related stone strength loss and provide correlations with strength loss caused by natural weathering.

Summary

The papers assembled in this book demonstrate a continuing advancement in the understanding of dimension stone use in building construction. Investigations of distressed stone installations, historical review of in-place performance, laboratory testing, and computerized analysis continue to improve the knowledge base from which designers of new buildings and restorers of older buildings can draw.

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SECTION I: STRENGTH TESTING

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Steven G. Naggatz[1] *and Edward A. Gerns*[1]

Full-scale Flexural Strength Testing for Stone Cladding Design

ABSTRACT: Beginning with the Industrial Revolution and rapidly increasing following World War II, building cladding systems changed dramatically. Exterior wall construction that had previously consisted of thick mass masonry evolved into contemporary curtain wall systems. As curtain wall systems evolved, the economics of reducing the thickness of curtain wall components became more important. The use of thin stone veneers on buildings in the modern era was not associated with increased knowledge of stone material properties based on testing and research. Rather, the reduction in thickness is attributable to advancements in fabrication technology, economy in construction, and a rational approach to design of veneer systems. This paper will discuss the evolution of the use of stone in curtain wall systems and the associated ASTM standards for building stone. Early ASTM standards for building stone were limited to tests used to evaluate the compression strength of stone which was consistent for mass bearing walls of the early 20th century. As stone was increasingly used more as a veneer, standards for modulus of rupture (ASTM C 99) were introduced. As systems continued to evolve, flexural strength (ASTM C 880) was introduced to address material variability. The evolution of ASTM C 880 included increasing specimen width to further incorporate variability of the stone. Recently, the authors have introduced full panel testing in the design and evaluation process. This paper will present data and discuss the differences in test results for three case studies where both ASTM C 880 specimens and full panel flexural strength testing was performed. The focus of this section will be to discuss the implications of performing full-size panel testing in addition to ASTM C 880 standard specimen testing on design of stone cladding systems for tall building construction as a natural evolution of the design and evaluation process for thin stone cladding systems.

KEYWORDS: flexural strength, full scale, specimen

Introduction

Stone has been used as a building material for thousands of years. Its aesthetics and sense of permanence have made it a popular material among society in general, and specifically for builders and architects. Historically, buildings of great significance, such as churches, civic, and government buildings were constructed of stone to reflect the importance of the building within the culture and fabric of the urban context.

History

Historically stone was used for both decorative and functional purposes. With few exceptions, building systems incorporate inexpensive backup materials in combination with more expensive facing. Early stone structures were typically multi-wythe load bearing assemblies that combined high quality stone finished to very tight tolerances with a looser rubble or brick backup.

Within the past 150 years advances in technology and the introduction of new building systems have changed how stone is incorporated into the building systems. The Egyptians used large blocks of rough-cut limestone arranged in a geometrically stable configuration to create the pyramids which were both functional and symbolic. The pyramids were clad with a veneer of dressed limestone and granite installed to tolerances of less than 1/25 in. (1.0 mm) to create the final form [1].

The Greeks also used large blocks of stone to build monuments of completely different forms from the

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Egyptians. The characteristic post and beam construction of the Greek temples combines structural function with visually pleasing forms. One of the earliest known uses of thin stone has been documented on the frieze portions of the Greek temples. These stones, used for decorative purposes, were hand-sawn and held in place with metal spikes.

The most dramatic change in building construction was the result of the industrial revolution of the 19th century. The development of new machinery facilitated the economical production of metal shapes that led to the development of the skeleton frame structural system. This system enabled the exterior wall to be used as a nonload bearing component of the building. As a result, the structural function of the exterior facade was no longer necessary. The facade could be treated as a skin that wrapped the skeletal frame. The skin still needed to transfer wind loads to the frame, but it no longer had to support interior floor loads.

Early skeletal frame buildings used numerous exterior cladding materials. Brick, terra cotta, and stone were all used, with economics frequently dictating both the location and quantity of material. Stone, still a relatively expensive cladding material, was frequently used only on the lower floors and interiors of high-rise structures. Early methods of anchoring were varied and frequently experimental. Very few of the early applications exist today due to remodeling and deterioration.

The building boom of the early 20th century and the resulting dramatic increase in building heights resulted in the need for increased economies of material. Early skyscrapers tended to use primarily brick and other unit type materials, but by the 1920s limestone slabs began to be used with greater frequency [2]. Many of the buildings of the 1920s and 1930s used granite and marble in the lower floors of the building and limestone panels on the higher portions of the facades. The uniformity of appearance of the limestone reinforced the architectural aesthetic of the Art Deco massing. The richness of the granite and marble color and veining accentuated the desire for human scale at the base of the buildings. The panels were typically at least 4-in. (102-mm) thick and stacked between floors. During this time and until the 1950s, each floor was typically designed individually with horizontal movement joints installed directly below the support.

With the development of the curtain wall system and the rise of modernism in the 1950s, stone began to be used as a panel within curtain wall or facade systems. The stone panels were used in vertical bands for column covers or horizontal strips for spandrel panels. Numerous techniques were employed to support the stone panels both within curtain walls as well as independent support techniques.

The 1960s and 1970s brought the development of composite systems which included stone-faced precast concrete panels and stone-faced composite panels. Development in technology during this period also led to the use of various prefabricated systems where stone panels were mounted to a supporting structural component in the shop and then transported to the site for erection on the building.

By the late 1970s, corresponding with the rise of the post-modern style of architecture, the resurgence in popularity of stone led to dramatic increases in the use of stone as an exterior cladding material. The material was still relatively expensive; thus designers experimented with systems utilizing stone cut to as thin as 1/4 in. (6 mm) in composite panels. Numerous support systems, many of which had been developed by the stone fabricators, were also available. These systems became widely used because they facilitated a rapid installation.

Today numerous systems are available to install stone on the exterior of buildings. Many factors must be considered by the designer in both the design and detailing of stone support systems to prevent premature failure and ensure long-term durability. Lessons learned from investigation of older thin-stone clad buildings provide valuable lessons in both the design and restoration of thin-stone clad buildings.

Stone Types

Unlike other materials used in construction, the physical characteristics of stone vary greatly between geologically different stones as well as between stone of the same type. These variations contribute to the inherent beauty of stone as well as potentially varied physical characteristics. Stone, used in building construction, is categorized in one of the following types [3].

Sedimentary

These include limestone, travertine, sandstone, brownstone, and shale. This type of stone is the product of deposits of sediment materials in prehistoric river and lake beds. The sediment is the product of decomposition and erosion of other rocks, minerals, and organic matter which is bonded together through compaction and naturally created cementitious products. Sedimentary stone is characterized by distinct bedding planes between individual layers of material and is characterized based on its grain size.

Igneous

These include granite and schist. This type of stone is the result of volcanic activity and the consolidation of molten magma. Igneous rock typically contains quartz, the crystal form of silica. Classifications of igneous rocks are based on the silica content within the stone.

Metamorphic

These include marble and slate. Metamorphic stones are the result of sedimentary or igneous stone being subjected to millions of years of heat and pressure resulting in a recrystallization of pre-existing rock. Two types of metamorphic processes can occur to change rock. The first is thermal metamorphic 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 and 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. Slate is the most commonly used example of this type of stone. Metamorphic stone is categorized based on the premetamorphosed rock.

Stone Characteristics

Because stone is not a manmade product its physical and aesthetic characteristics can vary significantly even within the same quarry. These unique features of stone include:

1. Natural planes of weakness, such as cleavage planes, bedding planes, and rifts occur within any quarry. These features are essentially discontinuities within the matrix of the stone.
2. The physical properties of an individual stone will vary depending on if it is tested in a wet or dry condition.
3. Stone is not an isotropic material and therefore its strength will vary depending on the orientation of load.
4. Stone is a heterogeneous material, which contributes to variability.

Stone within each of the geologic categories has its own physical characteristics. Within the past 30 years, fabricators and stone distributors have established test procedures and minimum standards for material properties. Historically, however, stone was used as very compact shapes that were subjected primarily to compressive forces. The building as a whole was massive enough that lateral loading on individual components was not an issue. The lateral loading was resisted by the geometry of the structure rather than individual components. Early stone cladding systems relied on empirical techniques rather than known material properties. The later curtain wall systems required greater attention to the behavior of the system components and the load path. More specifically, the stone was subjected to bending stresses in systems with minimal redundancy. Since the load path is more complex, yet defined, more conventional analysis techniques can be used to assess stresses. While conventional techniques are commonly used for materials which are relatively consistent and readily standardized, stone is natural and variable.

Traditionally recognized stone properties include compression, flexural, shear and tensile strength, density, abrasion resistance, absorption, coefficient of thermal expansion, and modulus of elasticity. Table 1 illustrates the range of properties which exist for various stones. The table provides an indication of the variability that can exist even with the same geological category of stone.

ASTM Standards

One of the earliest ASTM standards introduced related to the strength of stone was compression testing, ASTM Standard C 170, Test Method for Compressive Strength of Dimension Stone. Like many of the early ASTM standards, compression was a commonly and easily tested property. The first version of ASTM C 170 was published in 1941 which represents a time of transition between traditional load bearing

TABLE 1—*Range of properties for various stones.*[a]

	Compressive Strength (C 170)		Modulus of Rupture (C 99)		Absorption (C 97)	Coefficient of Thermal Expansion	Young's Modulus (E)	
(Units)	(ksi)	(MPa)	(ksi)	(MPa)	(% max) (weight)	($\times 10^{-6}/°F$)	(psi $\times 10^6$)	(GPa)
Marble	7.5–28	51.7–193.1	1.0–4.0	6.9–27.6	0.2	27–51	2.0–15	13.8–103.4
Granite	19–52	131.0–358.5	1.5–5.5	10.3–37.9	0.4	37–60	2.0–10	13.8–68.9
Limestone	1.8–32	12.4–220.6	0.4–2.9	2.8–20.0	3.0–12	17–63	0.6–1.4	4.1–9.7
Sandstone	2.0–37	13.8–255.1	0.3–2.0	2.1–13.8	1.0–20	37–63	1.0–7.5	6.7–51.7

[a]Portions of this table were excerpted from Marble and Stone Slab Veneer, published by the Marble Institute of America.

wall systems and curtain wall or hybrid systems. The relative value of compression strength for a stone was of some significance to designers, but generally provided a somewhat quantitative assessment of the "hardness" of the stone. Consistent with the common perception of the time, harder materials were deemed as better materials. In reality, the compression strength values almost never are approached with modern cladding systems under normal conditions. Even historical load bearing cladding rarely approached these values since prior to 1900, building rarely exceeded 250 ft (76 m). Literature from the 1900s states that compression stresses in bearing should be no greater than 1/10 the crushing strength of the stone (factor of safety of 10). Anticipated loads were stated to be 60 000 to 70 000 psf (417 psi to 486 psi) (2.9 to 3.4 MPa). The published crushing strength of virtually all stone at that time exceeded 6000 psi (41.4 MPa) with the exception of a few very soft sandstones [4]. Various machines were developed for testing compression strength, with no consistency to the specimen size or aspect ratio. Noncubic specimens inevitably introduced inconsistencies related to buckling rather than pure compression.

Historical references are also made to cross bending tests, but these are identified for stone beams with a midspan point load applied to failure of the specimen [5]. This loading pattern was intended to assist in the design of stone lintels and later evolved into modulus of rupture testing adopted by ASTM. The understanding that thinner cladding panels were also subject to bending to resist wind loads led to the introduction in 1931 of ASTM Standard C 99, Test Method for Modulus of Rupture of Dimension Stone. This standard remains in use today and provides a less conservative value of the bending capacity of a particular stone. This is a common reported value by quarries since the value tends to be higher than the more conservative ASTM C 880 procedure. ASTM C 99 consists of applying a single point load to the midspan of a specimen. This results in a single point of maximum moment and in many instances the point of failure of the specimen. Since stone is heterogeneous, this may not actually represent the weak point of a specimen.

In 1978, ASTM Standard C 880, Test Method for Flexural Strength of Dimension Stone, was introduced which was intended to more precisely determine a flexural strength of a particular stone. Rather than a single point of loading, point loads are applied at the quarter points of the specimen to create a uniform moment across the middle half of the test specimen. Thus, rather than a single point load, the heterogeneous nature of the stone can be better evaluated by subjecting a greater area of the specimen to the full moment. This standard initially required that specimens be tested parallel and perpendicular to the bedding plane (sedimentary) or rift.

Initial versions of the standard stipulated that the specimens were to be 1 1/2-in. (38-mm) wide and ten times the thickness of the material in length. In 1993, the specimen width was increased to 4-in. (102-mm) wide with the length remaining ten times the thickness. The increased width of the specimens, at least in part, was introduced to address the heterogeneous characteristics of the stone.

The natural progression of this trend is to increase the width of the specimens to further incorporate variability of the stone. Increasing specimen width may lead to test procedures that are inconsistent with the application if the aspect ratio of the test specimen varies significantly from the actual panel geometry. For this reason perhaps the most appropriate test specimen is a full-scale panel. But is this approach appropriate and reasonable? Full-scale panel testing is not always practical and may not be necessary depending on the application. Conventional detailing and stone with significant test data, or both, likely do not warrant this type of testing. However, for less conventional applications and detailing, and for projects with significant quantities of stone to be used for cladding materials, consideration should be given to include full-scale testing in the design process.

TABLE 2—*Averages, standard deviations, and coefficients of variation obtained from average values.*

Condition (Units)	Specimens Tested (No.)	Nominal Width (in.)	Nominal Width (mm)	Nominal Depth (in.)	Nominal Depth (mm)	Span (in.)	Span (mm)	Flexural Strength (psi)	Flexural Strength (MPa)	Standard of Deviation	Coefficient of Variation
Dry	5	8	203	4	102	40	1020	591	4.1	63	10.6 %
Wet	5	8	203	4	102	40	1020	415	2.9	33	7.9 %
Dry	5	8	203	3	76	30	760	683	4.7	62	9.1 %
Wet	5	8	203	3	76	30	760	435	3.0	46	10.7 %
Dry	3	4	102	1.25	32	12.5	320	781	5.4	52	6.7 %

Case Studies

The following case studies are presented by the authors based on previous investigations performed by Wiss, Janney, Elstner Associates, Inc. In the projects presented, various combinations of standardized testing and full-panel testing were performed. The specific goal of each testing protocol varied by project, therefore making specific definitive conclusions is difficult without more data. The intent of presenting these case studies is to provide a starting point for further testing and data evaluation to better assess individual specimen testing compared to full-panel testing.

Stone Testing Results

Flexural strength results were analyzed among stone used for three different buildings. Testing was performed on Italian limestone, Italian marble, and a North American marble. The limestone material was tested in support of specifying panel sizes for a building during the design phase. The marble was tested in each case to assess flexural strength of building panels on existing buildings.

Italian Limestone

Three different panel size thicknesses were tested to determine the flexural strength of an Italian limestone to be used on the facade for a new building. Three test specimens each measuring 1.25-in. (32-mm) thick were tested in the dry condition only in accordance with ASTM C 880. Since the job thickness had not been determined for this project, five additional specimens each were tested in both the wet and dry condition for both 3-in. (76-mm) thick and 4-in. (102-mm) thick material. Results of the flexural strength testing are presented in Table 2.

Italian Marble

Italian marble was tested to determine the flexural strength of panels that had been in service on an existing building for approximately 15 years. An in-situ test apparatus was built to test the flexural strength of whole panels on the building. Each 1 1/4-in. (32-mm) thick panel was approximately 3-ft. 8-in. (1.1-m) wide by 4-ft. 2-in. (1.3-m) tall. In-situ load tests were performed on whole panels until failure. Subsequent to the in-situ load test, each of the whole panels was cut into 1 7/8-in. (48-mm) wide by 14-in. (356-mm) long prisms parallel and transverse to the span of the panels. With the exception of a few panels, at least five C 880 specimens were tested parallel to the span of the original panel.

Pilot testing of a panel and experience with other building stones indicated that when this marble is tested in a wet condition it has a lower strength than when tested in the dry condition. Accordingly, all C 880 tests were performed after the marble was soaked in water for 48 hours as specified by ASTM C 880. All of the specimens were tested with the outside face in tension, which is the more critical loading condition on the building. The flexural strength results for full-panel testing and for C 880 specimens tested parallel to the span of the original panel are presented in Table 3.

North American Marble

North American marble panels were tested to determine the flexural strength of panels that had been in service on an existing building for approximately 30 years. Full-scale test panels were slightly over 62-in. (1.6-m) tall by 15 3/4-in. (400-mm) wide by 3-in. (76-mm) thick, and were subjected to quarter point

TABLE 3—*Flexural strength of C 880 specimens tested perpendicular and parallel to span, flexural strength of full panels, and percent difference between full panel and C 880 specimen tested parallel to span.*

Avg. Flexural Strength Perpendicular to Span		Specimens Tested	Avg. Flexural Strength Parallel to Span		Specimens Tested	Flexural Strength Full Panel		Percent Difference: Full Panel and Parallel to Span
(psi)	(MPa)	No.	(psi)	(MPa)	No.	(psi)	(MPa)	
NA	NA	0	722	5.0	5	768	5.3	+6.4 %
844	5.8	4	949	6.5	5	942	6.5	−0.7 %
786	5.4	3	1196	8.2	6	1188	8.2	−0.7 %
650	4.5	1	806	5.6	2	1043	7.2	+29.4 %
414	2.9	2	349	2.4	5	662	4.6	+89.7 %
NA	NA	0	404	2.8	4	681	4.7	+68.6 %
NA	NA	0	283	2.0	5	504	3.5	+78.1 %
NA	NA	0	508	3.5	6	672	4.6	+32.3 %
NA	NA	0	882	6.1	7	1093	7.5	+23.9 %
890	6.1	1	992	6.8	5	1046	7.2	+5.4 %
570	3.9	1	659	4.5	2	708	4.9	+7.4 %
690	4.8	4	832	5.7	5	1084	7.5	+30.3 %
1338	9.2	4	1197	8.3	4	1159	8.0	−3.2 %
582	4.0	4	569	3.9	3	654	4.5	+14.9 %
635	4.4	2	852	5.9	6	920	6.3	+8.0 %
671	4.6	4	1255	8.7	5	1403	9.7	+11.8 %
752	5.2	3	926	6.4	5	881	6.1	−4.9 %
NA	NA	0	948	6.5	5	833	5.7	−12.1 %
584	4.0	2	517	3.6	6	649	4.5	+25.5 %
619	4.3	1	1107	7.6	4	1201	8.3	+8.5 %
647	4.5	4	631	4.4	6	723	5.0	+14.6 %
766	5.3	2	803	5.5	5	819	5.6	+2.0 %
931	6.4	8	866	6.0	6	799	5.5	−7.7 %
708	4.9	3	698	4.8	6	561	3.9	−19.6 %
708	4.9	3	698	4.8	6	742	5.1	+6.30 %
789	5.4	1	673	4.6	7	675	4.7	+0.30 %
888	6.1	3	630	4.3	3	809	5.6	+28.4 %
664	4.6	2	560	3.9	7	786	5.4	+40.4 %
609	4.2	4	543	3.7	4	698	4.8	+28.6 %
657	4.5	6	657	4.5	5	720	5.0	+9.6 %
533	3.7	2	636	4.4	5	687	4.7	+8.2 %
738	5.1	1	568	3.9	7	514	3.5	−9.5 %

loading. Of the ten full-scale panels tested, half of the panels were tested with the inside face in tension, and half of the panels were tested with the outside face in tension. All of the full-scale panels were tested as received under ambient laboratory conditions.

Six test specimens with dimensions of approximately 16-in. (406-mm) long by 4-in. (102-mm) wide by 1 1/2-in. (38-mm) thick were cut from each of the ten full-scale test panels. Of the 60 specimens, half were removed from panels that had been tested with the inside face in tension and half were removed from panels that had been tested with the outside face in tension. Of the six specimens cut from each panel, two were removed from the outside face, two were removed from the inside face and two were removed from the center portion of the 3-in. (76-mm) thick panel. Half of the specimens from each region were oriented vertically and half horizontally.

Test procedures described in ASTM C 880 were used to determine the flexural strength of test specimens cut from the full scale marble panels. The specimens cut from the outside and inside face of the panel were tested with the exposed face in tension, while specimens cut from the center portion of the panel were typically tested with the outside face in tension. Half of the specimens from each region within the panel were tested to induce flexural failure parallel to the natural veining of the marble, and half perpendicular.

C 880 specimens were tested at room temperature in the wet condition after the marble was soaked in water for 48 hours. The specimens were loaded at a rate of 600 psi (4.2 MPa) per minute using 120 kip (533.8 kN) Satec Universal Test Machine. Table 4 presents a summary of the flexural strength test data.

As the 1 1/2-in. (38-mm) thick specimens were cut from the 3-in. (76-mm) thick panels, some of the

TABLE 4—*Averages, standard deviations and coefficients of variation obtained from average values.*

Tension Face of Full-scale Panel	Full-scale Flexural Strength		Nominal Width		Nominal Depth		Span		C 880 Flexural Strength		Standard	Coefficient	Percent Difference: Full Panel and
(Units)	(psi)	(MPa)	(in.)	(mm)	(in.)	(mm)	(in.)	(mm)	(psi)	(MPa)	of Deviation	of Variation	C 880 Specimen
Inside	399	2.8	4	102	1.5	38	16	406	368	2.5	60	0.164	+8.4 %
Inside	512	3.5	4	102	1.5	38	16	406	323	2.2	100	0.309	+58.5 %
Inside	516	3.6	4	102	1.5	38	16	406	413	2.8	103	0.249	+24.9 %
Outside	585	4.0	4	102	1.5	38	16	406	517	3.6	181	0.351	+13.2 %
Outside	836	5.8	4	102	1.5	38	16	406	730	5.0	131	0.179	+14.5 %
Inside	461	3.2	4	102	1.5	38	16	406	277	1.9	83	0.3	+66.4 %
Outside	238	1.6	4	102	1.5	38	16	406	218	1.5	44	0.202	+9.2 %
Outside	747	5.2	4	102	1.5	38	16	406	586	4.0	60	0.103	+27.5 %
Inside	409	2.8	4	102	1.5	38	16	406	599	4.1	74	0.124	−68.3 %
Outside	590	4.1	4	102	1.5	38	16	406	479	3.3	34	0.071	+23.2 %
Averages	529	3.6							451	3.1	161	0.357	+17.3 %

specimens and scrap material bowed. The bowing may indicate an internal stress release, which may have skewed the C 880 test results. In order to reveal any distortion in the flexural strength of 1 1/2-in. (38-mm) thick specimens caused by slicing the panel thickness, five additional test specimens with dimensions of 3-in. (76-mm) thick by 4-in. (102-mm) wide by 32-in. (813-mm) long were cut from five of the full-scale test panels. Three of the full-scale panels from which these specimens were removed had been tested with the inside face in tension, while two had been tested with the outside face in tension. The 3-in. (76-mm) thick test specimens could not be removed from the same test panels as the 1 1/2-in. (38-mm) thick specimens due to lack of material. All five specimens were oriented parallel to the original span and were tested with the outside face in tension. Flexural strength results for 3-in. (76-mm) thick panels tested are shown in Table 5.

Comparison Between Full Panel and C 880 Test Results

There is a significant size difference between the standard ASTM designated specimens and the full-scale panels tested. The standard ASTM designated size sample for a 3-in. (76-mm) thick panel is 32 to 34-in. (813 to 864-mm) long and 4 1/2-in. (114-mm) wide. It is likely that the bending behavior of the full-scale panels is close to two-way plate bending while the ASTM specimens are closer to beam bending behavior. This is a possible explanation for the higher average values of the full-scale tests when compared to C 880 testing.

TABLE 5—*Averages, standard deviations, and coefficients of variation obtained from average values.*

Tension Face of Full-scale Panel	Full-scale Flexural Strength		Nominal Width		Nominal Depth		Span		C 880 Flexural Strength		Percent Difference: Full Panel and C 880
(Units)	(psi)	(MPa)	(in.)	(mm)	(in.)	(mm)	(in.)	(mm)	(psi)	(MPa)	Specimen
Outside	754	5.2	4	102	3	76	30	760	978	6.7	−22.9 %
Outside	611	4.2	4	102	3	76	30	760	630	4.3	−3.0 %
Outside	643	4.4	4	102	3	76	30	760	620	4.3	+3.7 %
Outside	534	3.7	4	102	3	76	30	760	575	4.0	−7.1 %
Outside	369	2.5	4	102	3	76	30	760	402	2.8	−8.2 %
Average	582	4.0							641	4.4	−9.2 %
Standard Deviation	143								210		
Coefficient of Variation	24.6 %								32.7 %		

Discussion

To some extent, similarities exist between stone design and wood design. In wood design, allowable stresses decrease as the cross section of the member increases. The reduction is intended to account for increased probability of hidden defects within larger dimensional units. Checks, splits, and knots reduce the capacity of wood members in much the same way that inclusions, veins, and bedding planes will reduce the capacity within a stone specimen. However, the data presented above tend not to support such analogies. Direct comparison between allowable stresses in wood design and stone design is not entirely appropriate since the wood members are linear elements while stone is more planar.

In general the ultimate strength of the full-scale panels exceeds the C 880 values. This tends to reinforce the notion that stone panels behave more similarly to two-way plates than to simple span beams. Obviously aspect ratio of the full panel will have an impact on the test results. The current version of the C 880 standard is still primarily testing linear specimens. Depending on the thickness of the test specimen, the amount of two-way bending varies. Finally, metamorphic stone, such as marble which has an oriented microstructure, may complicate the issue since the behavior of oriented microstructures in plate bending has not been widely evaluated for stone.

Conclusion

Upon analysis of the three case studies presented, there is some indication that trends potentially exist when comparing standardized testing results to full panel tests. The statistical viability of these trends is difficult to assess without further testing and data evaluation. Ultimately, the trend or determination of a more exact ultimate strength of a particular stone may be outweighed by the cost to perform such rigorous testing. Variability of the stone, behavior of the system, and issues such as tolerances and installation variability may offset more specific design values. In many cases, the economics and time constraints of a project may offset justifying full-panel testing. Ultimately, safety factors and engineering judgment are critical in most design situations.

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Testing of Composite Stone Faced Aluminum Honeycomb Panels

ABSTRACT: Physical and mechanical testing was conducted on composite stone faced aluminum honeycomb panels manufactured in the United States for use as exterior veneer panels on buildings. Testing was conducted on specimens cut from new stone faced panels as well as stone faced panels removed from service. For comparison purposes specimens cut from new unfaced honeycomb panels were also tested. The purpose of the testing was to measure certain strength, durability, and performance characteristics of the faced honeycomb panel's composite construction. The stone faced specimens tested included those faced with travertine, limestone, and granite. Old travertine faced specimens had been in service for more than 18 years. The results very strongly suggest that the structural performance of the composite panel is dominated by the aluminum honeycomb portion. The stone portion is essentially a thin veneer adhered to the honeycomb that provides little if any structural strength.

KEYWORDS: aluminum, honeycomb, composite, stone faced, travertine, limestone, granite, flexural strength, tensile bond, accelerated weathering

Description of Stone Faced Honeycomb Panels

The panels tested are composed of 4.8 mm (3/16 in.) to 6.4 mm (1/4 in.) thick stone adhered to a 19.1 mm (3/4 in.) aluminum honeycomb. The aluminum honeycomb is faced on both sides with a fiber-reinforced epoxy skin. During fabrication aluminum honeycomb is adhered to both sides of 19.1 mm (3/4 in.) thick stone, the stone is cut through the center of its thickness, and the exposed surface of each panel is finished.

Description of Test Program

The laboratory testing performed included testing newly fabricated honeycomb panels faced with travertine, limestone, granite, and honeycomb panels faced with travertine that had been in service for more than 18 years. Flexural strength testing and tensile bond strength testing of faced honeycomb panels specimens exposed and not exposed to accelerated weathering was performed. Testing of honeycomb panel anchors was also performed but the results are not included in this paper as the anchor strength is not directly related to the composite aspect of the panel construction. The test program included the following tests, the results of which are summarized in Tables 1, 2, 3, and 4:

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Test Program

ASTM designation	Test procedure	Size	Facing/anchor type	Number of tests
C880	Flexural Strength	25.4 mm (1 inch) ×50.8 mm (2 in.) × 30.5 cm (12 in.)	No Stone	6
			New Limestone	52
			New Travertine	40
			Old Travertine	40
			New Granite	52
N/A	Freeze/thaw 100 cycles (from −10 degrees F to 170 degrees F)	25.4 mm (1 in.) ×50.8 mm (2 in.) × 30.5 cm (12 in.)	No Stone	5
			New Limestone	19
			New Travertine	20
			Old Travertine	19
			New Granite	20
Modified D897	Tensile Bond Testing	25.4 mm (1 in.) × 25.4 mm (2 in.) ×50.8 mm (2 in.)	New Limestone	6
			New Travertine	6
			Old Travertine	6
			New Granite	6

The purpose of this test program was to measure, via testing, certain strength, durability, and performance characteristics of the stone faced honeycomb panel's composite construction for a sample of new panels as well as a sample of panels that had been in service for several years. The test program included specimens removed from new and old stone faced aluminum honeycomb panels as well as specimens removed from panels having no stone facing.

Flexural Strength

Test Procedure—Test procedures described in ASTM C880 Standard Test Method for Determining Flexural Strength of Dimension Stone [1] were used to determine the flexural strength of test specimens cut from faced honeycomb panels. This test method is for determining the flexural strength of stone through quarter point loading of simple beam specimens with a span/thickness ratio of 10. The faced honeycomb panel test specimens measured 30.5 cm (12 in.) long by 50.8 mm (2 in.) wide by 25.4 mm (1 in.) thick. The ASTM designated specimen width of 101.6 mm (4 in.) was replaced with the 50.8 mm (2 in.) specimen width to maximize the number of conditions that could be tested with a limited amount of material. The span was 25.4 cm (10 in.) and the load points were 63.5 mm ($2-1/2$ in.) in from the support points. The specimens were loaded at a rate of 74.8 kg (165 lb) per min using a 54 431 kg (120 000 lb) Universal Test Machine. Figure 1 is a view of the test set-up. Specimens were tested at ambient indoor temperature unless noted otherwise.

For the ASTM C880 (flexural strength) testing a total of 160 faced honeycomb panel specimens were tested at room temperature in either "wet" (soaked in water for 48 h) or "dry" (oven dry) conditions; and 24 specimens were tested either at hot 76.7°C (170 °F) or cold −23.3°C (−10 °F) temperatures. Specimens tested at hot temperatures were preheated in an oven and transferred to an insulated box prior to testing. A thermocouple, applied to the stone face of each specimen prior to testing, measured temperatures within the insulated box which were maintained within approximately −5°C (5 °F through the use of strip heaters. Specimens tested at cold temperatures were pre-chilled using dry ice. During cold testing, the temperature within the insulated box varied from an average of −27.9°C (−18.2 °F) upon initiation of the test to an average of −19.2°C (−2.6 °F) at the conclusion of the test due to heat gain through openings in the insulated box (for the load head and deflection instrumentation).

Of the 160 wet or dry room temperature specimens, 40 specimens were tested with each type of stone facing (new limestone, new travertine, old travertine, and new granite). From each group of 40 specimens, 20 were tested wet and 20 were tested dry. From each of the wet and dry groups, ten specimens were tested with the honeycomb oriented in the longitudinal ("l") direction and ten were tested with honeycomb oriented in the transverse ("A") direction. Of the 24 cold and hot specimens, twelve specimens were tested with the new limestone facing and twelve were tested with the new granite facing. From each group of twelve specimens, six were tested in the hot condition and six were tested in the cold condition, half of which were tested with the honeycomb oriented in the longitudinal ("1") direction and half in the transverse ("A") direction. Six additional tests were conducted on the aluminum honeycomb "core" material in

TABLE 1—*Summary ASTM C880 flexural strength of stone faced and unfaced honeycomb panels.*

Condition		Honeycomb Aluminum-Unfaced			New Limestone Faced			New Travertine Faced			Old Travertine Faced			New Granite Faced		
		"A"	"l"		"A"	"l"		"A"	"l"		"A"	"l"		"A"	"l"	
Wet	Average	788	734	lb-in	778	815	lb-in	756	742	lb-in	646	727	lb-in	1,012	851	lb-in
	Standard Deviation	49	5	lb-in	27	22	lb-in	48	41	lb-in	61	23	lb-in	113	41	lb-in
	Coeff. of Variation	6.2	0.7	%	3.4	2.6	%	6.3	5.5	%	9.4	3.1	%	11.1	4.8	%
Dry	Average				812	801	lb-in	793	762	lb-in	690	764	lb-in	982	897	lb-in
	Standard Deviation	NA	NA		25	26	lb-in	29	36	lb-in	80	32	lb-in	121	32	lb-in
	Coeff. of Variation				3.1	3.2	%	3.6	4.7	%	11.6	4.1	%	12.3	3.5	%
		"A"	"l"		"A"	"l"		"A"	"l"		"A"	"l"		"A"	"l"	
−10 Deg. F	Average				906	888	lb-in	NA	NA		NA	NA		1,021	881	lb-in
	Standard Deviation	NA	NA		33	32	lb-in							76	39	lb-in
	Coeff. of Variation				3.6	3.6	%							7.5	4.4	%
170 Deg. F	Average				583	582	lb-in							581	635	lb-in
	Standard Deviation	NA	NA		14	5	lb-in	NA	NA		NA	NA		11	31	lb-in
	Coeff. of Variation				2.4	0.9	%							1.9	4.8	%

Condition		Honeycomb Aluminum-Unfaced			New Limestone Faced			New Travertine Faced			Old Travertine Faced			New Granite Faced		
		"A"	"l"		"A"	"l"		"A"	"l"		"A"	"l"		"A"	"l"	
Wet	Average	89	83	N-m	88	92	N-m	85	84	N-m	73	82	N-m	114	96	N-m
	Standard Deviation	5	1	N-m	3	2	N-m	5	5	N-m	7	3	N-m	13	5	N-m
	Coeff. of Variation	6.2	0.7	%	3.4	2.6	%	6.3	5.5	%	9.4	3.1	%	11.1	4.8	%
Dry	Average	NA	NA		92	90	N-m	90	86	N-m	78	86	N-m	111	101	N-m
	Standard Deviation				3	3	N-m	3	4	N-m	9	4	N-m	14	4	N-m
	Coeff. of Variation				3.1	3.2	%	3.6	4.7	%	11.6	4.1	%	12.3	3.5	%
		"A"	"l"		"A"	"l"		"A"	"l"		"A"	"l"		"A"	"l"	
−23 Deg. Celsius	Average	NA	NA		102	100	N-m	NA	NA		NA	NA		115	100	N-m
	Standard Deviation				4	4	N-m							9	4	N-m
	Coeff. of Variation				3.6	3.6	%							7.5	4.4	%
77 Deg. Celsius	Average	NA	NA		66	66	N-m	NA	NA		NA	NA		66	72	N-m
	Standard Deviation				2	1	N-m							1	3	N-m
	Coeff. of Variation				2.4	0.9	%							1.9	4.8	%

Note: "A"-Transverse
"l"-Longitudinal

TABLE 2—*Summary ASTM C880 flexural strength of stone faced and unfaced honeycomb panels.*

		Aluminum Honeycomb-Unfaced				New Limestone Faced				New Travertine Faced				Old Travertine Faced				New Granite Faced			
Condition		"A"	% Loss	"1"		"A"	"1"		% Loss	"A"	"1"		% Loss	"A"	"1"		% Loss	"A"	"1"		% Loss
0 Cycle	Average	788		734	lb-in	778	815	lb-in		756	742	lb-in		646	727	lb-in		1,012	851	lb-in	
	Standard Deviation	49		5	lb-in	27	22	lb-in		48	41	lb-in		61	23	lb-in		113	41	lb-in	
	Coeff. of Variation	6.2		0.7	%	3.4	2.6	%		6.3	5.5	%		9.4	3.1	%		11.1	4.8	%	
50 Cycle	Average						771	lb-in	5.4 %		612	lb-in	17.5 %		595	lb-in	18.2 %		854	lb-in	−0.4 %
	Standard Deviation	NA	NA	NA		NA	32	lb-in		NA	37	lb-in		NA	64	lb-in		NA	13	lb-in	
	Coeff. of Variation						4.1	%			6.1	%			10.8	%			1.5	%	
75 Cycle	Average						736	lb-in	9.7 %		612	lb-in	17.5 %		609	lb-in	16.3 %		810	lb-in	4.8 %
	Standard Deviation	NA	NA	NA		NA	51	lb-in		NA	86	lb-in		NA	54	lb-in		NA	23	lb-in	
	Coeff. of Variation						6.9	%			14.0	%			8.8	%			2.8	%	
100 Cycle	Average	691	12.4 %				738	lb-in	9.4 %		646	lb-in	12.9 %		623	lb-in	14.3 %		808	lb-in	5.1 %
	Standard Deviation	31		NA		NA	56	lb-in		NA	90	lb-in		NA	22	lb-in		NA	43	lb-in	
	Coeff. of variation	4.5					7.6	%			13.9	%			3.5	%			5.3	%	

		Aluminum Honeycomb-Unfaced				New Limestone Faced				New Travertine Faced				Old Travertine Faced				New Granite Faced			
Condition		"A"	% Loss	"1"		"A"	"1"		% Loss	"A"	"1"		% Loss	"A"	"1"		% Loss	"A"	"1"		% Loss
0 Cycle	Average	89		83	N-m	88	92	N-m		85	84	N-m		73	82	N-m		114	96	N-m	
	Standard Deviation	6		1	N-m	3	2	N-m		5	5	N-m		7	3	N-m		13	5	N-m	
	Coeff. of Variation	6.2		0.7	%	3.4	2.6	%		6.3	5.5	%		9.4	3.1	%		11.1	4.8	%	
50 Cycle	Average						87	N-m	5.4 %		69	N-m	17.5 %		67	N-m	18.2 %		96	N-m	−0.4 %
	Standard Deviation	NA	NA	NA		NA	4	N-m		NA	4	N-m		NA	7	N-m		NA	1	N-m	
	Coeff. of Variation						4.1	%			6.1	%			10.8	%			1.5	%	
75 Cycle	Average						83	N-m	9.7 %		69	N-m	17.5 %		69	N-m	16.2 %		92	N-m	4.8 %
	Standard Deviation	NA	NA	NA		NA	6	N-m		NA	10	N-m		NA	6	N-m		NA	3	N-m	
	Coeff. of Variation						6.9	%			14.0	%			8.8	%			2.8	%	
100 Cycle	Average	78	12.4 %				83	N-m	9.4 %		73	N-m	12.9 %		70	N-m	14.3 %		91	N-m	5.1 %
	Standard Deviation	4		NA		NA	6	N-m		NA	10	N-m		NA	2	N-m		NA	5	N-m	
	Coeff. of Variation	0.5					7.6	%			13.9	%			3.5	%			5.3	%	

Note: "A"-Transverse
"1"-Longitudinal

TABLE 3—*Summary ASTM C880 flexural strength of stone faced and unfaced honeycomb panels design load after exposure to 25 cycles of accelerated weathering ultimate load after exposure to 100 cycles of accelerated weathering.*

		Aluminum Honeycomb-Unfaced		New Limestone Faced			New Travertine Faced			Old Travertine Faced			New Granite Faced		
Condition		"A"	"1"	"A"	"1"		"A"	"1"		"A"	"1"		"A"	"1"	
25 cycles Design Load, 100 cycles-Ultimate Load	Average				752	lb-in		571	lb-in		641	lb-in		820	lb-in
	Standard Deviation	NA	NA	NA	48	lb-in	NA	27	lb-in	NA	24	lb-in	NA	22	lb-in
	Coeff. of Variation				6.3	%		4.7	%		3.7	%		2.7	%
		Unfaced		New Limestone Faced			New Travertine Faced			Old Travertine Faced			New Granite Faced		
Condition		"A"	"1"	"A"	"1"		"A"	"1"		"A"	"1"		"A"	"1"	
25 Cycle Design Load, 100 Cycle-Ultimate Load	Average				85	N-m		64	N-m		72	N-m		93	N-m
	Standard Deviation	NA	NA	NA	5	N-m	NA	3	N-m	NA	3	N-m	NA	2	N-m
	Coeff. of Variation				6.3	%		4.7	%		3.7	%		2.7	%

"A"-Transverse
"1"-Longitudinal

the wet condition for comparison with stone faced specimens.

During flexural strength testing, composite specimens were tested with the stone face in tension, where it would contribute the least to system strength. Loads were distributed across the width of each specimen at quarter points through 9.5 mm (3/8 in.) thick by 19.1 mm (3/4 in.) wide steel shims and bearing tape to provide a uniform loading area and reduce localized crushing of the aluminum honeycomb. Tables 1–3 present a summary of the flexure strength test data.

Accelerated Weathering

Testing and Exposure—Specimens tested to determine the effects of accelerated weathering were the same dimensions as the flexural strength specimens described above. The load rate, load points, and test span were the same as those not exposed to accelerated weathering. Comparisons were made between results obtained from specimens exposed and not exposed to accelerated weathering.

The accelerated weathering (freeze/thaw) conditioning consisted of exposing the flexural strength specimens to a cyclic temperature range of −23.3°C (−10 °F) to 76.7°C (170°F), while the specimens were partially submerged 6.4 mm (1/4 in.) in a 4 pH sulfurous acid solution which simulates exposure to acid rain. For flexural strength testing, a total of 78 specimens were tested after accelerated weathering exposure. Specimens were tested with each type of stone facing after each 25, 50, 75, and 100 cycles of accelerated weathering exposure.

Simulated Design Loading During Accelerated Weathering—Table 3 shows flexural strength data obtained for specimens that were loaded to their maximum design bending moment after 25 cycles of accelerated weathering and then loaded to failure after 100 cycles of accelerated weathering. The manufacturer's allowable span for wind load was used to calculate the equivalent design moment, shear, and deflection of a 50.8 mm (2 in.) wide test specimen. The limiting factor was determined to be a design bending moment of 12.1 N/m (107 lb/in.). This bending moment was applied to the 50.8 mm (2 in.) wide test specimens to determine if micro-cracking of the stone would occur prior to achieving the design bending moment, and evaluate the effect this simulated in-service loading would have on the strength of the composite specimen over time. A penetrating dye was applied to the surface of each stone before and after loading the specimen to design moment to aid in observation of micro-cracking.

Tensile Bond Strength

Test Procedure—Figure 2 is a view of the tensile bond test performed. Tensile bond tests were conducted in general accordance with ASTM D897, Standard Test Method for Tensile Properties of

TABLE 4—*Summary tensile bond strength of stone faced honeycomb panels.*

Condition		New Limestone Faced		% Reduction	New Travertine Faced		% Reduction	Old Travertine Faced		% Reduction	New Granite Faced		% Reduction
Amb., Initial	Average	174	psi		190	psi		207	psi		299	psi	
	Standard Deviation	14	psi		73	psi		24	psi		2	psi	
	Coeff. of Variation	8.3	%		38.2	%		11.7	%		0.5	%	
Amb., 100 Cyc.	Average	161	psi	7.4 %	146	psi	23.2 %	199	psi	3.8 %	234	psi	21.7 %
	Standard Deviation	18	psi		31	psi		17	psi		45	psi	
	Coeff. of Variation	11.2	%		21.3	%		8.5	%		19.3	%	
Amb., Initial	Average	122,300	Kgf/m^2		133,600	Kgf/m^2		145,500	Kgf/m^2		210,200	Kgf/m^2	
	Standard Deviation	9,840	Kgf/m^2		51,320	Kgf/m^2		16,870	Kgf/m^2		1,410	Kgf/m^2	
	Coeff. of Variation	8.3	%		38.2	%		11.7	%		0.5	%	
Amb., 100 Cyc.	Average	113,200	Kgf/m^2	7.4 %	102,600	Kgf/m^2	23.2 %	139,900	Kgf/m^2	3.8 %	164,500	Kgf/m^2	21.7 %
	Standard Deviation	12,660	Kgf/m^2		21,800	Kgf/m^2		11,950	Kgf/m^2		31,640	Kgf/m^2	
	Coeff. of Variation	11.2	%		21.3	%		8.5	%		19.3	%	

FIG. 1—*ASTM C880-flexural strength test.*

Adhesive Bonds [2], to determine the ultimate tensile bond strength of the adhesive which bonds the stone facing to the aluminum honeycomb. For testing of bond between the stone face and aluminum honeycomb, modifications were made to ASTM D897 including adjustments in specimen sizes and bond surfaces (stone to aluminum honeycomb instead of steel to steel). The tensile bond test was conducted by attaching a metal plate to the front and back face of each 50.8 mm (2 in.) by 50.8 mm (2 in.) specimen and applying tension load to two rods that were attached to the center of each plate by tensile grip to the Universal Testing Machine. Loads were applied at a rate of 363.6 kg (800 lb) per minute to specimens until failure. Tests were conducted at ambient indoor temperatures. Twenty-four total tensile bond tests were conducted. Six specimens were tested with each type of stone facing. Half of the tensile adhesion tests for each group were performed on test specimens not subjected to accelerated weathering and half were performed on test specimens subjected to 100 cycles of accelerated weathering (described above). The tensile adhesion of the stone facing to the honeycomb portion of the panel for the two test groups is compared in Table 4

Test Results

Flexural Strength Results

Testing of Stone Faced Honeycomb Specimens—Table 1 shows a comparison between unfaced aluminum honeycomb material tested in the longitudinal (“1”) orientation and the transverse (“A”) orientation. The unfaced honeycomb specimens tested in the transverse orientation had slightly higher flexural strengths than specimens in the longitudinal orientation. The coefficient of variation for the honeycomb core material as well as for each stone facing is consistently low, regardless of specimen age and test exposure condition. Comparison of the honeycomb core material with stone faced specimens shows that, in general, stone faced specimens had comparable flexural strengths to unfaced honeycomb material. However, the new limestone with the honeycomb core in the longitudinal orientation and new granite faced specimens had slightly higher tested strengths than the core material. The old travertine with the

FIG. 2—*Modified ASTM D897-tensile bond test.*

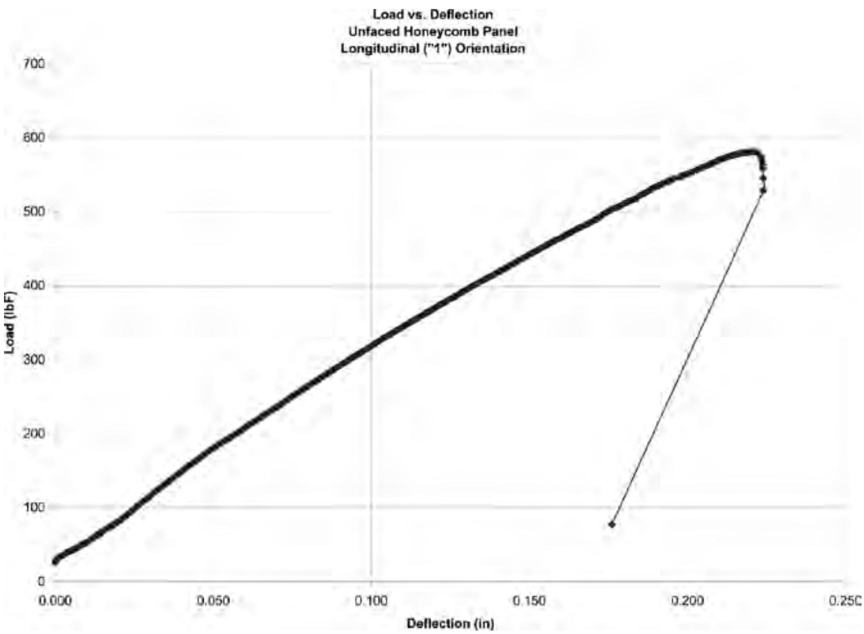

FIG. 3—*Load versus deflection unfaced honeycomb panel longitudinal ("1") orientation.*

honeycomb core in the transverse orientation had slightly lower tested strengths, on average, than the new core material. The granite faced specimens show the highest flexure strength compared to the honeycomb core.

Wet/Dry and Hot/Cold Testing—Table 1 also shows a comparison between wet and dry test samples and the effects of high and low temperature on the test samples. Test data indicates that dry samples have similar tested strengths to the wet samples and the coefficients of variation are about the same. Comparing the data for flexure strength of specimens tested at hot or cold temperatures shows that new limestone faced specimens have a higher average strength at cold temperature and a lower average strength at high temperature. The granite faced specimens did not have higher strength at low temperatures but did have an approximately 35 % lower strength, on average, at high temperature.

Review of Old and New Travertine Faced Panels—Travertine faced panels that have been in service for more than 18 years have lower strengths in the transverse orientation than new travertine faced specimens. New panel specimens faced with travertine have similar strengths in the longitudinal orientation to travertine faced panel specimens more than 18 years old. Travertine faced specimens tested dry have similar flexural strengths to those tested wet.

Load/Deflection Measurements—Load and deflection were measured during flexural strength testing. Deflection readings were measured through the use of a Linear Variable Deflection Transducer (LVDT) with a precision of ±0.0254 mm (0.001 in.)/V and plotted against the corresponding test loads. To prevent interference of the LVDT with the load head of the test machine, deflection measurements were taken at the end of a projecting aluminum angle which was glued to the top side of each specimen. Figure 3 shows the load/deflection curve for an unfaced honeycomb core specimen. Figure 4 shows the load/deflection curve for a new travertine composite panel specimen. Figure 5 shows the load/deflection curve for an old travertine composite panel specimen. Figure 6 shows the load/deflection curve for a new granite composite panel specimen. Figures 3–6 represent specimens tested with the honeycomb oriented in the longitudinal ("1") direction. All the graphs show similar load deflection curves. There are slight offsets in the curves for

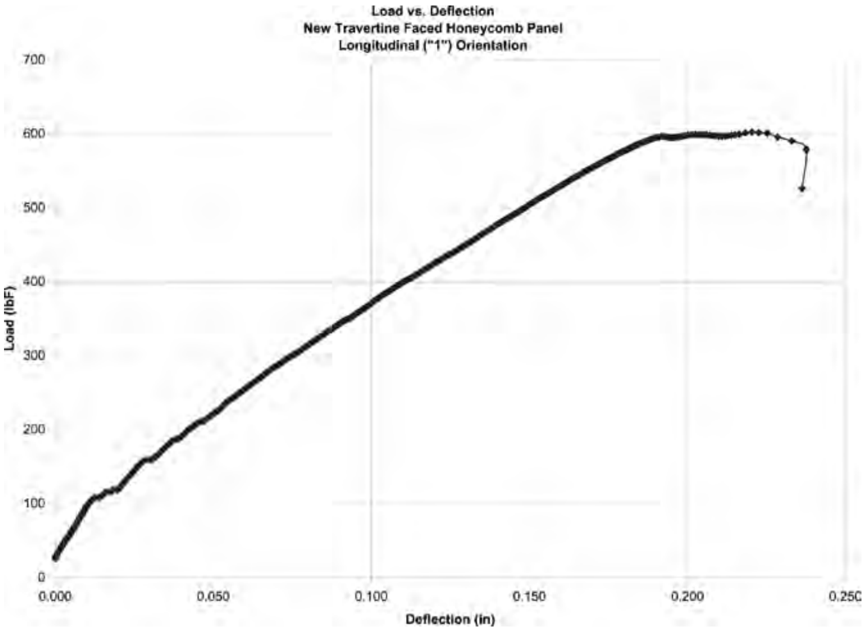

FIG. 4—*Load versus deflection new travertine faced honeycomb panel longitudinal (“1”) orientation.*

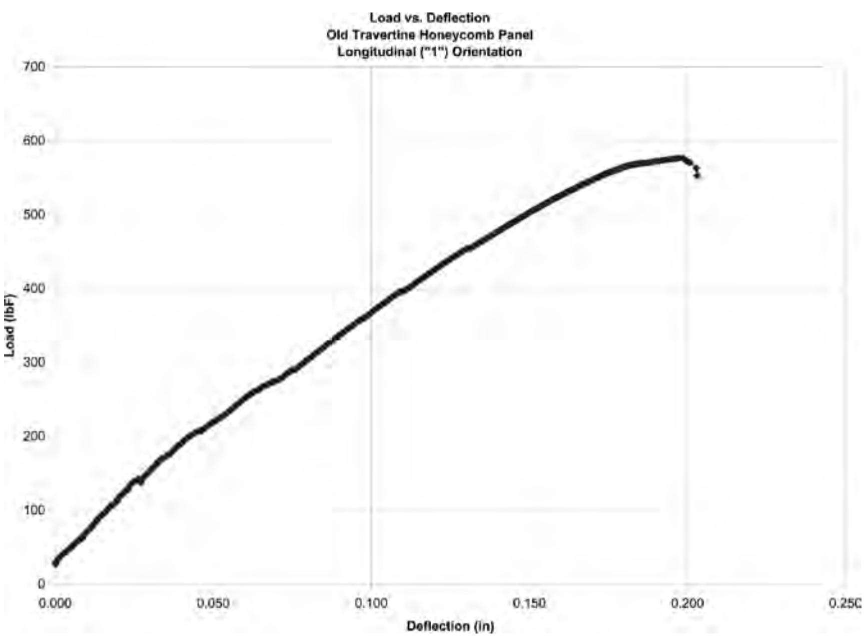

FIG. 5—*Load versus deflection old travertine honeycomb panel longitudinal (“1”) orientation.*

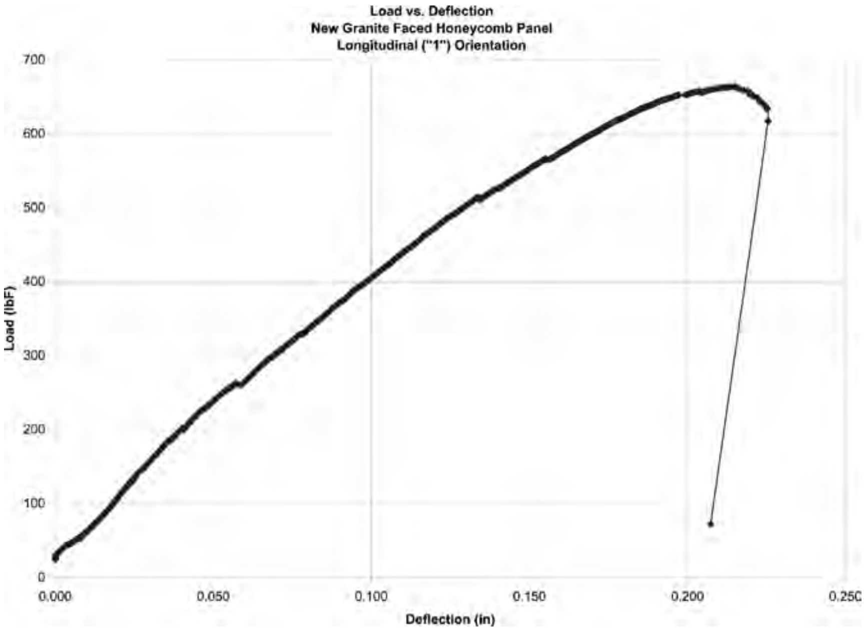

FIG. 6—*Load versus deflection new granite faced honeycomb panel longitudinal (“1”) orientation.*

stone faced specimens (Figures 4–6) which show a sudden increase in deflection without an increase in load. It was noted that the offsets correspond to observed cracking of the stone facing. Similar offsets are not present in the curve for the unfaced honeycomb core (Fig. 3).

Figure 7 shows a load deflection curve for a granite composite panel specimen tested at 76.7°C (170 °F). Failure occurred at a load of approximately 226.8 kg (500 lb) or a lower strength than specimens tested at nominal room temperature. After initial deflection, the slope of the load/deflection curve for the specimen tested at 76.7°C (170 °F) is shallower than the slope for granite faced specimens tested at room temperature. This indicates that, at high temperatures, there is a slight reduction in stiffness. Figures 6 and 8 show load deflection curves for granite composite panel specimens tested at ambient and cold, −23.3°C (−10 °F), temperatures. Failure occurred at a load of approximately 317.5 kg (700 lb) or a higher strength than specimens faced with limestone or travertine. The slope of the two curves is about the same. This indicates that at low temperature testing there is a similar stiffness of the composite system to the stiffness of the composite system at ambient temperatures for granite faced specimens.

Accelerated Weathering

The flexural strength of specimens exposed to accelerated weathering was compared to that of “unweathered” specimens in order to evaluate potential strength loss over time due to weathering. Flexural strength results indicate that specimens exposed to 100 cycles of accelerated weathering exhibit a varying degree of reduced strength depending on the stone facing.

Table 2 shows the average reduction in flexural strength for each type of specimen due to accelerated weathering testing as compared to those not exposed to accelerated weathering. While granite faced specimens exposed to 100 cycles of accelerated weathering had minimally lower strength than those not exposed to accelerated weathering, granite specimens exhibit significantly less strength reduction, as a percentage, as compared to the aluminum core without a facing. Limestone faced specimens exhibit slightly less strength reduction than the honeycomb core material alone and travertine faced specimens exhibit a strength reduction comparable to the core material. While limestone faced panel specimens

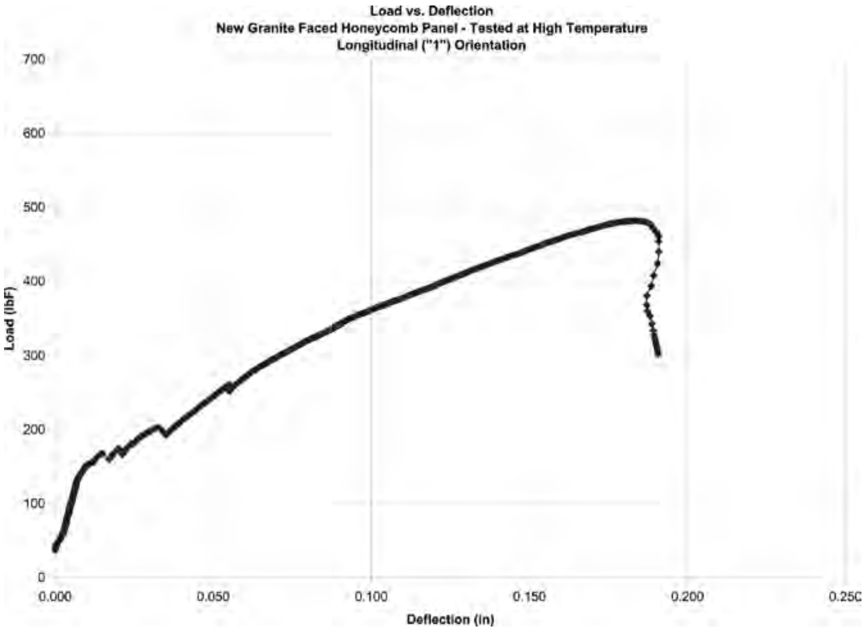

FIG. 7—*Load versus deflection new granite faced honeycomb panel-tested at high temperature longitudinal ("1") orientation.*

experienced strength reductions during the first 75 cycles of accelerated weathering, both the new and old travertine experienced strength reductions during the first 50 cycles of accelerated weathering as compared to unexposed specimens.

Simulated Design Loading During Accelerated Weathering—No additional cracking was observed after loading the 25 cycle specimens to design moment. This observation is supported by the load/deflection graphs in which there is no indication of cracking prior to the 12.1 N/m (107 lb/in.) design moment, see Figs. 3–8. The results show that the flexural strength data for the composite panel specimens exposed to their design moment at 25 cycles of exposure (summarized in Table 3) is not significantly different than companion specimens not exposed to their design moment at 25 cycles of exposure (data presented in Table 2).

Tensile Bond Strength

The tensile adhesion of the stone to the backup panel for the two groups is compared in Table 4. In most of the specimens failure was due to internal fracturing of the stone rather than at the bond of stone to honeycomb. This stone failure occurred in five out of twelve travertine faced specimens, while the bond failed in the remaining seven specimens. For limestone faced panels the limestone failed in every test while bond failure was not observed. The same stone failure occurred in five out of six new granite bond tests. The bond failure was observed most frequently for travertine specimens; likely because there is significant variation in the size and amount of void area in the travertine, resulting in significant variations in bond contact area from specimen to specimen.

Discussion

Although the subject test program was not comprehensive enough to support rigorous statistical treatment of the variables considered, the data provide substantial information concerning key character-

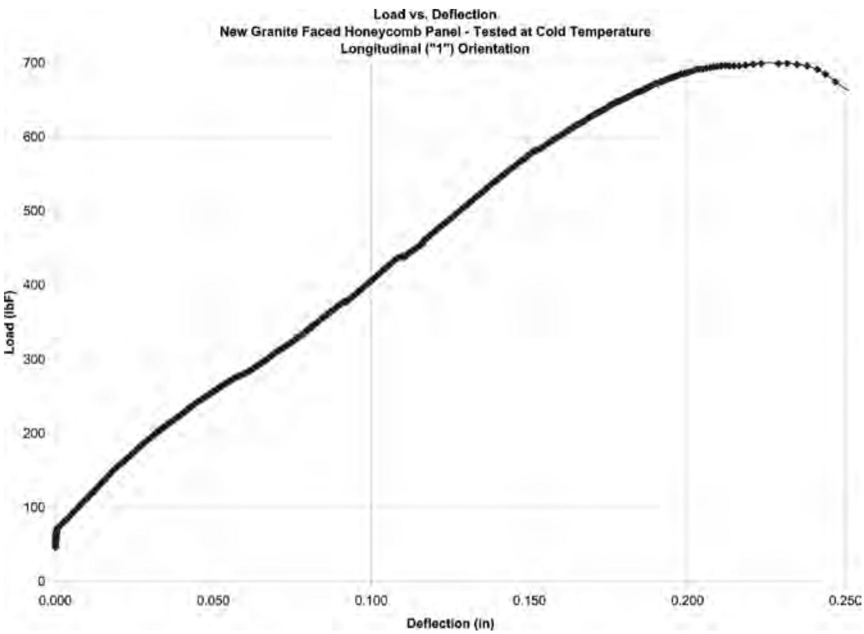

FIG. 8—*Load versus deflection new granite faced honeycomb panel-tested at cold temperature longitudinal ("1") orientation.*

istics of the stone faced honeycomb panels that were tested. Relevant observations are summarized in this section.

Flexural Strength

Flexural strength is an important structural property of cladding panels. Observations suggested by the flexural testing performed under this program are discussed below.

General Strength Variability—The test data indicate that faced honeycomb panels flexural strength variability is substantially less than that commonly found in panels made only of stone. This is most likely due to the fact that stone faced honeycomb panel flexural strength is dominated by the manufactured honeycomb material (discussed below). The significance of this is that design "safety factors" (when "allowable stress" principles are used) or "strength reduction factors" (when "load and resistance factor" principles are used) are, or at least should be, proportional to variability. In other words, as variability decreases, safety factors and strength reduction factors may be decreased. Quantification of "customized" safety factors or strength reduction factors ?or both consistent with the variability of the stone faced honeycomb panel would require extensive additional testing of both new and weathered product.

Faced versus Unfaced Panels—Panels with and without stone facing were tested in order to evaluate the contribution of facing materials. The test data suggest that the reliable strength of a composite panel is provided exclusively by the honeycomb backup. With the exception of the granite-faced panels at ambient and cold temperatures, there appears to be little difference between the flexural strengths of faced and unfaced panels.

The indication that the granite-faced panels have higher strengths at ambient and cold temperatures is difficult to explain at this time. It may be due to better stiffness compatibility between the stone and the honeycomb which would allow for more efficient load sharing between the two elements. The higher degree of variability of granite faced specimens relative to specimens faced with the other new stone supports this notion. However, since most panels will experience relatively high service temperatures, the apparent lack of significant granite facing contribution at elevated temperatures makes this more of an academic rather than a practical concern. Furthermore, even where the granite appears to be contributing to higher average strengths, an associated higher degree of strength variability would mitigate the significance of this contribution.

Effect of Temperature—The test data indicate that raising the temperature decreases panel strength; and that the effects may be significant. For a temperature increase from ambient to 76.7°C (170 °F), the data for the new limestone product indicates a flexural strength reduction from approximately 800 lb in to approximately 580 lb in, or about 25 to 30 %, while the reduction for the granite-faced product was much higher see (Table 1).

As discussed above, granite facing appeared to contribute to overall panel strength at ambient temperatures, while limestone facing did not. Although testing of the aluminum honeycomb material was not conducted at high temperatures due to a limited number of unfaced panels, test data suggests that there is little difference between the flexural strengths of faced and unfaced panels, with the exception of the granite-faced panels at ambient and cold temperatures. Since the magnitude of flexural strength at the higher temperatures appears to be unaffected by the facing material, overall consideration of the test data suggests the following:

- The honeycomb panel material loses flexural strength as temperatures increase above ambient, possibly due to a softening of the fiberglass skin at the top and bottom of the aluminum honeycomb composite; and that a temperature increase of 37.8°C (100 °F) degrees F) relative to ambient can cause a strength loss in the range of 25 to 30 %.
- Flexural strength contributions from granite facing is lost at the elevated temperatures

Honeycomb panel strength loss is most likely due to softening of the reinforced epoxy facing or its bond, or both, to the aluminum grid. Since, at ambient temperatures, the granite facing appears to significantly contribute to overall panel strength, the data for the limestone-faced panels more accurately represents the effects of temperature on the honeycomb material.

The greater loss of strength with temperature increase sustained by the granite-faced panels is most likely due to the additional loss of the contribution of the granite. This phenomenon is consistent with a softening of the granite/honeycomb adhesive. It is also consistent with the previously mentioned load sharing mechanism based on stiffness compatibility, which is mitigated as the honeycomb panel stiffness drops (i.e., as the difference between honeycomb stiffness and facing stiffness increases as temperatures increase).

Effect of Moisture—The test data indicate that moisture has little affect on flexural strength. Again, given the dominance of the honeycomb backup in this context, this is not surprising.

Effects of Exposure—The test data related to exposure clearly indicates that there is an associated loss of strength. The magnitude of this effect is very difficult to even estimate at this time, primarily because there is only one small set of data representing actual aged panels. While the data from the accelerated weathering specimens are useful in a generally qualitative sense, accelerated weathering conditions (e.g., temperature, moisture, number of cycles) may not necessarily correlate well with any particular set of actual exposure conditions.

Comparison between the new and aged travertine material indicates a flexural strength reduction of about 10 %. Results show that unfaced test specimens experienced a 12.4 % percent strength loss from accelerated weathering exposure. Flexural strength results for travertine faced specimens exposed to accelerated weathering varied and were not conclusive. The flexural strength of travertine specimens varies considerably more than limestone or granite specimens. This is likely because there is significant variation in the size and amount of void area in the stone, resulting in significant variations in bond contact area from specimen to specimen. This variability explains why testing of new travertine indicated a 17.5 % apparent strength loss after 50 cycles and only 13 % after 100 cycles. It also explains why testing of old travertine indicated an 18.2 apparent strength loss after 50 cycles and only 14.3 % after 100 cycles. Limestone and granite faced specimens had an apparent strength loss of 5 to 10 % from accelerated weathering exposure. The limestone and granite flexural strengths tended to stabilize after 75 cycles of exposure.

What can be said with considerable confidence is that any significant loss of panel strength caused by exposure (either real or via accelerated weathering) is due primarily to degradation of the aluminum/fiberglass honeycomb panel. Therefore, panel structural designs must be based on consideration of the durability of the fiberglass material and its bond to the aluminum.

Deflection Data—The load-deflection data corroborates the stiffening effects of lower temperatures. It also reveals a modest level of ductility relative to solid stone samples. The ability to sustain some degree of post-yield loading is probably a significant reason the overall ultimate strength variability is so low. In other words, the stone faced honeycomb panels are apparently much less sensitive than panels made of solid stone to minor imperfections, whether they are inherent or caused by fabrication/handling/erection.

Tensile Bond Strength

The tensile bond strength tests are not conclusive relative to strength loss from accelerated weathering and differences between stone facing types. This is because in most of the specimens failure was due to internal fracturing of the stone while the bond of stone to honeycomb did not fail. Since the bond strength often exceeds stone strength, a conclusion regarding bond strength could not be reached other than the bond strength was at least equal to or greater than the results obtained.

The testing in this program also involved areas that were quite small relative to the size of most actual panels. Therefore, the effective surface area that is mobilized for a larger panel likely has a much greater fracture strength or bond strength than the lower bound test values. Furthermore, even the lower bound values appear adequate for practical purposes.

Summary

A series of structural tests on new and exposed stone-faced honeycomb panels was performed, revealing what appear to be some significant general characteristics of the stone veneer/honeycomb system. Although the testing was done in general conformance with common stone testing methods, the results very strongly suggest that, structurally speaking, the panels are not stone panels. The test data suggest that panel structural performance is dominated by the aluminum/fiberglass backup panel, and that structural design of the panels using conventional stone panel design methods may be very conservative. However, like most stones, the panel flexural strength appears to be adversely affected by exposure. Unlike stone, the material primarily affected appears to be the fiberglass facing of the honeycomb backup or its bond, or both, to the honeycomb.

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SECTION II: DESIGN

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Paving Design: Is Rigid-Fix External Stone Paving the Way to Go?

ABSTRACT: Recent investigations into a number of paving failures in Australia have shown that very little is understood about the many aspects intrinsic to rigid-fix (stuck down) paving. This paper examines and documents some of the factors that can determine the success or failure of paving (assuming a high standard of workmanship). Additionally, it has become clear that most engineers who investigate paving failures fail to fully understand the complexities and interplay of the many factors. Such lack of knowledge leads to poor design, poor supervision, and ultimately poor performance. There are basically three types of paving systems—adhesive/mortar fix, sand-bedded, and setts/cobblestones in a soft mortar. All three systems are being used in Australia but there is undoubtedly an overriding rigid-fixing mentality. By fixing stone to a solid concrete base so that it cannot move is probably seen by most engineers and architects as the most straight-forward and most controllable solution. However, on analysis, it is a system that is subject to numerous potential lines of failure. Not only is there the nature of the stone itself such as the mineral composition, strengths, thermal characteristics, and porosity, but also the environment into which it is placed, the types of loading it is subject to, and how it is fixed. Stability of the base (including shrinkage), expansion/construction joints, size, thickness and shape of the pavers, the laying pattern, type of adhesive, thermal conditions at the time of fixing, influence of salts and water, and sealing, all play a significant role in determining the success or otherwise of rigid-fix paving.

KEYWORDS: stone, paving, failures, external, design

Introduction

Although paving is an important element in modern construction it was an *essential* element in historical times for the movement of people and goods. In more recent times, paving is often ornamental, rounding off the design and complementing the construction. But even today some paving is meant to be functional as well as providing the high aesthetic value afforded by the use of natural stone.

Unlike the historical paving which was often somewhat boring and admittedly a bit rough (in more ways than one) modern processing techniques allow for a much greater range in the type of stone and in the dimensions of the stone paving. Furthermore, there have been many significant advances in bonding media (adhesives, grouts, sealants) and protective coatings. Modern engineering test facilities are available to test stone and computers can be used to simulate load conditions. Yet there have been numerous significant paving failures in the past 20 years all over the world. Usually these failures have been due to a combination of contributing factors—not just a single factor—and some of the failures have been surprisingly elementary.

There are many key factors that need to be considered in the design and successful construction of standard paving. For more demanding paving, involving factors such as exposure to the elements and exposure to complex loadings, the analysis needs to be substantially more rigorous. Stone paving is not just a simple matter of sticking pavers to a prepared concrete bed or placing them onto compressed decomposed sand. Each major application is different and involves a combination of different factors. Expert advice needs to be sought on the stone to be used, how it is to be laid, by what pattern, and on the maintenance of it. Shortcuts cannot be taken and in-house advice is inherently inadequate. Persuasive advertising, extended warranties, long associations with suppliers, and that ever-pressing need to win contracts by those suppliers should not be determining issues.

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Recent involvement in a number of paving applications has prompted me to examine and document some of the factors that can determine the success or failure of paving (assuming a high standard of workmanship). Additionally, it has become clear that investigations into paving failures in Australia have failed to fully understand the complexities and interplay of many of the factors.

There are basically three types of paving systems—adhesive fix, sand-bedded, and setts/cobblestones in a soft mortar. All three systems are being used in Australia but there is undoubtedly an overriding rigid-fixing mentality. By fixing stone to a solid concrete base so that it cannot move is probably seen by the engineers and architects as the most straight-forward and most controllable solution. However, on analysis, it is a system that is subject to numerous potential lines of failure. Sand-bedding is a system that relies on a compressed graded sand or deco base upon which are placed smaller and thicker stone pavers in a particular pattern. Special sand is used between narrowly jointed pavers to lock the pavers into place. This is a system that is widely used in Europe and North America. Among the benefits are ready access to underground services, easy replacement in case of failure, and easy maintenance. The sett/cobblestone paving system utilizes small, usually square or cubic units varying in thickness from 30 mm to at least 150 mm. Their use tends to be restricted to areas of limited traffic and they can be laid in a variety of straight and radial patterns. Rectangular block-shaped units are generally deployed for specific applications, particularly where substantial load-bearing is anticipated, e.g., bus-turning areas.

A paving project generally starts with a client's or architect's, or both, vision. The drawings then tend to go to an engineering section and progress to the budgeting section. None of these sections really know much about the performance of any stone and hope that they can get most of the information from a supplier (who hopes to make as much profit as possible, with the least amount of work and commitment, and with fingers crossed that everything will work out). But often the supplier does not know much about the stone (e.g., calling a marble a bluestone or calling a basalt a granite is not unusual) and because most of the stone is now being imported there is quite often a language problem between an Australian buyer and overseas supplier. Surprisingly, many overseas quarries *do* know what they are selling but this is not always conveyed to the purchaser in Australia. Unfortunately, there are also some overseas suppliers who see Australia as a target for dumping inferior material because of a perception that Australians have little expertise with respect to stone.

Assuming the client and architect have agreed on a color and pattern for the stone, and between them the engineers and budgeting section have agreed on a size and quantity they go to tender (unless there is a preferred supplier). Competition is fierce and potential suppliers will break their necks trying to get the contract. This usually means applying financial pressure to the overseas supplier as much as they can get away with. And that is where quality is often compromised, not only in the stone quality but in the quarrying, processing, and delivery.

Once the contract is let how much does the client really know about the quality of the stone that is to be delivered and how can he be assured that delivery will be of the standard expected from the conditions written into the contract? A financial stick is usually successfully applied to the Australian supplier, but what happens when the overseas supplier encounters a change in the type of stone in the quarry, gets into financial trouble, has machinery breakdowns, or demands more money for his product? Some of the human aspects cannot be controlled but at least the quarrying, production, quality control, and shipping of the stone can, and must, be organized *before* supply commences. An experienced stone scientist should be involved in every project dealing with a substantial amount of stone whether it be for paving or for any other application. This will ensure that the stone is available to be quarried, that the quarrier has the capacity and expertise to quarry it, that it can be successfully processed in one or more designated factories, that quality control procedures are put in place and rigorously followed, and that proper, unbiased, and meaningful testing of the stone will be carried out to Australian requirements. So for the small additional expense (in relation to the overall cost of the stone) the client can be assured that he gets delivered the stone at the time required and of the quality required.

Factors that Commonly Assume a Secondary Role in Paving Integrity

Stone Composition

Most stone sold for paving is compositionally sound provided that it is not placed in a chemically and physically demanding situation without undue stress. Once placed into an external environment the stone becomes subject to a variety of stresses. If mechanical loading is placed on that stone another set of stresses are added. This now requires a rigorous evaluation of the stone product to assess its suitability for that situation.

Although there are approximately 2700 granitic rocks available on the world market and their compositions cover almost the entire spectrum of possible granite compositions on this earth they rarely consists of more than a half dozen minerals out of the dozen or so common minerals that make up the majority of common rocks. Quartz and the two varieties of feldspar (alkali and plagioclase) dominate most paving stone with small amounts of ferromagnesian minerals (biotite, amphibole, pyroxene), and traces of iron-titanium oxide completing the bulk of the primary mineralogy. There are usually some secondary minerals in small amounts (e.g., clay, white mica, hydrated iron oxide, epidote, carbonate, and chlorite) and provided these remain as small amounts the stone is usually sound mineralogically. The basalts, dolerites, and gabbros differ from granites in their mineralogy by having little or no quartz (hence softer), little or no alkali feldspar, and minor amounts of biotite and amphibole. Instead, they contain abundant plagioclase feldspar and pyroxene (usually the calcic variety), lesser olivine, and variable but usually minor amounts of iron-titanium oxide. So, unless there are minerals or structures, or both, in a certain stone that might prove deleterious to its short- and long-term performance (such as reactive pyrite, zeolite, and feldspathoids) the mineralogy of most dimension stone is innocuous and plays little part in paving failures.

Limestone and sandstone are also used as paving. However, because they tend to be more porous than the igneous and metamorphic rocks and are made of minerals that are either more susceptible to weathering (by chemical and physical attack) or are held together by a variety of possible cements, they are recognized as being less resistant to environmental and mechanical stresses. Their use as paving tends to be dominated by noncommercial situations or in a form that takes into account the intrinsic properties.

Stone Strength

The texture of the stone, together with its mineralogy and structure play a major role in determining the strength of the stone. There are several ways of expressing the strength of stone depending on whether it is under compression or tension, in a dry state, or saturated. Early tests relied on compressive strengths but the advances in processing the stone into slabs meant that there was a need to also test the stone in tension. The degree of saturation appears to have little influence on the strength of most dense crystalline rocks but the type of surface treatment given to stone can cause a serious reduction in strength (e.g., exfoliation). Tests have demonstrated that there can be a 30–40 % reduction in the strength of certain granites due to the generation of thermal fractures [1]. Sandstones (and to a lesser extent porous limestone) commonly show a substantial decrease in strengths (40–70 %) when saturated but, surprisingly, many granites yield test results that are higher than when the samples are tested in a dry state.

Most granites exceed the compressive strength of 131 MPa and 10.34 MPa modulus of rupture values recommended by ASTM Standard C 170 [2] and ASTM Standard C 99 [3], but some do not because of features such as secondary veining, closed joints, systematic microfractures, hydraulic fracturing, thermal fracturing, hydrothermal alteration, and weathering. Some of these features are not visible macroscopically and testing of all stone must involve petrographic examination by a highly skilled petrographer experienced in dimension stone (*not* in mining, coal geology, engineering, or oil reserves).

Most common granites have compressive strengths over 150 MPa and modulus of rupture values in excess of 14 MPa. Fine-grained black granites tend to be at the high end with values around 250 MPa and 25 MPa, respectively [4]. These values exceed by far the values for general commercial concrete which is required to have a compressive strength of at least 20 MPa and preferably 32 MPa. For comparison, the dry compressive strengths for a range of sandstones varies from around 40 MPa to 75 MPa and limestones from less than 20 MPa to more than 120 MPa.

In terms of choosing granite paving, absolute strengths *above* the recommended values play only a minor role in most applications, i.e., there is little real difference in the performance of 160 MPa granite compared to 190 MPa granite. The most important aspect of stone strength is that it comfortably exceeds the minimum values. Granites not meeting these values are inherently weak, generally have an unacceptably high porosity, and should not be used for exterior commercial paving.

Stone Color

The color of the stone is basically a function of its collective mineralogy. Light colored granites contain an abundance of felsic minerals (quartz, plagioclase feldspar, and alkali feldspar) whereas dark-colored granites contain mafic minerals (calcic pyroxene, amphibole, biotite, olivine, orthopyroxene, and magnetite). Variations in the proportions of the minerals and occasionally certain textures, structure, and type of finish can also influence the color of stone. Because each mineral has its own diagnostic physical characteristics large abundances of certain minerals can influence the color of a stone. However, there is usually a considerable overlap in the modal abundance of some of the minerals and often it is only a relatively small abundance of some minerals which determines the color. The color might also reflect the chemical and physical stability of the constituent minerals and provide some insight into the weatherability or durability, or both, of the stone. For example, brown is not a natural granite color. There are no fresh, brown minerals that make up a granite. Brown indicates alteration and this can be effectively masking the true color of the stone. With increasing depth (usually coinciding with a decrease in weathering) the brown color may give way to fresh, green- or blue-colored, or both, granite.

Imbibition Coefficient

This is a measure of the absorption capacity of the stone. It is an indicator of stone quality and stone strength by providing a measure of the pore space and degree of fracturing. In stone that has been partly altered or weathered some of the water can also be taken up by extremely fine-grained clays and micas. The water absorption of granites typically varies from 0.10 % (by weight) to around 0.45 % [5]. These values are recorded after full immersion for 48 hours. The maximum amount of water absorption recommended by ASTM Standard C 97 [6] is 0.4 % (by weight). The dark, strong plutonic rocks are typically the most dense (least porous) and also the strongest. With increasing amount of felsic (light-colored) minerals the degree of hydrothermal alteration, weathering, pore space, and structural defects tend to be higher. Felsic granites are often not as strong as mafic plutonic rocks.

The porosity of stone is often expressed visually and can become an aesthetic issue. A darkening of the stone along expansion joints, a patchy darkening, and a grayish discoloration are signs that resident subsurface water is penetrating the stone from the base up. Very little water, if any, penetrates through the stone from the surface. While it is subsurface (mostly in the screed, mortar bed, or adhesive) the water is accumulating dissolved calcium and iron compounds. These substances (among others) are then drawn through the stone by the thermal “engine” causing reaction and remobilization of chemically susceptible and unstable minerals en route to the surface where they precipitate. A high porosity will enhance this process and is one reason why beige granites appear to oxidize/discolor more than dark-colored granites.

It has been suggested that the absorption capacity of certain granites can have a bearing on the setting/curing characteristics of mortar or adhesive, or both. This is highly unlikely because most granitic stones would not be saturated at the time of laying nor would they have the capacity to draw large amounts of moisture out of grout before it has time to cure. The suggestion is true to an extent in the more porous limestones and sandstones which can suck up moisture or they can become substantially saturated after fixing.

Thermal Conductivity

It has long been observed that dark-colored stone appears to become quite hot at the surface when exposed in hot, sunny conditions and it has been suggested that the high conduction of heat through even 40-mm thick black stone could be partly responsible for paving failure. The argument suggests that because of elevated surface temperatures there is a rapid conduction of heat through the stone, thereby affecting the setting characteristics of the adhesive.

Scientifically, thermal conductivity is a measure of the quantity of heat Q transmitted through a unit thickness L in a direction normal to a surface of unit area A due to a unit temperature gradient $\bullet T$ and when the heat transfer is dependent only on the temperature gradient, i.e., $1 = Q \times L/(A \times \bullet T)$ and measured in watts per metre-kelvin $\mathrm{W/(m\text{-}K)}$. Although this is applicable to theoretical physics it is not strictly applicable to the changes of temperature from the top of a paver to the bottom. Even the simpler concept of heat conduction (transmission of heat across matter) is not entirely applicable in the case of paving.

It was interesting recently to observe a situation where this argument was used against a failed 40-mm thick stone, yet adjacent, identical 20-mm stone (using identical adhesive) remains sound.

Another scientific fact is that with increasing silica content (as in light-colored granite) there is an increase in the thermal conductivity compared to that of black granite but concomitantly there is a decrease in the thermal conductivity of stone as the temperature increases. This tends to have an evening effect between two apparent compositional extremes.

Heat Retention of Concrete Base and Stone

It has also been speculated that retention of heat by dark-colored stone might adversely affect the bonding characteristics of the stone to concrete presumably due to a softening of the adhesive or causing it to become brittle. Thermal retention is essentially governed by three heat transfer mechanisms; namely, conduction, convection, and radiation and for a wide range of granites there is likely to be little difference between their thermal retentivities principally for reasons outlined above.

Heating of the stone during the day is also likely to have a minor effect on the temperature of the concrete slab and therefore prolong the heat retention of the stone. Although unlikely to contribute to any degradation of the adhesive, elevated thermal conditions will slightly accelerate drying (and shrinkage) of the concrete base.

Thermal Expansion

All natural stone and masonry materials expand as the temperature increases. In engineering terms it is known as the coefficient of linear thermal expansion and measured directly as mm/mm/°C. Most stone falls within the range of 4×10^{-6} to 9×10^{-6} with slightly higher values for concrete (10×10^{-6} to 12×10^{-6}, depending on aggregate composition), over the same temperature range [7].

The thermal expansion of stone is one property that is usually understood by engineers who normally make adequate allowances for this property by the introduction of expansion/contraction joints at generous intervals.

Contrary to common beliefs there is no absolute relationship between the color of a stone and its thermal expansion. A gray-colored granite can expand more than a black granite even though the surface of the black stone might be hotter. Black stone often has a tight structure (few grain boundary imperfections, few microfractures) compared to say a mica-rich, partly altered, gray or beige-colored granite with numerous microfractures that collectively allow substantial bending and flexing.

Stone Growth

Some porous stone such as a low-quality limestone, slate, or sandstone have mineralogies and a structure which could be considered dimensionally unstable over time. Absorption of water into clay or mica-rich sedimentary rocks (especially with some dissolved chemicals or inorganic impurities or both), introduction of sulfates, and reactions between pollutants in the air with some minerals (e.g., gases of sulfur acting on carbonate) can gradually cause an increase in the overall dimensions of a susceptible stone. To achieve this, the stone must be at least moderately porous, must have an abundant supply of pollutants, is chemically reactive, and usually requires a long time. Most granite used for paving has a fairly dense and strong crystalline structure and does not fall into the susceptible category.

Dimensional stability tests carried out in Australia [8] have demonstrated a high degree of stability for granites compared with some porous sedimentary stone (albeit over a limited time).

Climate

There is a perception that tropical climatic conditions impact on the performance of paving. This means generally elevated temperatures but with a restricted diurnal range, frequent rainfall during summer, heavy rainfall, extended periods of rain, and moderate to high humidity. Although these conditions could be considered uncomfortable and even difficult at times they are certainly not harsh or overtly detrimental to the performance of stone. For example, although temperatures can be in the low 30s (°C), and the number of sunlight hours/day quite high, there is only a small diurnal range. Many places at latitudes around 30°N or S in say Perth, Adelaide, and Sydney, or Cairo, Baghdad, and Las Vegas, have similarly long daylight hours and significantly hotter daytime temperatures than in the tropics [9]. They can also have somewhat higher diurnal temperature ranges. This translates to greater stresses acting on the stone, adhesives, and base than would be the case in the tropics. Moreover, the location of some paving is adjacent to the seafront and subject to persistent tradewinds in winter and sea breezes in summer. Huge areas of stone paving also grace many of the streets and footpaths of large cities, such as in Singapore, Hong Kong, Malaysia, Portugal, Saudi Arabia, and the UAE.

A factor that is raised at times is the sudden quenching effect of relatively cool rainfall onto hot stone paving. Presumably the argument suggests that a rapid decrease in surface temperature will create stresses that can fracture the rock, cause a dramatic shortening of the stone surface which will affect the bonding with the mortar, concrete base or with the flexible construction joints, or which will in some way affect the adhesive. This argument has problems in three areas. First, there will be a large "inertia" within say 50-mm thick pavers in that the thermal conductance is substantially slowed with prolonged wetting of the stone surface. In time, the heat transfer through the stone will equilibrate and should the stone surface cool to temperatures below the bottom of the stone, there will indeed be a reversal of heat transfer. It is important to realize that it is only the top few millimetres of the granite paver that is substantially affected during the initial cooling period and not the entire paver. Because of the low temperatures involved in the "quenching," most igneous paving stone is capable of dealing with such stresses.

Second, to have any effect numerous repetitions of these rapid cooling events need to occur over decades. Failure of paving due to these climatic conditions is highly unlikely to occur within one or two years. And third, the "quenching" effect should be more severe on both stone and grout on thin (say 20 mm) paving, a situation that has not been observed.

Critical Factors that Can Influence the Integrity of Rigid-Fix Paving

Having considered a large number of factors which *might* exert some influence on stonework in some places, in a variety of situations, using stone of wide compositional range and employing a system that does not follow the strict guidelines and standards that are ostensibly followed in Australia, it is necessary to examine a range of factors that *are* likely to play a significant role in the success or failure of rigid-fix stone paving.

Instability of the Concrete Base

Numerous investigations of paving failures in Europe have shown that the dominant cause of failure has been instability/failure in the supporting/underlying slab. Admittedly, northern Europe and other high latitude countries do face additional difficulties in concrete placement and durability due to subzero temperatures. Nevertheless, there are many common factors, such as shrinkage, cracking, warping (curling), and reactive aggregate. Any dimensional instability in the concrete will affect the integrity of rigid-fix paving.

Below every concrete slab is a subbase which also has to be effectively stabilized in order for the concrete slab to be stable. However, there are some situations that are challenging such as sand-fills, steep slopes, unconsolidated material, old river channels, acid sulfate soils and mangrove flats, and sometimes the in-house engineers do not get it right. Occasionally, problems do not present themselves in the first few years of service in these difficult locations because of changes in environmental conditions and usage, e.g., unexpected vibrational effects. Furthermore, effective stabilization can be rather expensive especially where piling to bedrock is required.

It is interesting that when paving fails within a few years one of the first aspects to be investigated is the concrete base. Cores are customarily taken, to confirm what—its thickness? Was this not being supervised during construction? Other geotechnical and scientific tests may follow, such as the possibility of ASR (alkali-silica reaction), poor mix, or chemical attack, but rarely is the quality of the concrete to blame for physical slab instability.

Curing/Shrinkage of the Concrete Slab

It has been observed that some thick concrete slabs are cured for only seven days before paving commenced. This is contrary to general practice and the Australian Standard [10] which recommends a curing time of 28 days for concrete with compressive strengths in the range of 25 to 40 MPa. However, adhesive manufacturers are providing assurances that their products are capable, *indeed designed*, to accommodate the shrinkage of the slab even during the early periods of maximum shrinkage.

It is widely recognized that 30 % shrinkage will occur in the first 28 days, about 50–60 % over the first three to four months, and the remainder over the remaining year. It is also recognized that thick slabs (180–300 mm) take longer to cure (and longer to shrink) than thinner slabs. The amount of shrinkage is substantial with strongly reinforced concrete averaging around 1 mm every 2 m. This considerably exceeds thermal expansion of the stone. A strong grout between the pavers would ensure rigidity in the paving and there must be a possibility that shrinkage of concrete over 10 m (5 mm) could stress the bonding of the adhesive, especially while thermal expansion is occurring in the granite.

As concrete cures and the volume of the slab decreases due to any or all of the four recognized shrinkage mechanisms (drying, autogenous, plastic, and carbonation), tensile stresses are inevitably created in the slab and fractures develop. In the presence of seawater chloride ions will diffuse through the slab and cause corrosion of the steel. This is generally a longer-term factor of concrete instability.

Expansion Joints

Expansion/contraction/construction joints are usually spaced as per standard requirements. Depending on the design, most joints are arranged in a square or squat rectangular pattern. Construction joints tend to be longer but little distinction is made from the more-common expansion joints. Curling of the concrete surface is commonly observed at construction joints due to differential shrinkage between the core and the top surface.

Most of the sealants used for these joints are two component polyurethane sealants with an expected life of around ten years. But depending on the stability of the concrete surface, the adhesive used, the vehicle loading, and the environmental chemistry, sealants not uncommonly show signs of deterioration after only a few years in service. An obvious compression of sealant at a joint might clearly demonstrate that the joint is working but also indicates that there are likely to be considerable stresses on the bonding of the adhesive.

Because these joints are rarely fully sealed, especially in paving >30-mm thick, they often serve as a source of water ingress and tracking. This permits the formation of subsurface resident water which causes many problems (see section on Imbibition Coefficient).

Size, Thickness, and Shape of the Stone Pavers

Large, square paver sizes (600 by 600, 400 by 400) can be used for domestic use, shopping centers, and any other application where there is limited (if any) light vehicular traffic. This size is economical to produce and easy to lay but there is little variation possible in the paving pattern (stacker and stretcher). A 30–50-mm thickness of that size reduces considerably the options of use and the most common applications for that size are footpaths, sidewalks, garden paving, low level cladding, and domestic driveways (i.e., where the traffic is light and infrequent).

A single 400 by 400 paver is sufficiently large to accommodate an entire wheel loading even for wheels as large as a 40-seater bus. Provided that it is rigidly affixed to a stable, flat substrate without any ridging or variation in thickness, trucks and buses could drive over granite paving with a low risk of failure because the compressional and tensional loads are well within the capacity of most competent stone. However, any unevenness of the stone, unevenness in the concrete base, structurally inhomogeneous bedding/adhesive, the presence of introduced material, or the removal of any supporting material will cause point loading. Point loading can create pressures up to 20 times that of even loading and even 40-mm thick granite and basalt cannot withstand this, particularly in the presence of dynamic (shock) loading. Shock loading is where there is a rapid impact on the stone paver (e.g., from a rapidly moving/braking vehicle) akin to striking the stone.

Virtually all significant external paving projects in cities around the world use rectangular pavers. Sizes will vary according to the strength of the stone, textures and structures, the maximum weight of vehicles likely to drive onto them, the frequency of these vehicles, the type of subbase, the type of base, the thickness of the paver, and other considerations such as shipping, ease of production, ease of handling, and ease of laying. Many different stone varieties are used for paving and considerable testing is done by most authorities/end users to establish the suitability of the stone in relation to its intended application.

In Europe, and to a lesser extent North America, there are arbitrary codes/categories for the use of stone paving and methods of laying. These categories recognize the engineering factors likely to have an effect on the durability of the stone paving. For example, bus stations are bracketed with airport landing zones in terms of the high dynamic loading of the pavement and, to ensure stability, small rectangular paving units with thicknesses up to 225 mm are used. A more typical thickness specified for heavy vehicular traffic is between 80 and 150 mm. For lesser carriageways and roadways, rectangular paving units remain small but with thicknesses typically 70 to 80 mm depending on the volume and weight of the traffic. The British Standard BS 6717 states that a minimum of 60 mm is compliant for stone pavers for areas with vehicular access.

The most common application of square pavers is in the form of setts and cobblestones. La Mar Diamant, located in Singapore, has over 50 years of experience in laying pavers, having placed over 1 000 000 square metres in that time. For general purpose roadways, their typical unit is a cube of 100 mm. Cobblestones can be 200 by 200 but with a thickness of at least 100 mm. One of the most frequently used stone types for setts and cobblestones is porphyry—an extrusive (volcanic) or high-level intrusive rock type characterized by scattered large crystals (phenocrysts) in a very fine-grained groundmass. Alteration of the rock is usually extensive but because it is used as a solid unit it tends to maintain its integrity. Colors are usually gray (Australia) and deep burgundy (Italy), but there are other colors such as green, mauve, and brown. The color is a reflection of the type and intensity of alteration; haematite gives red, chlorite and epidote give green, and weathering gives brown. However, it is important here to emphasize that there is nothing "magic" about a porphyry in terms of its use as paving. Because of its mode of formation most commercial porphyries do not occur in solid enough rock masses that are amenable to block extraction and slabbing. They are often dug out mechanically and then simply presented to a hydraulic splitter. In effect, porphyry is a low cost product that has been adapted for this use because it is very difficult to find any other uses. Any other reasonably competent igneous rock can be used for this type of application. Indeed, in Portugal, many kilometres of roads and sidewalks are graced by good quality limestone that shows little signs of wear.

Even more so than with rectangular setts, cubic setts in trafficable areas are arranged in circular or fan-shaped patterns, or both, that are not only aesthetically decorative but serve to deflect forces in a complex way.

It must be emphasized that most of the thicker paving done in Europe, Canada, and the United States is not bonded to concrete but laid on a compressed grit/sand base. This is particularly useful in urban areas where there is a need to access services. Rather than requiring a jackhammer approach to remove/destroy firmly attached paving, the pavers laid on sand with a sand grout are simply removed and then replaced after the service work is completed. This approach has been demonstrated to dramatically reduce the cost of maintenance. The first section of the mall redevelopment in Perth (Hay St.) has recently been completed using 75-mm thick granite paving, laid on a compressed deco base. The remaining malls will be done the same way. The preferred Australian granites were tested in many configurations for strength but did not achieve the required rating. Even with the much stronger, imported granite many tests were undertaken to select the most appropriate, rectangular paver dimension.

I was astounded recently to witness an engineering requirement on the Gold Coast for an exfoliated finish to the underside of a brick-shaped granite paver in order to achieve less slip on a deco base.

Laying Pattern of the Stone

It has long been recognized that some paving patterns perform better than others. Tests and experience around the world have shown that the herringbone pattern is superior in many functions of paving (e.g., deflection, creep). However, this pattern requires rectangular units similar in shape to that of a household brick. In this pattern, static loads, dynamic loads, and torsion are distributed over several paving stones with forces acting in different directions. The stacker pattern of paving is usually restricted to domestic use and internal use where there are no stresses or substantial loads. It is the least desirable and least effective pattern for external paving.

In some designs a circular pattern is demanded with square pavers. The only way this can be achieved is by a progressive increase in the grout widths towards the outside circumference so that grout widths range from 20–30 mm. The integrity of the paving is severely compromised because it is substantially reliant on the grout but which has a strength of only 20–25 % of the stone. A breakdown of the grout (such as dynamic vehicle loadings on the pavers, chemical disintegration, salt attack) will lead to paving failure. Considerably greater strength and stability can be provided by supplying the stone pavers cut to a trapezoidal shape. The added strength of interlocking shapes in stone structures has been known for over 2000 years.

Adhesive

The effective bonding of stone paving to its base is essential in a rigid-fix system. There are many ways of achieving this and the preferred method is by using an adhesive system. The traditional way of fixing stone to a cement screed in a wet slurry mix has generally been superseded for several reasons, one of which has been long-term integrity.

Adhesives tend to be preferred because of the time constraints placed on projects, the likelihood of concrete base shrinkage, thermal expansion of pavers, and possibly, prolonged saturation of the cementitious screed. One other major financial consideration is the availability of adhesives that do not require lengthy curing times for the concrete base.

The adhesive market is very large and the competition among manufacturers to secure contracts is intense. Indeed, the choice of adhesives to fix tiles and pavers is very extensive and the decision-making process for the choice of adhesive is difficult. This is especially so because of numerous advances in this industry in the past ten years and frequent changes in formulations to remain competitive.

Because of the competitive nature of the industry the advertising commonly exceeds scientifically and technically proven specifications. Manufacturers can get away with this because in the large majority of cases the product serves its purpose (by keeping the tiles and pavers stuck to the floor) even if the quality of adhesion is poor. One method of securing a contract is to warranty the product for extended periods and even warranty the products for uses that are outside their own specifications. Careful examination of some of the warranties reveal ambiguities and conditions that enable the manufacturer to be absolved from blame in case of a failure. It must be remembered that there is no way that manufacturers of a wide range of products can adequately or conclusively research the efficacy of all their products in a very wide range of situations with an enormous range of natural and man-made paving materials.

There are several categories of adhesive system available for stone paving. One is a two-component system where a cementitious base is mixed with a polymer (synthetic rubber latex). Another is a flexible powdered adhesive containing fragments of rubber. A third uses a combination of thin but strong priming and bedding mortars. Epoxy adhesives could be added as another category. All have their strengths and weaknesses and it is essential to use the adhesive that is best suited for the intended application. Then it is important to choose an adhesive that is compatible with the working conditions. A hot, humid exposed environment can impose severely limiting conditions to the successful use of many adhesives. Open time, pot life, and mixing time can be down to minutes in such environments. Where back-buttering is required in addition to the normal adhesive spread, increased handling time and final positioning time is very limited when the weight of some pavers can be up to 20 kg.

Adhesive with a substantial rubber component is widely regarded as suitable for domestic tile applications and possibly large areas of terrazzo or ceramic tiling, or both, in interior public places. In these conditions there is negligible dynamic loading and certainly negligible torsional loading. However, for external stone paving under severe and frequent loadings this type of rubber-based adhesive is totally unsuitable.

The thin, sandwich type of adhesive is used in Europe in an attempt to minimize the mismatch of strengths between the different elements in a composite paving structure and to direct the loadings through the stone and into the concrete base. For this to work effectively a high quality concrete base is required.

It is customary in Australia to present the stone product to a prospective adhesive manufacturer (along with a description/inspection of the project) for the formulation of specifications. The manufacturer then warrants the adhesive for whatever period is required. A critical analysis of the specifications shows that the descriptions and requirements are often poorly worded, without a good understanding of the paving circumstances, and basically “marketing talk.” As noted earlier, the manufacturers of these products “hope” that the adhesives will perform satisfactorily as they have done in a number of other situations and circumstances.

Some manufacturers go further and require that all work carried out in the system recommendation must be carried out by a manufacturer-approved specialist employing skilled personnel under the direction of an experienced supervisor.

Jointing Widths and Jointing Material

Due to the regularity in size of stone paving (governed by standard tolerance requirements) the typical jointing widths in stone paving is somewhere between 3 and 5 mm. However, it has been noticed that some designs call for a jointing width of 10–25 mm on external stone paving. A jointing width of 10–12 mm is typical for porcelain, terracotta, and ceramic tiles in domestic situations but not for stone pavers affixed to a concrete base by adhesive. Interior stone tiling often demands narrow joints of 1–2 mm which introduces some difficulties in grouting. Customary grouting practice is to sponge in the grout but this is poor practice because of the difficulties in getting the grout down the edge of the tile. Grout should have a bond with the bedding mortar or adhesive bed and the only way this can be done effectively is to butter the edge of the tiles prior to fixing (i.e., more work). Butt-jointing stone tiles is another poor practice that frequently leads to failure.

Because of the contrasting physical behavior and chemistry between stone and grout it is inevitable that fractures develop in grout, especially in a dynamic environment. Even though some grouts are fairly stiff with a compressive strength of about 40 MPa (when freshly made) they are weak when compared with an average strength for stone pavers of around 160–200 MPa. Increasing the jointing widths reduces the overall unit strength.

Two types of fractures develop in almost all jointing materials, namely (a) hairline partings along the stone, and (b) transverse fractures across the grout. Fractures clearly further reduce the paving unit strengths and allow easy ingress of water and chemicals. Grouts are also quite porous (3–5 %) so can accommodate a range of dissolved substances such as salts and sulfates which can gradually lead to growth, efflorescence, and internal disintegration. A physical breakdown of the grout and its removal from the joints contributes to a loss of paving integrity.

Dynamic Loading/Torsional Effect on Pavers

Provided the grout remains intact in its original form, a reasonable degree of rigidity could be expected even when flexible adhesive is used. However, because of its reduced strength compared to stone it is undoubtedly a weak link and differential compression of the grout by heavy or torsional (radial), or both, loading (due to the angle of approach of vehicles) will inevitably lead to grout failure. Once the grout has been crushed or broken it is easily removed physically and the paver that was held in place by the grout is now less constrained. Any subsequent dynamic or torsional force on the unconstrained paver will lead to shearing along the adhesive/stone interface and impingement by the paver onto an adjacent paver or a flexible control joint. Once the stone paver adjacent to the flexible joint has sheared there is rapidly a progression of shear failure in the second line of pavers, and so on until the entire paving has failed.

A trafficable, paved slope presents a particularly demanding scenario in terms of a combination of different stresses. Roundabouts or bus terminals are rarely paved with stone fixed to a concrete base. Instead, because of the severe tangential deflection in such situations, setts, cobblestones, or thick pavers are preferred, bedded on a compressed base. The frequency, spacing, and thickness of the lateral restraints required for such situations need to be ascertained. Interestingly, in southern Europe, many narrow winding roads are paved with stone setts or cobblestones, or both, and have remained trafficable for centuries.

Influence of Salts

Salts and other chemicals (e.g., sulfates) have a physical property which can be very detrimental to some types of stone. Salts have a high crystallization strength, i.e., when they come out of a saturated solution, they can exert considerable forces on the pores of some natural stone. Porous limestone and most sandstones are potentially susceptible to damage by the entry and subsequent crystallization of salts. The same stones are also susceptible to damage through freeze-thaw action and appropriate tests need to be carried out to determine suitability for a certain application or at least determine the level of risk, or both.

Although granites have very little pore space compared to porous limestone and most sandstone, the grouts, mortars, and adhesive have porosities and rates of water absorption similar to those of these sedimentary rocks. The location of a project immediately adjacent to the waterfront means that there is an inevitability that some salt is carried from the sea to the stone paving. Because of its solubility, salt is carried in solution into the pores of the bonding media where it has the opportunity to buildup and concentrate. Although it is unlikely that in isolation it has had a significant impact on the strength and integrity of the mortars and adhesive, it can contribute to an accelerated degradation of these bonding media once there is ready ingress of water through hairline fractures or failures in the stone. Chemically, salts are often not compatible with adhesives and react gradually to cause a weakening of the bonds in the adhesive.

In cold climates such as Scandinavia and parts of North America salt is used as a deicing substance on many roadways and footpaths. This introduces very high concentrations of salt into the concrete, the subbase, any stone paving, stone kerbing, and into the adjacent soils. In these locations it is customary to use high-strength, low-porosity stone such as granite.

Influence of Water

In the absence of mechanical factors, water is generally the principal factor causing problems with natural materials, including stone. And thermal change is the driving mechanism that activates the water. The interaction between water and stone takes many forms. Some of these have already been mentioned previously. At first appearance, water could be ideally seen as simply falling onto the stone from rain and then being taken away by the drains. Upon drying, the effects of water appear to have disappeared. However, below the surface many reactions are occurring because of the residence and persistence of the water. Not only is the water the result of rain but nightly in many paved shopping/transport areas there is a cleaning regime in place which hoses down some of the paving, cleans windows, and waters gardens. In other words, the grout, adhesive, and concrete base are subject to water daily. Furthermore, it is not unusual for the paving design to allow water to freely track along construction joints where it can then move laterally away from those joints and be absorbed by the grout and adhesive.

Prolonged contact with water can damage the bonding in the adhesive, particularly if there are dissolved chemicals such as salt, sulfates from vehicle exhausts, detergents and surfactants from the window and floor cleaning, phosphates from the planter beds, and hydrocarbons from vehicle leaks.

Water will also dissolve small amounts of calcium from the concrete base, grout, and cementitious adhesive especially at elevated temperatures (around $35-40^\circ C$) and precipitate the calcium at sites where the redox conditions are different or suitable, or both. Iron is also mildly soluble in its hydrated forms and this can show as yellowish to light brownish stains on the stone, in contrast to the grayish stains of the calcium.

Apart from the chemical aspects of the interaction between water and cementitious products, water can act as a powerful hydraulic medium. If a soft, absorbent, or porous substance contains fluid, a strong compressive force will cause the water to be violently channeled away from the area of applied force. A large force acting on small amounts of free water in a porous/soft medium can create a hydraulic jetting effect that will rapidly break down the host medium. A multiple jetting effect caused by a series of bus or truck tires can effectively remove the adhesive that is/was bonding the paver to the concrete. White blooms and stains appearing on the stone paving attest to the mechanical, physical, and possibly chemical breakdown of the adhesive.

Sealers

The application of a water-repellent substance to the surface of the stone might seem to be a savior in a bottle that protects the stone from the effects of water and chemicals. This is far from the truth. Sealers can be detrimental to the performance of the stone as well as compromising the aesthetics. Salts, sulfates, calcium compounds, and any other soluble chemical can be drawn up through the stone in response to the thermal engine noted previously. However, when the dissolved substances hit the sealer they can go no further. The “breathability” often referred to in advertising is applicable only to vapor, not liquid. Gradually, enough pressure can build up to cause the surface and edges of the stone to crumble, fret, and spall. Before that, in granite that is porous, the soluble substances can accumulate under the surface of the sealer and cause a patchy, usually grayish discoloration that is difficult to remove. Not surprisingly, some of the substances remain somewhat soluble and can be remobilized to appear elsewhere in a different form if the substrate becomes periodically saturated.

Workmanship

If any of the above factors have been overlooked in the planning and design, and there is a paving failure, there is always the intangible factor at the bottom of the pecking order which can be blamed—workmanship. Unfortunately this is all too common and the contractors are often in no position to argue the case against the architects, engineers, and construction "experts." Some contractors do follow the documentation to the letter and comply with all the rules and regulations and the quality of workmanship is at least as good as most overseas projects in similar situations.

However, with the mushrooming of projects using stone, many tiling contractors, experienced only in domestic porcelain, ceramic, and terracotta tiles, have tried their hand with stone, sometimes following written advice. With little or no understanding of the properties and requirements of a natural material, many disasters have ensued and in this highly competitive world there is little that can be done about those inexperienced contractors who successfully win their jobs by quoting cheaply, promising the earth but delivering substandard work. A more rigorous selection process for stone paving tenders is certainly warranted in Australia and most European countries. Work practices appear to be stricter in the U.S. where a prequalification process can exclude inexperienced contractors.

There is little doubt that workmanship is an important factor in successful paving. It may appear to be an intangible attribute—and one that is difficult to qualify; however, the remarkable preservation of many old areas of paving after more than 30 (and often 50) years (e.g., Nuremberg Airport), using the same or similar varieties of stone still in use today, attests to the skills of the old stonemasons. Time and care may be more valuable than generally credited—two factors that are commonly missing in the present-day pace of construction.

Summary

There are many key factors that need to be considered in the design and successful construction of external paving. An understanding of the stone, particularly its behavior in its intended application, is essential. Consisting of minerals that have come out of solution (igneous rocks), have recrystallized under elevated thermal conditions, pressure and stress (metamorphic rocks), or been the subject of erosion, attrition, sedimentation, and then lithification (sedimentary rocks), building stone is a complex natural material that has a range of physical characteristics, just like glass or steel framing. When placed under load or put into demanding situations these characteristics must be scientifically analyzed to predict performance. Expert advice needs to be sought on the stone to be used, how it is to be laid, by what pattern, and on the maintenance of it. Shortcuts cannot be taken and in-house advice is often inadequate. The "she'll be right" attitude is no longer acceptable.

When stone paving fails it generally fails due to a combination of factors and on analysis it is often the basic choice of paving type that is incorrect. Although rigid-fix paving might seem the most "controllable" option it is usually the one that registers the most failures.

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Material Strength Considerations in Dimension Stone Anchorage Design

ABSTRACT: One of the most important aspects of designing dimension stone cladding involves determining the configuration, size, and spacing of the anchorages that will affix the stone panels to the underlying building substrate. Information from many sources, including material strength testing, anchorage strength testing, and knowledge of the stress states created within stone cladding panels by loaded anchorages, is crucial to this process. In this paper, relationships are examined between material strength, anchorage strength, and induced stress states for four common anchorage configurations: edge dowels, Type 31 back anchors, and edge kerfs with strap and split-tail anchors. For each of these four anchor types, the relevant material and anchorage strength test data obtained for a medium-grained granite, in conjunction with finite element computer analyses of the stress states induced in the stone panels by the anchors and within the material strength test specimens, are presented. Material strength test configurations evaluated include ASTM C 880 Standard Test Method for Flexural Strength of Dimension Stone, ASTM C 99 Standard Test Method for Modulus of Rupture of Dimension Stone, and a proposed test for punching shear currently under consideration by ASTM Subcommittee C18.01. All anchorage strength tests were performed in accordance with ASTM C 1354 Standard Test Method for Strength of Individual Stone Anchorages in Dimension Stone.

KEYWORDS: dimension stone, cladding, design, anchor, anchorages, connections, testing

Introduction

The authors recently performed strength testing of four anchorage configurations for a granite cladding system being installed on a commercial high-rise building. These tests, in conjunction with absorption and flexural strength testing of the granite, were performed as part of the preconstruction verification process to assess whether the proposed material, cladding panel sizes, and panel anchorage details met the requirements set forth in the project specifications. At the time the tests were performed, the cladding system design had been completed, shop drawings had been prepared and approved, production of the granite cladding panels was nearly complete, and panel installation had already commenced. As happens more often than many designers would like to admit, the strength values from the anchorage tests were lower than expected, causing much consternation and requiring design changes for certain panels and the installation of supplemental anchors on some of the panels already installed.

Once the dust had settled and the project had been completed, the authors found themselves with surplus odd-sized granite pieces for which neither the building owner nor the design team had any use. The combination of the available anchorage and material strength test data, additional stone from the same production run, and the lower-than-anticipated anchorage strength values presented the authors with an opportunity to research the designs and assess the potential for predicting anchorage strength from material strength data. Adding to the already available data, the authors conducted additional tests following a proposed procedure for punching shear currently under consideration by ASTM Subcommittee C18.01. Additionally, finite element computer analyses were made of certain anchorage and material strength test configurations in order to compare stone stress states present. Findings from these studies are presented in the following order: (1) anchorage strength test results; (2) analytical results for the tested anchorage configurations; and (3) material strength test data and test specimen analyses.

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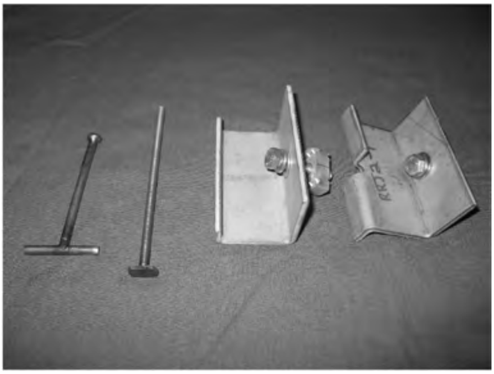

FIG. 1—*Dowel, Type 31, strap and split-tail anchors (left to right).*

Anchorage Strength Tests

All of the anchorage strength tests discussed herein were performed according to the ASTM C 1354 test method [1] using medium-grained granite specimens measuring nominally 12-in. (30.5-cm) square by 1.2-in. (3-cm) thick and having one polished and one sawn face. The tests included four common anchorage configurations: edge dowels, Type 31 back anchors, and edge kerfs with strap and split-tail anchors (Fig. 1). The failure shapes and sizes were documented along with the failure loads with the objective of comparing these data with failure shape observations from the material strength tests for flexure and punching shear all performed using the same stone type.

Kerf Anchors

The test configuration for discrete-length (noncontinuous) kerf anchors installed in a continuous kerf utilized 12-in. (30.5 cm) square stone specimens with one edge supported using the anchor, the opposite edge on a roller support, and the test load applied on the top surface of the stone as close as possible to the strap anchor but at least the thickness of the stone plus the kerf depth from the edge in order to avoid interference of the load application apparatus with the kerf failure surface. The actual load applied to the anchor, which was less than the total applied load, was calculated using simple statics.

Tests were performed using stainless steel strap anchors and split-tail anchors installed in a continuous kerf (refer to Fig. 2 for anchor dimensions) to simulate both wind pressure and wind suction. The test configuration for wind suction is shown in Fig. 3. The stone rift orientation was not indicated on the samples received. All specimens were tested after being in dry storage for over one month, with no additional wetting or drying procedures used. The anchors were bolted to a proprietary cold-formed channel support which was attached to a heavy steel reaction frame using threaded rods. During testing, the authors verified that the cold-formed channel support did not undergo appreciable vertical or lateral

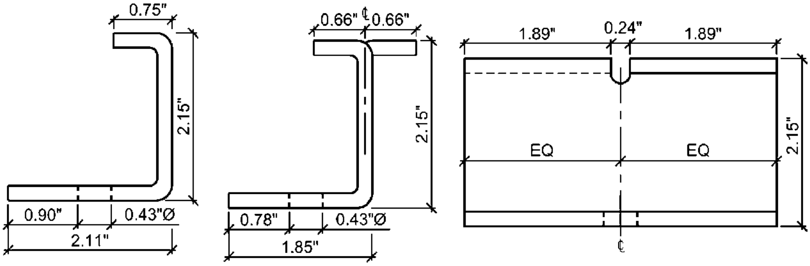

FIG. 2—*Four-in. long strap anchor (left), split-tail anchor end view (middle), and split-tail anchor side view (right). Anchors are 0.19-in. thick.*

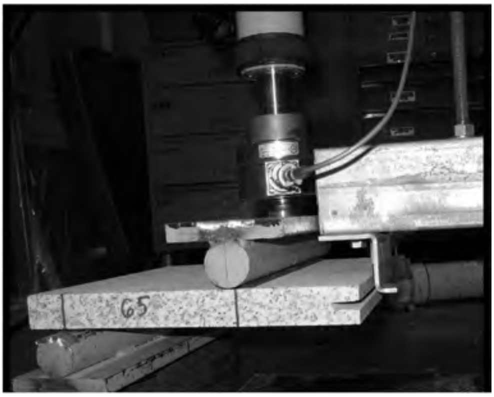

FIG. 3—*ASTM C 1354 test configuration simulating wind suction with a split-tail anchor and 12-in. square stone specimen.*

displacement. A 0.145-in. (0.368-cm) thick shim was placed between the anchor and the stone edge to simulate the field installed condition. The results of the wind suction strap and split-tail anchor tests are presented in Table 1. A typical spall generated by a 4-in. (10.2-cm) long strap anchor test is shown in Figs. 4 and 5. The spalls from the 2-in. (5.1-cm) long split-tail anchor test were similar in general shape but shorter in length. The orientation of the spall failure surface when viewed in section (refer to Section C of Fig. 5) typically was between 30 and 45 degrees from vertical. The effective span-to-depth ratio of the short cantilever legs adjacent to the kerf slot loaded during this testing was typically around 3.5. This value is determined by taking twice the kerf depth divided by the leg thickness which converts the cantilever behavior into an equivalent simply-supported beam.

Type 31 Anchors

The Type 31 slotted back anchor testing performed used a test arrangement where the anchor, installed from the back of the test specimen, was coupled to a threaded rod and load was applied directly along the anchor axis using a hydraulic ram. A cylindrical ring and tube section were used to support and provide a reaction surface for the hydraulic ram. The anchor load was read directly from an in-line load cell. The average anchorage failure load for the nine tests performed was 1443 lb (6419 N) with a standard deviation of 151 lb (672 N). The average of the anchorage failure load divided by the horizontal projection of the spall area was 121 psi (834 kPa) with a standard deviation of 12.9 psi (88.9 kPa).

The spalls created during the Type 31 anchor tests were traced, digitized, and the projected spall area calculated using a common computer aided drafting (CAD) software. A theory for predicting spall dimen-

TABLE 1—*Average values of ASTM C 1354 tests performed to simulate wind suction.*

	Strap Anchor		Split-Tail Anchor	
	Value	Standard Deviation	Value	Standard Deviation
Number of tests	13	NA	13	NA
Anchorage failure load	538 lb 2390 N	36 lb 160 N	321 lb 1430 N	28 lb 125 N
Interior stone face to rear kerf face	0.455 in. 1.16 cm	0.012 in. 0.030 cm	0.466 in. 1.18 cm	0.017 in. 0.043 cm
Rear kerf face to front kerf face	0.273 in 0.693 cm	0.005 in 0.013 cm	0.273 in. 0.693 cm	0.006 in. 0.015 cm
Kerf depth	0.819 in. 2.08 cm	0.031 in. 0.078 cm	0.812 in. 2.06 cm	0.025 in. 0.064 cm
Anchor embedment into kerf	0.432 in. 1.10 cm	0.007 in. 0.018 cm	0.461 in. 1.17 cm	0.000 in. 0.000 cm

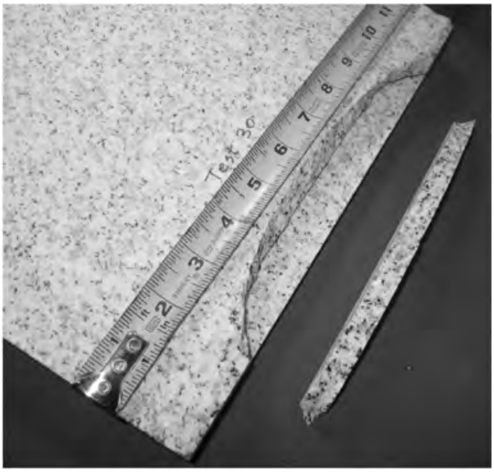

FIG. 4—*Spall generated during an ASTM C 1354 test using a 4-in. long strap anchor.*

sions was developed based upon these digitized spall traces as shown in Fig. 6. A failure surface 20 degrees from the stone plane was assumed. This orientation generally agrees with test observations except the failure surface typically started out steep adjacent to the location loaded, and the slope became shallower moving radially away from the location loaded, generating a concave shape. The failure surface starting location is at a distance d_f from the stone surface as shown in Fig. 6. A circular spall perimeter is predicted where the center of the circle is located at the longitudinal axis of the Type 31 anchor. The area determined based upon the spall radius was reduced by the slot area. The following equation was used to predict the horizontal projection of the spall area:

$$A_{spall} = \pi[d_f \tan(70°) + G_W/2]^2 - (G_w G_L + \pi G_W^2/8) \tag{1}$$

Note: Refer to Fig. 6 for variable definitions

The average ratio of the area predicted by Eq 1 to the area determined by the digitized spall trace for the nine tests performed was 1.01 with a 14 % coefficient of variation. The actual spalls were somewhat elongated and more elliptical than circular; however, Eq 1 is still a good predictor of the spall area; therefore, a procedure more closely matching the actual spall shape is unnecessary.

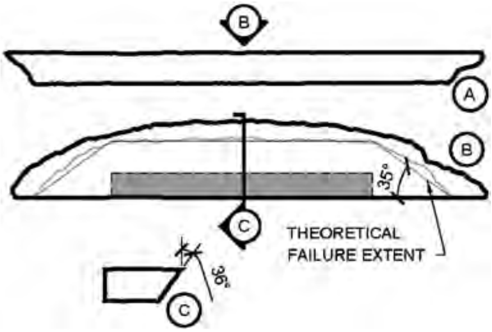

FIG. 5—*Tracing of a spall generated during an ASTM C 1354 test using a 4-in. strap anchor. Edge view (A), surface view (B), and section view (C) shown. Shaded area represents anchor location in kerf slot. Light line in B shows theoretical failure extent.*

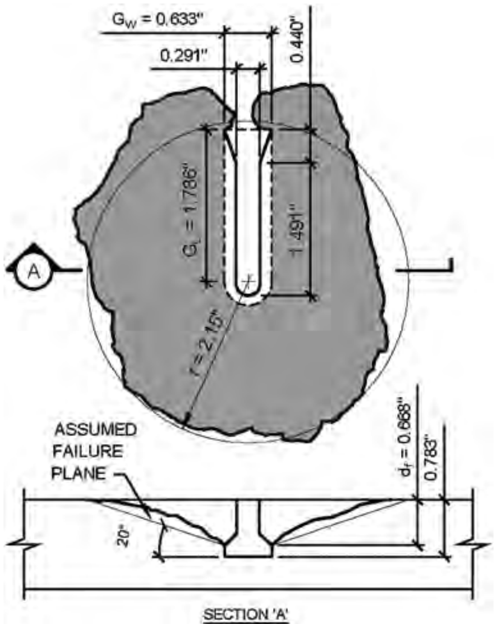

FIG. 6—*Spall trace for Type 31 anchor test with stone section view (bottom) and surface view (top). Idealized spall location shown with lighter lines.*

Edge Dowels

The test configuration for dowel anchors inserted into the edge of a dimension stone cladding panel can take a variety of forms depending upon the type of hardware being tested. The tests used a configuration where 1/4-in. (0.64-cm) diameter dowels (Fig. 1) were inserted 3/4 in. (1.91 cm) into nominally 1-in. (2.54-cm) deep by approximately 0.32-in. (0.81-cm) diameter holes in two adjacent pieces of stone separated by a 1/2-in. (1.27-cm) wide open joint. Load was applied directly along the anchor longitudinal axis by coupling a threaded rod to the anchor and applying axial load to the threaded rod using a hydraulic ram. The actual anchor load in this configuration is equal to the load obtained from an in-line load cell divided by two. The average anchorage failure load from the 13 tests performed was 501.8 lb (2232 N) with a standard deviation of 46.2 lb (206 N). Dimensions from one of these tests are shown in Fig. 7 and a photograph of a spall generated from an edge dowel anchor test is shown in Fig. 8.

The spalls created during the edge dowel anchor tests were traced, digitized, and the spall area calculated using CAD software. A theory for predicting spall dimensions was generated based upon these digitized spall traces. As viewed from the side of the stone, the spalls were typically oriented at 20 degrees from the primary surface plane of the stone. The spall was assumed to initiate at the drilled hole location where a horizontal chord drawn across the hole has a length equal to the anchor diameter. This location, S_1, is given by:

$$S_1 = E + \frac{D_H}{2} - \frac{\sqrt{D_H^2 - D_A^2}}{2} \tag{2}$$

Note: Refer to Fig. 7 for variable definitions

Lines drawn at 20 degrees from the horizontal from the assumed initiation location to the stone edge define the spall width. The spall width, W, is given by:

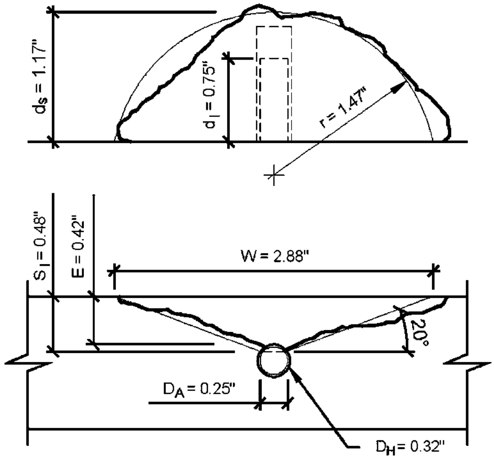

FIG. 7—*Surface view (top) and edge view (bottom) of edge dowel test showing spall geometry and predicted failure.*

$$W = \frac{2 * S_1}{\tan 20^\circ} + D_A \tag{3}$$

Note: Refer to Fig. 7 for variable definitions

The spall depth, d_s, is taken as the dowel embedment depth into the stone plus the distance from the stone surface to the top of the drilled hole. This assumes a 45-degree failure plane initiating at the top of the hole at the anchor embedment distance. The spall depth, d_s, is given by:

$$d_s = d_1 + E \tag{4}$$

Note: Refer to Fig. 7 for variable definitions

The anchor most likely does not bear for the full embedment distance on the stone surface due to anchor deformation and misalignment. If bearing occurs primarily at the stone edge, other methods may serve as a better predictor of d_s. However, for the testing performed by the authors, Eq 4 proved to be an adequate predictor.

A circle segment can then be fit to the three points defined by W and d. This method can only be applied when $d \leq W/2$. The radius, r, of this circle is given by:

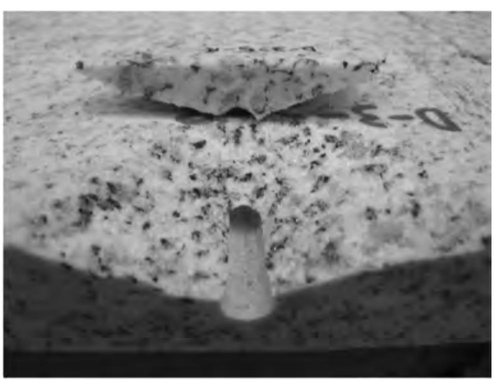

FIG. 8—*Spall from edge dowel test.*

$$r = \frac{W^2 + 4d_s^2}{8d_s} \tag{5}$$

Note: Refer to Fig. 7 for variable definitions

The projected horizontal surface area of the spall, A_{spall}, is given by:

$$A_{spall} = r^{2*}\tan^{-1}\left(\frac{W}{2(r - d_s)}\right) - \frac{W}{2}(r - d_s) - D_A d_1 \tag{6}$$

where calculations are performed in radians. Fitting of other shapes, such as a triangle or half ellipse, to approximate the spall surface leads to simplified equations; however, accuracy in the prediction is sacrificed. Note: Refer to Fig. 7 for variable definitions.

Each test generated two spalls (one from each stone) and these areas were added together giving the gross spall area. The area defined by the measured anchor breakout widths and breakout depths was subtracted from the gross spall area, resulting in an area representative of the projected failure surface. The average ratio of the theoretical area determined using Eq 6 to the area determined by the digitized spall trace for the 13 tests performed was 1.035 with a 15 % coefficient of variation.

Anchorage Stress States

The states of stress present in kerf anchorages and edge dowel anchorages were investigated using finite element computer models. The material properties used in the models assumed linear-elastic, isotropic, and homogeneous behavior, whereas dimension stone, being a naturally occurring material, is heterogeneous and anisotropic. However, the models presented are used to gain a general understanding of the stress states present. This understanding will assist with determining the material strength test data best suited to predicting anchorage performance.

Kerf Anchors

A two-dimensional finite element model representative of a kerf anchorage configuration was developed in order to investigate the states of stress present in the stone in the vicinity of the anchor. The maximum principal stresses from the finite element model, which utilized triangular membrane elements, indicate a stress concentration occurs at the 1/32-in. (0.794-mm) radius fillet shown in Fig. 9. Testing by the authors and work by Stecich et al. [2] confirm that the effect of the stress concentration is not as pronounced as predicted by linear-elastic analysis methods. Stecich's et al. method of predicting the maximum stress is based upon a formula presented in *Roark's Formulas for Stress and Strain* [3] with a reduction applied to the stress concentration factor. The values obtained using the previously mentioned methods, normalized by the tensile stress predicted at the root of the kerf using fundamental beam theory, are presented in Table 2.

Three-dimensional finite element models (Fig. 10) representative of discrete strap anchors in a continuous kerf were developed to investigate the behavior of the ASTM C 1354 test configuration similar to that shown in Fig. 3. The anchor and stone geometry were modeled to approximately match the stone and 4-in. (10.2-cm) wide strap anchors previously discussed. Additional strap anchors 2, 6, and 8-in. (5.1, 15.2, and 20.3-cm) wide were also analyzed. The anchor bearing surfaces were modeled as perfectly planar and parallel to the stone kerf surface. This is an unlikely condition to encounter in the field or even in laboratory tests due to anchor imperfections and installation misalignment. Each model included the following components: a load spreader modeled as a frame element, a stone specimen modeled as a combination of solid elements and shell elements, a steel strap anchor modeled as shell elements, and the contact region between the stone and the anchor modeled using compression-only springs. Only a portion of the test configuration was modeled by using a symmetry boundary condition in order to control model size. The region of the stone adjacent to the continuous kerf was modeled using solid elements, and the remaining portion of the stone was modeled using shell elements in order to further control model size. The transition between the solid and shell elements was made by using constraints. The interaction between the stone and the anchor was modeled using springs that provide compression resistance in the axial direction only (refer to Fig. 10), and resistance that may be provided by friction or sealant restraining the anchor from moving further in or out of the kerf slot was neglected. Unlike the two-dimensional

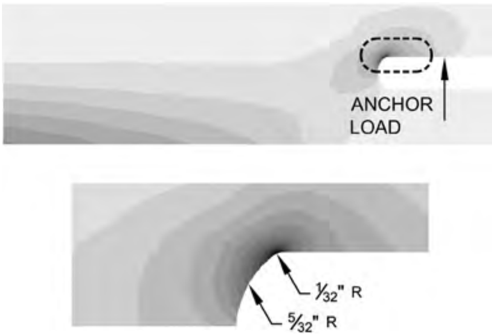

FIG. 9—*Kerf anchorage two-dimensional finite element model using membrane elements. Shade of gray depicts principal tensile stress magnitude (darker means higher tensile stress).*

models used to evaluate kerf root stress concentrations, the root of the kerfs in these models was modeled as a right angle and the mesh was not adequately refined to accurately capture stress concentrations.

The models were restrained to represent the boundary conditions imposed by the ASTM C 1354 testing configuration. Supports provided vertical restraint parallel to and approximately 10-1/2 in. (26.7 cm) from the anchored stone edge. Fixity was provided at the anchor centerline where the tested anchors were bolted to a proprietary cold-formed steel channel backup. The load spreader was modeled as frame elements, which were located 2 in. (5.1 cm) from the anchored stone edge. Load was applied as a single downward point load acting at the intersection of the model plane of symmetry and the load spreader, which simulated use of a hydraulic ram.

A dry-set anchor condition was simulated by assigning a very large stiffness to the spring elements in the strap anchor models. The bearing pressure between the stone and the anchor was calculated by taking the axial force in each spring element and dividing it by the bearing surface tributary to that spring. As shown in Fig. 10, four lines of springs were modeled along the anchor lengths. The edge springs were in compression for all of the anchor lengths, and the first inboard line of springs (0.14 in. (0.36 cm) from the stone edge) were in compression only for the 6-in. (15.2-cm) and 8-in. (20.3-cm) anchors, and at levels much lower than the pressure determined along the outboard spring line. The springs in the third and fourth lines were not in compression in any of the models. This demonstrates that anchor rotation relative to the stone causes dry-set anchors to bear primarily at the kerf edge.

TABLE 2—*Stress predictions at the root of a kerf normalized by the stress predicted by fundamental beam theory (My/I).*

Method	Normalized Stress
Stecich, Chin, and Heidbrink	1.4
Roark	2.3
Finite Element Model	3.0

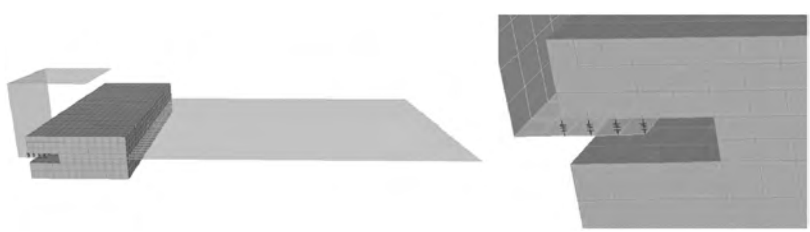

FIG. 10—*Kerf anchorage three-dimensional finite element model using a symmetry boundary condition along the centerline of the anchor.*

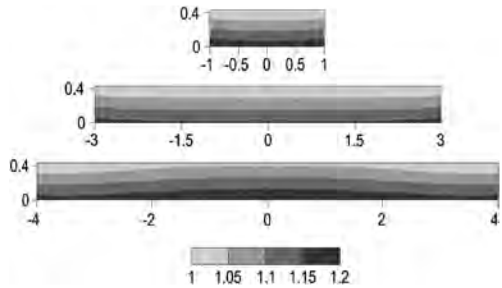

FIG. 11—*Bearing pressure contours for sealant-set strap anchors in a continuous kerf for 2-in. (top), 6-in. (middle), and 8-in. (bottom) anchors. Pressures are normalized by the anchor minimum bearing pressure. The origin is at the intersection of the anchor centerline and the stone edge.*

A sealant-set kerf condition was modeled by adjusting the spring stiffness values to represent the stiffness of sealant placed in a 0.04-in. (0.10-cm) gap between the stone face and anchor face. A confined compressive sealant modulus of elasticity of 454 psi (3130 kPa) was used based upon published values for a common silicone sealant. For the sealant-set condition, all of the springs in the various models were in compression. The spring forces were converted to pressures as previously discussed for the dry-set anchors which provided bearing pressures along the anchor surface. These bearing pressures were converted into contour maps for the 2-in. (5.1-cm), 6-in. (15.2-cm), and 8-in. (20.3-cm) anchors as shown in Fig. 11. The origin of each contour map represents the intersection of the centerline of the anchor-bearing surface with the kerf edge. The contour plots depict the bearing pressures normalized by the minimum bearing pressure. The sealant-set anchors have a trapezoidal bearing-pressure distribution on the stone surface with significantly lower peak pressures than the dry-set anchors.

Type 31 Anchors

The geometry of the Type 31 anchorage configuration makes creating a finite element model cumbersome. Although a model for this configuration was not developed, some general characteristics of the stress states believed to be present based upon the observed failure shapes merit discussion. Section A of Fig. 6 shows the failure plane intersection with the slot transition. A region of higher stress is assumed to be present at this location due to the change in geometry, which suggests that failures are initiated at this point. In many of the tests conducted by the authors, the stone did not spall in the region outboard of the anchor entry point (Fig. 6). This region is isolated from the anchor load by the presence of the installation slot, reducing the applied stress.

Edge Dowels

A three-dimensional computer model was generated using solid elements to investigate the stress distribution present in the typical edge dowel anchorage configuration load tested by the authors and described earlier. For simplicity, load was applied directly to the solid element nodes at the top of the anchor hole. It was assumed that the anchor contacted the stone surface over the entire dowel diameter and for the full depth of penetration into the hole.

Figure 12 depicts the calculated maximum principal stone tensile stresses within the edge plane at the hole. Stress concentrations are present in the vicinity of the hole and quickly dissipate, suggesting that edge dowel anchorage failures are influenced by a nonuniform stress distribution over the failure surface. The direction of the maximum principal stresses located adjacent to the hole are approximately perpendicular to the failure surface observed in the laboratory tests previously discussed, suggesting that failure initiates at this location. When moving away from the location of maximum stress, the principal stresses are no longer oriented orthogonal to the failure plane. A redistribution of stress must occur following the initiation of failure and during crack propagation.

The finite element model was modified to include the steel dowel anchor. The interaction between the dowel and the stone surface was modeled using compression-only gap elements. The gap elements only

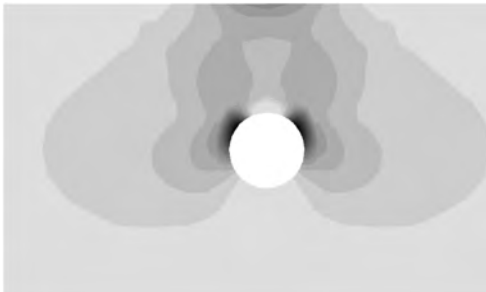

FIG. 12—*Edge view of three-dimensional solid model of edge dowel anchor hole depicting principal tensile stress magnitudes (darker means higher tensile stress).*

provide axial stiffness once the gap between the anchor surface and the stone face is closed. This model showed the dowel bearing upon the stone surface for only a portion of the anchor diameter due to the different radii of the edge hole and the dowel. Additionally, the steel dowel exerted the largest bearing pressure at the anchor entry location in the stone edge hole, and the bearing pressure decreased moving further into the hole until reaching zero at approximately 40 % of the anchor embedment. The failure shapes observed from the tests previously discussed suggest that the anchor bears upon the stone surface for nearly the full anchor diameter at the anchor entry location when the final failure occurs. Also, the depth of anchor bearing varies due to anchor shape imperfections and installation misalignments. The differences between the model predictions and test observations are most likely due to nonlinear fracture behavior that occurs in the stone material and anchor imperfections that are not accounted for in the model. The two finite element models discussed previously represent two different dowel anchor bearing conditions. The observed failure shapes from the test specimens suggest that the actual bearing condition is somewhere in between the bearing conditions found in the two models.

Overview of Material Strength Testing Procedures and Associated Stress States

ASTM publishes standardized testing procedures to characterize dimension stone strength under several types of loadings to assist design professionals with the selection and design of dimension stone used for building construction. The material property information obtained from unit strength tests can be used to assess the flexural capacity of thin stone veneer, stone modulus of rupture, and stone compressive strength. The ASTM C 170 [4] compressive strength test procedure will not be discussed in this paper because this stress state is not generally relevant to dimension stone cladding installations. A test for punching shear currently under consideration by ASTM Subcommittee C18.01 is described later. After reaching an understanding of the stress states present in the material strength tests, their applicability can be considered for use in stone anchorage design.

ASTM C 880—Flexural Strength

The ASTM C 880 [5] test method is used to determine the flexural strength of dimension stone. Testing performed by the authors on the same granite as used for the other tests presented in this paper is summarized in Table 3.

TABLE 3—*Average values from ASTM C 880 flexural strength testing.*

Specimen Condition	Number of Tests	Flexural Strength	Standard Deviation
Dry	25	2068 psi (14 260 kPa)	52 psi (360 kPa)
Wet	25	1808 psi (12 470 kPa)	51 psi (350 kPa)

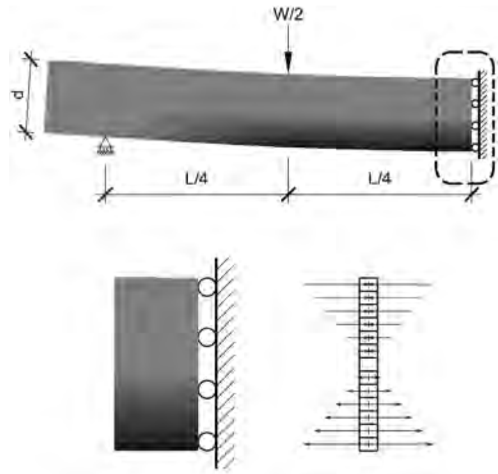

FIG. 13—*Finite element model of ASTM C 880 test using a symmetry boundary condition. Arrows shown indicate direction and magnitude of the principal stresses at the plane of symmetry. Shade of gray depicts principal tensile stress magnitude (darker means higher tensile stress).*

The samples received for initial testing were all marked indicating that the rift orientation of all of the samples was the same. Subsequent testing by the authors performed on a limited quantity of samples cut in orthogonal directions indicated that the samples presented in Table 3 were tested in the stronger rift orientation and that an approximately 12.5 % flexural strength reduction could be expected in the orthogonal direction.

The ASTM C 880 test specimen has a span-to-depth ratio of approximately 10 to 1, which maximizes the flexural behavior of the specimen while minimizing the deep beam behavior associated with small span-to-depth ratios and large shear deformations. The ASTM C 880 test also uses quarter-point loading, resulting in a constant moment region located between the loading points. Upon determining the failure load, the flexural strength of the specimen is calculated using the formula:

$$\sigma = \frac{3WL}{4bd^2} \tag{7}$$

where W is the maximum applied load, L is the span length, b is the specimen width, and d is the specimen depth. This formula is derived using the fundamental beam theory formula:

$$\sigma = \frac{My}{I} \tag{8}$$

which relies on the assumption that flexural stresses vary linearly with the distance from the neutral axis, an accurate assumption for beams exhibiting primarily flexural behavior. With Eq 8 established as governing the ASTM C 880 test specimen behavior, it follows that a region of constant tensile stress is present along the specimen bottom fiber between load points.

In the region of constant moment and stress, an equal probability exists for failure at each plane orthogonal to the specimen's longitudinal axis assuming nonvarying material properties. However, because dimension stone is naturally formed and naturally variable, the failure plane occurs wherever the weakest transverse section lies. Therefore, inherent in the ASTM C 880 testing procedure is a search for this weakest location.

A finite element model using a mid-span symmetry boundary condition is shown in Fig. 13. Although most commercially available stones are naturally anisotropic and heterogeneous, the finite element model constructed by the authors assumed a homogeneous and isotropic material. Stress output values from the model were normalized by the flexural stress predicted by Eq 7. The normalized principal stresses along

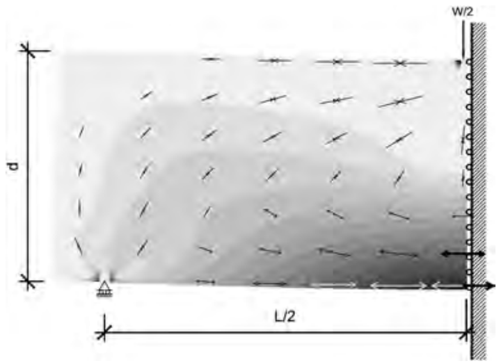

FIG. 14—*Finite element model of ASTM C 99 test using a symmetry boundary condition. Arrows shown indicate direction and magnitude of the principal stresses of greatest magnitude. Shade of gray depicts principal tensile stress magnitude (darker means higher tensile stress).*

the bottom fiber of the ASTM C 880 specimen from mid-span to a distance $L/4$ from mid-span are equal to 1.0 in the finite element model.

In order to assess the effect of friction at the supports, the finite element model discussed above was modified so that the support was restrained against horizontal displacement. Support fixity represents an extreme case of support friction and was modeled to check whether the stone was likely to slip at the support bearing surface due to flexural deformations introduced during the application of vertical test loads. With support fixity in place, the calculated support horizontal reaction was 2.6 times greater than the vertical reaction, indicating an impossibly high stone-to-support static coefficient of friction of 2.6 would be required to preclude slippage and that, in practice, the stone must slip relative to the support.

Using a simple horizontal pull test, the kinetic coefficient of friction was determined between the polished surface of the medium-grained granite being studied and a steel roller support to be approximately 0.14. Using this data, the ASTM C 880 finite element model was again modified to remove the previously applied horizontal restraints at the support and instead apply inward horizontal loads at the support equal to the support vertical reaction times the kinetic coefficient of friction in order to assess the effect of friction-induced axial compression. The maximum principal stress normalized by Eq 7 along the bottom fiber of the stone equaled 0.965 in this model, a reduction from the 1.0 value previously discussed without frictional effects. In other words, the ASTM C 880 test likely overestimates the actual stone flexural strength by approximately 3.6 % due to frictional effects.

ASTM C 99—Modulus of Rupture

The ASTM C 99 [6] testing procedure is used to determine the modulus of rupture of dimension stone. ASTM C 99 testing was not performed for this project because material samples of the appropriate thickness were not available. ASTM C 99 test specimens are 2-1/4 in. (5.72-cm) thick, have a span-to-depth ratio of 3.11 to 1 and are loaded with a single point load at mid-span. The ASTM C 99 standard provides the following formula derived from Eq 8 to determine the modulus of rupture:

$$R = \frac{3Wl}{2bd^2} \tag{9}$$

where W is the breaking load, l is the span length, b is the specimen width, and d is the specimen depth.

The ASTM C 99 test specimen has a single location of maximum stress at mid-span, which is where specimen failures typically occur. Stone, being a natural material, has variable tension capacities at different bottom fiber locations. The ASTM C 99 method effectively tests only the stone at mid-span.

A finite element model using a mid-span symmetry boundary condition is shown in Fig. 14. The maximum bottom fiber stress occurs at mid-span and, when normalized by the modulus of rupture deter-

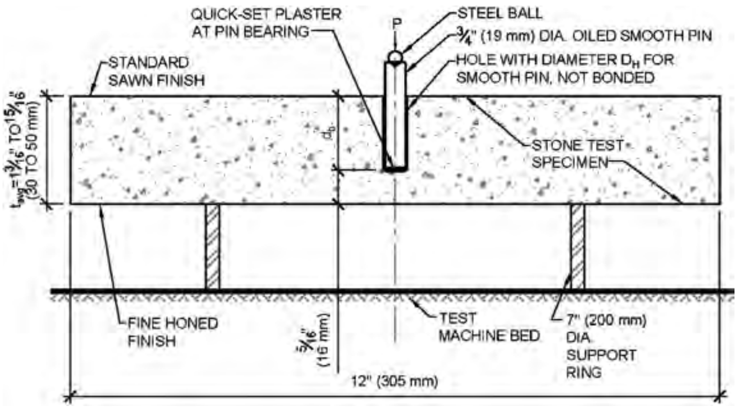

FIG. 15—*Punching shear test configuration.*

mined by Eq 9, a value of 0.944 is found. Based upon this finite element model, deep beam behavior alone causes the modulus of rupture test to overpredict the actual stone flexural strength by approximately 5.9 %.

As with the ASTM C 880 finite element model, the ASTM C 99 model was modified so that the support was restrained against horizontal displacement in order to check the effect of friction at the support. With support fixity in place, the calculated support horizontal reaction was 86 % of the vertical reaction. The static coefficient of friction between stone and the metal support is less than 0.86, indicating that the stone must slip relative to the support.

Applying an inward horizontal force at the support equal to the support vertical reaction times a 0.14 kinetic coefficient of friction, as discussed for the ASTM C 880 finite element model, resulted in a maximum principal stress normalized by Eq 9 at the mid-span bottom fiber of the stone of 0.884 in this model. This represents an approximately 6 % reduction from the 0.944 value previously discussed without frictional effects and indicates the ASTM C 99 test likely overestimates the actual stone flexural strength by approximately 13 %.

The stone-to-support kinetic coefficient of friction could vary from the 0.14 value used for this model depending upon the surface characteristics of the stone and support. This point is significant in that it is important to minimize friction at the support of ASTM C 99 and C 880 tests because frictional forces lead to an overprediction of the specimen flexural capacity due to reduction in the maximum bottom fiber stress.

Historical test data indicate ASTM C 99 modulus of rupture test results typically exceed ASTM C 880 flexural strength test results. Depending on the stone, this increase can be as much as 20 %. This difference can be attributed in part to deep beam behavior, support frictional effects, and the bottom fiber stress distribution. Deep beam behavior in the ASTM C 99 test, as discussed previously, results in a 6 % overestimate of flexural strength, and support frictional effects are more prevalent in the ASTM C 99 test than in the C 880 test. When deep beam behavior and support frictional effects are considered, the ASTM C 99 test procedure overpredicts flexural strength by 13 % whereas the ASTM C 880 procedure overpredicts by 3.6 %. Comparatively, therefore, deep beam and support friction effects readily account for ASTM C 99 test results exceeding ASTM C 880 test results by approximately 9 % (C 99/C 880 = 1.13/1.036 = 1.09).

Additional reported differences between ASTM C 99 and C 880 test results beyond the 9 % attributable to deep beam and support friction effects are likely related to the previously described "searching" by the ASTM C 880 test configuration for the weakest stone location along the constant moment region, a feature the ASTM C 99 single point loading configuration does not provide.

Punching Shear

The punching shear test currently under consideration by ASTM Subcommittee C18.01 was developed to evaluate the strength of stone in a configuration similar to the states of stress present in certain stone

FIG. 16—*Punching shear spall trace digitized in CAD software.*

anchorage designs. The test configuration is shown in Fig. 15. In summary, a 3/4-in. (1.91-cm) diameter oiled smooth pin with a steel ball bearing placed in a 0.76-in. (1.9-cm) to 0.80-in. (2.0-cm) diameter hole is used to apply load to the specimen such that the punching shear failure occurs in approximately two minutes. Upon failure, the spall shape is traced on graph paper and the spall area is determined by digitizing the tracing and determining the projected area using CAD software (Fig. 16). The punching shear strength is then calculated by dividing the spall failure load by the projected spall area (minus the area of the bored hole). An equivalent circular spall diameter, D_E, is determined using the formula:

$$D_E = \sqrt{(4A_G/\pi)} \tag{10}$$

where A_G is the gross projected spall area determined from the spall tracing. An equivalent spall angle relative to the bored hole axis, φ, is determined using the formula:

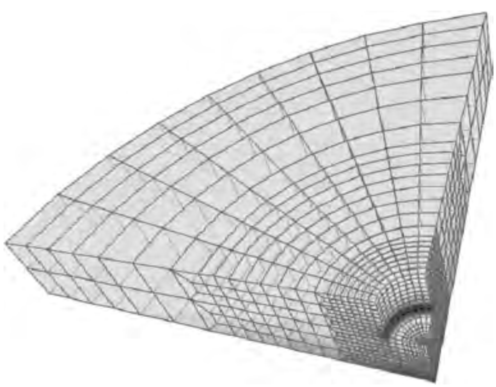

FIG. 17—*Finite element model of punching shear test using solid elements. Shade of gray depicts principal tensile stress magnitude (darker means higher tensile stress).*

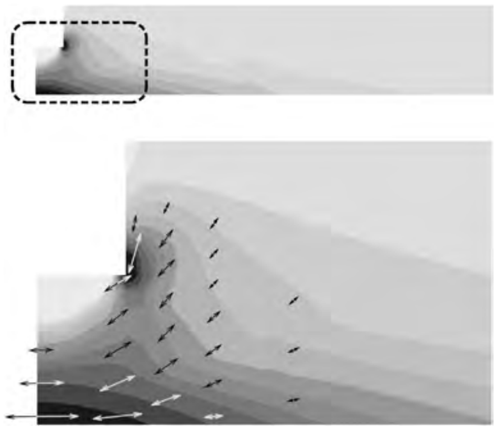

FIG. 18—*Section view of the solid model shown in Fig. 17. Arrows indicate direction of the principal stress of greatest magnitude. Shade of gray depicts principal tensile stress magnitude (darker means higher tensile stress).*

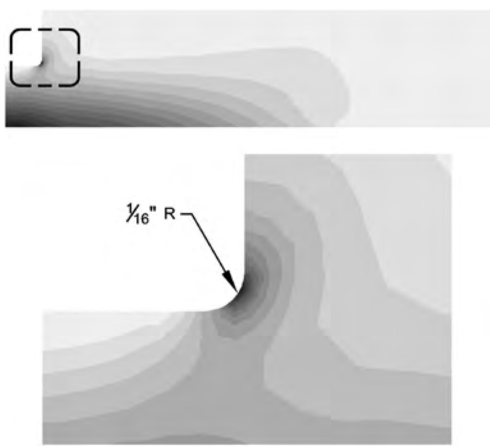

FIG. 19—*Two-dimensional finite element model of punching shear test modeled using a symmetry boundary condition. Shade of gray depicts principal tensile stress magnitude (darker means higher tensile stress).*

$$\phi = \arctan\{(D_E - D_H)/[2 * (t_{avg} - d_h)]\} \tag{11}$$

where D_E is defined in Eq 10, D_H is the diameter of the bored hole, t_{avg} is the average specimen thickness, and d_h is the depth of the bored hole.

Nine tests were performed on the same medium-grained granite used for the other tests discussed herein. The average punching shear strength was 145.1 psi (1000 kPa) with a standard deviation of 17.4 psi (120 kPa), and the average spall angle relative to the hole axis was 70.3 degrees with a standard deviation of 1.3 degrees. The actual spall fractured surface was concave. The slope became shallower moving radially away from the loaded location, similar to the Type 31 test spalls. The bottom surface of some of the spalls exhibited incipient flexural cracking.

A finite element computer model of the punching shear test configuration was generated where a "slice" of the stone was modeled and symmetry boundary conditions were used along two planes (Fig. 17).

TABLE 4—*Comparison of average test "failure stress" values with standard deviation given in parentheses.*

Test Description	Number of Tests	"Failure Stress"	Normalized by Punching Shear
Punching Shear	9	145.1 (17.4) psi 1000 (120) kPa	1.00
Edge Dowel	13	202.8 (32.6) psi 1398 (225) kPa	1.40
Type 31 Anchor	9	121.2 (12.9) psi 836 (89) kPa	0.84

For simplicity, the stone specimen was modeled as circular in plan instead of square, which does not have an appreciable effect on the calculated punching shear stresses. Homogeneous and isotropic material properties were assumed to approximate the stone material behavior.

The model principal stresses (Fig. 18) show a large stress concentration around the perimeter of the bored hole base. The stress concentration was overestimated by the model because the cylindrical hole perimeter-to-base intersection was modeled as a right angle without adequate refinement of the element size and geometry to accurately capture stress concentrations. However, the presence of a stress concentration at this intersection is still expected. The orientation of the principal stresses (Fig. 18) is shown to be around 70 degrees from the horizontal in the vicinity of the stress concentration, consistent with the results of the authors' previously discussed punching shear tests. The orientation of the stress rotates away from the pin axis when moving in the radial direction away from the hole perimeter. Additionally, large tensile stresses are present at the base of the model which corresponds to the region of the stone below the loading point. The magnitude and direction of the tensile principal stresses at the model bottom surface are in agreement with the observed incipient flexural cracking observed on the test specimen spalls.

Two-dimensional models were generated to study the effect of the radius present at the cylindrical hole perimeter where the loading pin bears on the quick-set plaster, as shown in Fig. 15. The maximum principal stresses of a model with a 1/32-in. (0.794-mm) radius were 33 % greater than a model with a 1/16-in. (1.59-mm) radius (Fig. 19). The principal stress differences are based upon linear-elastic models which may overestimate the stress concentration effect. These models indicate that the results of punching shear tests can be affected by fabrication variations, resulting in a different transition radius present at the base of the loading pin hole.

The distribution and orientation of stresses calculated by the finite element models suggest that the punching shear specimen failure is initiated at the hole perimeter and propagates outward to form a spall. With this said, the punching shear strength determined using the presented procedure will underestimate the stone punching shear strength due to the uneven stress distribution. Reducing the radius present at the bottom of the hole will reduce the indicated punching shear strength due to an increased stress concentration effect.

The average failure loads and spall areas determined from the authors' punching shear, edge dowel, and Type 31 back anchor tests were used to prepare Table 4. Ratios of failure load to measured spall area ("failure stress") are shown for each test type. These "failure stresses" represent a fictitious stress state that assumes a uniform distribution of force over the failure surface. As previously discussed, this is not the case due to stress concentrations and the uneven stress distributions that result from the test geometries. The "failure stress" values do, however, provide insight into how significant the uneven stress distributions are in each case. The Type 31 anchor has the lowest "failure stress" value, the punching shear test is in the middle, and the edge dowel test value has the highest. The influence of an uneven stress distribution is, therefore, most pronounced in the Type 31 anchor, the punching shear test is in the middle, and the edge dowel is the least influenced, according to these data.

Conclusions

Determining the configuration, size, and spacing of the anchorages that will affix dimension stone cladding panels to the underlying building substrate requires information regarding material strength, anchorage strength, and knowledge of the stress states created within the cladding. In this paper, the relationships between material strength, anchorage strength, and induced stress states for four common anchorage configurations and three material property tests were examined. The findings are based on a limited quantity of testing performed on a single stone type (a medium-grained granite). Additional tests on a variety of stone types and with varying anchorage configurations need to be performed to see if the presented conclusions apply to other stones and configurations. Additionally, large variabilities frequently present in stone strength test data warrant caution when predicting anchorage strengths, and a statistically significant sample of anchorage tests is recommended to verify adequate strength. Specific conclusions based on the authors' experience and the testing and analytical work described herein include:

- The projected area of the stone failure surface for Type 31 back anchors can be reasonably approximated by a circle, less the area of the intersected installation slot. The radius of the circle, r, can be determined by assuming a failure surface inclined 20 degrees from the stone plane and initiating at the transition on the side of the installation slot.
- The projected area of the stone failure surface for an edge dowel can be reasonably approximated by a circular segment. A failure surface inclined 20 degrees from the stone plane on the stone edge and a 45-degree failure plane initiating at the embedded dowel end can be used to define three points to which a circular segment can be fit. Some modification to the point predicted by the 45-degree failure plane may be necessary depending upon the anchor bearing locations.
- The stress states and general appearance of the stone failure surfaces for edge dowels and Type 31 back anchors are similar to those produced by the proposed punching shear test. Unfortunately, punching shear test values determined using the currently proposed test configuration are fairly sensitive to the shape of the transition present at the interface between the bottom and sides of the bored hole. Variability of indicated punching shear strengths due to test specimen fabrication tolerances may be too great for this test method to provide reliable, reproducible data. Further study of this issue is warranted before the proposed test method is accepted by ASTM. If the specimen fabrication sensitivity issue is overcome, punching shear test results could be used to predict failure loads of edge dowel and Type 31 anchorages if modification factors are introduced that account for the uneven stress distribution present in different configurations. For example, a modification factor of 0.84 (refer to Table 4) could be used to predict the Type 31 anchorage failure load from punching shear test results.
- C 99 test results are inflated by approximately 5.9 % due to deep beam behavior caused by the relatively low test specimen span-to-depth ratio. When deep beam behavior and support friction effects are considered for the ASTM C 99 test, the apparent failure stress is overestimated by 13.1 %. The ASTM C 880 test overestimates the apparent failure stress by 3.6 % due to support friction effects. Overall, deep beam and support friction effects readily account for ASTM C 99 test results exceeding ASTM C 880 test results by approximately 9 %.
- The use of cured sealant to provide a cushion between the anchor and stone bearing surfaces in kerf anchor configurations can significantly reduce peak bearing pressures when compared to dry-set hardware.
- Test results and finite element models suggest that the failure of discrete kerf anchors installed in a continuous kerf is primarily associated with flexural stresses. Stress concentration at the kerf root appears to play a significant role in limiting the failure load; however, the effect of the stress concentration is overestimated by linear-elastic analysis methods. The span-to-depth ratio of the kerf leg converted into an equivalent simply-supported beam (approximately 3.5 to 1 for tests performed by the authors) is closer to the span-to-depth ratio of ASTM C 99 test specimens (3.11 to 1) than ASTM C 880 test specimens (10 to 1). Additionally, the orientation of the principal stresses in the finite element model of the ASTM C 99 test specimen is more similar to the kerf anchorage principal stress orientation than the ASTM C 880 model. However, the influence of kerf root stress concentrations overwhelms any differences present between the ASTM C 99 test and ASTM C 880 test results. Since ASTM C 880 tests are typically required to address the flexural capacity of thin dimension stone cladding, kerf anchorage strength predictions based upon ASTM C 880 test results can reduce the number of tests required to develop material strength data for new designs by eliminating the need to conduct C 99 tests.

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Stiffness Considerations in Dimension Stone Anchorage Design

ABSTRACT: Modern building facade design concepts consider all cladding materials, including dimension stone cladding, as nonstructural elements designed to transmit localized gravity and lateral loads to the primary building structural elements. In practice, the structural backup behind dimension stone cladding can take a wide variety of forms, including cast-in-place concrete shear walls, brick and concrete masonry units, aluminum curtain wall system framing, hot and cold rolled steel subframes, and precast concrete panels and members. The flexural stiffness of these backup systems can vary widely, with cast-in-place concrete shear walls on one end of the stiffness spectrum and aluminum curtain framing on the other. The stiffness of dimension stone cladding relative to the backup system can have a significant effect both on the stresses induced in the cladding and the loads transmitted through the cladding anchors. Likewise, the stiffness of the anchor elements and the backup system can affect the loading and stress distribution within the cladding panels. This paper addresses some of the issues associated with the interaction of dimension stone cladding panels, panel anchors, and metal backup structures, and the effects of relative stiffness on load and stress distributions. Information is presented that was obtained from laboratory tests and analyses for new designs, as well as investigations of dimension stone cladding failures.

KEYWORDS: dimension stone, cladding, design, anchor, anchorages, connections, stiffness

Introduction

The Egyptian pyramids, Roman aqueducts, and ancient Greek temples all stand as testaments to the long-standing popularity and utility of stone as a construction material. These early uses of stone utilized cubic pieces either dry-set or mortared into their final configuration without the use of metal anchors. Stone construction continued in this same basic form for centuries, resulting in castles, cathedrals, and other bearing-wall-based structures that are still in use and admired today.

The introduction of the modern "safe" passenger elevator by Elisha Otis in 1853 ushered in a new era of "high rise" building construction by providing the necessary vertical access to make these structures feasible. Increasing building heights quickly found the practical limits of bearing wall construction as demonstrated by the 6-ft thick masonry walls of Chicago's Monadnock Block constructed between 1891 and 1893.

The use of structural steel to form an internal supporting frame to carry the weight of construction materials and occupants solved the building height limitation inherent with bearing wall construction, allowing true "skyscrapers" to be built to ever increasing heights. With the load carrying duties removed from the walls and now handled by the structural frame, exterior wall designers could limit their focus to keeping the weather from entering and allowing light into building interiors. This allowed the exterior wall materials to be designed as a "cladding" fitted around and attached to the structural frame and enclosing the interior space.

Modern building facade design concepts consider all cladding materials, including dimension stone cladding, as nonstructural elements designed to transmit localized gravity and lateral loads to the primary building structural elements. In practice, the structural backup behind dimension stone cladding can take a wide variety of forms, including cast-in-place concrete shear walls, brick and concrete masonry units, aluminum curtain wall system framing, hot and cold rolled steel subframes, and precast concrete panels and members. The flexural stiffness of these backup systems can vary widely, with cast-in-place concrete

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shear walls on one end of the stiffness spectrum and aluminum curtain framing on the other. The stiffness of dimension stone cladding relative to the backup system can have a significant effect both on the stresses induced in the cladding and the loads transmitted through the cladding anchors. Likewise, the stiffness of the anchor elements and the backup system can affect the loading and stress distribution within the cladding panels.

The Problem

During full panel structural load tests performed to verify the load capacity and safety factors provided by various dimension stone cladding designs, the authors have from time to time witnessed failures at loads significantly lower than predicted by the cladding designer. A rational review of the failure mode frequently reveals evidence that the actual load distribution to the stone anchors did not match the designer's assumptions. Similar failure patterns have also been observed on completed facades, resulting in expensive repairs and, in some cases, costly civil litigation.

Frequently, review of the design calculations for dimension stone cladding systems which have experienced unexpected laboratory or in-service failures has revealed extremely simplistic assumptions regarding the distribution of out-of-plane stone panel loads to the supporting stone anchors. Often, the simplified calculation for anchor load distribution takes the form shown in Eq. 1, for multiple discrete anchors, or Eq. 2 for full-width kerf anchors:

$$\text{Anchor Load} = \text{Total Applied Load/Number of Anchors} \tag{1}$$

$$\text{Anchor Load per Unit Length} = \text{Total Applied Load/Total Kerf Anchor Length} \tag{2}$$

Equations 1 and 2 are both based on the assumption that all anchors, or all portions of a continuous anchor, will equally participate in supporting the attached stone panel. Likewise, both equations neglect the realities of variations in anchor placement, engagement, and stiffness, as well as stone panel and backup structure stiffness and load/deformation behavior. Any plate structure supported at more than three points is, by definition, an indeterminate structure for which support reaction forces cannot be easily calculated without making simplifying assumptions. Increasing the number of anchor points on a stone cladding panel both increases the number of assumptions that must be made and the degree of difficulty to calculate accurate support reactions.

Example A—Eight Discrete Anchor Points

Structural performance tests were conducted for a project with granite cladding panels nominally 4-ft 3-in. wide by 7-ft high and 1-5/8-in. thick (1.30-m wide by 2.13-m high by 4.13-cm thick). Out-of-plane panel support was provided by four pairs of 1/4-in. diameter (0.64 cm) Type 31 slotted back anchors (eight anchors, total) located along two lines as shown in Figs. 1 and 2. Type 40 adjustable aluminum brackets were attached to each pair of Type 31 anchors (four brackets, total). The Type 40 brackets engaged aluminum J-sections attached at each end to a steel frame.

A significant difference between the upper and lower connections was that while the upper J-section spanned the 4-ft-3-in. (1.30-m) distance unaided, the lower J-section had been stiffened by attachment to a 4-1/2-in. high by 3/8-in. thick (11.43-cm high by 0.95-cm thick) steel plate at the ends and at mid-span in order to allow the lower brackets to also carry gravity loads.

The tests were performed in accordance with ASTM C 1201-91 Standard Test Method for Structural Performance of Exterior Dimension Stone Cladding Systems by Uniform Static Air Pressure Difference [1] with target performance levels of three times the facade wind pressure and suction design values. Initial tests indicated adequate performance under wind pressure loads but premature failure when wind suction loads were applied. A consistent pattern emerged in the failures, with one or the other of the upper outboard Type 31 anchors spalling the surrounding stone at loads as low as one-half of the target performance. The initial failure was immediately followed by spalling of the adjacent inboard anchor (Fig. 2). Observations of the interface between the upper J-section and the back of the stone panel while suction loads were applied revealed significant bending of the J-section. While the J-section remained in contact with the middle of the stone, relatively large gaps formed between the stone and J-section near the panel

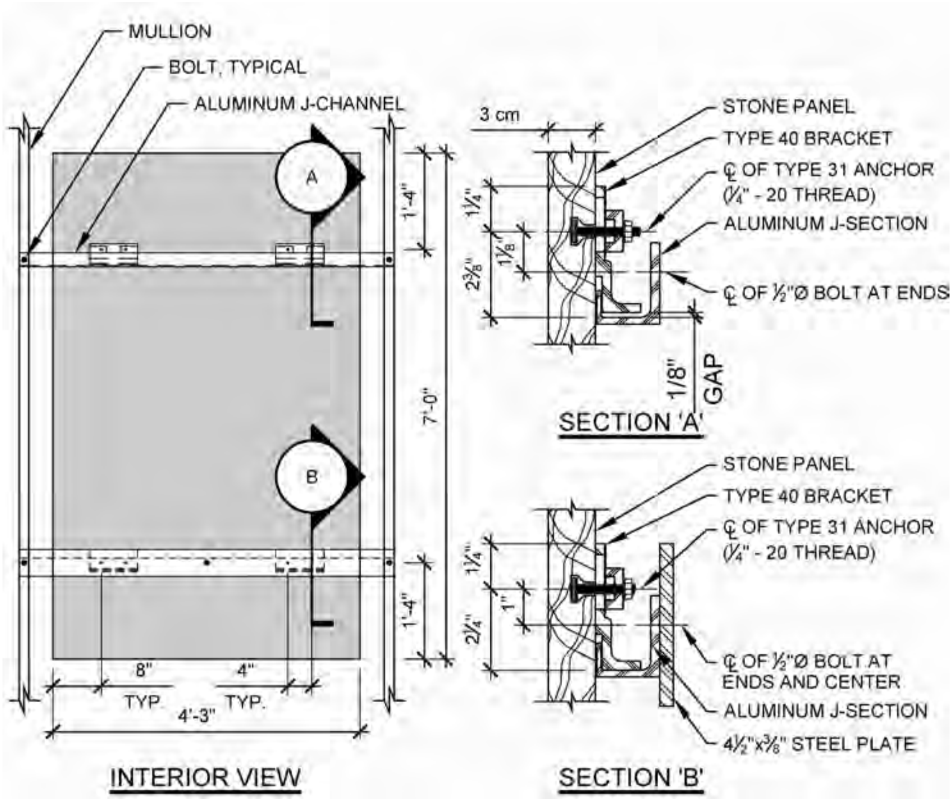

FIG. 1—*Details of panel with Type 31 anchors used in the Example A structural performance tests.*

edges. Post-test examination of the anchor hardware and the aluminum J-section revealed no permanent distortion. Based on the test results and observations, the authors posited that the unstiffened J-section was simply too flexible to span the 4-ft-3-in. (1.30-m) panel cladding width, resulting in uneven load distribution within the upper Type 31 anchor pairs, with greater loads applied to the outboard anchors which were closer to the J-section supports. Stiffening of the upper J-section was recommended.

In order to verify the theory that the relatively low stiffness of the upper, unstiffened J-section was the cause of the premature failure of the upper outboard Type 31 anchors, additional tests were performed with the upper J-section stiffened with the same 4-1/2-in. high by 3/8-in. thick (11.43-cm high by 0.95-cm thick) steel plate used for the lower anchors. This revised arrangement met the project wind pressure and suction test requirements but would have added significant construction cost to install the upper steel plate stiffener. An alternate, and much cheaper, stiffener arrangement using back-to-back J-sections fastened together with self-drilling self-tapping screws was successfully tested and ultimately incorporated into the facade design (Fig. 3).

Example B—Full-width Top and Bottom Kerf Anchors

A far more common dimension stone cladding anchorage configuration than the eight-point arrangement discussed in Example A involves the use of full-width top and bottom kerf anchors. Designs of this type are often used in curtain wall systems incorporating both stone and glass, with the glass in either "punched" or "strip" window openings (Fig. 4). In this type of system, aluminum mullions span vertically between floors. Horizontal extrusions fastened to the mullions are used to support the weight of the glass

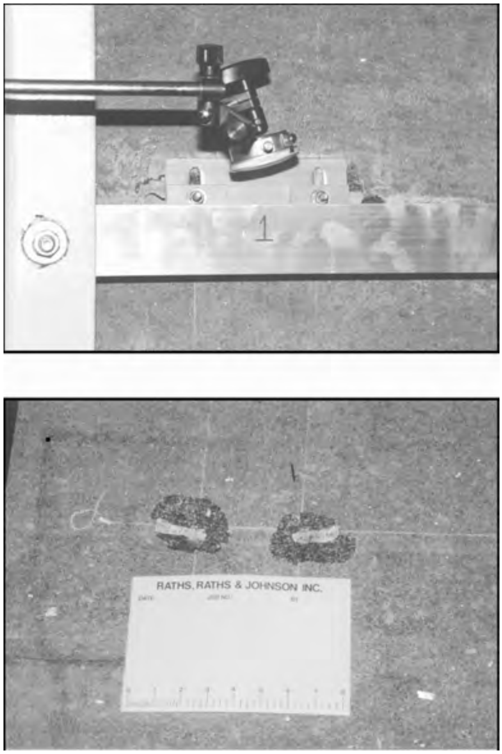

FIG. 2—*Spalling located at Type 31 anchor under negative wind pressures.*

FIG. 3—*View of aluminum J-section used to stiffen the upper J-section.*

FIG. 4—*Curtain wall system that incorporates stone and glass.*

and stone cladding pieces. Retention of the glass may be achieved through a variety of means, including mechanically attached exterior pressure bars, removable interior glazing stops, and structural glazing. The dimension stone panels, however, are typically restrained against out-of-plane loads from wind and seismic forces through the use of full-width "J" or "T"-shaped anchors engaging slots, known as "kerfs," cut into the top and bottom edges of the stone (Fig. 5).

The authors have observed both laboratory and in-service kerf failures of curtain wall systems with full-width aluminum extrusion kerf hardware. When they occur, these failures typically manifest themselves in the form of multiple short kerf breaks starting at the outer panel edges and proceeding inward toward the panel center (Fig. 6). In reviewing the underlying cause of this type of failure, commonly called an "unzipping" failure, lessons can be learned from Example A. The stone cladding panel in question can be thought of as a plate supported along two edges, but unlike the simple assumption in Eq. 2 not all portions of the supporting aluminum extrusion provide the same amount of restraint. This is because the aluminum kerf extrusion is free to deflect between the support points provided by the aluminum curtain wall mullions. The amount of deflection under a particular load level, and therefore the amount of restraint provided by a particular segment of the kerf extrusion, is dependent on many factors, including: the moment of inertia provided by the kerf extrusion cross-section; the length of the kerf extrusion pieces relative to the curtain wall mullion spacing; the configuration of the connection between the kerf extrusion and the curtain wall mullions; the cladding panel width and thickness; the flexural modulus of elasticity of the dimension stone used for the cladding panel; and the stiffness of any kerf filler material present. The many possible combinations of these factors virtually guarantee that the degree of support provided along the length of a full-width kerf extrusion will not be uniform, will be the least at mid-width, and will be greatest at the curtain wall mullions. The magnitudes of the maximum and minimum kerf loads relative to an average load, as determined by Eq. 2, and the strength of the stone are necessary to determine the likelihood of kerf failure for a particular design.

A More Refined Analysis

Recently, the authors were afforded the opportunity to provide a peer review of the stone cladding design for a new high rise with granite cladding panels supported by full-width aluminum kerf extrusion hard-

FIG. 5—*Edge view of full-width kerf anchors.*

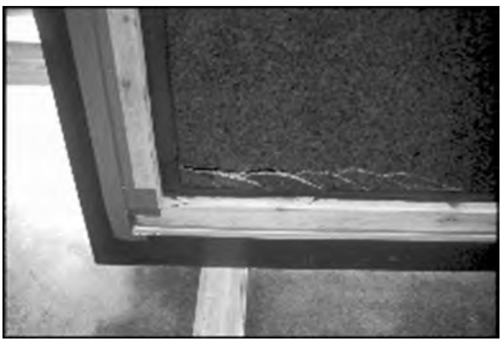

FIG. 6—*"Unzipping" failure at a panel edge.*

ware. A battery of tests for standard material strength parameters had already been performed, as had instrumented full-panel uniform load tests conducted according to the methods of ASTM C 1201-91 [1]. The full panel load tests successfully resisted differential pressure proof loadings of ±4 times the 22 psf (1053 kPa) design wind load for that portion of the building, despite having been designed using the approach presented in Eq. 2. Since the full-panel load tests had been performed on dry granite pieces representative of the typical cladding module, and not the worst case combination of panel size, design wind load, and wet conditioning, one of the questions posed to the authors was whether the test results could be extrapolated to represent the likely performance of all of the cladding panels on the building. The authors responded that the variations from the worst case combinations and the use of proof loading instead of loading to failure made extrapolation of the test data unreliable as a means for assessing other panels. Additional tests representing worst case combinations were recommended.

As is the case in many construction projects, the project schedule could not accommodate the lead time necessary to fabricate and test additional cladding panels, and a different approach was needed. In response, the authors undertook a series of finite element computer analyses to simulate the untested conditions. The general approach used was to develop a computer model whose load/deflection behavior would reasonably predict the behavior of the load-tested cladding panels. The analytical parameters developed for this model would then be used to develop additional models for other conditions.

Model Definition

The instrumented full-panel load tests had been conducted using granite panels 47-in. wide, 47-1/2-in. high, and 3-cm thick (1.19-m wide by 1.21-m high by 1.18-cm thick) with full-width kerfs at the top and bottom. The kerf cuts had been centered within the stone thickness and were 1/4-in. wide by 11/16-in.

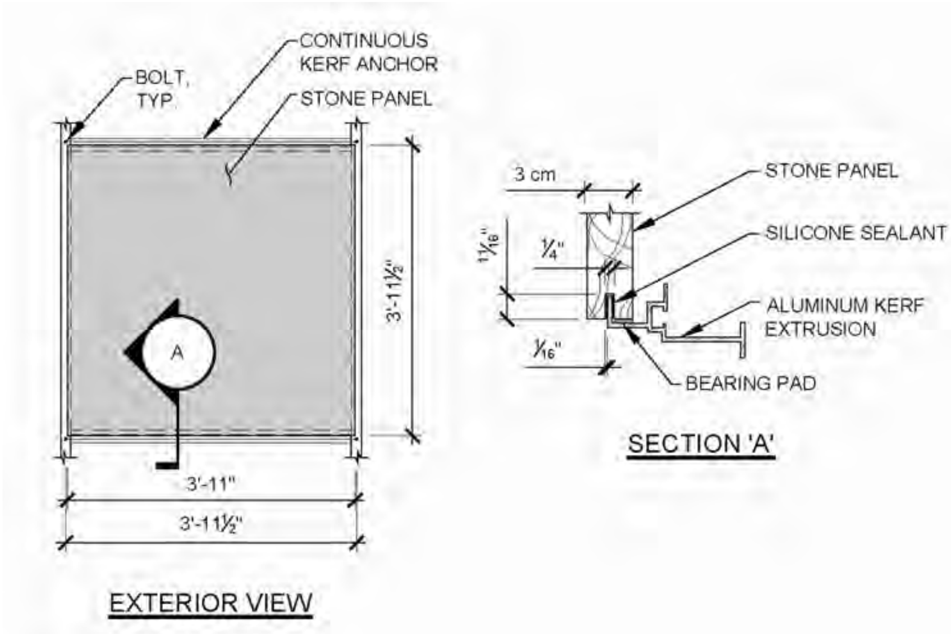

FIG. 7—*View of stone panel and construction used in full-panel load tests.*

deep (0.64-cm wide by 1.75-cm deep). The full-width aluminum kerf extrusions were simply supported at each end by a single bolt into vertical mullions spaced 47-1/2 in. (1.21 m) center-to-center (Fig. 7) and the stone kerfs had been filled with silicone sealant prior to inserting the kerf extrusions.

The computer model was developed using SAP 2000 finite element analysis software following the general approach described by Hoigard et al. [2] to simulate the tested cladding panels. Plate elements were utilized to represent the granite panel. A flexural modulus of elasticity of 4 687 000 psi (32.32 GPa), derived from the test data, was used in modeling the granite. Kerf extrusion dimensions and material and section properties were obtained from the curtain wall designer and translated into beam elements within the computer model with a moment of inertia of 1.310 in.^{4} (54.69 cm^4) and a 10 100 000 psi (69.64 GPa) modulus of elasticity. The beam elements were positioned so their nodes would be aligned with, but offset 1/16 in. (0.16 cm) from the plate element nodes along the top and bottom edges of the granite panel (Fig. 8). The interface of the kerf hardware with the sealant-filled stone kerf was modeled as a 1/16-in. thick (0.16-cm thick) strip of silicone represented by spring elements whose stiffness was determined based on tributary area and a confined compressive modulus of elasticity of 454 psi (3130 kPa) provided by the sealant manufacturer.

Because the kerf extrusions in the full-panel load tests had been installed as simply supported lengths with single bolts at each end, the authors initially assigned no end fixity to these elements. Deflections for this configuration exceeded those measured during the load tests, causing the boundary conditions to be reconsidered. Rerunning the model with full end fixity resulted in deflections that were too small. Ultimately, the kerf extrusion-to-mullion connections were modeled with 20% fixity, resulting in calculated stone panel mid-span and support deflections reasonably close to those measured during the full-panel load tests.

Test Panel Kerf Loads and Stresses

Once the load/deflection behavior of the computer model was shown to reasonably simulate the deflections measured during the instrumented full-panel load tests, a uniform load of 22 psf (1053 kPa) was applied to the plate elements representing the granite panel. The spring elements representing the silicone kerf

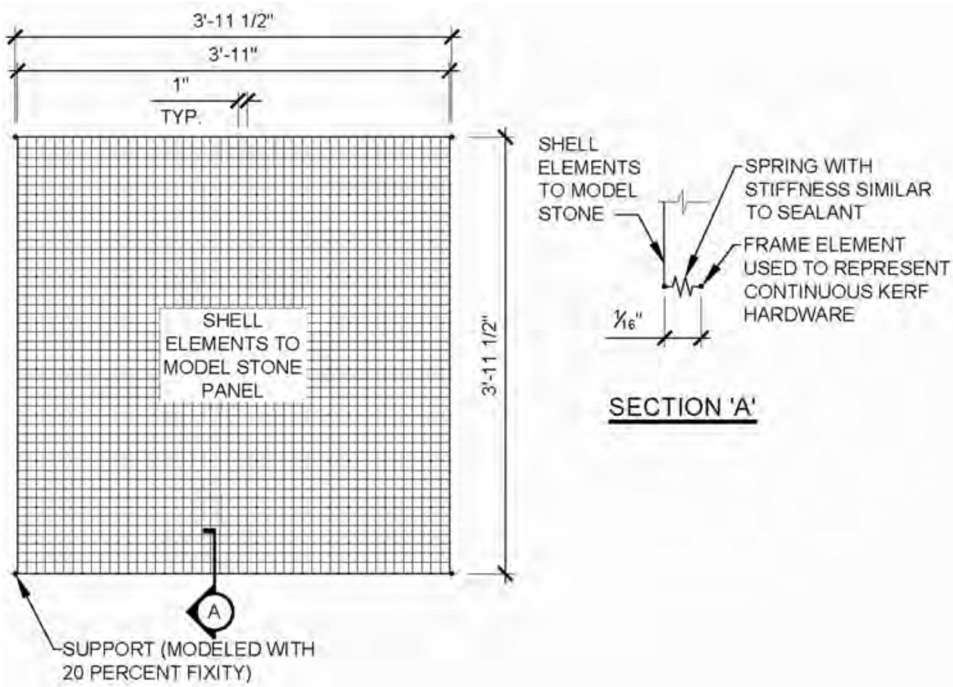

FIG. 8—*View of SAP2000 computer model used to simulate cladding panels.*

filler were then polled to assess the magnitude of the kerf contact forces generated along the length of the aluminum kerf extrusion. As shown in Fig. 9, the contact forces along the length of the kerf extrusion were not uniform, with maximum and minimum kerf loads of 13.64 and 1.29 lb/in., respectively (23.89 and 2.26 N/cm), 376 and 36% of the average value of 3.63 lb/in. (6.36 N/cm) determined by Eq. 2.

In order to assess the likelihood of kerf failure, the remaining stone flanking the kerf slot was analyzed as a short cantilever. Based on the straight insertion leg of the kerf extrusion and the presence of the silicone sealant filling the remaining kerf space, an inverted triangular distribution of the kerf contact load was assumed to be present at each point along the kerf length. Nominal bending stresses computed for this configuration were further increased by 45% based upon the work presented by Stecich et al. [3] in order to account for the stress concentration associated with the relatively sharp corner (1/32 in. = 0.79 mm radius) found at the bottom of the saw-cut kerf and the small span-to-depth ratio of the cantilever. Using this approach, the maximum flexural tension stress applied to the stone kerf by a 22 psf (1053 kPa) wind load was calculated to be 274 psi (1889 kPa). This stress level compared well to the reported 1310 psi (9032 kPa) dry flexural strength value determined for the project granite and the project-required 4.0 safety factor for stone anchorage design where the calculated safety factor = 1310 psi/274 psi (9032 kPa/1889 kPa) = 4.78.

Other Cladding Panel/Wind Load Combinations

Using the computer modeling parameters and kerf analysis approach developed for the load tested cladding panel, the authors analyzed four granite cladding panel sizes located in areas of the building with a higher predicted maximum wind load of 34 psf (1628 kPa). These panels were all fabricated from the same granite and supported in the same manner as the load-tested panels, and measured 47-1/2-in. high by 14, 29, 47, and 59-in. wide (1.21-m high by 0.36, 0.74, 1.19, and 1.50-m wide). As shown in Fig. 10 and Table 1, increasing the cladding panel width while maintaining a constant kerf extrusion moment of inertia resulted in increasingly nonuniform kerf contact forces, significant increases in maximum kerf load levels,

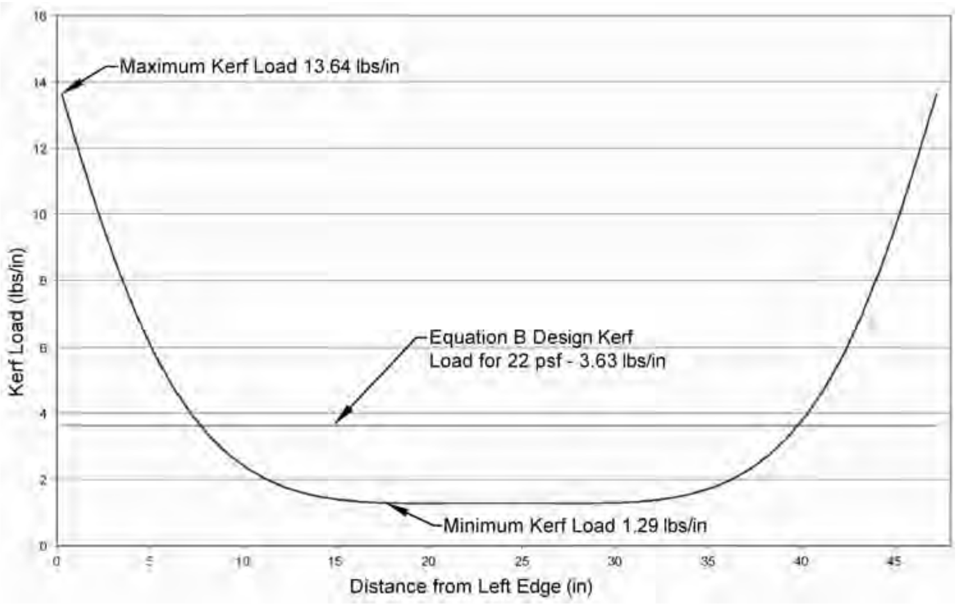

FIG. 9—*Kerf loads obtained from SAP2000 model along a 47-in. wide panel with a uniform load of 22 psf*.

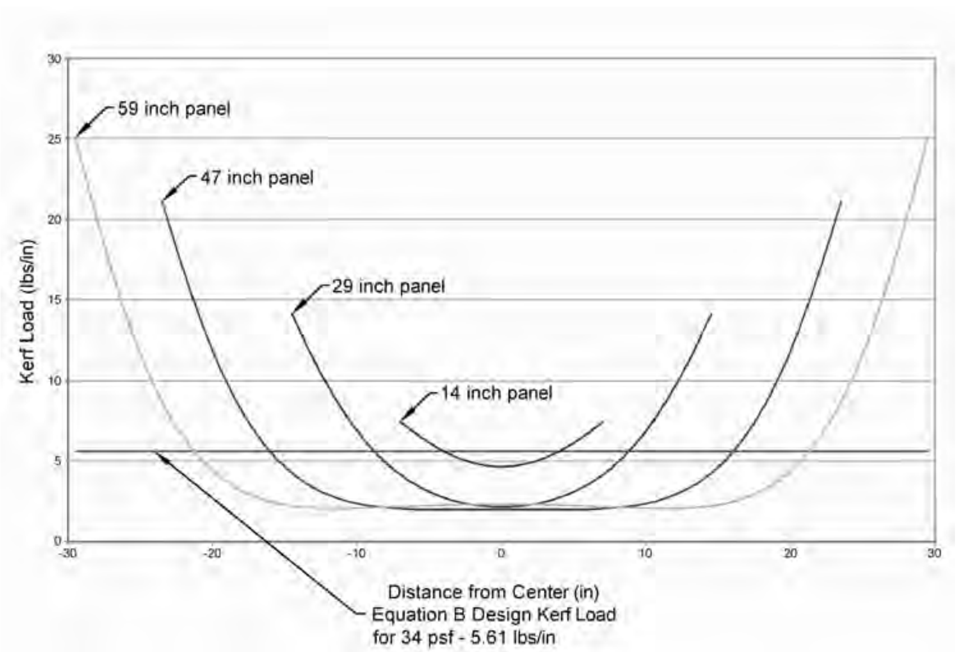

FIG. 10—*Kerf loads obtained from SAP2000 for various panel widths.*

TABLE 1—*Comparison of maximum stone kerf stresses for 34 psf wind load.*

Width in. (m)	Equation B Kerf Load lb/in. (N/cm)	Max. Kerf Load lb/in. (N/cm)	Max./ Eq 2 (percent)	Max. Kerf Stress psi (kPa)	Safety Factor
14 (0.36)	5.61 (9.82)	7.42 (12)	132	149 (1027)	8.8
29 (0.74)	5.61 (9.82)	14.08 (24.66)	251	283 (1951)	4.6
47 (1.19)	5.61 (9.82)	21.08 (36.92)	376	423 (2916)	3.1
59 (1.50)	5.61 (9.82)	25.08 (43.92)	447	503 (3468)	2.6

and reduced safety factors. Although beyond the scope of this paper, consideration of additional factors, including cladding panel fabrication tolerances and wet conditioning stone strength reduction, further reduced the available kerf safety factor sufficiently to warrant adding tube stiffeners to the 47 and 59-in. wide panel kerf extrusions.

Kerf Extrusion Stiffness

Brick masonry veneer design is based on the assumptions that the brick carries no gravity load except for its own weight and shares out-of-plane loads, such as those from wind and seismic events, with back-up structural elements, which are frequently metal studs. The design of the metal studs is based on the further assumption that limiting the metal stud mid-span deflection under service loads, neglecting any load-sharing contribution from the brick masonry, to a value equal to the span length divided by 600 ($L/600$) will provide an acceptable support structure for the brick masonry. An underlying premise of this approach is that cracks in the brick masonry up to 0.015 in. (0.04-mm) wide are acceptable [4].

Some designers have advocated adapting the brick masonry/metal stud design approach for use with dimension stone cladding, suggesting the minimum stiffness of anchor hardware and backup support elements be defined as a fixed ratio of span length (L/X). In this approach, as with brick masonry/metal stud walls, deflections of the stone cladding support system are calculated assuming no flexural stiffness contribution from the stone. Various values for X have been suggested, with 1000 being the most common.

Using the computer model developed for the previously described 47-1/2-in. high, 47-in. wide (1.21-m high by 1.19-m wide) granite panel with full-width top and bottom kerf anchors, and an applied wind load of 34 psf (1628 kPa), the authors varied the moment of inertia of the beam elements representing the kerf extrusion to study the validity of the L/X design approach. Starting with the existing model, with a kerf extrusion moment of inertia of 1.310 in.^{4} (54.69 cm^4), the maximum extrusion deflection under the applied 34 psf (1628 kPa) load was calculated to be 0.024 in. (0.61 mm) after removing the plate elements representing the stone from the model. This deflection, when compared to the kerf extrusion span of 47-1/2 in. (1.21 m), can be expressed as $L/1978$. As discussed earlier, and shown in Fig. 10 and Table 1, this configuration resulted in significant nonuniformity of the kerf-to-extrusion contact force, with the maximum kerf load of 21.08 lb/in. (36.92 N/cm) being 376% of the 5.61 lb/in. (9.82 N/cm) value calculated using Eq. 2. Additional computer models were configured to simulate the behavior of kerf extrusions sized for maximum extrusion-only deflections ranging from $L/1000$ to $L/10{,}000$. The calculated kerf contact loads for these models are shown graphically in Fig. 11 and summarized in Table 2. Of note is the rapidly decreasing benefit derived from increasing the kerf extrusion moment of inertia. While increasing the moment of inertia from an $L/1000$ to an $L/2000$ configuration reduced the maximum kerf load by 5.98 lb/in. (10.47 N/cm) (approximately 22%), an increase from $L/2000$ to more than $L/5000$ is necessary to achieve a similar kerf load reduction.

Conclusions

The authors have observed field and laboratory dimension stone cladding distress and failures attributable to simplistic assumptions made by the cladding designer regarding the distribution of anchor loads. Failure to account for the stiffness of dimension stone cladding panels relative to anchor and backup structure stiffness can result in stone loads and stresses significantly higher than anticipated, reducing the intended design safety factors. Specific findings based on the authors' experience and the analytical work described herein include:

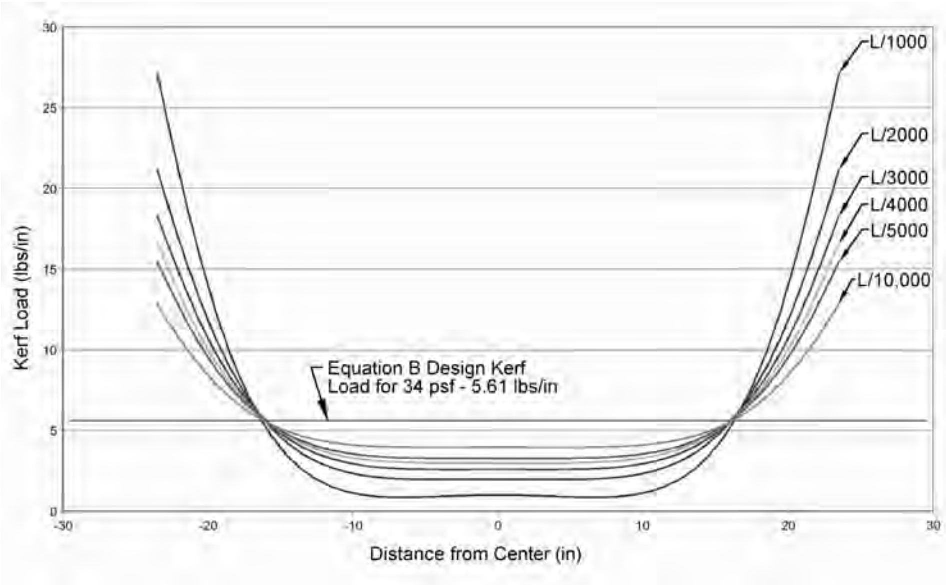

FIG. 11—*Kerf loads obtained from SAP2000 model along a 47-in. wide panel with varying kerf extrusion stiffnesses.*

- With the exception of four bi-symmetrically placed anchors, the use of more than three discrete anchors per piece of dimension stone cladding can easily create circumstances where the load carried by each anchor is not equal to the total load divided by the number of anchors present (Eq. 1), with some anchors having significantly higher loads than others. Differences between the actual anchor load distribution and the average load determined by Eq. 1 can be large, result in significant reductions in the apparent anchor safety factor, and result in unanticipated spalling failures at the stone anchors. The project described in Example A was one such case.
- Kerf loads within dimension stone cladding panels utilizing full-width kerf extrusions for out-of-plane support are not uniform. Instead, kerf contact forces are greatest near the kerf extrusion support points and least in between. The maximum kerf contact forces are typically significantly greater than the average load determined by Eq. 2, so much so that the difference can readily exceed the apparent anchor safety factor.
- The actual distribution of kerf loads in dimension stone cladding panels utilizing full-width kerf extrusions is dependent on many factors, including: the moment of inertia provided by the kerf extrusion cross section; the length of the kerf extrusion pieces relative to the curtain wall mullion spacing; the configuration of the connection between the kerf extrusion and the curtain wall mullions; the cladding panel width and thickness; the flexural modulus of elasticity of the dimension stone used for the cladding panel; and the stiffness of any kerf filler material present.

TABLE 2—*Comparison of maximum kerf loads for various kerf extrusions 47-in. wide panel, 34 psf load.*

Extrusion Deflection (in.)	Extrusion in.^{4} (cm^4)	Equation 2 Kerf Load lb/in. (N/cm)	Max. Kerf Load lb/in. (N/cm)	Max./Eq 2 (percent)
$L/1000$	0.651 (27.097)	5.61 (9.82)	27.1 (47.46)	483
$L/2000$	1.302 (54.193)	5.61 (9.82)	21.12 (36.99)	377
$L/3000$	1.952 (81.248)	5.61 (9.82)	18.28 (32.01)	326
$L/4000$	2.603 (108.345)	5.61 (9.82)	16.58 (29.04)	296
$L/5000$	3.254 (135.442)	5.61 (9.82)	15.44 (27.04)	275
$L/10\,000$	6.508 (270.883)	5.61 (9.82)	12.84 (22.49)	229

- Increasing the stiffness of full-width kerf extrusions can reduce the maximum kerf contact loads in dimension stone cladding panels. However, the relative benefit derived from increasing the kerf extrusion moment of inertia rapidly decreases with increasing stiffness. Even in cases with the kerf extrusion deflection limited to $L/10\,000$, the peak load near the supports can still be more than twice the average kerf load determined by Eq. 2.

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Available online at www.astm.org

Michael D. Lewis[1]

Status on Development of Code Requirements for Exterior Stone Cladding

ABSTRACT: Existing building codes legislated to set minimum standards for construction state dangerously little about stone used to clad buildings. Without standards, failures have proliferated from the use of new materials in unproven applications and familiar materials in new systems. In response, the American National Standards Institute called upon the industry to develop standards that could be adopted into code. Past reliance on judgment without adequate experience must be replaced by uniform standards that include fundamental principles for the design and installation of contemporary systems to protect the public. To improve responsible use of stone as cladding, building codes must address differences from other veneers. Masonry veneers are more homogenous in production, erection, and behavior than stone, yet stone is presently governed by the same provisions of the building code. The in-progress Code Requirements will include parts of the present ASTM C1242 Standard Guide for Design and ASTM C1528 Standard Guide for Selection. The contents will address minimum material properties by stone type, engineering evaluation, attachment types, safety factors, joint design, and weather barrier integration. This is the status of the document's development, which upon completion, is intended to be incorporated into the building code. ASTM Committee C18 on Dimension Stone has compiled the industry practice on behalf of producers, engineers, architects, contractors, and owners for over 80 years. While its standard test methods, material specifications, and guides are invoked by many projects' construction documents, the documented knowledge is not currently incorporated into any of the model building codes, including the International Building Code. To develop the new standard Code Requirements for ANSI, professionals representing all interests of the stone building process are compiling the fundamental aspects of recommended stone cladding practice that should be mandatory.

KEYWORDS: building code, stone, cladding, ANSI, attachment, safety factor, ASTM C18

Introduction

Several years ago, the American National Standards Institute called the industry to develop "Standard Requirements for Stone Cladding." The Institute and the entities relying on ANSI standards, like building codes that incorporate ANSI standards by reference, recognized the dearth of formal mandatory instructions specific to modern methods of stone use on building exteriors. The current code addresses stone as masonry in mortar-set applications, not as thin cladding on backup framing with soft joints, the more common application of stone in contemporary construction. Proliferating diversity of cladding applications, greater global sources of stone materials, and the trend for design professionals to delegate design responsibility to the constructors add more urgency to the need for "Code Requirements" and to present them for formal adoption into a building code.

ASTM Committee C18 on Dimension Stone responded to the call because its membership represented all facets of the stone construction industry. C18 members represent the broadest expertise in the stone industry available in any active consensus organization. Material producers, system installers, architects, engineers, consultants, and national specifications authors comprise the professionals that have promulgated ASTM standards used all over the world in the stone industry for more than a half-century. Also, its body of standard specifications, test methods, and guides for recommended practice are already widely recognized and frequently incorporated into many significant project construction documents. Current ASTM documents establish the basis of sound judgment set by the consensus of experts in stone cladding, some international. But in contrast to existing ASTM C18 work which is voluntary, "Standard Require-

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ments for Stone Cladding" would not be voluntary; it would be mandatory. Upon its adoption by ANSI, the document could also become the basis of stone construction in model building codes. To transition from current voluntary content to mandatory practice, prescriptive language needs to replace the wider-based suggestions and recommendations in each Standard Guide, and issues not critical to minimum safety need to be deleted. Committee members' knowledge of ever-increasing frequency of failures and problems further emphasizes the need for "Code Requirements." Members are using their forensic experience to segregate which aspects of each guide are critical to keep as "Code Requirements," and eventually become available to be adopted into a model building code.

Recognized Sources for Starting the New Document "Code Requirements"

Two existing ASTM C18 standards contain the content that needs to be converted from recommendations to "Code Requirements." The first is ASTM C1242 Standard Guide for the Selection, Design and Installation of Stone Attachments (Design Guide), and the second is ASTM C1528 Standard Guide for Selection of Dimension Stone for Exterior Use (Selection Guide). Because ASTM C1242 contained the engineering content most critical to minimum safety requirements for "Code Requirements," Subcommittee C18.06 on Attachment Components and Systems began the first draft of the new document by editing C1242. After three ballot cycles, the content of the new "Code Requirements" includes the following technical sections: scope, references, documentation standards, engineering evaluation (including minimum thickness, material quality and testing), safety factors, attachment types, joints, and drainage. Key elements of ASTM C1528 Selection Guide will also need to be added later once those elements are determined. The following describes the document in its present state and the chairman's interpretation of the subcommittee towards future refinements, considering ballot responses and meeting discussions.

Scope of "Code Requirements"

Presently entitled "Building Code Requirements for Stone Cladding," the proposed document intends to prescribe minimum requirements for natural stone used as cladding. The requirements apply to the three generic components of the cladding system: stone material, attachments, and support. Involvement of a qualified professional experienced with stone and its support systems is required. Definition of "professional" is being debated. Exemplars, meaning existing construction having similar features and systems, must be evaluated when they exist, and compatibility with all other interfacing systems must be proven. For example, integrity of other adjacent cladding systems, insulation, flashing, and the weather barrier must be maintained; otherwise durability of support would be compromised. And most critically, the stone system must be able to accommodate movements of the structural frame without altering planned load paths.

Engineering, Safety Factors, and Testing

An engineering evaluation of a stone cladding system ideally begins with an objective comparison of the intended application to existing structures. Stone material, attachment, and support should have a previous successful performance in similar applications. All applications require evaluation by a licensed engineer or architect substantiating that they should perform successfully in their planned application if constructed as designed. The evaluations must address movement, adjustability, changes in properties over time, exposure to environmental effects, and integration with building envelope weather barrier, flashing, and perimeter fire containment systems. Durability is best evaluated by examining the performance of existing structures; however, unprecedented materials and systems require additional analysis that may include special testing for weatherability. The present draft has yet to resolve these thresholds.

The second step of the engineering evaluation must derive loads and analyze capacity to resist them. As with other cladding systems, wind and seismic effects must be calculated as lateral loads with self-weight, which influences seismic effects and comprises gravity load. Use conventional structural analysis techniques with physical testing to check capacity, making sure the techniques match expected behavior. Unplanned distress and sometimes failure can result from over-simplified analysis, misunderstanding panel and system behavior, or confusing their interaction. Avoiding such a failure requires insight of an experienced professional early in the engineering evaluation process.

Determining adequacy of a stone cladding system has traditionally involved comparison of probable capacity to probable loads. The margin between capacity and loads, called a safety factor, has historically been dictated primarily by the stone material type. However, professionals experienced with investigating contemporary stone system failures recognized stone type is less influential in causing failures than anchors, support, or differential movement. The natural stone material is not always the most variable component in the cladding system. Nonetheless, the design and construction community recognizes different safety factors for different geologic stone types. The proposed "Code Requirements" cite values already approved in the ASTM C1242 Design Guide as minimum safety factors. To reflect our industry's experience with system failures, an introductory provision is added to increase safety factors by 1.5 times if a new material or unproven system is involved in the project.

Testing may be the most misused aspect of the stone cladding engineering process. Material requires testing to not only learn a material's average strength, but also the variability of the material currently being quarried. Responsibly assessing stone material reliability requires knowing strength and variability, plus durability. Examination of existing construction could address durability for most applications where precedents exist. Once material properties are evaluated, the most critical part of cladding system performance, attachment integrity, must be established. Tests of attachments are required to learn those capacities, their variability, and mode of failure. Engagement of an anchor into a stone does not automatically constitute effective support, and contact between the stone panel and anchor does not always occur where expected. While philosophies regarding testing programs vary widely among experienced stone professionals, those philosophies almost unanimously believe some type of material and attachment tests should be a part of every project.

Design and Installation Documentation

Like other building systems, stone cladding systems require thorough documentation and assessment to assure dependable performance. Unfortunately, few projects are properly documented, and some result in failures. Lack of a designer's knowledge should never be an excuse to omit system concepts or details from construction documents. If the professional responsible for the building design delegates development of cladding system concepts to the constructor, the process could jeopardize public safety. Such a process does not always provide proper engineering evaluation. A process involving architects that prescribe performance requirements to assure conformance that are not easily comprehended and interpreted fails to provide reliable long-term safety. Stone cladding is unlike any other building system, and requires specialized expertise within specific professional disciplines to provide appropriate performance thresholds. "Code Requirements" will require them to be documented. For instance, the structural engineer under the architect's direction designs beams, columns, and floors; the mechanical engineer under the architect's direction designs mechanical equipment, ductwork, controls, and wiring. In contrast, stone cladding design is too frequently delegated to the mason, which is not prudent when the project is not design-build.

At a minimum, documentation standards require concepts to be developed by the licensed architect or engineer as part of the design documents. The concept must include pertinent aspects of the system that resolve load path and movement compatibility by showing attachments between stone, support, and structure. Then, once the constructing entity adopts the concept, full detailing can occur on fabrication and installation drawings according to the engineering process and prescribed thresholds. If the initially-presented concept is not adopted by the constructor, the concept at least represents the characteristics required to function properly. Then, to close the engineering cycle with the licensed professional, the analysis substantiating the full detailing must be received, reviewed, and approved before being installed.

Relating to installation and constructability, designed systems must be built to accommodate tolerances in interfacing work, and those tolerances must be documented. Use tolerances recognized by the interfacing trade, do not impose limits that change established practice. Anchor material compatibility and engagement into the stone must also be documented. For instance, anchors must be metal, noncorrosive long-term, and must mechanically engage the cladding and support. Backup support must be protected from moisture and deleterious effects long-term. If using an adhesive instead of anchors, applications are extremely limited due to inherent workmanship consistency problems with the systems and lack of precedent long-term good performance in many climates.

Material Quality and Minimum Thickness

Quality of stone material must meet objectively-assessed properties established in ASTM material specifications to establish that the material's variable properties are sufficient for its intended exterior use. Different specifications exist for individual geologic stone types. For decades, by their incorporation into project specifications, material specifications have voluntarily set minimum material properties. In addition to material property tests, the system must also be checked by engineering analysis, and where available, by evaluation of similar example construction.

Minimum thickness must also be controlled so fabrication of the stone does not compromise panel capacity. The present draft states that when used as cladding panels, granites, slates, dense limestones, and Group A marbles must be at least 1 1/4-in. thick. Quartz-based stones and medium-density limestones must be at least 2-in. thick.

Attachment Types

Many types of attachments have evolved for contemporary stone systems that will successfully connect stone cladding to its backup. "Code Requirements" expects to list basic requirements that should apply to all projects. Some general requirements include the following: within the sequence of assembly, only use anchors that can be visually seen to engage the stone during installation. While best practice recommends cladding panels should be individually supported, panels can also be stacked if done carefully, particularly thicker panels. Heights of stacked panels are limited. "Code Requirements" generally describes four different anchor types with specific characteristics for each one: precast, edge, bearing, and back. Different devices can work in each of those applications.

Joints and Drainage

This section in the draft addresses elements in the system not directly responsible for stone cladding support, but frequently contribute to support failures if not done properly. These include elements involved with joint design, internal moisture management, and fire containment. Movement joints in a system that intend to prevent stacking must be open or filled with a soft material. Water penetrating a cladding must be redirected outside to protect support. Contemporary walls usually include a cavity behind the cladding to help keep water off the interior wall, thus bridging across the cavity should be minimized. However, flashings, firesafing, support, compartment dividers, and other elements bridging the cavity must be designed and built to prevent deleterious effects.

Progress

Based on ballot activity and improvements incorporated since the first draft in 2004, it is possible that "Code Requirements" could be approved by ASTM Committee C18 with its design content sometime in 2008. At that point the ASTM document will be presented to ANSI for their adoption, and presented to the model building code bodies for adoption consideration. Until then, present design content will be refined before adding selection content from ASTM C1528 Selection Guide. Specific limits on applications included in C1528 will then be deciphered and added where appropriate.

SECTION III: EVALUATION AND INVESTIGATION

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Investigation of Masonry Failure of a Granite and Limestone Clad Historic Church in Eastern Pennsylvania

ABSTRACT: The use of dimension stone was commonplace in turn of the century churches and St. George's in Shenandoah, PA, is an example of granite and limestone construction set in the anthracite belt of Eastern Pennsylvania. The structure's two towers, over 100 feet tall, have recently succumbed to age, the elements, and poor long-term repairs and maintenance that veiled shifting limestone and granite. Early detection did not occur due to the lack of periodic inspection following ASTM's Standard E 2270 "Standard Practice for Periodic Inspection of Building Facades for Unsafe Conditions," and the oldest Lithuanian Catholic Church in the United States has temporarily closed. Field observations have revealed a variety of failures. Limestone and the front entrance has outwardly displaced from the mass masonry brick backup nearly 1 in. at several locations. Granite masonry at the top of the towers has outwardly displaced 3/4 in. and has caused adjacent decorative limestone elements to displace up to 2 in. away from the wall. Spalled fragments of limestone of different sizes have fallen to the ground from varying heights at the rate of nearly one piece per month during recent years. A condition assessment examining the masonry deficiencies has been completed and the building now faces substantial repair options that may impact the building's operational viability.

KEYWORDS: granite, failure, masonry failure, church masonry, limestone, investigation, improper maintenance, inadequate repairs, lack of inspection

Introduction

Many stone façade buildings face similar life cycle challenges that include minimal preventive maintenance, limited long-term repairs, and infrequent inspections. This essentially limits intervention to only times when substantial problems arise. A good case study is St. George's Catholic Church, which has now approached that critical threshold.

Saint George's Catholic Church (see Fig. 1) is located in Shenandoah, Pennsylvania, and its cornerstone was laid in 1891. The church has twin bell towers rising above the sanctuary, which top out at over 100 ft above grade. The original towers consisted of brick mass masonry walls with sandstone embellishments. Various additions and renovations have occurred over the years, but most importantly, the renovations in 1915 clad the brick tower façade with granite and cut limestone. Each masonry tower supports a timber-framed spire covered with copper roofing; the timber framing continues partially down the tower and integrates with the masonry.

During the past several years, small pieces of limestone have fallen to the ground. During a much needed cursory condition assessment, elevated concern regarding the façade was noted and pedestrian protection bridging was installed at sidewalks and entrances. Several pieces of limestone fell on top of the bridging during the short time it was in place. A close proximity visual evaluation was then performed to gauge the extent of the deterioration of the masonry. Observations indicated that multiple areas posed unsafe conditions and the church was not safe to remain in operation. Several large granite and limestone sections had displaced and were poised to fall into pedestrian areas below, including the nave. The church was shut down to normal operations and scaffold was erected at the south tower. Structural forensic engineers performed a detailed field investigation to determine the causes of deterioration and develop repair and stabilization options. The results of the evaluation indicated that the masonry and limestone veneer was not properly attached to the brick backup, significant masonry displacements developed and

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FIG. 1—*Overview of towers.*

were exacerbated by water infiltration, adequate repairs and maintenance to limit repairs were not performed, and substantial repair efforts are now required.

Previous Repairs

Preventive maintenance and repairs are an important element in a building's performance; owners need to consult with design and repair professionals with specific knowledge and experience to properly educate and guide them through maintaining their structure. Unfortunately, those consultations do not always occur and the consultant doesn't always have the expertise required. For example, building owners do not normally understand that portions of mortar joint repointing and flexible sealant replacement are maintenance items and work that should be anticipated approximately every 10 to 20 years (depending on materials, exposure, and environment). The only times when the masonry receives much needed attention is when the maintenance window passes and more substantial repairs are required or severe damage occurs, or both. Repairs are often focused more toward addressing deficiencies with a short-term approach versus overall impact of façade deficiencies on the long-term stability of the structure. Repair means and methods are often dictated by budget constraints, which do not always align with the best interests for the building. Although well intended, many of the repairs at St. George's may have adversely affected the long-term performance of the façade and caused premature deterioration of the masonry components. A formal long-term repair and maintenance program was not in place to maximize the service life of the structure; a good program can be created and tailored to the specific needs of individual buildings.

Common of older buildings, limited repair records were available for the majority of the church's existence. Although it appears that during the recent 15 years, the frequency of repairs was increased to apparently address the worsening conditions at the exterior masonry. A general masonry maintenance

FIG. 2—*Areas of previous mortar joint repointing that were applied too thin and have deteriorated.*

program consisting of periodic mortar joint repointing and safety inspections was not instituted at the church.

Many areas of selective mortar joint repointing had been completed; however, some was shell-pointed (applied too thin) and has subsequently failed (see Fig. 2). If repointing work isn't completed in accordance with applicable portions of the Brick Industry Association guidelines [1] and the Secretary of Interior Standards [2], it by-and-large does not perform properly and provides minimal value. The proper procedure includes deteriorated mortar removal to a uniform depth that is approximately twice the joint

FIG. 3—*Displaced limestone opening at joint, nearly 2 in. wide. Note sealant was installed covering widened joint.*

FIG. 4—*Outwardly displaced limestone, nearly 3 in. Note joint opened by displaced masonry was filled with mortar during repair efforts, which will both conceal and cause further outward displacement.*

width or until sound mortar is reached, new mortar installation applied in multiple (approximately 1/4 in.) layers as each previous layer becomes “thumbprint” hard, and tooling of the last layer to match the existing mortar joint profile.

Flexible polyurethane sealant was installed at many cracks, openings, and joints; in particular at or around the limestone. The sealant was mostly found to be cracked and deteriorated, with holes or openings at multiple locations. Some sealant joints were several inches wide due to re-sealing of moving (widening) joints (see Fig. 3). Sealant application over mortar joints is generally not recommended, except for exposed skyward facing joints. No matter how well built and maintained, masonry walls will always allow a certain amount of water into the wall, which will then dry by evaporation through the exterior face of the wall. In the case of the granite façade of St. George and granite’s relatively low absorption properties, the evaporation will occur primarily through the mortar joints. Sealant joints are frequently not maintained, and upon their deterioration, will allow larger volumes of water into the masonry assemblies. Once water enters the masonry, the sealant traps it from effectively evaporating and it remains in the wall. This water presence in the wall causes accelerated deterioration and breakdown of the mortar and masonry from within. The expansive effects of freeze-thaw and recrystallization of salts further amplify the deterioration.

A few through-bolts and plates were installed at select locations of the granite during previous repair projects. These anchors were apparently installed to stabilize outwardly displacing granite at upper areas of the towers, to which their effectiveness was unable to be determined. Regrettably, this was a temporary solution that never ascertained the underlying cause of the problem.

As masonry elements shifted and displaced, cracks expanded in size and repair efforts in-filled the openings with mortar or sealant, or both (see Figs. 4 and 5). These “band-aid” type repairs only provided temporary waterproofing and stability for the masonry and the effects of water infiltration and freeze-thaw

FIG. 5—*Displaced granite at tower corner. Note sealant was installed around granite at mortar joints.*

FIG. 6—*Outwardly displaced (nearly 3/4 in.) granite at top of tower.*

cycles continued. This repair cycle hid displacements and exacerbated their condition, allowing deterioration to grow and accelerate.

Observations

The front west elevation is comprised of two corner towers clad with quarry-faced, coursed ashlar granite. The towers are mass masonry and consist of a brick inner core that was later covered with granite, accompanied by decorative limestone trims and embellishments. The masonry rises approximately 100 ft above grade where the copper primary spire roof extends an additional 40 ft; each primary spire is surrounded with four minor masonry spires. As expected during period construction, no vertical expansion joints, horizontal soft joints, other stress relief measures, or through-wall waterproofing measures were implemented. Lack of these elements significantly reduces the building's ability to resist stresses within the wall and water infiltration.

Observations at the church were completed in general accordance with ASTM Standard E 2270 "Standard Practice for Periodic Inspection of Building Façades for Unsafe Conditions" [3] and ASTM Standard E 2128 "Standard Guide for Evaluating Water Leakage of Building Walls" [4]. The use of a 120-ft boom lift and scaffolding were key elements in tactile access and exploratory probes during the detailed inspections.

The smaller, minor spires were displaced, exhibited masonry spalling, and appear to be out of plumb. Corner masonry of the upper portion of the tower is outwardly displaced. Granite masonry surrounding what originally was thought to be a header was observed to be displaced out of plane between 3/4 to 1 in. (see Fig. 6). Limestone trim bands at the tower exhibit shear/stress-induced spalling. The limestone trim band components at lower levels of the tower exhibit cracking and lateral displacement particularly at changes in plane of the primary granite masonry. At the main entrance side sanctuary alcoves, cracking,

FIG. 7—*Spalled limestone at alcove adjacent to granite.*

FIG. 8—*Cracked and spalled limestone at upper portion of tower.*

lateral displacement, and crushing of limestone (see Fig. 7) were observed at the approximate granite to backup masonry interface. Many small probes were completed during the inspection, and the mortar inside the joints was found to be wet in most locations. Observed patterns in deterioration indicate that as granite components are displaced, stress is transferred to the weaker limestone causing the observed cracking, crushing, and spalling (see Fig. 8). Additionally, the stress was induced by shifting load paths of the granite and limestone veneer due to lack of ties or other positive attachment to backup brick.

Some minor cracking was observed at the interior of the towers at the brick backup, but no compelling evidence of settlement issues was observed. Efflorescence, indicating water infiltration through the wall was also observed at certain areas. Observations and probes at several areas of the brick backup walls at the upper portions of the tower revealed unstable and deteriorated mortar and masonry. Additionally, the timber spire and roof framing interface/embedment with the masonry has deteriorated wood at multiple bearing locations attributed to water deteriorating adjacent masonry. Many gaps and openings in the masonry, especially at upper and exposed assemblies, combined with lack of flashings, allowed water to easily migrate into the wall system. The lack of preventive long-term repair at regular intervals to properly address these openings was a direct contributor to the water infiltration and associated deterioration.

Exploratory probes indicated that granite and collar joint depth (approximately 10 in. combined) varied and no suitable bond or attachment with the brick backup was observed. Although the collar joint was filled with mortar and masonry debris, no substantial bond between the smooth-faced brick backup and granite veneer existed. Nor were any other mechanical ties, anchors, headers, or other means to connect the granite to the brick observed. No areas of flashing, coping, or other waterproofing to direct water away from the wall interior were observed; the tower was a mass masonry structure consisting of a thick granite veneer and limestone over a multi-wythe brick wall.

Key Issues

Lack of Periodic Inspections

Over the course of the 90-year existence of the granite towers, no documented periodic inspections were performed until recently. Unfortunately, recent inspections discovered serious deteriorations and safety issues that are not easily or cost effectively corrected. A more proactive inspection plan could have provided early detection of the masonry issues at a point in time where reasonable corrective actions could have taken place to extend the façade's useful life. The cracks, displacements, evidence of water infiltration, and other deterioration at St. George's Church could have been identified at their beginning so appropriate additional evaluation or repairs could have been completed. Close proximity and tactile inspections of masonry is critical to a long-term, safe, and properly functioning life of building façades.

Lack of Appropriate Rehabilitation Efforts

Past remedial efforts to repair cracks, spalls, displacements, and other deterioration at the façade were improperly implemented as short-term repairs. For example, the causes of the problems were overlooked and a larger bead of sealant was installed, cracks were merely filled with mortar, or mortar joints were shell pointed. The short-term repairs were well intended and not negligent, but limited to the extent of the knowledge of the parties involved. These temporary type repairs were replicated over many years and at many areas, which accelerated deterioration by providing more avenues for water infiltration and means for displacement. Examples of more appropriate direction for repairs would have been remedial anchor installation, flashing installation, proper repointing, and high quality limestone patching. Many deteriorated and displaced areas of granite and limestone were accelerated and degenerated by water infiltration and freeze/thaw cycles (many per year during the extended Pennsylvania winters).

Aging Façade

The integrity of an aging masonry façade declines in a progressive manner over time, particularly with minimal effective maintenance. Additionally, the fact that the granite and limestone was clad over the brick inner core years after original construction questions the long-term effectiveness of this system with no substantial means of interconnection. This aspect is particularly concerning due to aging façade elements and chronic water infiltration issues.

Recommendations

A condition assessment was completed at the towers to determine the existing conditions of the masonry and to provide recommendations for repair. As part of the investigation, system scaffold was erected around the south tower (currently in the worst condition) to facilitate detailed inspections, probes, and sample masonry removal. It was determined that no suitable connection between the brick backup and granite and limestone veneer was present and the granite was beginning to peel away from the backup. No flashing or other waterproofing measures were in place and masonry deterioration was accelerating due to water infiltration. Embedded timber framing and brick backup walls were deteriorated at upper portions of the tower and granite and limestone displacement was severe at several locations. Observed prior repairs had minimal impact on slowing or mitigating the deterioration mechanisms present.

Currently, pedestrian access near the towers and in the church is restricted. Observations do not indicate that a tower structural collapse is imminent at this point in time; however, significant portions of granite or limestone masonry, or both, could fall from the structure at any time. Emergency structural containment netting was required and completed at one of the worst corners of the south tower. Thereafter, temporary containment at the upper portions of masonry at both towers was completed to limit the potential for debris falling to the ground. The containment consisted of a fine and course structural netting woven together with steel cables anchored with threaded rods set in adhesive into the backup brick.

After the containment projects had begun, various repair options were proposed for the church's consideration, although none have been selected yet. Recommended options include:

Option 1—Truncate the two towers by removing the spires and the upper portion of masonry and timber framing resulting in towers approximately at half height. Reconstruct a portion of the limestone and granite façade at the front elevation and install through-wall flashing at key locations of masonry to remain. Pin the remaining granite and limestone to the backup brick with restoration anchors at regular intervals and perform 100 % mortar joint repointing. Also included in this work is capping the towers with a roof system and main nave roof repairs/alterations to limit water infiltration. All masonry and timbers would have to be carefully removed, documented, labeled, and stored for a possible future reconstruction project. Also note that a substantial amount of limestone replacement will be required and the majority of units are specially cut or carved.

Option 2—Tower reconstruction consisting of significant masonry veneer and brick backup demolition at the upper portions of the tower. Subsequent rebuilding of the masonry walls using CMU backup and the original granite exterior along with a combination of original and new limestone will then follow. Substantial structural timber and ring beam bolstering and modification to support the spires during masonry removal would be required along with timber repairs to address water damaged deterioration. Installation of through-wall flashing at key locations of new and existing construction to help control water flow would be utilized. Granite and limestone veneer reconstruction at a large area of the front elevation would be needed to correct displacement. All remaining granite and limestone wall areas not affected by the rebuild portion of the work would be pinned to the backup brick with restoration anchors at regular intervals, with 100 % mortar joint repointing to follow.

Option 3—Demolition of the towers is an unfortunate realistic option to the owner due to the high costs of restoration. Although due to the volume of heavy masonry at the building and tight site constraints, demolition is not an inexpensive option.

Option 4—Mothballing the structure in accordance with the Secretary of Interior guidelines is a potential option to extend the time required to make an ultimate decision or raise funds for restoration. However, mothballing requires effort and funding to maintain the building in reasonably useful condition for reuse.

If a restoration option is selected, a phased restoration of both towers could then proceed using materials and methods appropriate for long-term repairs. Budget costs for the repairs are currently being finalized and the owner is reviewing their options and operational needs.

If the repairs are completed, a course of periodic (every five years) inspections should be completed at the church to ensure the façade condition is properly monitored. An overall preservation plan should be developed to outline at what intervals and to what extent preventive maintenance should be completed. Additionally, maintaining an accurate and descriptive log of repairs and repair materials is an important element to overall building maintenance and planning.

Summary

Various influences, including poor long-term preservation efforts, lack of periodic inspections to provide early detection, and an aging granite façade have impacted the service life of the church. The condition of the stone façade was generally poor, with multiple deficiencies permitting water into the wall and deteriorating the masonry. Deficiencies included displaced granite and limestone, spalled and fallen limestone fragments, no positive connection between granite and brick backup, lack of appropriate waterproofing, structural timber deterioration, deteriorated mortar joints, and inappropriate or failed prior repairs. As these influences combine, the detrimental impact of water infiltration and masonry deterioration enter into a damaging cycle for the granite façade. If periodic inspections and repair interventions were completed regularly during the building's life cycle, many of the serious deteriorations could have been avoided or delayed.

Stabilization repairs have been designed and implemented. A substantial comprehensive exploratory evaluation was conducted by an experienced engineering firm and repair options outlined. The owner can select from full tower and masonry restoration, tower truncation and partial masonry restoration, full demolition, or mothballing. Each option has its own inherent advantages, disadvantages, and impact on the church and surrounding neighborhood that are specific to the owner. The church is currently reviewing their options to determine the most appropriate course of action. The exterior walls are currently a patchwork of prior repairs that are not consistent with long-term preservation of a building. If restoration is completed, a long-term maintenance and preservation plan consistent with historic building materials and craftsmanship should be implemented to ensure future leaks are minimized and the life cycle of the building is extended. Most buildings intended to be in service for many years can benefit from a tailored preventive maintenance plan. It is often not the building owner's desire to perform limited building repairs, but the lack of knowledge and funding. With the help of the design and construction community and papers such as this, owners can be better aware of their building's needs, consequences of decisions, and make more informed decisions.

Once a masonry façade reaches this point of deterioration and conditions become unsafe, minimal choices aside from a significant restoration effort or decommissioning remain. Stabilization or stop-gap measures will only afford the building a short-term reprieve. Many facilities in the country face similar deterioration and circumstances with their stone facades. If measures are not implemented to periodically inspect and properly repair the masonry, problems will continue to develop.

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Characteristics That Affect the Integrity of Existing Thin Stone Cladding

ABSTRACT: Prestigious structures began using thin natural stone panels as wall cladding to give them historic character in the late 1960s. Easier fabrication methods increased stone production and enabled larger, thinner panels that were lightweight. Unfortunately, engineering and testing did not keep pace. Inadequate initial evaluation of material durability and panel strength resulted in varying degrees of distress developing in some claddings. Because early stone cladding was installed with little structural analysis, current evaluation should predict capability by checking characteristics initial engineering likely ignored. Comprehending all characteristics that affect the integrity of thin stone cladding must be the objective of a responsible maintenance program. The program must evolve with each evaluation's findings to adapt to the specifics of that building and its changes over time. Maintaining the safety of a building's skin is essential to extending the service life of the whole structure. Verifying safety requires more than a cursory visual check for exposed elements to find them before they fall. In the past, stone cladding was completely removed from too many structures that experienced distress. Some evaluations misunderstood or misdiagnosed affect of distress on integrity. They failed to investigate some characteristics, or did not relate others, yielding an incomplete assessment. Adding evaluation of characteristics common to traditional structural assessment to a cladding program adds rigor and objectivity, avoiding misdiagnosis and unjustified expense. In rare cases, remove-and-replace-all may still be required, but only if gross problems exist with support. However, in most cases, an appropriate program will not only identify the parts creating the highest risk, but the program can prescribe selective replacement instead of complete recladding as a long-term remedy.

Background

Since the late 1960s when stone frequently became thin to fit within skyscraper curtain walls, buildings have used many varieties of natural stone as wall cladding. Then and now, material properties critical to structural performance and durability vary significantly even within the same stone type and quarry. While Michelangelo recognized this centuries ago as he painstakingly selected blocks for soundness, the building industry continues to have to re-learn this lesson by fixing problems principally caused by deficient evaluation. Problems resulted from insufficient engineering and ineffective quality control of characteristics, other than appearance, that affect structural integrity. No quality control tests to verify consistency rendered widely varying material strengths. In the past, stone testing typically was done only before fabrication, and tested only a few specimens. Fabricated stone was almost never tested for key properties during fabrication after those original preconstruction tests. Incorrect or missing analysis of the stone panel as a structural member rendered systems with insufficient capacity. Analysis of panels and anchors rarely occurred, and tests of actual anchor devices were even rarer, even on 70-story towers. Aggravating the analysis, safety factors thought to be conservative 20 years ago were less stringent than current practice.

The integrity of an existing cladding should be verified during the service life of the building whether it was originally designed and built properly or not. This philosophy is not unique to thin stone. Inspection of cladding by experienced professionals with correlation to testing and analysis are the key ingredients of a program that expects to evaluate cladding *integrity*, not just its general *condition*. Content of the inspection and how its findings are used is the difference between a condition assessment and an integrity evaluation. Determining integrity integrates a comprehensive visual inspection that compiles not only

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condition characteristics, but component characteristics that affect capacity such as support configurations, thicknesses, and spans. The inspection findings are then correlated to a conventional testing regimen to show relationships that can be compared to loads and safety factors. Analysis must show actual panel capacity exceeded required capacity, i.e., simply, *strength must exceed load*, with some margin of safety. The concluded evaluation would determine whether cladding or support are sufficient and would indicate whether any parts required replacement. Removing potentially weak components improves safety of the cladding system and extends the service life of the building.

Correlating testing to the inspection findings is the ingredient of the program that is unique to thin natural stone. The variability of the natural material, methods of installation, and workmanship creates a diverse population of conditions to evaluate. Reducing risk and thus increasing reliability requires eliminating weak components from the population. To find the capacity of the weakest existing panels and establish their immediate condition, bending stresses should be checked against material tested from removed panels in the worst visibly *distressed* conditions. Those tests should be correlated with measurements of panel *thickness* and material *porosity* as principal properties influencing *strength*. Comparing strength to applied loads established panel safety.

From experience inspecting and surveying individual stone panels that clad many buildings in various climates the last twenty years, the process of evaluating each panel's physical condition has become more objective and comprehensive. Inspections revealed several common characteristics that affect panels' integrity, meaning their *immediate* capacity and their *long-term* durability. Observance of certain types and patterns of distress indicated whether original engineering and quality control provided a proper system in their exposures. While determining a stone panel's capacity to resist loads without physically testing it to destruction is not possible, the author has employed multiple in-place visual and physical checks of panels' characteristics to identify *visible* critical weaknesses in panels that had low capacity. Those characteristics will be explained in detail under *panel strength characteristics*. In the past few years, this process added nondestructive evaluation using acoustic sounding with a specialized pulse-velocity device incrementally improved since its early 1980s introduction by Dr. John Logan [1]. This special instrument, developed specifically for thin stone cladding, augments previous techniques to help locate *hidden* faults in panels that could significantly reduce their capacity. The objective evaluation process that determined the approximate capacities of retained panels correlated visible characteristics to acoustic sounding measurements and also to actual material and panel strength testing. Experienced interpretation is required, for a single characteristic rarely showed a panel to be inadequate. Because all characteristics affect panel capacity, all must be considered together to generally [1] predict whether a panel's capacity may be lower than required. Understanding that weathering effects and loads imposed vary across each building face depending on orientation to prevailing weather and architectural shape of the building, vulnerable areas must be studied. Actual panel capacity must adjust to those varying conditions to maintain the margin, called a *safety factor*, against those loads.

Evaluation Scope

The objective of an integrity evaluation is to determine whether a cladding is safe. What factor is safe is a debated subject outside the scope of this paper. This paper instead presents the objective evaluation of actual and required capacity. Both are derived to reduce unknowns, permitting safety factors to be intelligently set and applied.

Many characteristics of each natural stone cladding panel affect its actual capacity, or ultimate *strength*. A panel's approximate safety factor is the ratio of its ultimate *strength* compared to maximum *loads* that it must resist. The safety factor is approximate because the characteristics controlling strength can never be precisely measured for every panel. Thus, both *strength* and *loads* are approximated, but are determined as closely as current engineering science can feasibly predict with reasonable certainty.

Panel Strength Characteristics

Practice has determined that six primary physical panel characteristics need to be evaluated in-place during an inspection to begin an analysis: anchors, joints, cracks, thickness, sounding, and bow. Each characteristic would have its individual threshold correlating to unacceptable capacity, based on the material and project circumstances. However, all characteristics need to be measured or quantified because their interaction affects panel strength, and combined minor deficiencies could also correlate to unacceptable capacity. The six characteristics that predominantly determine each panel's ultimate capacity, or *strength*, either evaluate the panel's condition as a structural body or the strength of the material that compose that body. Panel *anchors*, perimeter *joints, cracks,* and panel *thickness* directly affect the panel body. Most distress related to these characteristics is directly observable from the exterior. Acoustic *sounding* and panel *bow* can measure *relative* and *comparative* material strength by indirectly measuring changes in the material. The characteristics affect panel strength as described below.

Anchors

Each panel connects to the building with anchors, and their integrity dictates panel stress. By their distribution near the edges or corners of the panel, each anchor resists a proportion of the panel's total lateral load. The bottom anchors also typically carry the panel's weight. Similar to a beam spanning a floor between two columns or a floor slab spanning between four columns, a cladding panel spans between its anchors on the building's wall. Anchor layout and spacing determine flexural behavior, and thus distribution of internal stresses, which dictate panel capacity. Not only is the location of the anchor device connecting to the stone important, but knowing what that anchor device is attached to is also important. Relative stiffness among all the supports dictates effective panel support. Distress of any type, including cracks entering the load path, influence panel support.

Joints

Perimeter joints should isolate the panel from adjacent, unplanned forces. Verifying boundary conditions is critical to an engineering evaluation. By structural design of their attachment system and soft perimeter joints, thin stone cladding panels should be designed and installed to resist only wind loads and their own weight. Their slender profile should not stack or carry axial loads. If axial loads occur because physical conditions changed at their boundary since they were installed to accidentally cause stacking, stresses in the panel would increase unintentionally, jeopardizing stability. An unintended stacking condition may be recognizable by irregularities in the sealant that fills top and bottom joints between panels.

Cracks

Cracks or any other type of separation in the solid body of the panel could reduce the panel's capacity, depending on their location and orientation within the panel. Because a panel spans between anchors usually positioned near its edges, stresses are highest near the middle of the panel between the anchors, and thus separations in the middle region affect capacity the greatest, whether the panel behaves as a plate in two-way bending or a shallow beam in one-way bending. Where cracks reduce the effective width of the panel body acting as a flat plate to resist perpendicular loads, their visually apparent size must be tracked.

Thickness

Greater panel thickness increases the section's theoretical capability to resist lateral wind loads. As a structural body, the panel's change in thickness increases or decreases its capacity at an exponential rate of the difference in thickness.

Sounding

Changes in material porosity, a characteristic not visible by hands-on on-site inspection, has been shown to correlate to changes in material strength. Porosity and absorption are not the same property. Exposure, weathering, and hysteresis described under *bow*, have been shown by previous research to increase porosity in some stones over time. Increased porosity correlates to reduced material strength even if no cracks are visible. Logan's acoustic sounding instrument measures relative porosity non-destructively in terms of transfer time of a pulse across the body and has been shown to detect not only significant changes in porosity, but also invisible weaknesses such as rift and seam separations due to geologic flaws in panels. Low instrument readings indicate low porosity and higher relative material strength. High readings indicate high porosity or an internal flaw, rendering lower material strength. For sounding readings to be meaningful measurements of changes in the stone material's strength, they require correlation to actual flexural strength tests.

Bow

Bow, also called dish, and technically termed hysteresis, is the panel's deviation from its original finished plane caused by inelastic differential volume changes induced largely by thermal cycling. This is *not* a phenomenon that occurs only in marble. Whether a panel bows concave or convex, as bow increases, material strength generally decreases. For bow measurements to be meaningful indicators of changes in the stone material's strength, they require correlation to actual flexural strength tests.

Loads on Panels

When computing panel adequacy, loads constitute the side of the equation opposite panel capacity. This evaluation assumes the thin stone panels are intended to be individually supported. Since cladding panels should *not* be stacked, primary loads imposed on the panels are lateral. Seismic-induced lateral loads that involve self-weight are usually exceeded by wind loads, and most analysis models do not require both to be applied simultaneously. Thus, the controlling lateral load is typically induced by wind.

Accurately predicting maximum wind loads on cladding can be a complicated exercise. Wind loads change with wind speed and direction, and vary with location on the building due to prevailing direction and vicinity to other nearby buildings. Recent cladding failures have emphasized the influence of local context in accelerating wind around certain features, significantly increasing loads in some wall areas. While code formulas set loads based on building height, basic wind speed, and edge distance, the most accurate way to predict all those variables is to physically model them in a laboratory wind tunnel. Individual pressure taps within a scale model measure pressure effects as wind direction changes. Laboratory results report schematic distributions of inward pressures and outward suctions after correlating to historic weather patterns, then translates them onto building elevations, showing loads on each stone panel when jointing is superimposed. Most tests derive maximum predicted pressures and suctions resulting from gusts with a 50 year occurrence and use the most recent 25 years of weather data from nearby stations. Effects can be evaluated within an azimuth sector, or direction, in 10 degree increments. This is the same procedure used to test new structures.

However, straightforward panel analysis using code formulas and even wind tunnel tests may not reveal actual stresses on cladding panels. Design of anchorage in the wall system design controls how those imposed loads are resolved and distributed within the panel. Equal relative stiffness of each anchor on its supporting backup must be attained to assume equal resistance by each anchor. Separate from anchor effectiveness, use of cavities, closed, open, or vented joints, and internal compartments will modulate pressure difference between the inner, inside panel face and the outside face. Effective equalization will reduce lateral loads on a panel, but determining the amount of reduction is complex. Study of pressure equalization and moderation resulting from venting, cavity volumes, air-barrier integrity, continuity around corners and across floors, and gust duration must be considered to determine effectiveness of equalization.

Engineering Implications of Panel Capacity Characteristics

Anchor

If a panel fails at one anchor either by anchor device failure or by stone fracturing at the device, load that would have been resisted by that attachment would distribute to the adjacent anchor. Depending on the layout of anchors within the panel and their support, changed load path caused by a failed anchor can drastically increase a panel's internal stress.

Joints

Establish whether the original stone support design provided for each panel to be independently supported. This means that the *weight* of each panel and the surface-applied *lateral load* due to wind is transferred to the building through the connection anchor and not through another stone first. Not only does this keep panel behavior structurally determinant, the technique also maximizes panel capacity. However, installation inconsistencies, building movements and stone panel distortions may have, in some locations, changed the physical conditions such that those originally assumed boundary conditions no longer exist. If panel edges contact each other through their anchors or shims, stacking could occur, meaning the weight of one panel rests on top of another. This in-plane axial load adds to the flexural stresses induced by lateral wind loads and reduces the panel's capacity if buckling could be induced. How much axial load is transferred, and how many panels stack, is usually difficult to determine in the field. For this reason, it is expected that axial loads should be eliminated. Keeping each thin panel independently supported maintains the original boundary conditions. To preserve those conditions, the perimeter joints are filled with compressible filler or left open to prevent transferring load in a way that affects panel behavior or capacity. When joints become compressed, frequently the sealant bulges. While bulging does not prove stacking, it must be checked to verify patterns and checked to see if anchors clamp hard between panels.

Cracks

The size of a structural member directly affects its potential strength, or capacity. The size of a panel's cross section perpendicular to its span is its width times its thickness. Decreases to either dimension, as caused by cracks, can reduce the panel's effective size and thus reduce its structural capacity depending on whether the cracks occur within elevated stress areas in the panel. Cracks in the middle third of the panel are critical because an edge-supported panel's ultimate capability to resist wind loads by flexural strength is controlled by the size of the cross section at midspan. That middle third could be its width or height or both, depending on anchor layouts, and assuming the panels do not cantilever. A thin panel, as a structural member resisting load that induces flexure, develops its highest stresses midway between its anchor supports. Cracks in the middle region reduce a panel's effective width and thus reduce its capacity *in direct proportion* to the cracked width. While cracks in the end-third regions of a non-cantilevering panel do reduce the effective cross section, they may not reduce ultimate capacity because less material is required in these regions to resist the lower stresses in that region. Understanding stress distribution in the flat panel is critical to interpreting whether cracks in certain locations, their size or their orientation could reduce panel capacity. Anchors located away from panel edges changes panel behavior, internal stress distribution, and thus locations where cracks are critical.

Adding to the difficulty of interpreting cracks is the range of actual crack types found within different stone panels and different geologic stone types. On a wall surface viewed from the exterior, only the outside exposed panel face can be seen. Compared to the back surface facing the cavity, this face is most susceptible to deterioration due to exposure and tension resulting from suction loads, and thus most likely to show cracks caused by imposed loads. However, cracks can occur on the concealed backside for several reasons: where panels bow concave inward, or the panel was shimmed improperly when set, or its anchor slips. Some cracks are hairline and visible only at hands-on range. Some cracks originate as eroded surfaces in the stone's natural bedding plane or geologic rift and do not penetrate the entire panel thickness; their real depth cannot be seen.

Thickness

Variations in thickness change the size of the panel as a structural member, directly affecting its capacity. Thickness typically varies with the original fabrication slabbing tolerance. Slabbing is the operation that cuts flat sheets from solid blocks cut from the quarry. While slabs would be sawed to a certain module thickness, some will be thinner than the module due to tolerances. Today, quality control procedures usually segregate and exclude under-thickness panels from a cladding project. However, older buildings or those clad in materials fabricated in places lacking quality control often included under-thickness panels.

Panel thickness is one of the most critical characteristics, because a structural body's flexural capacity varies with the square of the change in thickness. It is an exponential relationship. For instance, a 29 mm thick panel is only 3 % thinner than a nominal 30 mm panel, but has 6 % less capacity. And a 27 mm panel is 10 % thinner than specified, but has 19 % less capacity. We have found panels 20 % thinner than the designated thickness on projects where quality control was not stringent. Panels found thicker than the designated module have increased capacity and thus a potentially higher safety factor against flexure, but also weigh more. Increased thickness can offset deficiencies in other characteristics.

Sounding

Dr. Logan extended his non-destructive rock mechanic evaluation principles from subsurface research to natural stone building claddings beginning in the early 1980s. By constructing a device with a soft pulse transducer, receiver, and measuring timer, Logan developed an instrument that measured travel time without risking propagation of internal geologic flaws within the stone. The proven theory was that unfractured, uncracked, dense rock bodies transmitted pulses quickly through the body from transducer to receiver. Stone with those features had higher flexural strength. As porosity increased or fractures occurred, whether visible or microscopic, transmission times lengthened as pulses migrated around the faults. Stone with those features had lower flexural strength. The basis of the procedure on stone was established by testing of marble panels removed from facades 20 years ago. Those studies established a *general* correlation between acoustic sounding measurements and stone tensile strength for Carrara marble. Tensile strength is the critical principle stress developed by wind loads inducing flexure. When tensile stresses translate to flexure, they can be measured by standard test method ASTM C880 *Standard Test Method for Flexural Strength of Dimension Stone*.

It is not physically possible to independently isolate all physical boundary conditions that can affect acoustic sounding measurements with data currently collected. It may never be technically possible to the level of reliability necessary to call a panel safe or not safe by the acoustic reading alone. Therefore, it is not possible to expect acoustic sounding measurements to directly translate into material flexural strength with a necessary degree of engineering certainty. It is proven possible by past practice, however, to identify *anomalously* weak panels with acoustic sounding that may not be otherwise visibly identifiable. These are panels with faults that make them significantly different than the majority population. When significant separations occur within the panel body, whether they are frequent microscopic breaks in crystal bonds or laminar weaknesses that fail to transfer sound and stress, the acoustic sounding measurements will find those panels. Setting the threshold minimum value is a study unique to each project. However, readings below thresholds must be considered in combination other characteristics to determine integrity.

Bow

Most panels on a facade are not flat because over time, thermal cycling caused inelastic volume changes in the stone material. This geologic phenomenon occurs in several types of natural stone, and is most prevalent in thin marbles. Actual distortion is usually more three-dimensional like a dish, and occurs to some extent horizontally *and* vertically. Some panels dish inward and are concave; some dish outward and are convex.

Surveys Compared to Inspections

Two types of visually based evaluations, i.e., *surveys* and *inspections*, are valuable for different reasons. A *survey* entails a relatively quick visual scan that looks primarily for large cracks, bows, or discernable anchor distress that might threaten a panels' stability *near*-term. Any panel found with such a feature should be prudently removed or stabilized, though means to stabilize must be carefully studied to assure they do not actually worsen panel condition. If specific records of the feature are not kept, a survey might check between 1 000 and 2 000 panels in a long workday without weather interruptions. While not a complete assessment, a survey can find panels approaching potential failure if by principal mechanisms of anchor failure, cracking, or bowing, and present the overall general condition of the cladding. An *inspection* is quite different. It entails a thorough, close visual examination that checks and records all six characteristics of every panel, whether it appears precarious or not. The inspection can find less-obvious potential weaknesses and allows *long*-term monitoring of panel conditions to evaluate changes over time. More thorough and time consuming than a survey, 150 to 200 panels could be inspected in a long workday when acoustic sounding is also done. Frequent surveys of all surfaces should mix with incremental partial inspections to provide the most feasible approach to maintaining an ongoing assessment.

Conclusion

Current technology does not allow accurate or precise predictions of the service life of existing thin stone panels. The oldest facades clad in thin stone built 40 years ago provide the most reliable exemplars for understanding natural weathering. Therefore, monitoring the performance of retained panels is our best method for establishing trends and legitimately predicting future durability. Monitoring performance also increases safety by reducing risk.

The multitude of variations in architectural arrangements, climatic exposures, stone types, material properties, wind loads, and construction workmanship must be monitored in-place by a structured, objective program. Understanding variability of individual characteristics increases reliability and permits lower safety factors. Removing distressed panels and those potentially below accepted safety factors eliminates the known *under*-strength panels. The program will help mitigate complications resulting from general lack of familiarity with the distinguishing properties of thin stones used as wall cladding. Lack of familiarity was common on early-generation buildings due to a lack of experience. Similar mistakes occur on new buildings because constructors take it for granted systems work without evaluation.

Monitoring the condition of facades clad with thin natural stone by objective evaluation can reduce risk inherent in common practice, old and new. Only a short quarter-century ago, varied quality and mixed durability panels were commonly built into systems clad in thin stone. Unfortunately, some of those practices continue. The objective evaluation can not only mitigate inherent problems, it can also help avoid unnecessary expense prompted by an incomplete assessment that renders unsubstantiated risk. Findings from the process assist maintenance even if deleterious distress does not develop. Applied findings will maximize panel service by objectively checking if selection, design, testing, and quality control practices worked.

[1] Logan, J. M., "On-Site and Laboratory Studies of Strength Loss in Marble on Building Exteriors," *Fracture and Failure in Natural Building Stones*, S. K. Kourkoulis, ed., Springer Verlag, Berlin, 2006 pp. 345-362.

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Travertine: Successful and Unsuccessful Performance, Preconceived Notions, and Mischaracterizations

ABSTRACT: Travertine is a beige-colored stone with unique color variations, veining, and cavities that many architects worldwide find attractive. The word travertine was derived from an old Roman name for the town of Tivoli (Tibur) in Italy where large deposits of travertine exist. The ancient name for the stone was Lapis Tiburtinus (Tibur Stone) which later evolved into the word travertine.Travertine is a sedimentary rock that began as limestone. Underground water heated by the earth's core dissolved the limestone and brought it along with other minerals to the earth's surface to form mud beds. In time, these mud beds of limestone and other minerals cooled and crystallized into solid travertine. This cooling process caused the small unique cavities and open channels in travertine to develop and the minerals created the unique color variations and veining in travertine.Travertine has been successfully used in buildings for thousands of years. The Coliseum in Rome is among the larger buildings in the world that was constructed largely with travertine. However, the inherent small cavities, open channels, and veining in the stone that many architects find attractive are viewed by some people as cavities where water can collect and potentially cause deterioration of the stone due to freezing of this water. This preconceived notion has led to avoidance of the use of travertine in some buildings. While this condition has locally occurred with certain uses of travertine, it has not occurred on a systematic scale, and properly designed, constructed, and maintained travertine elements on buildings have performed successfully.Travertine is a form of limestone and is sometimes classified as travertine limestone because it is composed principally of calcium carbonate. However, sometimes it is commercially classified as travertine marble because it can be polished. This commercial classification of travertine as marble has sometimes led to confusion in the design of travertine on buildings because geologically travertine is a form of limestone and is not marble.This paper describes the successful and unsuccessful uses of travertine on buildings, and provides guidance on how to successfully design travertine on buildings. This paper also provides information that will help to clear up the preconceived notions and mischaracterizations of travertine.

KEYWORDS: travertine, marble, limestone, freeze-thaw, flexural strength, modulus of rupture

What is Travertine?

The word travertine originated from the old Roman name of the town of Tivoli in Italy near Rome where large deposits of travertine exist [1]. Tivoli was formerly known as Tibur in ancient Roman times. The ancient name for the stone from this town was Lapis Tiburtinus, which meant Tibur stone, and Lapis Tiburtinus, the ancient name for the stone, has subsequently evolved to travertine.

Travertine is a sedimentary rock that began as limestone. Due to geological shifting forces, some limestone deposits were forced deeper into the earth where some of these deposits were exposed to hot water from aquifers heated by the earth's inner core. The hot water dissolved the limestone and other minerals and this solution rose to the earth's surface as steam to form hot springs, geysers, and mud beds. Over time, these mud beds of limestone and other minerals cooled and crystallized into solid travertine. The cooling process caused numerous cavities and open channels to develop in the stone. These cavities and open channels can be filled with grout or be left in their natural form [2].

The mixture of limestone and other minerals created the unique color variations and veins in travertine. Travertine never occurs in one solid color. The color tones in travertine vary from white to brown with bands of different colors occurring in the stone. As shown in Fig. 1, the unique color and color variations along with the veins contribute to the attractiveness of travertine.

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FIG. 1—*View of vein-cut travertine wall panels in Museo dell' Ara Pacis in Rome, Italy.*

In the *Dictionary of Geological Terms* that was prepared under the direction of the American Geological Institute (AGI), travertine is defined as:

> "A finely crystalline, massive deposit of calcium carbonate, of white, tan, or cream color, formed by chemical precipitation from solution in surface and ground waters, as around the mouth of springs, esp. hot springs. It also occurs in limestone caves, where it forms stalactites and stalagmites. A spongy or less compact variety is tufa" [1].

Under "LIMESTONE GROUP" in ASTM C 119-06, "Standard Terminology Relating to Dimension Stone," the definition of travertine is referred to the "OTHER GROUP" where travertine is defined as:

> "A porous or cellularly layered partly crystalline calcite rock of chemical origin."

In ASTM C 119-01, the definition of travertine was previously listed under the "LIMESTONE GROUP" as:

> "a variety of crystalline or microcrystalline limestone distinguished by layered structure. Pores and cavities commonly are concentrated in some of the layers, giving rise to an open texture."

In ASTM C 119-01, the definition of travertine is also previously listed under the "MARBLE GROUP" as "travertine marble" where it is defined as:

> "a porous or cellularly layered, partly crystalline calcite of chemical origin."

As described above, ASTM C 119-06 has removed the definition of travertine from the "LIMESTONE GROUP" and from the "MARBLE GROUP" where it was historically defined and lists its definition under another group of stone.

Is Travertine a Limestone or is it a Marble?

There are three types of rock: igneous, sedimentary, and metamorphic. AGI defines these types of rock as follows:

> Igneous: "Said of a rock or mineral that solidified from molten or partly molten material, i.e. from magma; also, applied to process related to the formation of such rocks" [3].

> Sedimentary rock: "A layered rock resulting from the consolidation of sediment, e.g. a caustic rock such as sandstone, a chemical rock such as rock salt, or an organic rock such as coal. Some authors include pyroclastic rocks, such as tuff" [4].

> Metamorphic rock: "Any rock derived from pre-existing rocks by mineralogical, chemical, and/or structural changes, essentially in the solid state, in response to marked changes in temperature, pressure, shearing stress, and chemical environment, generally at depth in the earth's crust" [5].

AGI defines limestone and marble as follows:

FIG. 2—*View of cross-cut travertine bench slab in Museo dell' Ara Pacis in Rome, Italy.*

> Limestone: "A sedimentary rock consisting chiefly of the mineral calcite (calcium carbonate), with or without magnesium carbonate. Common impurities include chert and clay. Limestone is the most important and widely distributed of the carbonate rocks and is the consolidated equivalent of limy mud, calcareous sand, and/or shell fragments. It yields lime on calcination" [6].

> Marble: "1. A metamorphic rock consisting predominantly of fine- to course-grained recrystallized calcite and/or dolomite. 2. In commerce, any crystallized carbonate rock, including true marble and certain types of limestone (orthomarble), that will take a polish and can be used as architectural or ornamental stone" [7].

Since travertine is principally composed of calcium carbonate, it may be considered as a form of limestone. ASTM C 119-06 discusses in nonmandatory language that "travertine is sometimes classified for commercial purposes as limestone because it is composed principally of calcium carbonate…"

Since the formation of travertine did not include heat and pressure and subsequent recrystallization of limestone, it is not a marble. Geologically, travertine is significantly closer to limestone than it is to marble.

ASTM C 119-06 also discusses in nonmandatory language that travertine "is sometimes classified for commercial purposes as marble because it is capable of taking a polish." Although it is obvious that this commercial classification of travertine as marble is geologically incorrect, it can lead to the belief that travertine is marble, a metamorphic rock, and that travertine should therefore have the physical properties of marble.

Physical Properties of Travertine

General—Due to the natural cavities, open channels, and veins that developed during its formation, travertine has a strong directional veined appearance and a corresponding variation in flexural (bending) strength. Travertine panels for buildings can be cut perpendicular to its natural veins (vein-cut) to expose its natural cavities, open channels, and veins, as shown in Fig. 1; or can be cut parallel to its natural veins (cross-cut) to conceal these natural features, as shown in Fig. 2. Vein-cut panels are typically used in buildings for vertical applications such as walls. Cross-cut panels are typically used for horizontal applications such as flooring and benches.

ASTM Requirements:—ASTM C 1527-03, "Standard Specification for Travertine Dimension Stone" which was first published in 2002 requires travertine supplied under this specification to have the following physical properties:

Absorption by weight, max. (C 97):	2.5 %
Density, min. (C 97):	144 pcf (2305 kg/cu m)
Compressive strength, min. (C 170):	7500 psi (52 MPa) (exterior)
Modulus of rupture, min. (C 99):	1000 psi (6.9 MPa) (exterior)

Abrasion resistance, min. (C 241/C 1353):	10 Ha
Flexural strength, min. (C 880):	1000 psi (6.9 MPa) (exterior)

ASTM C 1527 also states:

"4.3.1 Some travertines may not be suitable for exterior use in areas subject to frequent freeze-thaw cycles."

"4.3.2 Travertine that is fleuri-cut (cross-cut) rather than vein-cut can be expected to experience certain problems, because some areas of the exposed surface will consist of only a thin layer of stone that covers a void in the stone."

Prior to the original approval of ASTM C 1527-03 in 2002, travertine was classified under "dimension marble" as "IV Travertine" in ASTM C 503-99, "Standard Specification for Marble Dimension Stone (Exterior)." Travertine supplied under ASTM C 503-99 was required to have the following physical properties:

Absorption by weight, max. (C 97):	0.20 %
Density, min. (C 97):	144 pcf (2305 kg/cu m)
Compressive strength, min. (C 170):	7500 psi (52 MPa)
Modulus of rupture, min. (C 99):	1000 psi (7 MPa)
Abrasion resistance, min. (C 241/C 1353):	10 Ha
Flexural strength, min. (C 880):	1000 psi (7 MPa)

The ASTM C 1527-03 required compressive strength of 7500 psi (52 MPa), modulus of rupture of 1000 psi (6.9 MPa), and flexural strength of 1000 psi (6.9 MPa) for exterior travertine are exactly the same requirements for marble as specified in ASTM C 503-99. As discussed above, geologically, travertine is not marble; therefore, the physical properties of typical exterior travertine and marble will not be and should not be expected to be the same.

The ASTM C 1527-03 required compressive strength of 7500 psi (52 MPa) and modulus of rupture of 1000 psi (6.9 MPa) are very similar to the requirements of "high-density" (160 pcf or greater) (2560 kg/cu m or greater) limestone specified in ASTM C 568-03. As discussed above, since travertine is principally composed of calcium carbonate and may be considered as a form of limestone, the physical properties of exterior travertine should be expected to be similar to limestone.

However, since the minimum density of travertine supplied under ASTM C 1527-03 is required to be 144 pcf (2305 kg/cu m), travertine should be expected to be more similar to "medium-density" limestone (135 pcf to 160 pcf) (2160 kg/cu m to 2560 kg/cu m) rather than "high-density" limestone (160 pcf or greater) (2560 kg/cu m or greater), as classified by ASTM C 568-03.

Laboratory Testing of Travertine:—ASTM tests were performed by the author and his colleagues on test specimens that were cut from vein-cut travertine panels removed from the exterior facade of six existing buildings. Four of these buildings, referred to as building A, B, C, and D are located in the northern part of the USA, and the remaining two buildings referred to as building E and F are located in the southwestern part of the USA. The age of the buildings when the travertine test specimens were removed from its facade and tested varies from 10 to 20 years.

The specimens were tested in a dry and in a wet condition. Generally, the compressive strength, modulus of rupture, and flexural strength of the wet specimens were found to be about 15 % less than those of the dry specimens. Consequently, since ASTM C 1527 specifies the minimum compressive strength, modulus of rupture, and flexural strength of travertine under this standard, only the results of the testing of wet specimens are presented. The flexural strength of vein-cut travertine panels bent parallel to its veins (weak direction) is about 20 to 30 % of its flexural strength when bent perpendicular to its veins (strong direction) [8].

The results of the tests were as follows:

Absorption (C 97):
Building A: 1.27 %, *N*(number of test specimens) = 6 CV = 19.9 %,
Building C: 1.75 %, N = 5, CV = 52.0 %
Building D: 0.86 %, N = 5, CV = 8.5 %

Density (C 97):
Building A: 151 pcf (2420 kg/cu m), N = 6, CV = 1.7 %
Building C: 149 pcf (2390 kg/cu m), N = 5, CV = 2.6 %
Building D: 153 pcf (2450 kg/cu m), N = 5, CV = 1.6 %

Compressive strength,wet (C 170):
Building A: 4220 psi (29 MPa),N = 5,CV = 30.9 %, (parallel to veins);
5440 psi (38 MPa), N = 5, CV = 31.8 %, (perpendicular to veins).
Building B: 7760 psi (54 MPa), N = 5, CV = 18.1 % (parallel to veins);
5700 psi (39 MPa), N = 5, CV = 40.7 % (perpendicular to veins).
Building C: 9340 psi (64 MPa), N = 5, CV = 11.9 % (parallel to veins); 6880 psi (47 MPa), N = 5, CV = 33.5 % (perpendicular to veins).
Building D: 9060 psi (62 MPa), N = 5, CV = 9.7 % (parallel to veins);
7290 psi (50 MPa), N = 5, CV = 18.3 % (perpendicular to veins).

Modulus of Rupture, wet (C 99):
Building A: 840 psi (6 MPa), N = 4, CV = 39.5 %, (weak direction); 995 psi (7 MPa), N = 3, CV = 16.4 %, (strong direction).
Building B: 840 psi (6 MPa), N = 5, CV = 38.0 %, (weak direction); 1260 psi (9 MPa), N = 5, CV = 9.3 %, (strong direction).
Building C: 190 psi (2 MPa), N = 5, CV = 60.0 % (weak direction); 1170 psi (8 MPa), N = 5, CV = 10.6 % (strong direction).
Building D: 1347 psi (9 MPa), N = 5, CV = 18.7 % (weak direction).

Flexural strength, wet (C 880):
Building A: 465 psi (3 MPa), N = 5, CV = 24.2 %, (weak direction); 1345 psi (9 MPa), N = 5, CV = 6.4 %, (strong direction)
Building C: 210 psi (2 MPa), N = 5, CV = 31.1 % (weak direction); 770 psi (5 MPa), N = 5, CV = 55.8 % (strong direction).
Building D: 790 psi (5 MPa), N = 5, CV = 7.9 % (weak direction).
Building E: 900 psi (6 MPa), N = 5, CV = 13.7 % (weak direction); 1575 psi (11 MPa), N = 5, CV = 5.2 % (strong direction).
Building F: 800 psi (5 MPa), N = 5, CV = 72 % (weak direction); 1030 psi (7 MPa), N = 5, CV = 36 % (strong direction).

The difference of the modulus of rupture of the travertines tested between the "weak" and "strong" directions varies between 15 and 84 %. The difference of the flexural strength of the travertines tested between the "weak" and "strong" direction varies between 22 and 73 %. This significant difference is due to the natural strong directional veined formation of travertine that developed during its formation.

In addition to the above testing of travertine specimens removed from the exterior walls of existing buildings, the author also performed ASTM C 880 flexural strength testing of specimens that were cut in the laboratory from travertine panels that were fabricated for a new building. The result of the testing of these specimens which were tested in a wet condition is as follows:

Flexural strength, wet, (C880): 560 psi (4 MPa), N = 5, CV = 13.0 % (weak direction).

Review of the ASTM C 880 flexural strength of various travertines that are presented in *Dimension Stones of the World, Vol. 1* [9] that was published by the Marble Institute of America in 1990, revealed that 11 of the 13 travertines listed have a flexural strength that is greater than 1000 psi (6.9 MPa) and two have a flexural strength that is less than 1000 psi (6.9 MPa). However, laboratory tests performed by the author's co-workers on one of the published travertines that is listed to have a flexural strength that is greater than 1000 psi (6.9 MPa), revealed that the published flexural strength was derived from testing the test specimens in the strong direction and that the ASTM C 880 flexural strength of that specific travertine when tested in the weak direction is less than 1000 psi (6.9 MPa) and is approximately 50 % of that of the published strong direction. The flexural strength of each type of travertines published in *Dimension Stones of the World, Vol. 1* was therefore most likely derived from testing the travertine specimens in the strong direction which is approximately twice the flexural strength in the weak direction.

The results of the laboratory tests performed by the author and his colleagues on travertine specimens removed from existing buildings as well as for a new building revealed that the ASTM C 1527-03 minimum compressive strength requirement of 7500 psi (52 MPa), and the minimum modulus of rupture and flexural strength requirements of 1000 psi (6.9 MPa) for exterior travertine, which is the same for marble and is similar to "high-density" limestone, are not achievable with typical exterior travertine. The specified C 1527-03 requirements for exterior travertine should therefore be reconsidered and changed to reflect its actual properties.

Based upon the results of the tests performed by the author and his colleagues and upon the fact that geologically, travertine is significantly closer to limestone than it is to marble, the physical properties of exterior travertine that are specified in ASTM C 1527 should be considered to be similar to ASTM C 568-03 requirements for "medium-density" limestone. These requirements are as follows:

Absorption by weight, max. (C 97):	7.5 %
Density, min. (C 97):	135 pcf (2160 kg/cum)
Compressive strength, min. (C 170):	4000 psi (28 MPa)
Modulus of rupture, min. (C 99):	500 psi (3.4 MPa)
Abrasion resistance, min. (C 241/C 1353):	10 Ha

In addition, in order for ASTM C 1527 to better reflect the actual range of properties of the typical exterior travertines that have been successfully used in the exterior facades on buildings, the specified minimum flexural strength of travertine should be 500 psi (3.4 MPa).

Use of Travertine in Buildings

Travertine has successfully been used in buildings for thousands of years. The Coliseum in Rome which was constructed about 2000 years ago is among the larger buildings in the world that was constructed largely with travertine. In the USA, travertine was first used at the interior of buildings beginning in about circa 1900, and in the exterior facades of buildings in about circa 1950 [10].

Travertine is mostly used in the exterior facades on buildings and at the interior of buildings as panels that are vein-cut to expose the natural cavities, open channels, and veins of the stone that many architects find attractive. Depending on the desired architectural expression, the cavities and open channels in the travertine are filled with a complementary-colored grout or are left unfilled. When used in building facades, the cavities, open channels, and veins in the panels are oriented horizontally or vertically.

If the decision is made to fill the cavities and open channels with grout, this work should be performed by the fabricator at the place of fabrication and be cured at the place of fabrication. This approach utilizes the experience of the fabricator and the better quality control obtained by performing the work at the place of fabrication.

The use of travertine panels in the exterior facades on buildings that are cross-cut to conceal the cavities, open channels, and veins is rare. Cross-cut travertine panels are primarily at the interior of the building as floor and wall tiles and furniture.

Will Normal Vein-cut Travertine Panels in Exterior Facades on Buildings Deteriorate When Exposed to Freeze-thaw Cycles?

When water freezes its volume rapidly expands by about 9 %. When water within a material freezes and the expansion of the water is accommodated within the unfilled pore structure of the material, no internal forces are developed within the material to cause freeze-thaw damage to the material. If the unfilled pore structure of the material cannot accommodate the expansion of the freezing water, internal forces are developed that can damage the material.

The presence of the exposed cavities, open channels, and veins in vein-cut panels has led to the belief by some members of the building industry that travertine may not be suitable for exterior use in areas subject to freeze-thaw cycles as reflected in ASTM C 1527. However, the inspection of vein-cut travertine panels in the exterior facade on several major buildings in the northern portion of the USA by the author has not revealed freeze-thaw deterioration of travertine panels on any of the buildings inspected.

Freeze-thaw deterioration of normal vein-cut travertine panels when installed vertically in the exterior facade on buildings has not occurred because the natural cavities and open channels in the travertine prevent the accumulation of water in the cavities and channels and thus prevent the travertine from becoming critically saturated. Critical saturation is the almost complete filling of the internal pore structure of a material with water. In addition, the cavities and open channels allow water in them to expand without confinement, and without confinement of the expanding water when it freezes there are no forces to cause deterioration of the travertine.

Common Causes of Distress in Travertine Panels in Exterior Facades on Buildings and How to Avoid Them

The evaluation of travertine panels on the exterior facade of several buildings by the author has revealed that as a material, the performance of travertine panels in the exterior facades on buildings has been successful. While some of the travertine panels on some buildings inspected by the author have developed distress conditions, these conditions did not occur as a result of natural deficiencies in the travertine material itself. They occurred as a result of improper connections between the panels and the structure of the building and improper use of travertine as described below:

Corrosion of Embedded Metal at Connections—When a piece of steel is fully corroded, it occupies up to ten times the volume of the uncorroded steel. When this expansion is confined by the travertine panel, significant forces develop that cause cracks and spalls to develop in the travertine. When these cracks and spalls occur at the connection between the travertine panels and the structure, the cracks will adversely affect the ability of the connection system to withstand loads and function properly.

To prevent this condition from occurring, any metal element that is embedded in or is contacted by the travertine panel should be a noncorrodible metal such as stainless steel.

Expansion of Embedded Materials at Connections—At kerf connections, a groove is cut in the top and bottom of the travertine panel into which a noncorrodible connection element is inserted. This type of connection is capable of resisting inward and outward wind loads as well and gravity loads. It is normal to cut the groove in the panel slightly wider than the inserted steel element to facilitate the construction of the connection. Consequently, in order to prevent inward and outward movement of the panel, the groove is normally filled with a material to fill the space between the inserted noncorrodible metal connection element and the stone. If this filler material expands when it becomes wet or it expands more than the travertine when exposed to increases in temperature, this expansion will be confined and the resulting internal forces at the kerf connection will crack the stone and make the connection system incapable of resisting loads.

To prevent this condition from occurring, sealant should be installed in kerf connection groove in the travertine panels to fill the space between the travertine and the inserted noncorrodible metal connection element. The type of sealant should be carefully selected to fulfill its intended purpose and not stain the travertine.

Reflective Concrete Cracks in Travertine Faced Pre-Cast Concrete Panels—The use of travertine-faced precast concrete panels in the exterior facade on buildings allows large prefabricated wall panels consisting of several small travertine panels and reinforced concrete to be fabricated and cured in a controlled shop environment and be transported to the site of the building for installation.

The construction of these panels generally consists of installation of stainless steel wire loop anchors in the back of the travertine panels, placement of the travertine panels face down in a formwork, installation of a bond breaker membrane on the back of the panels, pouring concrete on the back of the panels, curing of the concrete, and filling of the joints between travertine panels with mortar.

Whenever the bond breaker membrane is not installed as a part of the fabrication process, the travertine and the concrete will be bonded together. In this condition, shrinkage cracks that develop in the concrete portion of the panel will reflect into and crack the travertine panels.

Bond breakers should be used in travertine-faced precast concrete panels.

Cracking of Vein-Cut Travertine Pavement—When vein-cut travertine panels are used in pavements, water that gains access into the exposed cavities and open channels at the top of the pavement will not be able to freely and rapidly flow out of the cavities and open channels at the bottom surface of the panels, due to the contact between the bottom of the panels and grade. Freezing of this trapped water can deteriorate the stone at the cavities and channels.

The use of vein-cut travertine panels in pavements and other similar applications should be avoided.

Conclusions

1. Travertine is a form of limestone.
2. Travertine is not marble.
3. The minimum compressive strength, modulus of rupture, and flexural strength properties that are specified in ASTM C 1527-03 for exterior travertine are not achievable. These specified properties were derived from and are exactly the same requirements for marble that was specified in ASTM C 503-99.
4. In order for ASTM C 1527 to better reflect the actual range of properties of typical exterior travertine that has been successfully used in the exterior facades on buildings, the specified minimum compressive strength and the specified minimum modulus of rupture and flexural strength should be consistent with ASTM C 568 for "medium-density" limestone. These values are:
   (a) Compressive strength: 4000 psi (28 MPa)
   (b) Modulus of Rupture: 500 psi (3.4 MPa)
   (c) Flexural Strength: 500 psi (3.4 MPa)
5. Travertine has been successfully used in buildings for thousands of years. Travertine is mostly used in the exterior facades on buildings and at the interior of buildings as panels that are vein-cut to expose the natural cavities, open channels, and veins of the stone that many architects find attractive.
6. Freeze-thaw deterioration of normal vein-cut travertine panels in the exterior facade on buildings has not occurred because the natural cavities and open channels in the panels prevent the travertine from becoming critically saturated.
7. The evaluation of travertine panels on the exterior facade of several buildings by the author has revealed that as a material, the performance of travertine panels in the exterior facades on buildings has been successful. While some of the travertine panels on some buildings have developed distress conditions, these conditions did not occur as a result of natural deficiencies in the travertine material itself. They occurred as a result of improper connections between the panels and the structure of the building and from improper use of travertine.

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SECTION IV: DURABILITY

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Durability of Marble Cladding—A Comprehensive Literature Review

ABSTRACT: Natural stone has been used for centuries as building material. In historical time it was mainly used as load bearing elements, but within the past 50 years a new processing technique has made it commercially feasible to produce and use thin façade cladding. Unfortunately, a number of marble facades on buildings in both Europe and elsewhere have had serious problems with deterioration of the stone material. The TEAM (TEAM = TEsting and Assessment of Marble and limestone) project consortium represents nine European countries and comprises 16 partners, representing stone producers and trade associations, testing laboratories, standardization and certificate bodies, consultants, building owners and caretakers and producers of fixing and repair systems. The project had a budget of approximately 5 million dollars and was partly funded by the European Commission under the contract no. G5RD-CT-2000-00233. Two of the main objectives in the TEAM project were: — To understand and explain the mechanisms of the expansion, bowing, and loss of strength leading to degradation of marble and limestone clad facades. — To prevent the use of deleterious marble and limestone by introducing a draft for new European standards. This paper presents some of the important conclusions drawn from a literature review carried out within the TEAM project—and was based on an extensive review of literature on marble and limestone deterioration dating from the late 1800s to 2006 and the results of the TEAM project. The comprehensive information from more than 70 selected literature references is reviewed and discussed in order to describe the present knowledge on the causes and mechanisms responsible for the bowing and strength loss of thin marble cladding. In the following, the literature and TEAM findings are grouped under a number of headings proposed to explain observations. Thus, the information from the literature is compared and supplemented with the results from the TEAM project in order to present a good overview of the existing, most relevant, knowledge in the field. The literature review reveals that only few researchers have examined the durability problem from a broad perspective. In addition, no conclusive answer about the mechanisms and influencing factors can be given. The TEAM project has made it possible to identify several of the key influencing factors in marble degradation, the relative importance of various factors, and to gain a deeper understanding of the mechanisms involved.

KEYWORDS: marble, cladding, durability, limestone, Finlandia Hall, bowing, inspection, deterioration, thin veneer, design

Introduction

Numerous buildings with marble cladding facades have been subjected to investigations over the past couple of years [1–14]. The reason is that the marble cladding in some cases deteriorates; when exposed to weathering, some claddings are known to bow, expand, and lose their strength as shown in Fig. 1.

In 1991, Cohen and Monteiro [15] wrote a state-of-the-art review on the durability of marble cladding. One of their main conclusions was that the available tests at that time did not provide a reliable basis for a safe use of marble cladding. Another important conclusion was, that "*the design of marble cladding needs to be examined as an engineering problem in the area of structural mechanics and materials, rather than continue the architectural tradition of treating the specification of marble as primarily a design issue*."

Since then, a large amount of research (e.g., the TEAM project) has increased our knowledge significantly. This paper presents the development in our knowledge of durability of marble cladding based on an extensive review of literature on marble and limestone deterioration dating from the late 1800s to 2006. More than 400 articles have been screened and information was extracted from about 73 references. The

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FIG. 1—*Bowing of marble panels (1998) on the Finlandia Hall, Helsinki, Finland. Photo by Bent Grelk.*

references were found through a search in various literature databases, Internet search, and through a large network of scientist and technicians in Europe and America. A large amount of the screened references is thus not referred to in this paper for various reasons. The main reason was that the subject of the papers was found not to be directly related to the issue; however, some papers were also omitted because they described examples that the present authors chose not to present herein. This literature review revealed that only few researchers have examined the durability problem from a broad perspective and so far it has not been possible to present theories that explain all aspects of the marble deterioration. Thus, some controversy and uncertainties exist on the subjects treated here, where no conclusion can be drawn. This work presents the information from the literature, compared and supplemented with the results from the TEAM project in order to present a good overview of the existing, most relevant knowledge in the field.

Marble may deteriorate due to several processes. Much attention has been given to chemical and biological attack on old monuments and statues where it may cause serious decay [16–18]. However, in context with marble cladding on buildings the most serious problem is that of thermal hysteresis combined with moisture. This work deals with the aspect of marble durability and deterioration.

Natural stone has been used for façade applications for centuries. Originally, the stone was rather thick, when used as construction elements, and the durability was apparently good. Scientific research on properties of marble began in the late 19th century. In the years following, the thickness of natural façade stones decreased from over 40 in. to typically $1\ \frac{1}{2}$–3 in. as a result of new cutting technologies and equipment being developed by the industry. The advantage of the thin stone was savings in weight and cost. Even though most marble claddings perform satisfactorily, durability problems have begun to appear at an increasing rate after some 50 years of using thin cladding. Well known buildings such as the Amoco Building in Chicago, Richmond City Hall, Virginia, Houston Concert Hall, Florida, SCOR Tower in Paris, France, IBM Tower, Brussels, Belgium, and the Finlandia Hall in Helsinki, Finland have had their marble cladding replaced after less than 30 to 40 years at a cost of many millions of dollars. The deterioration gives a very conspicuous change in the appearance of the panels; they bow, warp, or dish.

The high profile cases with buildings suffering from severe problems with their marble façades have led to growing concern of its safe use, and many architects and building owners are unfortunately afraid of using marble as cladding material. This study has revealed that the marbles used for cladding may perform quite differently, even though they appear to be of the same quality and marble type, come from the same area, and are exposed to the same climatic conditions. We will emphasize that the technical quality (durability) of the marble can be established via laboratory tests.

The Problem

Scientists have been aware of the bowing of dimension stone for nearly 100 years [19–25]. Most reports about bowing are related to stone panels made of marble, but some examples of bowed limestones have also been described [26]. Few examples of bowed granite cladding have also been reported [26,27]. Laboratory experiments indicate that the mechanisms causing granite bowing probably are the same as in marble bowing [27].

Marble and limestone are carbonate rocks composed of mainly calcite and dolomite. The two rock types differ in their genesis, where limestone is a sedimentary rock, and marble is formed by metamorphosis of limestone during which calcite (or dolomite, or both) is recrystallized. The recrystallization of calcite is governed by the pressure and temperature conditions during metamorphosis and different degrees of recrystallization are therefore observed in different marbles. Thus, a range between unmetamorphosed limestone and completely recrystallized marble exist. Some of the members in this “range” of rock types are known to bow, when used as façade panels.

Even though most marbles tend to perform satisfactorily, numerous examples of marble bowing have been reported. It is mostly marbles consisting of calcite that are known to bow, but bowing of dolomitic marbles have also been reported [28]. It is not only fully metamorphosed marbles that exhibit bowing behavior, but also examples of partly recrystallized carbonate rocks [28] and unmetamorphosed limestone [25] have been found to bow.

Most cases of bowing involve marble from the Carrara area, simply because it is the most widespread and used marble type. It is, however, vital to emphasize that many building façades with Carrara marble perform well and furthermore that many other marble types from other areas all over the world (e.g., Vermont marble [29], Georgia marble [30], Porsgrunn marble from Norway [31], Christallino marble from Greece [32], Trigaches Excuro marble from Portugal [25]) also exhibit durability problems.

Marbles with varying grain size have also been found to bow. Erlin [33] stated in 2000 that “Marbles having coarser grain sizes and mosaic textures do not undergo similar thermal hysteresis effects as the Carrara marble because there are fewer crystals to interact.” Suenson [34], Winkler [4], Farrar [30], TEAM [28], and Alnaes et al. [35] observed, however, bowing in marbles with small or large grain sizes, or both, and Zeizig et al. [36] tested a range of different marbles, and found the same bowing potential in marbles with distinct grain sizes.

Deterioration of marble panels involves several parameters and properties. Shape deformation (Fig. 2) is the most obvious phenomenon, where the panels bow either convexly or concavely out of their original plane. Along with bowing follows also permanent volume changes (Fig. 3), i.e., the marble expands. In laboratory experiments, thermal cycles resembling that of temperate climates have been shown to result in permanent expansions of certain marble types of more than 0.2% after only 50 thermal cycles [37].

However, the most serious deterioration feature is the loss of strength (Table 1) which may progress as far as total decohesion of the grains (known as the sugaring effect). The above three features may cause bowing of panels (Fig. 2), spalling and cracking in connection with anchor points (Fig. 3), and in the worst cases, ultimately failure of the panel.

Laboratory studies performed in the TEAM project [28,37,43] clearly indicate that there is no correlation between the amount of bowing and the loss of strength. This is especially worrying since there is a potential risk of severe strength loss without any evident bowing of claddings.

Cases

A wide range of buildings with bowing problems has been reported in the literature [2,6,8,11–15,22,24,25,28,30,32,34,44–58]. An example of bowing marble is seen at the National Bank of Denmark, Copenhagen built in 1965–1978. The building was designed by the Danish architect Arne Jacobsen and has an exterior cladding made of Norwegian marble. The marble type, which is a contact-metamorphosed calcitic-silicic limestone with remnants of fossil corals, is because of its beauty very often

FIG. 2—*Bowing of marble panels on Zagrepcanka business tower in Zagreb, Croatia. Photo by Jan Anders Brundin.*

used as façade cladding on buildings in Scandinavia and Germany. However, after some 30 years of use the panels on this building show medium deformation in the order of 10–15 mm, the most bowed panels are situated on the southfacing façade, which is shown in Fig. 4. A loss in flexural strength of approximately 40% has been recorded on 35-year-old panels taken down from the south façade [41].

Serious bowing is observed on La Grand Arch de la Defénce in Paris, France from 1989 (Fig. 5). The building, which was designed by another Danish architect Johan Otto Von Spreckelsen, is covered with Carrara marbles from Italy. After only 15 years of exposure, the panels show a high degree of deformation [49]. Most panels are bowed 20–30 mm (3/4–1 3/16 in.) out of their original plane, and a total replacement of the cladding is expected in the near future.

Due to the risk of panels falling down from the building, a net has been placed on the upper (windy) parts of the façades, and the area close to the south façade has been closed to the public.

In the city of Malmo, Sweden, the house of “Sydsvenska Dagbladet” is also covered by Carrara marble. Even though the building is only approximately 20 years old the cladding exhibits severe deterioration as shown in Fig. 6. Numerous failures at anchor points, broken stones and stones falling down have been observed. The original marble cladding was replaced in 2005 [28,31].

FIG. 3—*Expansion of the marble panels creating cracking and spalling around the anchoring points, on a building façade in Malmo, Sweden. Photo by Bent Grelk.*

TABLE 1—*Examples of reported strength loss (flexural strength) of marble claddings from different buildings.*

Building	Marble Type (Origin)	Loss of Flexural Strength	Reference (Literature)
Finlandia Hall—Old façade Helsinki (FI)	Bianco Carrara (Italy)	~85 % after 21 years	[38]
Finlandia Hall—New façade Helsinki (FI)	Bianco Carrara (Italy)	~20–30 % after 3 years	[39]
Amoco—Chicago (USA)	Bianco Carrara (Italy)	~40 % after 15 years	[40]
Office building—Nyköping (S)	Bianco Carrara (Italy)	~75 % after 31 years	[28,31]
Office building—Köln (D)	Christallino (Greece)	~65 % on west and south façades, and ~10 % on north façade after 20 years	[32]
Hospital—Lünen (D)	Trigaches E. (Portugal)	~49 % after 14 years ~75 % after 28 years	[3]
Office building—(CH)	Bianco Venato (Italy)	~40 % after 3 years	[8]
Bank building—Copenhagen (DK)	Porsgrunn (Norway)	~40 % after 35 years	[41]
Office building—Copenhagen (DK)	Porsgrunn (Norway)	~75 % after 41 years	[41]
Office building—Lyngby (DK)	Marmorilik (Greenland)	~45 % after 60 years	[41]
Office building—Malmö (S)	Bianco Carrara (Italy)	~10 % after 25 years	[42]

Also, many other types of marble exhibit the same type of deformation problems. The façade of the hospital in Lünen is currently being renovated because of very strong bowing (20–50 mm (3/4–2 in.)). A Portuguese calcitic marble covers the building [3]. In 1991, a loss of flexural strength of the order of approximately 75 % was measured on the 28-year-old panels from parts of the façade.

Some parts of the University in Göttingen, the Juridicum and Oeconomicum have exterior cladding of a Swiss calcitic marble, which is also deformed in the magnitude of 20–30 mm (3/4–1 13/16 in.) [11].

The marble façade on Hotel Terraza in Ljungby, Sweden, clad with a Swedish dolomitic marble is now facing a renovation of about 1.5 million US$ due to deterioration (bowing and cracking) of the marble panels [28].

The 450-ft high Richmond's City Hall, Virginia, built in 1972, had similar façade problems with its approximately 23 000 pieces of marble panels. The marble used on the building was a coarse grained Georgia white marble [30]. Due to severe deterioration and bowing of the marble panels on the building, approximately 5000 fiberglass straps and a system of fiberglass corner supports were installed for safety reasons, in 1995, over the face of the building to temporarily ensure that all marble panels remained in place. In 2005 the original marble panels were replaced by a metal panel system.

One of the "worst" examples of deterioration (bowing and loss of strength) of marble panels can be found in Zagreb, Croatia, where one of the most extreme degrees of bowing and loss of flexural strength has been observed [28] (see Fig, 2). The 310-ft tall building, Zagrepcanka, is clad with white Carrara marble. About ten years after the completion in 1976, marble panels started to fall off from the façade.

FIG. 4—*Part of the south façade on the Danish National Bank. Photo by Per Goltermann.*

FIG. 5—*La Grand Arch de la Defénce in Paris. Photo Bent Grelk.*

Because of the risk of falling marble panels, the users of the building had to enter the building through an improvised tunnel made of wooden planks and steel bars (see Fig. 7). The building got the nickname “The hells tower.” The renovation of the façades started recently (2003–2005).

The above-described examples of bowing are not typical for marble cladding. In most cases there is actually no or only minor deformation observed.

A very illustrative example of marble performing well is seen in the city of Malmo located in the southern parts of Sweden. Here, the City Hall (Fig. 8) is located next to the above-described house of “Sydsvenska Dagbladet” (Fig. 6). It is also covered with the Italian Carrara marble Bianco and is furthermore of the same age (25 years). However, the marble panels of the City Halls are still in great shape, no bowing is recorded and the flexural strength is reduced by only 10% [12,28].

The City Hall of Borås, Sweden, which is more than 40 years old, also have façade cladding of Carrara marble, and no deformation has been detected. Another example of a durable façade with Carrara marble is a Bank in Brussels, Belgium [31], which is located only about 500 meters from the bowing façade panels of the IBM tower.

The City Hall of Aarhus, Denmark has an exterior façade of the Norwegian marble from Porsgrunn. The building is from 1934, but no serious deformation has been observed on this building.

The façade of Lyngby City Hall, Denmark is also in great shape (Fig. 9). This building is from 1941 and is covered with a calcitic marble type from Marmorilik, Greenland.

FIG. 6—*Part of the facade on an office building in Malmo, Sweden. Photo by Bent Grelk.*

FIG. 7—*An improvised "security" tunnel in front of the building, Zagrepcanka, Zagreb. Photo by Jan Anders Brundin.*

The Finlandia Hall constitutes the most famous and controversial example of marble façade clad bowing. The building, designed by the famous Finnish architect Alvar Aalto, was built between 1967 and 1971/72. The façades on the Finlandia Hall were clad with approximately 7000 m^2 of Carrara marble panels. The maximum size of the original panels was 140 cm and they were 30-mm (1 3/16 in.) thick, and each panel was fixed by four dowels, two on each side in the vertical joints. The flexural strength of the original marble was approximately 8.2 MPa [40].

Within a few years after its completion in 1972, the marble panels began to bow severely [39,44,50,59]. The problems with the marble façade were many: Deterioration of the marble panels

FIG. 8—*A Carrara marble façade on Malmo City Hall, Sweden in perfect condition. Photo by Bent Grelk.*

FIG. 9—*Lyngby City Hall, Denmark. The 60-year-old marble façade is still in very good condition. Photo by Bent Grelk.*

resulting in (1) bending of panels, (2) surface deterioration, (3) cracking and spalling around fixing points, (4) "inferior" appearance, and (5) safety problems. In 1983 some of the panels have bowed approximately 50 mm (2 in.) and the flexural strength was decreased by more than 50 %. In 1991, an angular marble panel fell off, and it was found that the flexural strength had decreased to 1.2 MPa, which was less than 15 % of the original flexural strength of the panels. It was found [44] that the flexural strength decreased and the bowing of the marble panels increases apparently almost linearly as a function of time. White metal braces and nets were then attached to secure the marble façades until the restoration could begin in 1999.

An intense debate began in the city of Helsinki, Finland, over the selection of a new cladding material [39,60]. But in 1997, after many discussions and meetings, the citizens of Helsinki approved a decision to use Carrara marble—again.

The requirements on the new marble façade were, besides that the appearance should resemble the original: (1) the flexural strength of the marble should be more than 9.0 MPa. (2) The fixing system should be flexible and allow for movements, and the dowels should be moved from the vertical joints to the horizontal joints. (3) The maximum size of the panels was reduced by 20%. (4) Better insulation and ventilation.

The completion of the recladding was finalized in 1999. However, after only six months it became clear that these criteria were insufficient in that the bowing of the new marble panels became apparent. Only a few years after installation measurements showed that the renovated walls of Finlandia Hall had deteriorated, and the reduction in the flexural strength of the marble was in the order of 20–30% [39].

The bowing behavior of the new marble panels is remarkably different from the original panels even though they consist of almost the same type of white Carrara marble. The original marble panels on the façades of Finlandia Hall bowed all almost uniformly concavely (inwards) (Fig. 10) with extremely high amplitudes—65 mm (2 1/2 in.) in 1989—after less than 20 years of exposure, but after the new panels were installed they started to bow in a convex direction (outwards) (Fig. 11) [12]. No explanation for the opposite bowing behavior has yet been established.

Observations

We can, from the literature, conclude that the phenomenon of bowing of marble is actually rather common. Deformation by bowing is experienced in buildings of various ages, in buildings exposed to various weather conditions, and for slabs of various thickness and dimensions and with different anchoring methods. It is also interesting to note that bowing is registered for marble of seemingly very various composition and structure.

In the TEAM project about 200 building projects with marble or limestone cladding around the U.S. and Europe have been recorded, and about 50 of these have bowing problems. Measurements of bowing amplitudes during the inspections of buildings in the TEAM project were carried out with a so-called "bow-meter" in accordance with the procedure described in NT BUILD 500 [61] (Fig. 12). The "bow meter" is basically a 1200-mm straight edge with a digital dial gage that allows the distance from the edge to the panel surface to be measured very accurately.

FIG. 10—*Part of the old marble façade on Finlandia Hall in 1999. Photo by Elmar Tchegg.*

Climatic Condition

There is no evidence that any particular climate is typical of the conditions that can result in long-term bowing, expansion, and loss of flexural strength. Cases of bowing have been reported from the most different climates, from Libya (Africa), Lebanon, and Cuba [12,21] in the south to Canada, Sweden, and Finland in the north. A daily temperature variation and a source of moisture are, however, common to all locations, see Fig. 13.

From most of the investigated buildings it is observed that the absolute temperature and the temperature variations seem to be very important factors for the bowing mechanism, i.e., the higher the temperature variations, the higher the degree of bowing. The humidity is also a key factor for bowing. Laboratory tests have shown that bowing only occurs when free water is available. "Dry" bowing and expansion tests only result in a small limited expansion and no bowing [37].

Dimensions of the Stones

Bowing has been observed for all panel dimensions (largest dimension ranges from 900 to 2000 mm) and all thicknesses ranging from 30 to 60 mm (1 3/16 to 2 1/3 in.) (see also Ref. [62]). Very large marble

FIG. 11—*Part of the new façade on Finlandia Hall in 2006. Photo by Bent Grelk.*

FIG. 12—*Measurements of bowing amplitudes on a marble façade with the use of a "Bow-meter." Photo by Björn Schouenborg.*

panels (>2 m^2) have been recorded with a perfectly plane and unaffected surface, while on other buildings, small (<1 m^2) panels have deformed, deteriorated, and fallen from the façade [12].

No correlation has been observed between bowing tendencies and the stone panel thickness [12]. Erlin [33,63] mentioned an optimal thickness for bowing to occur, but it has not been possible to verify this during the literature review. On the contrary, several examples of relative thick tombstones (up to 15-cm (6-in.) thick), which were strongly bent, have been observed around the world [21,25,28,29,34,43] indicating that no optimal thickness exists.

The inspections performed during the TEAM project [28] and the reports from the literature clearly show that larger panel dimension (width and height) will increase bowing, since the bowing amplitude (U) is related to the length (width) (l) and the thickness (t) of a panel: $U=\Delta\varepsilon l^2/8t$, where $\Delta\varepsilon$ is the difference in strain from the front to the rear side of the panel [28].

The loss of strength of smaller panels is of the same magnitude, even if the smaller panels show less visible bowing amplitude. The strength loss is, however, not equally significant because the smaller panels usually are affected by smaller external forces. An expanding panel with no visible bowing will also lose strength.

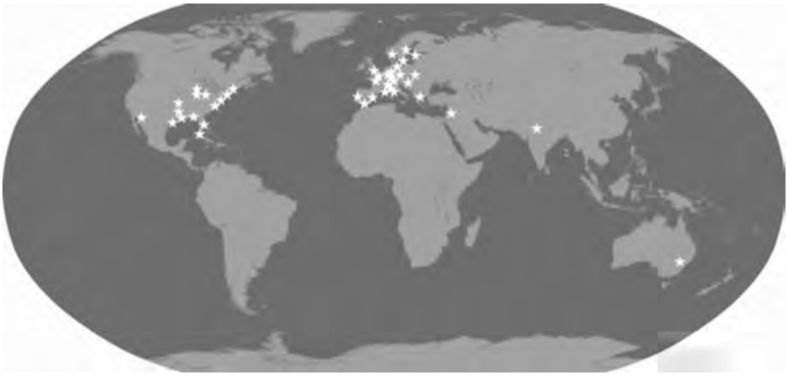

FIG. 13—*Geographical distribution of marble bowing registered by TEAM.*

The bowing and expansion lead to increased stresses at the fixing points, and the dimensions can therefore often influence the resulting forces that need to be transferred and thus to the risk of failures.

Façade Orientation and Height Above Ground

For the main part of the cases studied in the TEAM project, bowing was observed on all façade directions as well on all heights above ground. However, the observations indicate that there was a significant difference in the bowing amplitudes depending on the orientation of the façade. It appears that the bowing phenomenon is less significant on façades exposed to minor temperature differences during the day. The smallest bowing amplitudes are usually found on the north façades (in the Northern Hemisphere), because these are not exposed to large temperature cycles from the sun. Zimmermann [32] reported a case with a 20-year-old building in Köln, Germany, clad with a Greek marble. On the west and south façade the marble had lost 65 % of its original flexural strength, whereas the loss of strength on the north façade was "only" 10 %.

In some cases, a more pronounced bowing was observed on the east or west façades than on the south façades. A reason for this could be that the lower position of the sun in the east and west may cause larger temperature variations on a vertical façade during the day, compared to the high position of the sun in the south. Besides this, the sun angle varies from north/south to the equatorial zone, and this may give some explanation as to why the bowing behavior differs between different geographical zones (latitudes). The local environment/climate has also been identified as an important factor, where shade from, e.g., trees or adjacent buildings, provide shelter from the sun resulting in smaller temperature variations and thus a smaller degree of bowing.

For many of the buildings inspected during the TEAM project, it has been observed that the bowing phenomena increase in frequency with the height above ground. This may be explained as the local variation in wind, humidity, and temperature often becomes more extreme with the height above ground.

Age of Panels

Bowing and expansion will increase with age of the panels, and it is preliminarily assumed that these will develop proportionally to the exposure time. The inspected buildings and the literature show that already after a few years of exposure ($<5-10$ years), bowing and deterioration may occur in panels.

Some researchers [38,44] actually supports the assumptions that bowing (and strength loss) is linearly correlated with the age and time of exposure. However, the observed rate of development varies from one panel to another, even in the same building and from one marble type to another [28]. This rate can to some extent be determined by measuring the amount of bowing and the joint width on a larger number of panels.

Repeated measurements of, e.g., joint width or bowing, will over a longer period of time provide documentation for the expansion rate in the actual building. This may be obtained by permanent, long-term monitoring or by repeated inspection with 2 to 5-year intervals using a bow-meter. The concept of measuring the joint widths for a range of panels with 2 to 5-year interval was tried at the Danish National Bank in 2001 and 2004 where the additional three years exposure corresponded to 10–15% increase of the exposure time, but this indicated no decrease of the joint width over this period [28].

Fixing System and Cladding Design

Most of the inspected façades in the TEAM project have 30 or 40-mm thick marble panels mounted on various types of anchors and with a ventilated air cavity of 20 to 40-mm (3/4 to 1 1/2-in) width between the back face of the stone panel and the layer of insulation. Some façades are, however, installed in mortar beds with or without restraint anchors—this means that there is no ventilated cavity. Some projects have part of the installation done on freely standing railing systems allowing both faces of the panel to be "exposed" to the surroundings. To date, no link has been observed between the type of anchoring system and the bowing of marble panels with the exception of panels bedded in mortar which do not seem to be susceptible to bowing to the same extent as other fixing systems.

Bowing, concave and convex, is observed for all fixing systems represented in the investigated buildings. So far no specific fixing system can be pointed out as being more (or less) suitable to prevent bowing, expansion, loss of strength, or the development of damage.

Open/Closed Joints and Joint Widths

The use of open joints may lead to larger variation in the environment behind the panel, compared to the design with closed joints, where the air behind the panels may be sealed in and isolated from the external influence. No effect of this has been verified by the inspections in TEAM. However, façade claddings with elastic sealant in the joint will often reveal if an expansion has taken place.

For most of the buildings it is observed that the average widths of joints are smaller than the designed width. If a panel is strained when free movement in the joints is not allowed, normal forces will occur in the panel or it may lead to a pushing aside of the neighboring panels. The restraining may affect the shape of the bowing panel, e.g., bowing takes place predominantly in one direction and damage typically occurs near the fixings.

Concave or Convex Bowing, or Both

It has not been possible to explain why panels bow concavely or convexly on a building, nor is it possible to relate the bowing behavior to one specific influencing parameter. Panels on one side or in one location of the same building may deform in one direction, while on another side or location they deform in the opposite direction, see Figs. 14 and 15.

One theory put forward by some researchers [33] was that panels exposed in the same way and to the same environmental conditions, for example, on part of a certain façade would bow in the same direction, but it has repeatedly been observed that some panels bow opposite than the rest [28]—also on the same part of a façade.

It has in addition to this been observed that in cases where old panels were replaced with new fresh panels, the direction of bowing of the new panels were opposite the demounted panels, which is the case with Finlandia Hall, where the new panels are bowing convexly, while the old original marble panels were bowing concavely.

Bowing Shapes

It has been observed that the unrestricted bowing shapes are part of a spherical curve [28,63]. The shape is seldom an ideal spherical shape due to the fact that the material has different properties in different directions (orientation of foliation), or the panels are or have been restrained to some extent by the fixings or neighboring panels, or both.

Color of Marble

Among the investigated building projects exhibiting bowing problems, marble types with different color are represented such as light types from Italy, Switzerland, Russia, Macedonia, and Sweden, a green type from Portugal, dark gray types from Portugal and Norway. It shall be stressed that the same types of marble on other reported projects do not show any bowing. Thus there is no clear indication as to the effect of color range of the marble related to deformation by bowing.

Surface Finishing and Treatment

It has been discussed in the TEAM project, if the surface finish, the cutting processes, any chemical surface treatment, or the start conditions in the beginning of a panel's lifetime may influence the bowing behavior. Possible surface finishing could be mechanical (rough, smooth, or honed) or chemical (impregnation, soap, etc.), and the cutting process could be more or less "rough."

No conclusion concerning finishing could be reached from the building inspections of the TEAM project. However, the laboratory tests, including bowing tests, clearly indicate that the surface finishing could influence the bowing behavior on certain marble types.

FIG. 14—*Zagrepcanka building in Zagreb with concave bowing on the lower part and convex bowing of the upper part of the façade. Photo by Jan Anders Brundin.*

Pollution

The solubility of calcite in water is relatively small, but becomes significantly higher in acids. SO_2 oxidizes to SO_3 and may react with calcite to form gypsum [64,65]. NO_x is also known to accelerate weathering of carbonate rocks [66]. Furthermore, the cyclic process of dissolution and precipitation may create greater porosity and cracking.

However, Royer-Carfagni [50] found it hard to believe that CO_2, NO, NO_2, SO_2, and SO_3 in air could have any significant influence on the bowing of panels on the Finlandia Hall. "*Close observation reveals that the presence of traces of calcium sulphate crystals is limited to within a few grain-diameters distance from both sides of the façade surface. This consideration could indicate that chemical aggression should be considered a surface phenomenon.*" This observation is in accordance with findings in the TEAM project [28].

Fabric/Orientation of Foliation

On most of the investigated buildings in the TEAM project the orientation of foliation could not be observed except for one building. On the Oeconomicum in Göttingen it was possible to investigate the cutting direction with respect to the metamorphic layering, foliation, or macroscopic folding and state that it had influence on the bowing amplitudes and shapes [11,28].

FIG. 15—*Zagrepcanka building in Zagreb. Close-up of the façade with panels showing significant different bowing behavior. Photo by Jan Anders Brundin.*

Long-term Monitoring System

Among all the cases studied in this literature review only two examples of long-term façade monitoring system has been recorded. One system was developed during the TEAM project [12,28] and a second system was installed on Finlandia City Hall [67]. However, no final conclusions have until now been presented concerning these two systems.

Design Guidelines for Marble Claddings

European standards for stone products have been developed by the European standardization body (CEN), mainly within its technical committee (TC) no. 246: CEN TC 246 Natural Stones.

There are two principally different types of standards, standardized test methods and product standards. The latter defines the requirements on stones for different applications and also requirements on factory production control and CE-marking.

In Europe two sets of standards exist for outdoor façade cladding of natural stone. EN 12326 specifies a test program for façade panels of slates, while EN 1469 deals with the remaining stone types, including marble and limestone.

In some cases, when research projects have been granted a funding, they can provide new knowledge and raise the standardization work to a higher level with better and more relevant test methods and better specifications. The TEAM project is one example that has provided new knowledge to the above-mentioned product standards for cladding (EN 1469). The TEAM project has, e.g., produced two proposals for new test methods and presented them for TC 246: (1) a bow-test (accelerated laboratory aging) for façade cladding of marble, and a test for (2) moisture and thermal induced expansion for design of anchoring and dilatation joints.

The TEAM project has focused on the problem of marble panels that lose strength and, in some cases, display large bowing. The bowing is a function of differential expansion. The expansion of all stone panels, when exposed to elevated temperatures in the presence of water, may cause cracking around the anchoring. In the worst case they will fall down.

There are still no common European criteria for the evaluation of performance versus a certain climate and application. The performance of a marble in one climate cannot always be correlated to its performance in another climate since the fundamental mechanisms of, e.g., stability problems (bowing and expansion), frost damage, salt crystallization, etc., are not fully resolved.

There is still a serious need for guidelines on the use of marble for façade cladding. Many indications point in the direction that many of the above problems may be avoided if proper testing is carried out in the selection of the stone for new building/construction projects. The results from the TEAM project will hopefully lead to a revision of the European product standard for façade cladding of natural stone in the nearest future.

The choice of a natural stone must be based on proper examination of its physical and mechanical properties in relation to the intended architectural or structural function and its expected performance over the entire anticipated life of the construction. The aesthetic considerations are of secondary importance.

In cases where several types of natural stones are to be used it is important at the design stage to understand the nature and characteristics of individual types of stone. The mechanical properties, weathering characteristics, durability, and appearance are all material considerations when deciding which stone type to use in a particular application.

The choice of natural stone for vulnerable areas is especially important as it is here that failures or disfiguration are most likely to occur. Steps and paving, plinths and base courses, cornices, as well as decorative elements are all prone to attack and decay at a far greater rate than simple vertical cladding. Harder-wearing and more resistant stones are generally better suited to such vulnerable areas.

Durability and Conclusions

Cohen and Monteiro [15] stated that marble is particularly susceptible to acidic atmospheres, because it will initiate a series of oxidation reactions in the marble. This type of degradation of marble is a well documented fact, especially in old monuments and statues. However, the marble degrades from the surface and inward at a fairly slow speed, and cannot cause the magnitude of strength loss in the relatively short period of time; that is the reality for some marble façade claddings.

Another general perception of the past on marble deterioration is that freeze-thaw cycles cause significant decay of marble. This theory is not supported by the many observations made on buildings in the TEAM project, where marble bowing with subsequent strength loss has been found in warm (even tropical) climates [21,49] without frost action as well as in more temperate climates [11,28,32].

Today, it is generally accepted that deterioration of marble façade cladding is mainly caused by temperature variations in combination with moisture presence in the pore space of the marble. Some controversy still exists on the actual mechanism, but these extrinsic parameters (moisture and temperature variations) are generally recognized to be the key influencing factors causing the fast weakening of some marble facades.

It has long been known that only some marbles exhibited bowing behavior, while others remained intact over time, and that the main reason for this is in the structure of the marble. In the literature there are many observations indicating that the durability of marbles is closely associated with its microstructure. Already in 1940 Bain [29] found in his investigations of Vermont marble that marbles with straight boundaries between calcite grains tend to be much more susceptible to bowing and degradation compared to marbles with irregular interlobate grain boundaries. This observation has also been put forward recently by Cantisani [68], Molli [69,70], Barsotelli [71], Åkesson [72], and Royer-Carfagni [47].

In order to correlate the performance of the marble to its microstructure, Åkesson et al., [49] have worked out a method to numerically describe the different microstructures and have achieved a very good correlation between bowing observed on buildings and microstructure.

It is anticipated that the test methods (1) a bow-test (accelerated aging) [73], including loss of strength, for façade cladding of marble, and (2) moisture and thermal-induced expansion for design of anchoring and dilatation joints, developed by TEAM, will become European standards in the near future for all marble and limestone claddings [28].

Future Work

Guidelines for choosing, testing, and production of marble and limestone panels as well as a product control must be established. The importance of choosing a technically suitable marble for outdoor claddings cannot be too strongly emphasized.

If a marble is chosen that rapidly will lose strength the consequences are considerable—for example, in terms of increased maintenance costs due to the fact that such a marble will quickly get an open micro-structure that is more susceptible for soiling. Removal of graffiti will be more difficult and, of course, the risk of failure due to loss of strength will make it necessary to monitor the changes and finally replace part, or all, of the façade before the risks of panel failure becomes unacceptable.

Technically acceptable properties should therefore have very high priority when choosing a marble type for a building project, whereas today aesthetical properties are often considered as being of greatest importance even though the aesthetic properties will change rapidly for a nonsuitable marble as it deteriorates.

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Testing and Assessment of Marble and Limestone (TEAM)—Important Results from a Large European Research Project on Cladding Panels

ABSTRACT: The use of natural stone as facade cladding has been shown to have much lower life cycle costs and they are more environmentally friendly than comparable products of concrete, glass, and steel. Promoting the use of natural stone has therefore a great positive impact on the environment. However, the number of occurrences of bowing and expansion of marble and limestone panels has led to increased maintenance costs, significant safety risk, and negative publicity. The lack of knowledge of a solution to the problem of bowing marble has a large negative effect on the entire stone trade. In response, short-sighted and less durable construction solutions are used as an alternative, adding to the decreasing export figures and numbers of employees within the stone sector. The TEAM (TEAM=TEsting and Assessment of Marble and limestone) project addresses a problem with marble types, from several European countries, that display bowing on facades in both cold and warm climates. There is, therefore a need to develop harmonized European standards for differentiating between marble that is susceptible to bowing and marble that is not. Resolution No. 013, in May 1999 taken by the European Committee for Standardization (CEN), Technical Committee (TC) 246 Natural Stone states the urgent needs "to develop a direct test method of the bowing risk for marble cladding products." Thus, the project addresses the mandate for external wall coverings and the safety of panels. This paper serves to give a comprehensive overview of the main findings in the project. The main objectives were: • To understand and explain the mechanisms of the expansion and loss of strength, probably the most important phenomena leading to degradation of marble and limestone clad facades. • To prevent the use of deleterious marble and limestone by introducing drafts for European standards. • To develop a concept for assessment of facades, including a monitoring system in order to predict strength development and improve safety and reliability. • To analyze if surface coating and impregnation could prevent or diminish the degradation. • To address quality control aspects in order to optimize the production conditions. The TEAM project consortium, representing nine EU (European union) countries, comprised sixteen partners representing stone producers and trade associations, testing laboratories, standardization and certification bodies, consultants, building owners and caretakers and producers of fixing and repair systems. A state-of-the-art report has been written and is based on an extensive compilation of more than 400 papers on marble and limestone deterioration dating from the late 1800s to 2006. A survey of about 200 buildings has given a clear picture of the extent of the problem in geographical, geological, and climatological terms. Detailed case studies of six buildings have resulted in a methodology for assessment of facades including monitoring system and risk assessment. Research both in the laboratory and the field were performed on a large number of different stone types from different countries and used in different climates. This gave the explanation of degradation mechanisms and led to the determination of the critical influencing factors. Two tests methods, including precision statements: one for bowing [1] and one for thermal and moisture irreversible expansion have been prepared for submission to CEN TC 246. Repair techniques based on the use of surface coating and impregnation systems has been tested at laboratory and in the field. Positive side effects including increased durability and easier cleaning have been observed. Guidelines for production and product control have been proposed, and an instruction for stone sampling and description has been developed.

Introduction

The TEAM project is by far the largest European R&D project ever directed towards natural stones used for building applications. It has focused on the problem of expanding and bowing marble and expanding limestone cladding for outdoor uses. The main problem with unsuitable stone types is failure at anchoring points and the decrease of strength over time that is always associated with the bowing expansion, or both.

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The basis for the project results are, in addition to personal knowledge, a literature review of more than 400 articles, a survey of about 200 buildings, and a comprehensive program of laboratory and field work. The project has rendered numerous findings that are useful for stakeholders such as the stone industry, European standardization, testing and research organizations, building owners, consultants, and architects. The most important ones are discussed below.

The project started in March 2000 and ended in August 2005 and has had sixteen partners from nine different countries and a budget of about 5 million U.S. partly financed by the European Commission. The problems are clearly of interdisciplinary character, while the project engaged experts from all parties concerned, e.g., stone producers, trade associations, standardization bodies, building owners, consultants, testing and research laboratories, universities, and caretakers and producers of anchoring systems, surface treatments, and repair systems.

Project Elements/Work Packages (WP)

Literature Study—WP 1

The starting point of the project was a literature study aiming at collecting and evaluating more than 100 years of research relating to this specific problem. The existing hypotheses were compiled and evaluated and many of them tested in the laboratory phase of the project.

Some of the most common hypotheses/statements were:

- Fine-grained marble is good or bad (both statements occur) [2–5].
- Carrara marble is bad.
- Acid rain and pollution are the reasons for the problems.
- Frost action is the cause and marble should not be used in the “far north” e.g., in Finland.
- Marble with a specific range in strength is suitable [6].
- Anisotropic thermal expansion of calcite and dolomite causes granular de-cohesion [4,7].
- The influence of moisture (and possibly free water) and temperature variations are crucial [8].
- The release of locked-in rock stresses is important.
- A complex microstructure is favorable [5,6,9–13].

The only hypotheses generally valid are numbers 7 and 9. Number 6 contributes to the problem. However, without the presence of water/moisture the order of magnitude of degradation is quite small.

It has become very clear, through the findings of the project, that the major influencing parameters are thermal expansion in a wet condition and the microstructure of the marble. See more under Section II E below. A comprehensive literature review has been carried out and is also included in this special volume [14].

Survey of Stone Projects—WP 1

In parallel to the literature survey, about 200 buildings using natural stone were identified, mostly in Europe (Fig. 1), but also in other parts of the world [15]. Most of these buildings were clad with marble (both “good” and “bad” performing) and limestone. It has provided a unique opportunity to study and test many of the hypotheses on real buildings and not just in the laboratory. This has given us knowledge of the extent of the problem, both geographically and geologically. Suitable marble types can survive for many years in any climate (e.g., more than 100 years). It is equally clear that unsuitable marble comes from many countries all over the world. The building owners also provided TEAM with the possibility to continue with detailed studies on a few of them. The geographical, geological, and climatic spread of the 200 buildings is representative for all of Europe!

Detailed Cases Studies—WP 2

Six of the two hundred buildings were chosen for detailed studies. These buildings represent different climates and marble types: Danish National Bank in Copenhagen (DK), City Hall in Nyköping (SE), City Hall in Malmö (SE), Magenta Hospital in Turin (IT), University Theologicum in Göttingen (DE), and St. Marien Hospital in Lünen (DE) [7,15,16]. A special piece of equipment (the bow-meter) which can be used for high precision and repeatable measurements of the bowing magnitude of the panels has been devel-

FIG. 1—*Distribution of buildings in Europe, where bowing or distortion of the marble cladding has been recorded by the TEAM partners.*

oped. In addition to the parameters listed below, the influence on the bowing of the design, the anchoring system, the building physics (like ventilation and insulation), façade orientation, etc., were studied. The studied parameters included:

- Height above the ground.
- Climatic conditions, including microclimate.
- Cracks and breakouts in panels.
- Cladding design.
- Anchoring system.
- Dimension and thickness of panels.
- Open or closed joints.
- Width of the joints.
- Fabric/orientation of foliation of the stone.
- Convex or concave bowing.
- Surface finishing and surface treatment.

The detailed study of these six buildings, the approximately 200 buildings surveyed and the literature review led to the final selection of the marble types for the major laboratory research work in WP 5. A methodology for site investigations and also for sampling was developed. By including the results of WP 5 and WP 7 it has been possible also to give recommendations for remedial actions. One of them is to minimize the influence of water by applying a suitable surface treatment. The development of the laboratory bow-test (described later in this article) has clearly shown that each marble and building is unique. Every marble has its unique degradation curve (Fig. 2). The actual buildings and marble types are given in TEAM Final Technical report and Ref. [14]. Any prediction of the remaining service life for a specific building has, therefore, to be developed individually and depends on the specific marble type in combination with the microclimate (especially surface temperature variations).

The influence of different anchoring systems was checked and it could be concluded that there is no significant influence on the bowing phenomena as such. However, mortized panels seem to have a lower tendency to bow. This may be due to the smaller temperature gradient developed in such a construction. This observation was supported by monitoring on a test wall installed in Waldactahl, SW Germany (Fig. 3). In addition, the fischerwerke system with undercut anchors can also be recommended since it requires less strength to keep the panels in place compared to kerbs or pins/dowels.

Long-term Monitoring—WP 3

In addition to the test wall at fischerwerke, monitoring equipment was installed on three of the six buildings mentioned above. The following parameters were monitored:

- surface temperature on the external surface of the stone
- time-of-wetness/condensation on external surface of the stone
- strain in two directions on the external surface of the stone

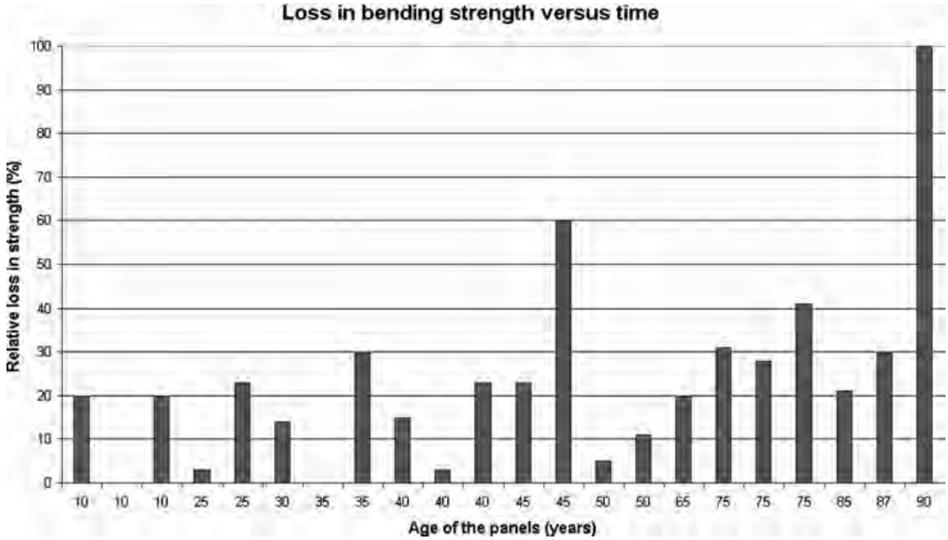

FIG. 2—*The relative loss in flexural strength, and modulus of rupture, recorded from different marble clad buildings. The initial strength values are generally data provided by the producer.*

- surface temperature on the internal surface of the stone
- air temperature in the gap behind the panel
- relative humidity in the gap behind the panel
- time-of-wetness/condensation on internal surface of the stone
- strain in two directions on the internal surface of the stone
- shade air temperature
- shade relative humidity

The bowing magnitude was measured four times a year with the bow-meter and correlated with the strain measurements. The monitoring has mainly generated input to the definition of the laboratory bow-test and the wet-expansion test and a greater understanding of the diurnal variations in temperature of the panels. By filtering these small scale changes it has been possible to establish a very good correlation between measurements of long-term residual strain and the manual measurements of the bowing magnitude with the bow-meter. A very important finding is that manual measurement on one occasion by use of, e.g., the bow-meter will provide almost random values due to the diurnal changes that can be up to 2 mm/m. Measurements of the bowing magnitude therefore has to be done on repeated occasions under similar conditions!

At one location the monitoring has been combined with an *in situ* test wall of different marble types, thicknesses, and chemical treatments (Fig. 4). See also Section II H below.

FIG. 3—*Test wall at fischerwerke in Waldactahl, SW Germany. One marble type in combination with five different anchoring systems were installed.*

FIG. 4—*Test site at the city hall of Nyköping. Different marble, different thickness, and chemical treatments are tested together with monitoring of temperature and strain.*

One very important deliverable from the combined work of monitoring, field inspection, and field exposure is the development of a methodology for risk assessment. The methodology includes the combination of results from laboratory tests and *in situ* nondestructive testing to enable a prediction of the remaining service life of existing panels.

Sampling and Influencing Parameters—WP 4

The case studies of buildings with suitable and unsuitable marble provided most of the input for the selection of marble types to include in the major laboratory research program. It was impossible to be completely sure of the exact quarry location for the marble on every building due to incomplete documentation. In total, 16 quarries have been sampled, giving 17 varieties of carbonate rock types. The samples represent 3 pure dolomite marbles, 11 pure calcite marbles, 1 ophicalcitic marble (containing serpentine), 1 limestone, and 1 silicate rich, contact metamorphic limestone. In addition, 86 marble types were sampled on different occasions at exhibitions, fairs, etc. See Table 1 below. These samples were used in the laboratory bow-test for evaluating the bowing potential and provided the additional necessary samples in order to ensure a broad geological spread of marble types in the laboratory test program.

The possible influence of quarrying and productions processes, together with geological aspects, were assessed in order to include such variables in the samples and at the same time increase the possibilities of explaining the test results. Detailed sampling and sample marking instructions were developed in order to ensure full traceability of all laboratory samples. Very few of the marble types had a visually observable foliation and these were sampled with respect to that in order to enable the assessment of this parameter's influence on the test results.

TABLE 1—*The origin of screening tested fresh samples.*

Country	No. of Samples
Italy	36
Greece	10
Portugal	13
Sweden	8
Norway	7
Greenland	1
Turkey	1
Bulgaria	1
France	1
USA	5
Macedonia	1
Germany	1
Spain	1
Schweiz	1
Austria	2
Total:	88

Full Scale Laboratory Testing, Including Quarry and Processing Variables—WP 5

Rock stress measurements were carried out in three quarries in the Carrara area. The original hypothesis for the rock stress measurements was that we would expect high stresses or a high anisotropic stress pattern in areas where strong bowing material is extracted and low stresses in localities where the quarried marble has not shown problems with bowing. The *in-situ* stress measurements clearly indicate that the marble in all test sites is subject to quite high, nongravitational stresses. The stresses are probably a combination of locked-in residual stresses and tectonic stresses. The results of the actual rock stress measurements show that our original hypothesis was somewhat simplified. High stresses are measured in all marbles, even in the two marbles where no bowing have been experienced. For these two quarries, it seems that the stresses are released during block extraction, which is not the case in the quarry where they have experienced bowing marble. In addition, some rock mechanical properties (Young's modulus and sonic velocity) of the bowing marble are much lower than the other two.

A summary of the findings related to quarrying and processing are:

- Some production and processing factors influence the bowing and deterioration pattern of marble cladding. The various factors depend to a large degree on the intrinsic properties of the marble itself.
- Even though various quarrying methods may give various impacts on the extracted rocks, it is very difficult to explain the different behavior in marble cladding as a result of the quarrying methods.
- There is no indication that diamond wire sawing introduces stresses within the rock material that may enhance the rock's bowing susceptibility. A marked stress relief may be experienced during diamond wire sawing, and not an introduction of stresses (or stress concentrations) as suggested in the literature.
- A relationship between the rock fabric on the one hand and bowing and expansion (and strength), on the other hand, has been found for some marble types. The cutting direction may therefore have an effect on the bowing and expansion potentials if the fabric of the marble is strongly anisotropic. However, it is most important to point out that marble types that have been found unsuitable for outdoor cladding should not be recommended regardless of extraction and processing directions. The decrease in strength may be equally high in such panels although in another direction and may therefore probably constitute an equally high a risk. In addition, it is, in most cases, not economically justifiable to experiment with different blocks in the natural stone industry.
- No conclusive answer can be given as regards the degree and speed of deterioration as a function of panel sizes. But thickness has been found to influence the bowing and deterioration of marble. A thick slab of unsuitable marble will bow eventually, but takes a longer time than a thin slab of the same marble. However, there is no safe thickness of slabs above which bowing will not occur. For a marble with favorable rock and mineral properties, the behavior is less dependent on thickness.
- Sawing, honing, and bush-hammering may cause slabs to bow during processing, typically towards the worked face. It has been verified that different marbles respond differently and that there is a relationship between the behavior during processing and the mechanical properties of the marble. Less stiff marbles with hysteresis in the stress/strain curve during unloading seem to be the most vulnerable. There is also a tendency for such marbles to be more prone to bowing as a result of stresses induced into the material from external factors like temperature and humidity gradients.

One of the most important findings in the project is the development of the possibility to quantify the microstructure of suitable calcitic marble types for cladding purposes. Adjacent grain analysis (AGA) is a quick and simple way of assessing whether a marble is likely to be suitable or not. AGA is a combination of grain size analysis and grain shape analysis. It will not quantify the magnitude of bowing or the strength loss that will occur over the years. But it will give a reliable basis for the decision whether it is justifiable to make more analyses, e.g., the laboratory bow-test, the wet-expansion strength tests, frost resistance testing, etc. Thus far, we have only been able to validate the method for calcitic marbles.

A lot of tests have been carried out trying to find out the potential relationship between lattice preferred orientation and bowing, and a clear correlation has been established. However, the influence on the magnitude of bowing is quite low for the marble types with a clear orientation. The difference is actually lower than the precision of the test method, but as it is systematic it cannot be ignored.

A huge number of other tests have been carried out, such as water absorption, capillary suction, flexural strength, breaking load energy, frost resistance, ultrasonic pulse velocity, long-term bending, heat

FIG. 5—*The same marble can display different bowing patterns on the same building. Horizontal panels are concave and the vertical ones are convex.*

capacity, heat conductivity and specific heat. One of the first important findings was the absolute necessity to dry marble and limestone samples at lower temperatures than those normally specified. Most standards specify the conditioning of stone samples to be done at 70°C. TEAM has observed a significant reduction in strength at considerably lower temperatures. In order to minimize the damage of the test specimens before testing we therefore recommend drying at only 40°C for one week. This procedure has been used throughout the project.

Many of the results from these tests have contributed to the verification or refuting of the numerous pre-existing hypotheses. The possibility to compare a result obtained in the laboratory with that observed on a building or at the field exposure sites has been extremely valuable. Many results and relationships obtained in the laboratory cannot be proven to be consistent with observations in the field simply because of the complex nature of the problem, with several influencing factors acting at the same time. The same marble has been observed to bow in both convex and concave directions on the same façade and in very nearby position (Fig. 5). In places where the temperature variations are less extreme, parameters such as fabric may play a more pronounced role. On the south-, east- and west-facing sides of the building, where the temperature generally is the highest, temperature is by far the single most important influencing factor.

One of the main objectives of the project was to try to determine the mechanism for the bowing. However, even though a lot of progress has been made we feel that there is still some work left before we can give a full explanation of the mechanism. However, this additional work may not be economically justifiable from an industrial point of view, particularly since TEAM has developed a test method that correlates very well with the performance of marble cladding. It is easy to say that granular de-cohesion is causing the expansion and the bowing. However, we are still unsure as to why this phenomena does not take place in all marble types and why it is more often found in calcitic than dolomitic marble. The differential expansion in different crystal lattice orientation is a property that marble shares with many other rock types and cannot explain the phenomena by itself, although it is seen as an important contributing factor. Note that one bowing granodiorite has actually been found in Ljubljana, Slovenia. However, the reason for this bowing is not yet elucidated.

A crucial parameter is the complexity of the grain boundaries and the grain size distribution of mineral grains in the rock (Fig. 6). This provides different bonding strength between the mineral grains due to the complexity of the arrangement of the grain boundaries in combination with the crystal structure. The irregular grain structure that all marble types suitable for outdoor cladding have in common is the product of the metamorphism that turned limestone into a marble, combined with a dynamic recrystallization event. This type of event or metamorphism will create a more or less irregular microstructure with complex grain boundaries and a wide span of grain sizes and a crystal lattice that may include defects.

This structure gives a stronger grain boundary bonding through a mechanically stronger contact (com-

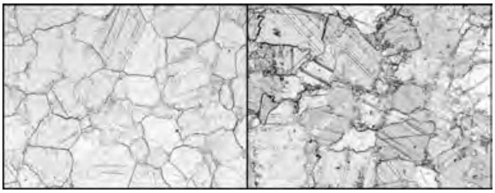

FIG. 6—*The favorable microtexture of the marble to the right is caused by complex grain boundaries in combination with a range of different sizes of the grains. The marble to the left is prone to bowing due its quite simple structure with, more or less, mono-sized grains and straight grain boundaries [17,18]. The image corresponds to an area of 1.3 by 1.0 mm^2.*

plex grain boundaries) and a larger contact area, with more bonds between individual grains, through a larger surface area compared to the "ideal" regular hexagonal crystals. The type of mineral grain/crystal with complex shapes and lattice defects, mentioned above, is more prone to a change through redistributing individual atoms than a defect-free ideally-shaped hexagonal grains/crystal. This is due to the fact that such crystals have a higher inner energy, due to the strain energy, compared to ideally shaped ones. We may look on these defects as internal stresses built up during a dynamic recrystallization. Stresses that want to be released can do so without primarily affecting the grain boundaries. The triggering event can be exposure to the elevated temperature and moisture cycling on a façade. The ideally-shaped crystals don't have the same ability (or willingness) to rearrange atoms within the lattice and are therefore primarily reduced to movements along the grain boundaries causing granular de-cohesion. Furthermore, crack propagation as well as expansion and coalescence of voids can take place more easily in a structure with straight grain boundaries. The hypothesis described above is logical from a mineralogical point of view but should still be verified by laboratory testing on a nanoscale.

The influence of the water is also not totally clear but we know that stresses are built up both during the wetting and drying processes and we know that most marble types with a bowing potential have more open grain boundaries compared to the nonbowing types (Fig. 7). This enhances the ingress of water and larger capillary forces to act on the micro-scale for the unsuitable marble types, both during wetting and drying. Furthermore, the energy needed for crack initiation and propagation is lower in a wet rock when compared to a dry one. The reason for this is that the energy increase is higher when a solid-solid contact is replaced with a solid-air contact compared to a solid water contact.

Once the microstructure is open it is also easier for more water to enter the stone, water that may cause frost damage during the winter time. In addition, the weathering through acid solutions from polluted rain is enhanced in an open structure.

The great complexity of the problem is the most likely reason for many of the misconceptions in previous projects. Tests on too few marble types, in combination with the lack of opportunities for validating the results in the field, have often resulted in erroneous or unsubstantiated conclusions.

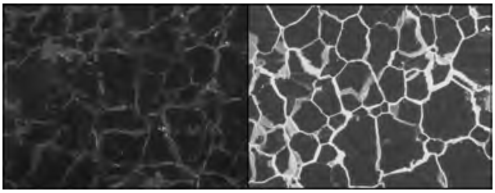

FIG. 7—*Fluorescent microphoto of nonbowing marble to the left and a bowing one to the right. The marble to the right has a high proportion of cracks between the individual grains. The image corresponds to an area of 1.3 by 1.0 mm^2.*

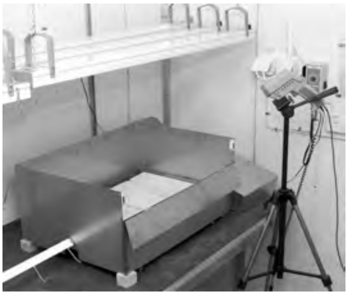

FIG. 8—*Laboratory bow-test equipment for combining a wet underside with cyclic heating from above. Sample sizes are 30 by 100 by 400 mm.*

Development of the Bow-test and the Wet-expansion Test—WP 6

The work to develop test methods to determine the bowing potential and the temperature expansion in the wet condition crossed two work packages (WP 5 and 6). It was clear from an early stage of the project that a moisture gradient is needed to obtain bowing. This phenomenon had previously been observed on many flooring installations where marble tiles have bowed soon after the placement in a grout. However, this bowing is not permanent. The bowing we can observe on the buildings is to a large extent permanent and caused by elevated temperatures in combination with the moisture gradient. A laboratory test method was therefore developed to try to simulate these conditions. It is necessary in order to reproduce bowing in the laboratory to create the same conditions as on a façade (see Fig. 8). The detailed building inspections, together with the long-term monitoring and the literature study, have given the necessary information about the maximum temperature of a stone surface on a façade (at least in Europe). About 77°C (171°F) has been measured on a vertical panel of dark limestone and approximately 60°C (140°F) on a white Carrara marble [19]. Normal temperatures on a white marble on a sunny summer day is about 55°C. It has also given information about the "ramp time," that is how fast the maximum temperature is reached and the time for the cooling of the panels. The resulting test method for bowing has produced results that correlate very well with the observations in the field. One important finding during the design of the method was the need to be able to repeat the temperature (T) cycle in every test and to ensure equal "climatic" conditions for any test specimen regardless of marble type, color, etc. The only way to achieve this was by using a black reference in accordance with ISO 4892-1 (Plastics—Methods of exposure to laboratory light sources). Note that it is the climate that is controlled and not the temperature of the stone!

The second test method developed is an expansion test that correlates better with the observed performance in the field than currently available standard methods. People responsible for designing a building generally favor a very small joint between stone panels. This joint is called a "dilatation" joint and is used to allow for some movement between the individual panels without causing any breakage. The usual input to the design of such a joint has previously been the "thermal coefficient of expansion" in the dry condition. The problem is that building façades are not always dry and the expansion in the wet condition is generally significantly higher than in a dry condition (not only for marble and limestone). Thus we frequently see damages like that in Fig. 9. Bowing is also a differential expansion and the causes of the two phenomena are, therefore, directly linked. The information input for the expansion test and the bowing test from other parts of the project is thus similar. It has proved insufficient to merely condition test specimens in different relative humidity. The test is, therefore, based on water saturated samples (Fig. 10).

Both test methods were used in an inter-comparison trial in accordance with the requirements stated in ISO 5725:1994 [20]. The repeatability (r) and reproducibility (R) were calculated accordingly.

Bowing Test—It should be noted that the conditions were not identical in all laboratories and the test results therefore vary more than expected. Both the stones and the bowing method are very sensitive to differences in temperature and not all the laboratories had the possibility to install the black reference during the project time. Slightly different layouts of the equipment were also reflected in different test results. However, it is clear that all laboratories have been successful in discriminating between bowing

FIG. 9—*Cracking at anchoring points due to excessive expansion in relation to the dimensions of "dilatation" joints.*

and nonbowing marble through this test.

Note that there is, in general, one clear difference between the bowing direction observed *in situ* and results of the laboratory bow. In practice the bowing direction is almost "arbitrary," while in the bowing test, the specimens always bow towards the heat (i.e., upwards). This phenomenon is difficult to explain but has also been observed by other researchers [21–23].

Expansion—One major problem with the expansion test relates to the practicalities of carrying out the test. For example, despite many trials with different types of glue, some of the installed measuring points tend to detach from the stone surface during the expansion in hot water. It is therefore important to start with a large number of test specimens. This is a common problem shared with many other expansion methods, e.g., expansion of concrete and mortar. Similar to the bow-test, the expansion test can be further refined but it has already proved to be able to give clear guidance on the suitability of a marble (and limestone) type for cladding. In addition, it provides an important input to the dimensioning of the "dilatation" joints. Another important finding is that some marble and limestone types tend to show a continuously increasing expansion while others display a limited expansion.

Both test methods are complemented by the determination of flexural strength after the temperature cycling. This provides a clear indication of the potential strength decrease over time and so provides an important input to the prediction of the service life. However, it should be noted that if a more precise estimate of the service life is needed, it is necessary to know the temperature and temperature variations of

FIG. 10—*Laboratory test for moisture and heat-induced expansion, providing relevant data for dimensioning of dilatation joints. Sample sizes are 30 by 30 by 200 mm.*

FIG. 11—*Field exposure site in Ljubljana, Slovenia. Sample sizes are 30 by 400 by 500 mm.*

the stone surface on the actual locality of the building. If a bowing marble has been chosen it may also be necessary to determine the frost resistance of that stone before and after temperature cycling.

Field Exposure and Possibilities to Prevent the Bowing or Decrease the Speed of the Aging—WP 7

Five field exposure sites have been installed in Sweden, Poland, Slovenia (Fig. 11), Italy, and the U.K. and one *in situ* on the City Hall in Nyköping (E Sweden) [24]. The test sites have provided essential information about the behavior of different marble types, and the influence of thickness and of chemical treatments, e.g., surface coatings and impregnation.

It has been concluded that all marbles, both calcitic and dolomitic, showed a degree of bowing that could be measured under field conditions; however, the magnitudes differ greatly. Comparable bowing was observed in all climatic zones of the participating countries with Itq2 (Gioia from Carrara) panels displaying the highest bowing and deterioration potential after one year's exposure at every field exposure site. Itq2 was the most sensitive marble but other types such as the Thassos marble, from Greece, showed slight bowing and especially cracking independently of the climate. It is essential to point out the substantial difference between bowing magnitude of the same samples measured at field conditions and after drying indoors (used as reference measurements four times a year). All results from *in situ* measurements gave higher bowing compared to measurements on dried samples. In some cases, marble panels showing bowing when measured on the field exposure sites showed no signs of bowing after drying in indoor conditions. This indicates that no permanent bowing could be observed for these panels. Therefore, it is recommended that bowing measurements taken outdoors should be taken when the temperature variations are as small as possible (summer and winter) and that the measurements are repeated on a number of occasions.

Analyses of how thickness and impregnation influence the bowing properties were performed on one type of fresh marble, Itq2, and on the old panels on the façade in Nyköping. The analysis of variance indicates that bowing depends on the thickness and impregnation. The highest bowing was observed for the 20-mm (3/4-in.) thick panels and lowest for the 30 mm (1 3/16 in.) impregnated thick panels. Both impregnation agents were shown to have an inhibiting effect on bowing process after one year of exposure. However, after three years of exposure it was clear that only the microcrystalline wax (AGS) was preventing bowing. Impregnation with a hydrophobic treatment decreased the bowing tendency of the fresh panels. The results from the field exposure were comparable to results from the laboratory experiment on the old panels from the Nyköping. The AGS impregnated samples showed small or no bowing at all. This effect could not be observed for the GS impregnated samples. No aesthetic change could be observed as a result of the impregnation process. Hydrophobic treatments had an inhibiting effect on the bowing and could be recommended as a remedial action for damaged panels, thought not as a final solution. It is important that each case is investigated independently.

Guidelines for Production and Product Control—WP 8

Guidelines for the production of panels and product control have been drafted. They are primarily directed towards the producers and suppliers, but also to designers. The importance of choosing a technically suitable marble for outdoor claddings is strongly emphasized. If a bowing marble is chosen or one that rapidly will lose strength, the consequences are considerable—for example in terms of increased maintenance costs due to the fact that such a marble will quickly develop an open micro-structure, especially in the surface, that is more susceptible for soiling. Removal of graffiti will be more difficult and, of course, the risk of failure due to loss of strength will make it necessary to monitor the changes and finally change part, or all, of the façade before the risks of panel failure becomes unacceptable. Technically acceptable properties should therefore have very high priority when choosing a marble type for a building project; whereas today aesthetical properties are often considered as being of greatest importance even though the aesthetic properties will change rapidly for a nonsuitable marble as it deteriorates.

The recommendations to the producers are to provide a geological map of their quarry and request a petrographical analysis including the AGA for all areas identified as significantly different. They should then carry out the laboratory bow-test and the wet expansion test to establish a correlation between those parameters and the petrography. By using nondestructive tests it should be possible to identify major changes at the rough block stage of production and thereby avoiding unsuitable material entering the processing stage. A detailed instruction for marking and identification of material at all stages is also strongly recommended for the sake of traceability and feed back to the production company. The sampling of test materials is very critical for any project. Detailed sampling and sample marking instructions have to be used for any sampling. Our findings, guidelines for sampling, have been reported to CEN TC 246 Natural Stone. Guidelines have also been given to designers and producers/suppliers to ensure a proper selection of suitable marble and limestone for outdoor cladding and to ensure production with a homogeneous and acceptable quality, respectively.

Dissemination—WP 9

A number of presentations and proceedings have been prepared for various conferences [19,21,24]. In addition, publications in scientific magazines and parts of doctoral theses are also results of the project. See the TEAM homepage for more information and download possibilities of brochure and technical reports (with test results), test methods, etc.: www.sp.se/building/team.

Conclusions

The main findings and conclusions are given below:

- Bowing is a worldwide phenomenon not confined to one type of marble or one type of climate, e.g., frost action is not necessary for this phenomenon to occur.
- The driving force of the degradation is the combination of elevated temperature in the presence of a moisture gradient. The latter is a crucial ingredient!
- Every marble is unique and has a unique response to these climatic stresses with its own degradation curve (i.e., loss of strength versus time). The acceleration factor of a laboratory bow-test is therefore different for different marble types.
- The most crucial intrinsic parameters are the degree of complexity of the grain boundaries combined with the grain size distribution of mineral grains in the rock. This provides different bonding strength between the mineral grains due to the complexity of the arrangement of the grain boundary and in combination with the crystal structure. The irregular grain structure that all marble, considered suitable for outdoor cladding, has in common is the product of the metamorphism that turned limestone into a marble combined with a dynamic recrystallization event. Weaker bonds will cause granular decohesion, "sugaring," of the marble and significant strength losses. Of the utmost importance is that this microstructure can be quantified. Limiting values can therefore be established.
- The laboratory test methods developed in TEAM enable a relevant evaluation of whether a marble is suitable for outdoor cladding or not and how to dimension dilatation joints for marble and limestone cladding. The bow-test can also be adapted and used for predicting the remaining service life of a specific marble on a particular building.
- One time *in situ* measurements of bowing amplitude, etc., are of little use due to large diurnal and seasonal variations. Repeated measurements therefore have to be carried out in order to enable a reliable risk assessment of a damaged façade.
- It is possible to inhibit or decrease the degradation of marble by coating the surface with a hydrophobic treatment. The effect is most pronounced on marble already exposed and it should not be used to support the selection of an unsuitable marble for a new building project. Whether or not this is a cost efficient solution has to be decided on a case by case basis.

The TEAM project has given our knowledge of marble and limestone deterioration processes a significant push forward and we hope that our findings will contribute to an increase in the use of marble and limestone for cladding and thus help in regaining some of the trust lost in these materials in particular climates and countries.

Acknowledgments

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Comparison of Field Testing with Laboratory Testing of the Durability of Stone

ABSTRACT: Samples of three granites (Texas Pink, Mount Airy White, Academy Black), one marble (Carrara) and Indiana Limestone have been exposed to 11 years of natural weathering on the roof of a building in the Chicago area. Twice a year, the samples are visually reviewed. In addition, the changes in physical properties have been monitored using non-destructive test procedures for determination of the dynamic (sonic) modulus of elasticity. The sonic modulus measurements were recorded using procedures as outlined in ASTM C 215, "Standard Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens." The sonic modulus measurements were compared to measurements of similar stone samples that were exposed to laboratory accelerated weathering testing. The exposed and protected surfaces have recently been examined petrographically, and the changes due to the natural weathering recorded. Based on this additional testing and evaluation, conclusions can be drawn which further indicate the use of accelerated weathering. Testing can effectively evaluate the long-term changes in the mechanical and physical properties of dimension stone when exposed to natural weathering.

KEYWORDS: Mount Airy, granite, sonic modulus, Texas Pink, Academy Black

Introduction

Durability can be defined as the ability to withstand wear or decay with time. No stone resists the action of atmosphere agencies indefinitely. In nature, the surface of the earth has a continuous cycle of changes due to the action of these atmospheric agents. Thus buildings and historic movements composed of natural stone are subjected through time to mechanical loads, temperature variations, and chemical attack, which can lead to degradation. Degradation is indicated by damage in the form of microcracks. Microcracks can be defined as cracks ranging in scale from microcracks (not visible to the untrained eye) to macrocracks (readily visible), and in the form of mineral degradation (such as feldspar alteration to clay). These microcracks and mineral degradation show up as increased absorption which manifests itself in greater porosity and lower strength.

This paper will show that a laboratory test can be used to estimate the degradation of stone. Comparisons are made which show the strength changes and microstructure changes are similar for the laboratory test and long-term exposure in a temperate climate, see Fig. 1. This work was begun during a research project for the National Association of Marble Producers at Armour Research Foundation (now the Illinois Institute of Technology Research Foundation) in 1958. The durability work has continued over the intervening years and is presently being supported by Wiss, Janney, Elstner Associates, Inc. as part of our internal research program. The original test work was based on the British Department of Scientific and Industrial Research, Building Report No. 18, The Weathering of Material Building Stones, by R. J. Schaffer. This document was published in 1937. Information used for the analysis given in this paper has been taken from work published in previous papers by the author of this paper, see Bibliography at the end of the paper.

Background

Building materials of natural origin are liable to considerable variation in quality; dissimilar types of decay may occur in different parts of the same building or even in different parts of the same block of stone. Thus

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FIG. 1—*Location of weathering regions.*

the selection of material for testing is difficult. This is the reason why test specimens should be taken from a variety of blocks to be used on a job. This allows for a statistical interpretation of the data. The weathering resistance of a material depends on its physical and chemical properties, which are expressions of the lithology and geological history of the stone. The effect of certain weathering agents is comparatively well understood and the result of laboratory experiments designed to demonstrate the effect of those agents are capable of being stated, at least approximately in terms of weather resistance.

The original test procedure attempted to include the various atmospheric conditions which the stone would encounter as follows:

1. Temperature changes
2. Rain
3. Acid
   (a) Sulfur compounds
   (b) Nitrogen compounds
4. Wind abrasion due to dust particles
5. Drying

Based on the above information a test program was developed which consisted of 30 cycles of the following exposures:

1. 1/2 hour water soak at room temperature
2. 1 hour drying at 170°F
3. 1/2 hour at room temperature
4. 1 hour fly ash abrasion at 30 psi
5. 2 hour acid dip, 0.02 molar H_2SO_3
6. 1 hour salt dip, 5 % NaCl
7. 1/2 hour forced air drying, room temperature

TABLE 1—*Results of durability study.*

Marble Type	Modulus of Elasticity $\times 10^{-6}$, PSI				Modulus of Rupture, PSI								
	Static	Dynamic	Dynamic After 30 Cycles	Change %	Standard Bars	Small Bars	Small Bars After 30 Cycles	Change %	Weight Change %	Increase in Absorption Percent	Change in Volume cc	Loss of Gloss, 30 Cycles	Remarks
Verde Antique	14.85	11.90	10.80	9.24	3820	3394	3149	7.22	0	0	+6.70	5.55	Polished surface remained
Clarendon White	4.33	4.88	2.32	52.46	1494	1543	515	66.62	0.73	76	−0.30	70.00	Slight sugaring
Cedar Tavernelle	9.45	9.80	5.20	46.94	1900	1869	1550	29.83	0.61	125	−10.00	90.76	Sugared coarse appearance with raised crystals
Florence	10.40	11.19	4.42	60.50	2160	2149	1508	29.83	0.59	58	+4.47	76.78	Smooth surface with veins raised
Batesville	9.15	9.01	5.34	40.73	2392	2399	2206	8.04	0.036	52	+11.37	90.19	Smooth surface with stylolites raised
Cliffdale	9.45	8.76	3.35	61.76	1224	1190	897	24.62	0.67	73	+5.04	84.00	Smooth dull surface
Ozark Famosa	7.25	7.35	2.41	67.21	1559	1741	1001	42.50	0.076	84	+13.00	89.28	Smooth dull surface
Emerald White	5.95	3.21	3.16	1.56	1385	793	639	19.412	0.72	125	−0.20	92.30	Excessive sugaring with pitting
Etowah	9.00	8.71	3.26	62.56	1990	1995	958	52.00	0.53	86	+0.80	77.00	Sugared course appearance raised crystals
Alabama White	10.40	12.09	6.91	43.11	2705	2844	1704	40.0	0.56	61	−0.90	72.00	Surface smooth with pattern raised
Tennessee Pink	10.60	10.84	6.06	44.06	2427	2308	1357	41.00	0.64	0	+1.80	91.00	Dull surface, stylolites crystals slightly raised
Cherokee	7.25	8.31	3.58	56.87	1640	1699	747	56.00	0.61	111	−15.20	78.00	Roughed crystal surface
Imperial Danby	7.43	8.02	2.38	70.52	1240	1403	638	54.00	0.56	122	−6.10	75.00	Sugared poor appearance
Tennessee Gray	10.40	11.26	7.67	31.89	2665	2799	2137	24.00	0.60	100	−9.30	85.00	Smooth dull surface
Ozark Tavernelle	9.45	10.58	7.99	24.53	2132	2345	1980	16	0.62	0	−11.70	82.00	Smooth dull surface

TABLE 2—*Changes in dynamic modulus at four and eleven cycles of heating and cooling.*

Marble Type	Number	Decrease in Resonant	Frequency Percent
Dovelle	194A	22.93	27.47
Ozark T.	205A	16.54	19.70
Silvetto	245A	9.64	14.77
Cedar T.	84A	27.32	34.91

8. 1 hour water soak at room temperature
9. 16 hours at −10°F

This test procedure was used to test 15 different commercial marbles. The results of this work are given in Table 1.

In order to monitor changes in the properties of the marble specimens being exposed to cyclic testing, the sonic-modulus method was used (ASTM C 885 Standard Test Method for Young's Modulus of Refractory Shapes by Sonic Resonance). This nondestructive method is used to a great extent in concrete research for measuring the effects of freezing and thawing on the properties of the concrete. The system requires the need to determine the resonant frequency, a size constant, and mass of the sample. The mathematical relationship is as follows:

$$E = C_1 w f^2$$

where C_1 is a shape constant, w=mass (weight) of specimen, lb, f=fundamental flexural resonance frequency, and b=width of specimen.

The shape factor C_1 can be found in ASTM C 885. Table 1 of the standard provides the C_1 shape factor based on the ratio $1/t$ where 1 is the length of the specimen and t is the thickness of the specimen.

The test cycle described in the early part of this section is cumbersome and each cycle lasts 24 hours. During the basic tests, observations indicated which aspects of the test were affecting the stone properties and other aspects were not. An experiment was initiated which consisted of subjecting the stone to eleven cycles of 170°F and −10°F. Both of these temperatures were observed on the surface of stone exposed on buildings in the Chicago area. Table 2 lists the results of this experiment. The strength loss appears to be greatest during the first few cycles and tends to stabilize with additional cycling. A second set of tests were performed with the full series of exposures and the results of the eleven cycles are shown in Table 3. Notice that the results of the exposure to the five agents are very similar to the same stone exposed only to freeze/thaw cycles. These tests led to the accelerated weathering test where the stone was exposed to 300 cycles of freezing and thawing in tap water.

At the same time similar specimens were exposed on the roof of one of the ARF Research Buildings. The purposes of these tests were to check the real time sonic modulus curves obtained over a long time period and compare them with the accelerated weathering data, Figs. 2–5 include twelve of the fifteen stones described in Table 1. They were exposed to natural weathering for eight years. The stone in Figs. 3–5 have been exposed for eleven years and are part of an ongoing study. Figure 6 shows how the different stones were exposed to real time weather. Note the similarity between the general marble long time exposure curves (Figs. 2 and 3). These will be compared to the field measurement in a later section.

An addition analysis was performed to determine the relationship between some elastic modulus strength, Fig. 7. The angle of the relationship between strength and sonic modulus is 40°. A 45° angle would indicate a 1 to 1 relationship. Based on the 40° measurement the strength to modulus relationship is approximately a 1+1 relationship.

TABLE 3—*Accelerated weathering of marble—eleven cycles.*

Marble	Number	Decrease in Resonant Frequency %	Decrease in Modulus of Rupture %	Increase in Absorption %	Increase in Volume %	Decrease in Weight %
Dovelle	195A	38.50	44.44	168.00	0.729	0.20
Ozark T.	204A	19.15	34.50	70.56	0.784	0.18
Cliffdale	216A	42.39	35.24‘	19.97	1.250	0.26
Ozark F.	226A	55.82	32.62	32.86	1.170	0.41
Silvetto	244A	12.92	21.90	39.24	0.706	0.03

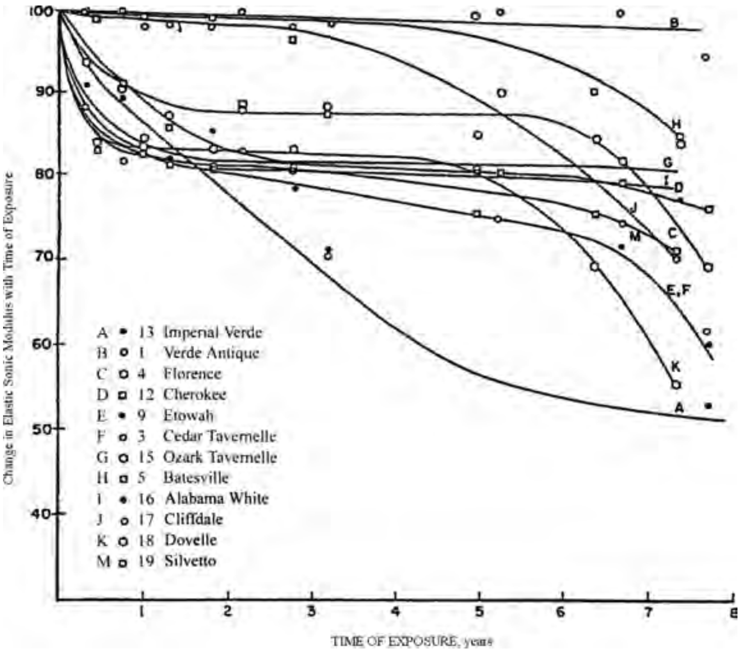

FIG. 2—*Outdoor weathering studies.*

Field to Laboratory Measurements

In the previous section the history of the development of a laboratory test for the durability of dimension stone was discussed. To establish a base line for comparison of exterior weather testing was presented which established the observation of the environmental deterioration of dimension stone using a nondestructive technique (measurement of sonic Young's Elastic Modulus) could be used. In this section results of a laboratory accelerated weathering test will be shown and comparisons will be made with the environmentally weathered stone. Before the comparisons are made we would like to point out that stone taken from different parts of the same quarry will not always test out the same.

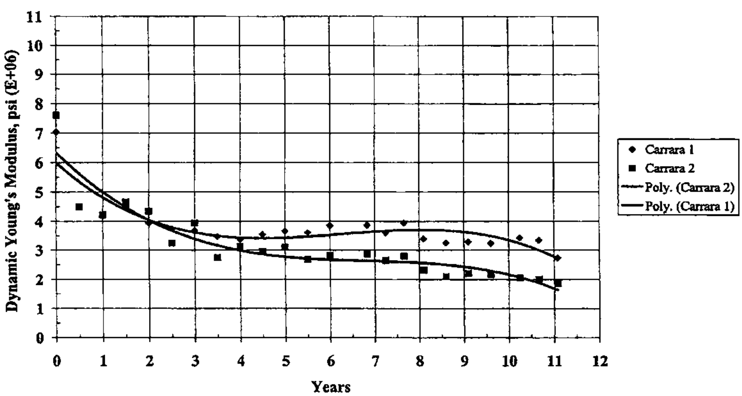

FIG. 3—*Natural weathering studies—Marbles.*

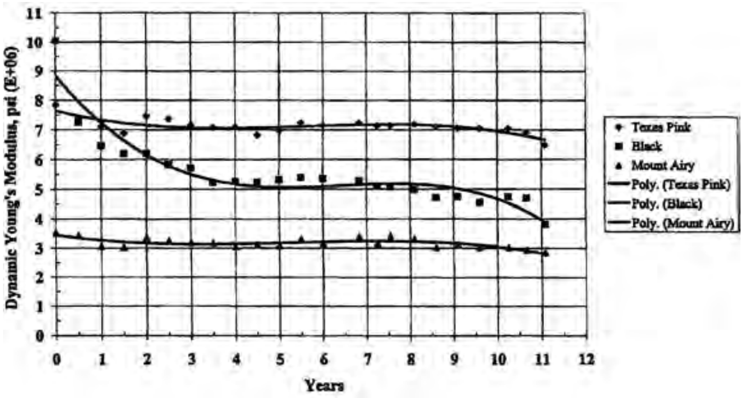

FIG. 4—*Natural weathering studies—Granites.*

Figure 8 shows the results of performing sonic modulus on Jura Stone. Jura Stone technically is a limestone which can be produced with a high polish. Table 4 provides the strength relationship of the Jura Stone taken from different parts of the quarries.

The locations in the quarry are shown in Fig. 9. The figures and graph are an example of how natural material properties can vary even when quarried within short distances between areas of removal. Examination of Fig. 8 not only shows approximately 33 % strength variation but differences in durability.

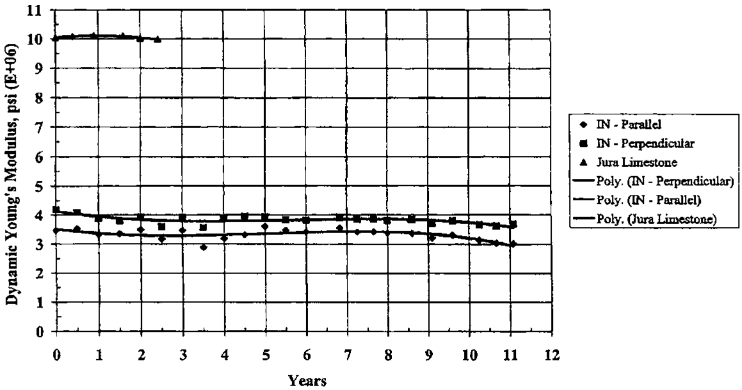

FIG. 5—*Natural weathering studies—Limestones.*

FIG. 6—*Roof top exposure of stone specimens.*

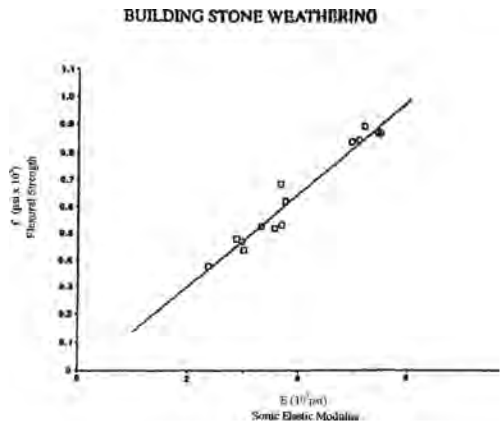

FIG. 7—*Modulus of elasticity (E) versus flexural strength (f) determined from marble specimens.*

Figure 10 is a plot of sonic modulus taken during exposure of Carrara Marble samples to accelerated weathering. Comparing this curve to the curve in Fig. 3, the sonic modulus of Carrara Marble for natural weathering and accelerated weathering are very similar. The accelerated weathering curve is beginning to lose strength at 200 cycles. Figure 11 is the accelerated weathering curve for Danby Marble and this curve can be compared to the plot in Fig. 2. This is shown in Fig. 12. The comparison plots are only good up to

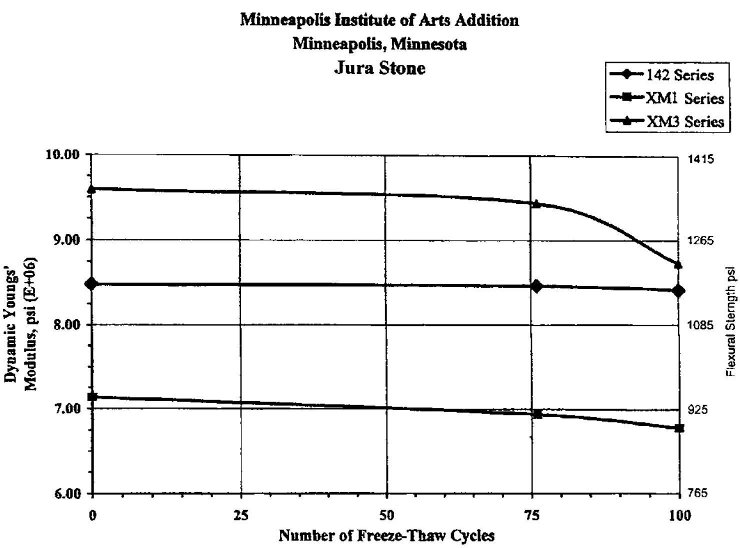

FIG. 8—*Summary of freeze-thaw test results.*

TABLE 4—*Strength changes with accelerated weathering.*

Group	Strength Before AW	CoV	Strength After 100 Cycles	CoV
XM1	923	26.5	926	33.7
XM3	1388	17.3	994	40.7
142	1364	16.3	1355	5.6

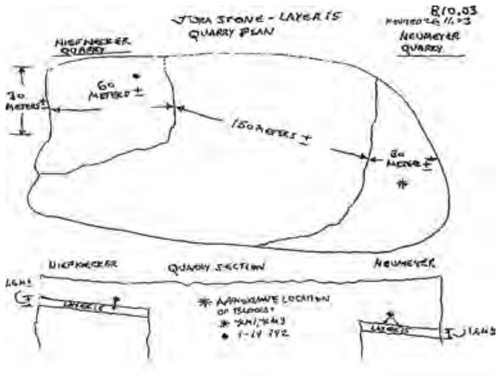

FIG. 9—*Hand drawn map of quarries.*

eight years (Fig. 2). Based on the slope of the curve in Fig. 12 it was extended to 20 years using a 50 % loss in elastic modulus as the end point.

Figures 13 and 14 present changes in sonic modulus with accelerated weathering exposure for limestone samples. Note that the limestone natural weathering over an eleven-year period, Fig. 5 indicates that limestone with limited amounts of impurities (i.e., clay seams) show similar behavior under accelerated weathering and natural environmental weathering. Figure 15 shows a limestone with clay impurities which shows signs of deterioration over a 100-cycle accelerated weathering.

Figure 16 shows the sonic modulus versus number of accelerated weathering cycles Academy Black Granite were subjected to. This curve can be compared to the Black Granite Curve shown in Fig. 4. The black granite shown in Fig. 4 is Academy Black Granite. Figure 17 indicates that the 4ph acid solution has no effect on Rockville Beige Granite. Figure 18 shows a plot of sonic modulus versus cycles of accelerated weathering which indicates little deterioration over 300 exposure cycles. These curves can be compared to the plots in Fig. 14 which shows eleven years of environmental weathering.

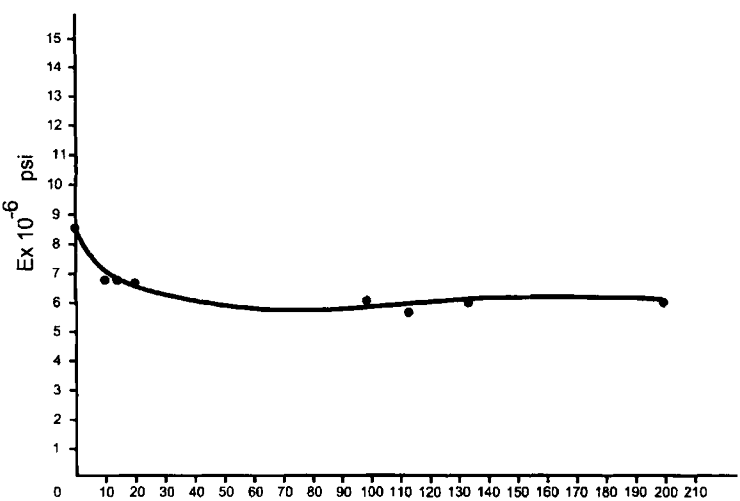

FIG. 10—*Heating and Cooling Cycles. 200 Cycles values modulus of elasticity for unweathered Carrara Marble.*

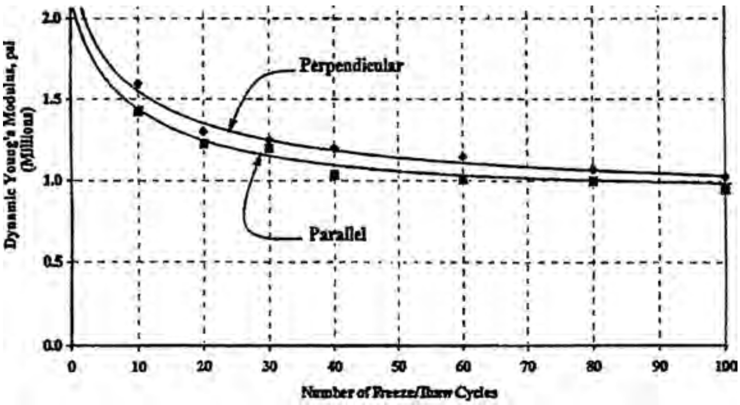

FIG. 11—*Accelerated durability results for Marquis Gray Danby Marble.*

Petrography

The petrographic studies were performed in general accordance with standard geological petrographic practice. The general study procedures are similar to those outlined in ASTM C 856, “Standard Practice for Petrographic Examination of Hardened Concrete” and C 295, “Standard Guide for Petrographic Examination of Aggregates for Concrete.” Lapped surfaces and freshly broken surfaces were examined at magnifications ranging from approximately 5X to 50X using a stereomicroscope. Scanning electron microscope (SEM) studies were performed using an ASPEX Personal SEM equipped with an energy dispersive X-ray spectrometer (EDS). The SEM acquisition parameters are given on the attached micrographs.

Petrographic studies were performed on stone that was exposed to natural weathering as well as accelerated weathering. Petrographic analysis of the environmentally weathered stone on the roof follows. The white Carrera marble prisms were designated KM1, KM2, KM3, and KM4. The prisms are 16-in.

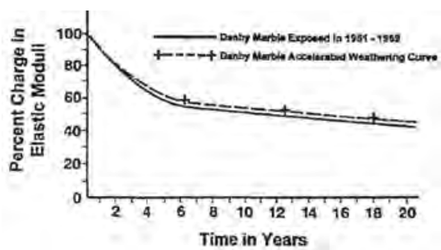

FIG. 12—*Natural weathering results for Marquis Gray Danby Marble.*

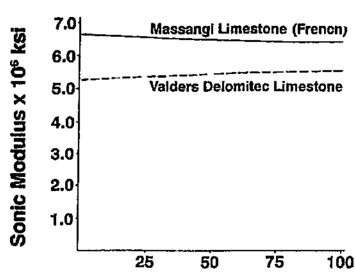

FIG. 13—*Accelerated weathering of limestone.*

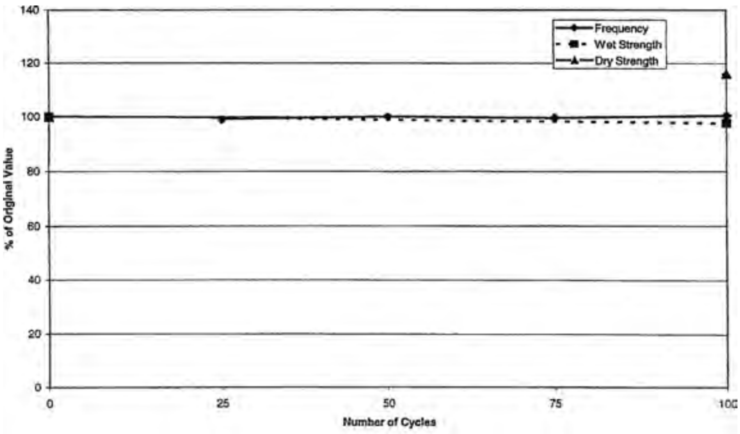

FIG. 14—*Cottonwood Falls, Kansas limestone—perpendicular frequency versus strength loss 100 cycles.*

long, 3-in. wide, and 7/8-in. thick, and appear to have essentially identical characteristics. The exposed surface of each prism is somewhat yellowed, and has small amounts of loosely adhered, black and brown particles scattered over the surface. The surface is slightly rough to the touch, and the surface texture is similar to medium-grade sandpaper. Stereomicroscope examination revealed widespread "sugaring" of the calcite crystals that are the major components of the marble. This phenomenon is caused by deterioration of the intergranular boundaries. Sugaring on the exposed surface of KM3 is illustrated in Fig. 19 and a closer view is shown in Fig. 20. Occasionally, small lenticular flakes are detected near the exposed surface. The depth of sugaring and flaking was typically approximately 1 to 2 mm (Fig. 21). The marble contains a few greenish gray veins that intersect the exposed surface at steep angles. These veins are slightly more eroded than the surrounding surface. The stone also contained a few elongated oval pits. The cause of pitting is not apparent. The pits are possibly derived from deterioration of pyrite streamers. Further investigation of the stone is required to be more definitive about the cause of pitting. The back (bottom) surface of the marble prisms are smooth (possibly ground). No evidence of sugaring was detected. The back (bottom) surface of KM3 is shown in Fig. 22.

The five Academy black granite prisms, BG-1, BG-2, BG-3, BG-4, and BG-5, have essentially identical characteristics. The prisms are 16-in. long, 3-in. wide, and 1-1/8-in. thick. The exposed surface is polished. Stereomicroscope examination reveals numerous microcracks, many of which are transverse to the long dimension of the prism. Plagioclase feldspar crystals exhibit rectilinear microcracking (Fig. 23). Radial microcracks emanate from clumps of magnetite granules (Fig. 24). Feldspar crystals appear to be more heavily cracked than other mineral species. The back (bottom) surfaces of the prisms are smooth. Figure 25 is a stereomicroscope view of an unweathered surface Academy Black prisms. The feldspar crystals reveal no sign of cracking. The Academy Black specimens show similar behavior with regards to cracking of the mineral crystals for both accelerated weather and environmental weathering.

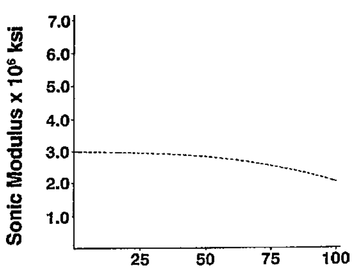

FIG. 15—*Accelerated weathering to limestone.*

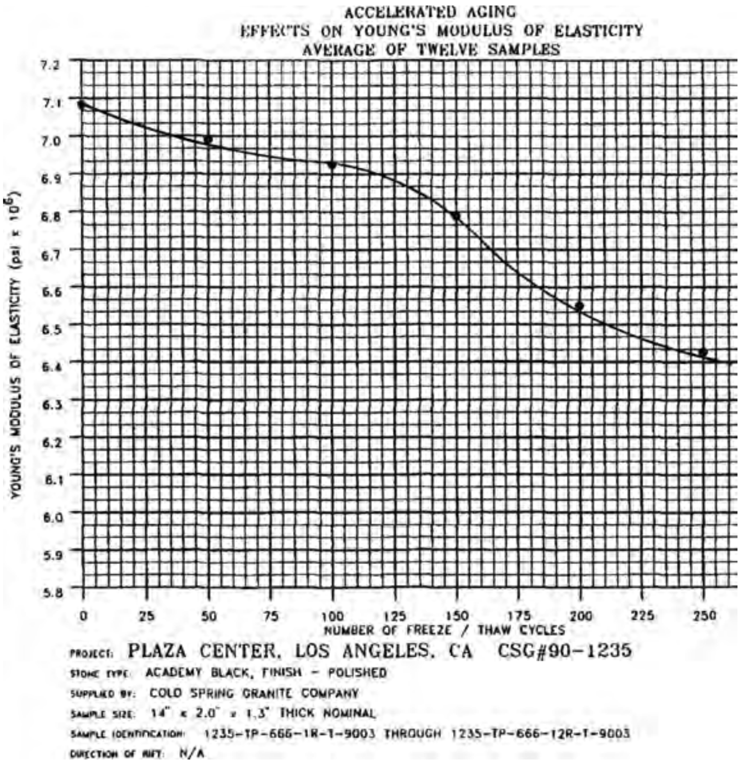

FIG. 16—*Accelerated durability test results for Academy Black Granite.*

We have no direct petrographic of environmental and accelerated weathering comparisons for Indiana Limestone. However, we can show that the limestone properties can vary measurably within a quarry, between quarries, and from one location to another. The Indiana Limestone used in the environmental studies (Fig. 5) can be combined to studies of the same generic stone which had a different petrographic

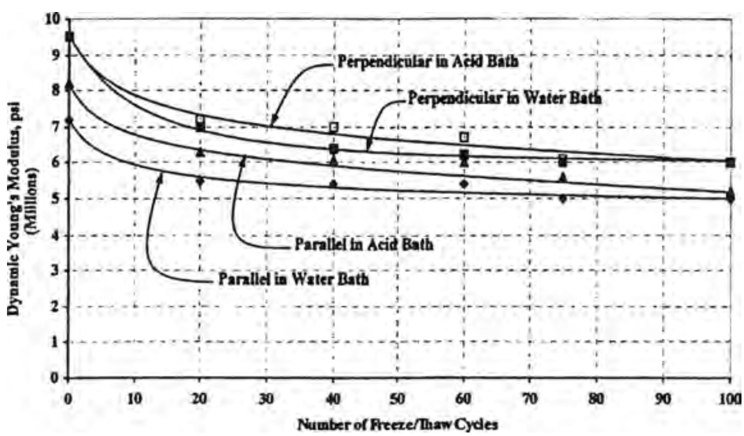

FIG. 17—*Accelerated durability test results for Rockville Beige Granite.*

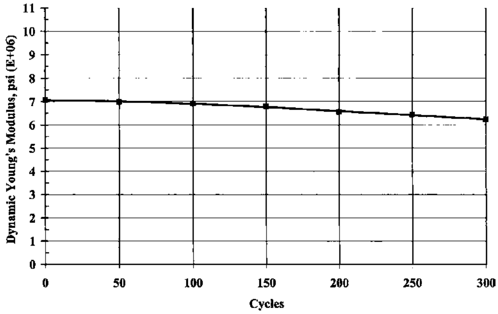

FIG. 18—*Weathering studies granite.*

picture. The stone aged in the field is a beige medium to coarse grained calcite structure with marine fossils, see Fig. 26, cemented to the calcite grains. Figure 27 shows cross bedding defined by the observed parallel orientation and size sorting of the fossils. The stone does not contain deleterious material or microstructure that would be expected to produce poor weathering behavior. The Indiana Limestone weathered on the roof has a similar structure to the field material. It also has a medium to coarse-grained calcite structure. It is composed of well-sorted marine fossils cemented fine and coarse grained calcite. The coarse-grained calcite crystals were moderately etched along cleavage planes, but the surface remains sound and no friability or microcracking was detected.

Conclusion

Based on the evidence presented for accelerated weathering and environmentally exposed marble, granite, and limestone, the accelerated weathering test presents a reasonable simulation of actual exterior exposure of stone in a temperate climate. Petrographic studies appear to verify the crystallographic changes observed for both the environmental weathered and accelerated weathered samples. For further information, see Refs. [1–24].

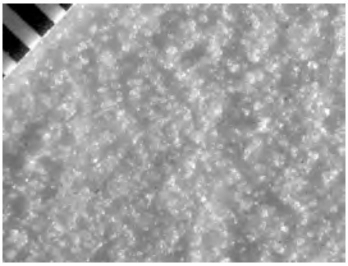

FIG. 19—*Sugaring of marbles surface. Millimetre scale.*

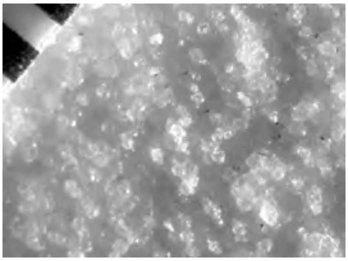

FIG. 20—*Close-up of sugared surface. Millimetre scale.*

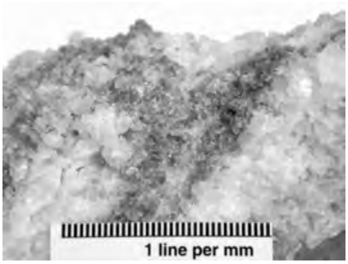

FIG. 21—*Sugaring and erosion at micaceous veins. Millimetre scale.*

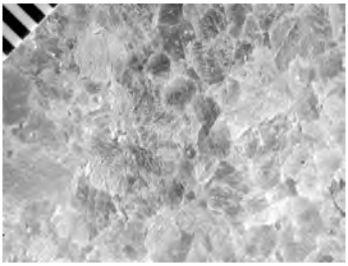

FIG. 22—*Unexposed marble surface showing interlocking grain boundaries. Millimetre scale.*

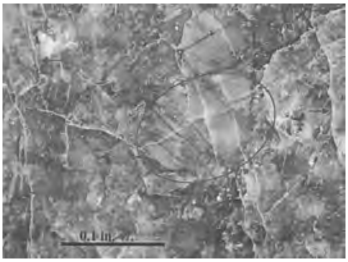

FIG. 23—*Photomicrograph of granite surface after accelerated weathering.*

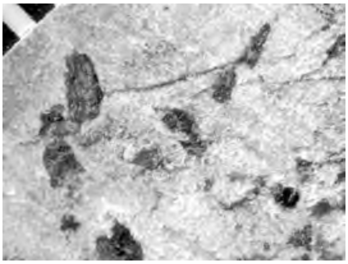

FIG. 24—*Photomicrograph of unpolished, dark-colored granite after accelerated weathering test showing cracking and alteration of iron-rich minerals. Millimetre scale.*

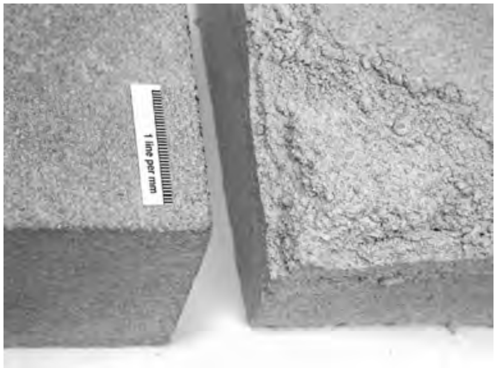

FIG. 25—*Photomicrograph of Indiana Limestone from two different locations shown after accelerated weathering.*

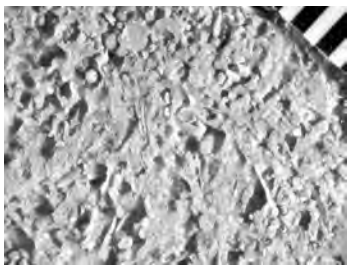

FIG. 26—*Photomicrograph of weathered surface of Indiana Limestone after eleven years of exposure. Millimetre scale.*

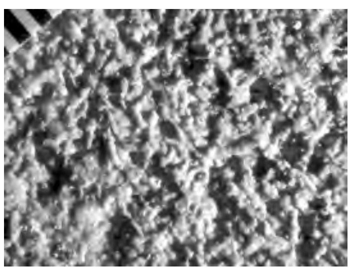

FIG. 27—*Photomicrograph of Indiana Limestone after accelerated weathering test. Millimetre scale.*

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