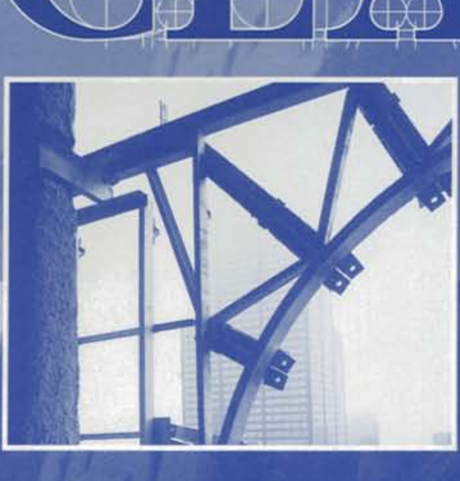
# MODERN STONE CLADDING

## Design and Installation of Exterior Dimension Stone Systems



**MICHAEL D. LEWIS** AIA

MANUAL 21

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*They sacrificed endless evenings and weekends*
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*and share the technology of ...*
*"rocks" on buildings.*

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# CONTENTS

# ABSTRACT

THIS book documents a sequenced procedure to design exterior dimension stone cladding. The design approach avoids arbitrary safety factors by considering performance variables that can establish true safety and durability. This text presents a process to select, design, and install dimension stone cladding and support systems.

Within a sequenced format, extensive explanations with new engineering applications enhance recognized industry practices and include successful exemplars to guide objective and rational decisions.

This approach increases awareness of the individual influences that affect exterior wall performance. These influences, termed "uncertainties," can each be researched to establish their impact on the risk of failure. They must be correlated to existing work. Evaluated individually, they formulate load and resistance factor design for dimension stone. This approach tends to provide safe and durable stone projects.

# FOREWORD

THE intent of this manual is to outline the process of selecting, designing, and installing stone cladding systems for exterior walls. Stone's physical nature and cladding retention systems vary widely. Their potential applications are widespread.

The engineering process should recognize exemplars before tests. Modern construction should include successful walls enduring in the real-world "laboratory." It should not duplicate the failures. These past lessons, not just fresh tests, should guide selection, testing, design, engineering, and installation. This approach identifies those variables known to influence stone cladding system performance. Each variable is considered separately within the process to optimize the solution. Applying this process results in better projects for all parties involved. Better walls are more efficient to construct and maintain. Well constructed walls are more durable. And more durable walls are safer and create more comfortable space for the public. This manual is not a code that formulates objective limits. Further structured practice and research can objectively measure the variables that influence performance. This manual organizes the principles that base such research on those variables.

Chapter 1, *Introduction to Modern Stone Cladding* and Chapter 2, *Precedents to Modern Stone Cladding,* discuss the history of stone as it evolved into modern "thin," non-loadbearing cladding.

Chapter 3 on *Determining Responsible Design Values* and Chapter 4 on *The Future of Stone,* outline a variable-based design procedure analogous to load-and-resistance factor design.

Chapter 5, *Guide Specification for Stone Systems,* advises owners, architects, engineers, and contractors about the specialty of modern stonework.

This manual comprises a process that assists users to rationally select, design, and install stone cladding for exterior walls.

This manual is sponsored by Committee C-18 on Dimension Stone.

# 1 INTRODUCTION TO MODERN STONE CLADDING:
## Approaching Design with Rational Principles

STONE is a prominent and desirable building cladding. It was first used in massive blocks stacked within loadbearing masonry walls. It is now commonly a thin-skin caulked cladding, which is only facing. As part of the exterior wall, it does not support the building. Stone's structural role is now flexural as facing instead of compressive as blocks, contrary to its natural strength and origin.

The newest stone assemblies seem more complicated than conventional masonry construction. Yet, they can still be simple and durable if executed with the proper design and installation techniques. The contemporary approach to engineering stone must consider stone's function and its environment in its intended exterior-wall applications.

This manual outlines a process for evaluating the aspects that influence stone cladding performance. The process considers existing systems and buildings, testing and engineering, and installation methods to predict performance. Designers and installers following this logical progressive analysis make objective design decisions to validate a design. Because the analysis is sequenced, consistency is reproducible. The results of the process offer consistent quality and safety appropriate for the intended application of the stone cladding.

Stone is a natural material that possesses variable properties. Using it as a cladding requires consideration of stone's unique characteristics. Also, the behavior of its supporting structure and previous uses exposed to the proposed building's environment must be considered. Both are important for proper performance. This manual describes how to evaluate these influences to maximize stone cladding system's economy, durability, and public safety.

## THE PROFESSIONALS' DESIGN RESPONSIBILITIES

Professionals intimately involved in the design and construction of natural stone skins for buildings know that there is a significant need for an objective process for completing those tasks. A uniform approach does not presently exist.

In this specialized field, a subcontractor is typically delegated design responsibility and absence of details is common. Contract documents specify a system with performance criteria and profiles, then subcontractors develop systems from these rules. Subcontractors guard their individual solutions to protect their ingenuity. Their design is their edge on cost, method, time, competition, and risk. This inevitably stifles innovation and prevents the current state-of-the-art stone technology from being compiled and disseminated. The specialty subcontractor, as a designer, a manufacturer, and an erector improves the process by encountering the difficulties of its own design during installation in the field, and then correcting those deficiencies. Repeating this improves wall quality.

The exterior wall physically encloses the building. Cladding contractors resolve errors in other contractors' previous work by others by covering them. This manual considers performance variables to help avoid interference problem conditions. It complements the design process by identifying potential conflicts and deficiencies in work that interfaces cladding systems. The characteristics of this surrounding work are the *boundary conditions* for the stone cladding system. Control of boundary conditions avoids engineering unknowns and construction interferences. Improving engineering makes installation more efficient and thus less expensive. A standardized approach gives greater confidence and thus a safer finished product.

This manual documents a process that comprehensively outlines stone, anchorage and support design. It begins by considering stone selection and continues through engineering and installation. Most professionals presently practicing in the stone field tend to protect their own "proprietary" ideas on stone and its anchorages. Ideas based upon empirical "experience" often lack justification by engineering or construction principles. Theoretical ideas often lack correlation with existing work. These personal experiences are unique. Individuals rarely share their insight. Their ingenuity is their edge on competing colleagues, who also sell their services and systems. Lessons learned from both good and bad exemplars must be balanced with scientific issues learned from tests. Each project requires different emphasis on the balance depending upon the type of stone and its intended application. If those issues are either inadequately observed or misapplied, failures occur and durability is reduced.

Owners and architects want buildings to fulfill their visions. They expect their investments to endure without losing appreciable appeal or performance, and certainly not safety.

Those expectations are not met if technical means and precedents are ignored or unbalanced. Architects and engineers should actively participate in the development and building of their project's envelope with the principal entities of the design and construction team. This begins with material selection and continues through facade system engineering and coordination, attachment installation, and maintenance methods. Officiating the design through submittals is untimely and inefficient.

The highest quality facade integrates each special interest expertise through all phases of concept, detailing, and installation. Legal anxiety and lack of expertise erroneously delegates this critical responsibility to others. Litigious intervention controls many aspects of design and construction. It underscores the importance of lasting durability. Its threat discourages the very ingenuity that improves exterior wall quality and the comfort behind it. The legal burden divorces once-qualified professionals from the role of technically designing cladding and directing its installation.

A knowledgeable single-source charged with building design should also actively govern its exterior walls. To minimize legal exposure, architects and engineers should practice a uniform approach to selecting, designing and installing dimension stone cladding, its anchorages, and support systems.

## THE DEVELOPMENT OF CLADDING FUNDAMENTALS

Implied programs for natural stone design are dispersed about the industry. This manual applies proven engineering and construction experience to add structure and discipline to the preceding centuries of traditional masonry mentality. Advanced analysis, curtainwall intelligence, and rehabilitated precedents bring many principles to stone cladding science. Still, present practice lacks coherent organization. The implicit ideas need to be compiled within the context of natural dimension stone. Actual stone and anchorage principles remain somewhat empirical and sometimes subjective. But the process of applying those principles should be academically objective. This manual compiles the new suggested concepts with past practices into a straightforward standard procedure to select, design, and install stone cladding systems.

Because both the nature of stone material and its use as cladding is diverse, engineering and installation methods are different from almost every other building component. Stone is inherently variable and brittle. Its natural strength characteristics must be determined first by testing. Its natural durability characteristics are best determined by studying exemplars in similar exposures. Stone material properties can not be specified for a project like most other materials, and themselves do not assume safety.

Structural skeletons are not built to finish tolerances. For cladding to fit onto the frame, its construction requires adjustability. This causes ranges in the final installed conditions that must still maintain strength. Exterior wall cladding covers all the visual, structural, and constructional "sins" of preceding work. The facade gives the visual impression of the building. Observers expect it to be true and accurate to convey high quality. Cladding's acceptable deviations from "theoretical" are small, practically imperceptible. But the structural frames concealed behind the cladding reach relatively larger locational errors. The building's skin system adjusts to the frame to attain a finished accuracy during installation. It must maintain both structural and environmental integrity after installation through the environmental extremes experienced during the building's life.

In addition to its structural functions, the stone cladding in a building skin must also resist environmental elements. It must successfully refuse air and moisture infiltration and filter the sun, temperature, and sound. Some of these exposures are predictable and some are not. All their effects on stone system durability and weatherability are not completely understood. Some effects are not yet known.

Stone as cladding protects the occupied interior from the exterior climate. It should repel those exposures through the wall's expected service life. Its appearance should "weather with dignity" by retaining its "original visual magnificence." While visual and textural characteristics are critical during initial selection, their changes over time are just as critical to perceived durability.

Structural permanence and architectural integrity share equal interest in the stone evaluation and erection process. Achieving proper stone panel and anchorage performance depends upon investigating and comprehending their individual and their joined behaviors. The building systems that interface the cladding should be matched. Their materials must be individually compatible and must remain so when their final assembled is whole. They must be symbiotic over time. Developing all systems together enables appropriate component selection to meet the exterior wall's attainable performance. The conscious study of the overall interaction between individual cladding components with interfacing systems is almost always underemphasized. These conditions then malfunction, causing deteriorated durability and performance.

## Boundary Conditions for Stone Cladding

Boundary conditions are the performance parameters for the systems surrounding the individual cladding stones. Cladding stones depend upon their support systems to maintain their structural integrity. Cladding stones also depend highly upon the thermal and moisture integrity of the wall behind to maintain their durability. A correctly selected stone that is supported soundly by a wall system with proper environmental qualities will remain beautiful. Preserving stone's aesthetic quality requires anchorage and envelope performance to be compatible with the selected stone material.

The behavior of the building systems that interface the cladding are the engineering boundary conditions for the stone cladding system. These boundary conditions are often underexamined. Few designers have enough experience with the many structural and environmental issues that influence stone cladding stability. Without this foundation, the boundary conditions cannot be stated or controlled correctly. Parameters critical to the exterior wall stone's performance are defined by specifying a sequenced list of considerations for these boundary systems. These considerations lead logical design decisions follow. The design can then be verified in actual construction of both the interfacing and cladding work. Only arbitrary overdesign, infrequent exposure to maximum loads, and the relatively young age of thin-stone-clad buildings have temporarily hidden problems of incorrectly built conditions in the past. However, exterior wall rehabilitation is quickly becoming a major industry as dilapidated walls show their wear.

There are many parameters that influence stone cladding performance. Each parameter, or uncertainty, should be checked during conceptual design to compute overall system adequacy. Once critical parameters are defined, they can be inspected and closely monitored while installing the work. Emphasis on tightly specified stone installation standards, because the stone is seen is unfair unless equal importance, is placed on the interfacing, preceding work by others.

Work adjacent to the stone panels create the engineering context for the stone panels. The exterior wall structure, the thermal and moisture envelope, and primary building frame determine the boundary conditions. How these systems interact control the function of the complete cladding system. Their reaction to climatic forces and building use, which are applied by the skin's reactions to those forces compose a complex, dynamic interrelationship that must change and adapt to endure.

Begin by considering the stone panel as a structurally isolated infill component in the skin; it behaves independently. Develop the interconnecting systems from the outside and move in. The severity of the exposures and the complexity of system components generally decrease penetrating toward the interior.

## Legitimate Testing and Comparison with Existing Skins

Stone is a natural product with varying properties. Different stones have different properties; even similar stones may have widely varying properties. The same stone likely performs differently depending upon its exposure and backup. Testing quantifies some engineering structural properties needed to prove strength. But it is impossible to duplicate nature by measuring durability in a laboratory. While some test procedures simulate certain parts of natural exposure, there are no better examples than existing buildings clad in stone.

There are many levels of testing, and many properties and capacities that might require tests to evaluate. Most stones were used as cladding previously, so few projects require extensive testing because sufficient historical and current data exists. Use recent data from previous projects using the same material. Examine exterior wall systems that use the anticipated anchor type. Investigate other buildings in the project location, regardless of its cladding type, to learn about that climate's effect on building skins. There are no tests for these examinations. There is no substitute for experience and sound professional judgement for these most important aspects.

Some of the presented test methods are accepted stone-industry standards. Others may be special techniques practiced by individual specialists. Know that the validity of these techniques has not yet been confirmed by the industry consensus. Methods to prove that customized assemblies and components perform adequately and are becoming standardized. Some of these nonstandardized methods' approaches adapt to the specific project systems. Any test's conjectures must be consistent with the project construction and its environment. Realize that combining structural and environmental challenges to a wall "sample" in a test, and then accurately measuring their effects, is difficult, if not impossible. People cannot yet suppose nor create the acts that buildings will endure.

Test interpretation relies on fundamental common sense and sound engineering judgement. Statistical conclusions must still be related to the performance of existing stone work to be relevant. Misunderstood, incorrectly applied, or misrepresented test mechanics cannot measure real capacities. Tests should simulate real-project conditions for structural, environmental, and assembly interaction. The results of any test are only as reliable as that test's ability to duplicate the condition it was intended to measure. Boundary conditions as well as stone panel criteria must be correct for tests to be accurate.

ASTM Committee C-18 on Dimension Stone governs standards and defines many methods for testing stone strength. The group of experts refines and develops standards for those using stone. But ASTM sample tests measure only unit-strength capacities. They are the first phase of testing that can be used to prove that the material is structurally adequate. These procedures do not suggest how to statistically derive appropriate values from their results. They also fail to suggest comparisons with precedents. For better predictability, probability analysis requires sample quantities proportional to the number of uncertainties (thickness, finish, rift, wetness, geological variability, heterogeneity, anchor types). Wider variability in any of those uncertainties further increases the need to study the scope of the testing program. Any testing must be related to existing work to be competent. Confident means of correlating test results with exemplars to obtain true material performance is the realm of an experienced stone professional.

The full panel procedure uses cycled loading to test the initial capacity of the panel as it will be anchored. The assembly's endurance over cycled weather extremes and cycled loading should also be considered. Accelerated weathering tests attempt to simulate the deleterious impact of climatic exposure to increasing moisture, chemical, and temperature. Their methods include extreme cycling and quick frequency to predict relative durability during an abbreviated period under the conditions of the test. They attempt to model months of "weathering" in hours. None duplicate nature. None are adopted ASTM standards because too many variables exist to present their methods as conclusive. What climatic exposures are most damaging, how to evaluate and interpret strength or material property change, supposed correlation of test results to existing stone cladding, and the many material and climatic combinations that exist presently impede standardization. Versions of prototype tests structure their procedures with consistent guidelines and parameters that attempt to predict relative weatherability of stone material. These tests should duplicate the performance of the actual field construction and intend to suggest a stone's relative durability. Be reminded that durability of stone cladding is as much, if not more, dependent upon wall systems' internal environment as it is the external environment. Tests do not yet address this. Study of existing buildings does.

## Significance of The Evaluation Process

This text organizes the incremental process that assists designers in selecting, designing, and installing facade systems clad in stone. The Guide Specification for Stone Systems presents the process in a format that a designer follows to reach a rational conclusion. It substantiates each step with recognized engineering and construction principles. It applies the previous step's conclusions to the next consideration.

Determining Responsible Design Values and Guide Specification for Stone Systems sequences and these explains these issues to help designers understand the reasons for particular conditions. The resulting design product becomes more deductive and objective and less personal and subjective. By practicing this process, the rational conclusions will lead to more standardized, safer, and more durable stone cladding solutions.

## Sequencing Decisions That Derive Designs

Traditional safety factors are not founded upon modern stone construction. Most are almost unchanged from loadbearing masonry applications. Their assumptions did not evolve along with changes practiced in current cladding engineering and construction applications. Some factors may actually be nonconservative, depending upon the cladding application. Present practice follows the "allowable-stress" approach. Design values are test values reduced by a safety factor coefficient. Instead of assuming that this coefficient covers all conditions, a rational safety factor should be derived in part from predetermined tests that quantify strength and variability. Other prime factors related to the stone's actual conditions of use and exposure that influence performance must be considered when determining a safety factor. Different issues are important for different applications. A safety factor should discriminate between those conditions that are important to that project.

Criteria for specific stress states need to be related to probabilistic risk in modern stone applications. Other structural design disciplines adopted load-and-resistance factor design to make designs specific to their applications. This text begins to formulate that approach for exterior stone cladding.

Rational load-and-resistance-factor engineering naturally fits stone design. Its many variables can and should be quantified independently and related to cladding performance. The section on Determining Responsible Design Values presents design and installation parameters in a load-and-resistance-factor format. Each of these variables, or uncertainties, can be independently considered, then combined to prepare a safe overall design. Proper construction methods for anchorage installation preserve presumed boundary conditions for the individual stone panel. The comprehensive system variables and conditions-of-use for work interfacing the cladding system. They can then be included in the cladding system's structural analysis to minimize risk.

The section *Guide Specification For Stone Systems* categorizes and examines design and installation variables. A case example parallels the specification to illustrate how these different influences fit a particular project's condition. That case example also follows the incremental testing sequence prescribed to prove the system's structural capacity. One could parallel the logic of this approach to confirm "allowable" stresses.

## Partnering Makes This Approach Successful

The owner, architect, engineer, and contractor share mutual ambitions for quality with profitability. These are the goals that build notable reputations. It is common for the contractor to have sole responsibility for conceiving the cladding system. "Inventing" its components, satisfying all compatibilities, and accommodating all predictable behaviors within the skin and a structural frame should be a team effort and a team responsibility.

When only criteria are specified, the architect delegates responsibility for the skin to the contractor. "Legalese" attempts to theoretically divorce the architect of specific accountability. The architect then "polices" proof of conformance to that criteria.

Contracts that separate exterior wall expertise from building design intent create animosity and cheat all parties except the attorneys. All parties must be re-joined to successfully build exterior walls. Safety factors and movement allowances typically specified are based on past practices that do not relate well with contemporary stone exterior cladding applications. Architects, engineers, and contractors should mutually interpret project conditions starting during design and continuing through construction. Mutual consideration during design resolves basic inconsistences and sets realistic expectations. Savings will result for the frame and the wall. Use the same test methods to support rationally developed criteria for a load-and-resistance factor format that support the present method. Variables vital to that project's conditions then factor into load-and-resistance factor design. A rational, project-appropriate design results. The building functions better and longer at a lower cost.

## HOW FUTURE ARCHITECTURE BENEFITS FROM MODERN STONE CLADDING

*Modern Stone Cladding* applies some of the most fundamental engineering philosophies practiced today by other construction disciplines to exterior stone cladding. Consistent, thorough application of the process removes handicaps to our understanding of the structural mechanics and how we analyze them. This text sequentially connects previously incomplete dimension stone logic by preparing and then justifying an analysis of design and installation variables. By comparing the analysis with existing work, they culminate in a safer, higher-performing work. With repeated practice, experienced professionals can eventually refine this process into a standardized load-and-resistance factor approach to stone cladding, anchorages, and exterior wall retention systems.

Present architectural styles reinterpret past forms in novel places with new materials. Their aesthetics explore architectural creativity in unprecedented ways. Stone cladding retention techniques struggle to progress to meet these challenges. To remain safe and durable, engineering abilities must advance to match cladding system characteristics with the new configurations. Construction methods already include modern stone panel and unitized system anchorages. Engineering should include some tests at times, and always comparisons with exemplars. The process supports architectural design. It promotes expanded aesthetic opportunities while it improves constructibility, building safety, and durability.

Designers and builders have slowly changed how they think about stone. Many underestimate modern stone production capabilities. Many misunderstand stone's natural structural properties. Durability can be optimized, but still not be finitely predicted. Aesthetic quality can be somewhat confined, but still not be discretely controlled. Reforming the incorrect assumptions that durability and aesthetics are concise characteristics requires realigned professional responsibilities and attitudes. Owners, architects, engineers, consultants and installers alike must be a team together. Quality and profitability increase with everyone's improved understanding of materials, means, and methods.

Comprehending the "nature" of stone will expand stone's involvement in modern architecture. While stone is brittle, variably inconsistent, and arduous to work, its unique beauty offers architects the opportunity to "signature" their designs like no other material can. Natural stone features an edifice to make it as spiritually permanent as it is physically enduring. Architects working with engineers, consultants, and installers with a standard design process guiding them can attain their mutual goals of inspiration, safety, performance, and profit.

The tradition of natural stone is as old as human existence. It is this presence that we know and feel from its traditional uses that gives stone its esteemed innate cultural value. But because stone cladding is fabricated and constructed so much differently now than in previous times, stone science must depart from those methods to preserve that solid cultural tradition. The sound fundamental approach begins by reviewing the precedents of modern uses of stone as cladding.

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# PRECEDENTS TO MODERN STONE CLADDING

## How Stone Became Thin on Building Skins

STONE has been present in our building culture since the beginning of human existence. It is important as a permanent, durable material because we perceive it as solid, stout, and secure for shelter; it was the strongest natural substance that seemingly lasted forever. We memorialize our heritage with stone monuments, we build our important institutional edifices of stone. Natural stone makes architecture art.

Stone experienced several distinct cycles of prominence in the last century. Incremental advances in stone manufacturing responded to style and technological changes in building construction. Stone's use parallels the level of technology and architectural fashion during each of those periods. The massive, blocky material, which for centuries was exclusively stacked in bearing walls, had difficulty evolving to fit into multistory curtainwalls. Exteriors were solid and monolithic. Its finishes and surfaces faired poorly in fire, casting doubt on its durability. Only thick slabs were available. Through the Classical Revival period and later the Art Deco period, entering the second quarter of the 1900s, modular masonry replaced that monolithic appearance with larger "stones." Lighter-weight, more sculptural terracotta served as fireproofing and then also cladding until fire protection of skeleton structural framing was fully conquered by other means. Terracotta flourished until it was learned that its durability was limited.

As mechanization made stone easier to manufacture, it also created more opportunities for stone applications. Stone fulfills fundamental spiritual needs by relating to past uses and past places. This feeling is inherited by tomorrow's ambitious architecture that includes natural stone.

Fabrication techniques did not advance quickly enough, though, to reduce stone's weight to compete with the facings of the European machinelike vision. The International Style influence on buildings began in the 1920s, but stone was still conceived as a heavy, blocky component. The traditional cultural value of stone revived when metal and glass envelopes failed. Early performance problems with metal and glass curtainwalls motivated stone's evolution from medieval traditions. Special machining then revolutionized stone fabrication to make it thin, competitively. Once thin, stone panels fit into lightweight curtainwalls by inventing a completely new way it could be installed. Dimension stone then joined other "modern" commercial construction materials on the cladding palate. Then engineering evolved to analyze the new construction techniques. This evolution continues.

Dimension stone entered the contemporary skyscraper age when New York City's A T & T Headquarters completed at the beginning of the last quarter of this century. While not the first thin-stone high-rise, its controversial "Chippendale" cap in Rhode Island pink granite was the most recognizable. Its architecturally and structurally influential image popularized "modern" stone cladding. Adapting stone correctly into the multiple types of versatile curtainwalls has been the engineering challenge ever since.

Fundamentally, stone must be designed as an independent structural element, typically it is not, which can cause failures. Understanding stone and anchorage behavior within an overall dynamic exterior wall system is critical for adequate performance. A symbiotic relationship with all interfacing building systems must be developed. One must apply that same technology that advanced curtainwalls to the once rudimentary use of stone. Stone is still only partially transitioned into modern construction. Appetite for stone architecture will continue to grow as it becomes even more economical, available, and is soundly constructed.

## STONE'S TRADITION AS SHELTER

The romance with stone began with a pragmatic appreciation for its durability—its original use in "building" was providing durable shelter. That permanence grew into an appreciation for stone's unique signature appearance.

Stone's heritage in construction began before the inception of "building" itself. Natural forces carved caves from rock and folded ground layers that formed seclusion and safe shelter. As the oldest indigenous material in existence, people found these fragments of earth's crust to be helpful tools as well as materials. Huts of lashed vegetation failed to provide protection from life-threatening elements. For better shelter, people began building their own shelters by gathering and stacking boulders and rubble. It was necessary to construct permanent boundaries and enclosures when natural enclaves were not found where their sustenance was.

The mason evolved as an artisan when people used tools to chip and break stones to make a different-than-found shape. Because of its hardness, stone tools worked other stones. The ensuing centuries invented myriad methods and eventually metal tools to sculpt stone. Ingenious talent using these techniques for re-forming stone improved how stones fit together. This greatly improved structural stability. Individual indestructible stones were useless if they could not be locked together. Better masonry workmanship, motivated by the need to survive and thrive, promoted trial-and-error learning about basic physics. Once gravity and stability were understood, our predecessors' constructive efforts accomplished greater building achievements.

## THE ASCENT OF THE BEARING WALL

Intuitive trial-and-error engineering improved awareness of structural principles. Bearing wall construction evolved slowly to attain greater capacity and height as a result of this empirical engineering. The very weight and durability that made stone so desirable also made it difficult to work and move, which slowed its development into new uses. More improvisation lead to more experimentation, which resulted in the mastery of masonry. Where tooling did not attain a tight fit, mortar of pozzolana and later, lime with sand filled creases and gaps to help form more monolithic structures. Simply stacking and bearing random form stones limits possibilities. Greek masons included lead-wrapped iron cramps between individual stones in the Parthenon before 400 BC. These devices applied the same idea as dowels in wood construction by keeping stones aligned. By improving both shape with fit, better structures were possible. This remained the main method for stone structure construction until novel "fasteners" were included after 1100 A.D. These developed with the taller, thinner verticals of the late medieval cathedrals. It then took centuries before those structural challenges pushed structures too tall for stone to stand.

Typically, an unsatisfied need in any field prompts most advancements in its technology. That idea could be a new or an existing practice applied in a new way. Knowledge from another discipline is adapted to respond to the need. The new science overcomes its previous limits and advances its abilities. Impediments removed, improved capabilities fulfill new opportunities. Significant progress comes from repeated cycles of this experience. In this way, stone architecture evolved slowly. Quarrying, hoisting, cutting, "engineering" and installing techniques all influenced each other to improve the entire process and therefore its product.

Bearing wall heights were limited without mechanical assistance from fasteners or reinforcement. Adding wood dowels, pouring lead "keys" and inserting metal rods between adjacent stones improved interlocking. This allowed ornamental shapes to be firmly constructed at greater heights and in longer spans. These devices fixed alignment for corbels, arches, and vaults to assure proper bearing. Linked iron loops, chains, or rods hooped to tie domes. These tension elements hidden in stone joints or exposed across spans held the thrust of horizontally spanning arches and vaults to push masonry skeletons as high as they would stand (Fig. 1).

## WALL METAMORPHOSIS CAUSED BY THE IRON SKELETON

Greater commercial demands forced greater building feats. The introduction of the iron skeleton to buildings marked the beginning of the departure of building construction from the centuries of gravity-dependent methods. Stone construction followed those same ambitions by resting on these frames instead of just the stones below. The skeleton was the first feature of the scientific revolution that transformed public construction. Larger floor area requirements for smaller urban lots consequently increased the number of floors that the structure needed. Increased heights then thickened bearing walls. But more retail required open ground floors. Traditional masonry could not satisfy these contradictory trends.

The search for alternative structural approaches began. Refined medieval architecture integrated structure and facade to heights limited by stone's ultimate strength. Joinery techniques borrowed from timber trusses and scaffolding adapted to hold alignments. With buttresses, ties, fasteners, and "sound" stone, "stability" engineering reached its pinnacle. Unlike the three-generation commitments undertaken to raise a middle-age edifice, even three years was too long to wait for a commercial-age structure. Completely new concepts were needed to meet the immediate time and height demands.

The railroads and their civil structures quickly escalated iron and steel capability. Directly applied to buildings, the composite metal-and-masonry construction allowed more height with a more open bottom floor. These materials were manufactured and connected easier than masonry and stone alone. More expedient to construct, metal's reinforcing strength reduced the masonry mass with fast methods and less labor. The first adaptations of cast iron were actually as storefront facades. But changed to a frame, the metal skeleton could become structurally and aesthetically independent of the skin. Beyond simply applying ornament to the facade, efforts then slowly separated the skin as a system from the building's structural frame.

Even perfect bearing wall construction could not achieve the unprecedented heights that the new structures demanded. Their thicker base walls encroached upon the ground retail storefront. Larger windows for better views and ventilation plus the less formidable open sidewalk level required lighter, not heavier facades on the increasingly taller buildings. Clients requested more prominent skyline profiles. Individual corporate "caps" pushed to higher altitudes.

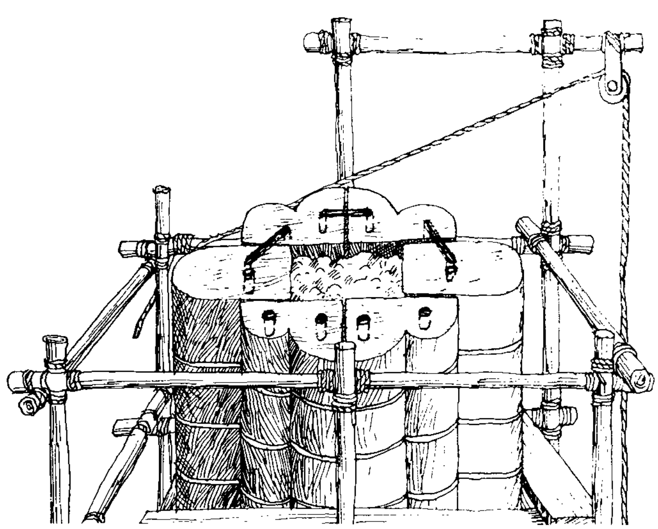
By structurally dividing the exterior walls into floor-by-floor horizontal bands, building envelopes departed from conventional bearing wall construction. Each floor edge relieved pier and spandrel weights. Instead of successively stacking a floor's worth of wall onto the next floor below, and that floor on the one below it, each floor's perimeter "curtain" is only one floor high. This eliminated the accumulating load causing cavernlike bases in the later multistory bearing walls. Base wall thicknesses became so massive in supporting walls above that nearly no penetrations were possible. Foundations grew even further from that. These burdens exceeded the distribution capacities of masonry foundations on almost any substrate except bedrock.

## SLENDER IRON MEMBERS REPLACE MASSIVE MASONRY PIERS

Multistory iron framing actually originated in England in 1792. William Strutt's Calico Mill used internal wrought iron posts instead of brick piers. By 1844, refinements replaced the traditional bearing masonry wall with thin infill behind its iron structure in the Portsmouth Royal Navy Dockyard. American James Bogardus introduced bolted connections to the iron frame in an 1847 New York factory that had sufficient stiffness to omit bracing infill walls and cross-bracing. It was clad in glass. He extrapolated the framing concept from Henri Labrouste's Biblioteque Ste-Genevieve in Paris begun in 1843 (Fig. 2). Joseph Paxton's 1851 Crystal Palace outside London glorified the glazing application.

George Johnson adapted clay tile to encase iron framing members to resist fire, as inspired by Paris' new fireproofing code. This separation of skin and skeleton and use of glass and metal infill first hinted the revolution to come. Bogardus' later 1855 McCullough Shot Tower used its eight-story hexagonal frame to support independent brick infill panels at each floor's beams. Thus masonry was born to the curtainwall age. Stone lagged because it was fabricated too thick and was too heavy.

These new metal frames required lightweight facades to minimize their own size. This inherently eliminated "thick"



◀ FIGURE 1: ***Sketch of Gothic Cathedral Column Construction, (13th C.)*** **To maximize the lightness of structure, interior stone columns were as slender as possible. Ribs from vaults were articulated in the column cross-section shapes by being carved as individual stones. Individual stones were smaller to make them easier to handle, hoist, and fit. The stacked stones were tied by metal rods not only to keep stones positioned, but resist outward thrust away from the core due to what we later termed "poisson's effect" Ties are added to the sketch drawn by David MacCauley from** ***Cathedral.***

◀ FIGURE 2: ***Biblioteque Ste-Genevieve in Paris, France (1844).*** Designed by Henri Labrouste, this structure marked one of the earliest applications of prefabricated iron column and vaulting elements in building structure. Iron was used for primary structural elements which has been stone or masonry before. The iron-framed structure incorporated the advantages of metal's connectivity with its minimal required cross-section (relative to other materials). It maximized openness to let natural light inside. Individual framing of slender columns and vaults marked one of the first steps towards developing metal skeletons as structural frames for buildings.

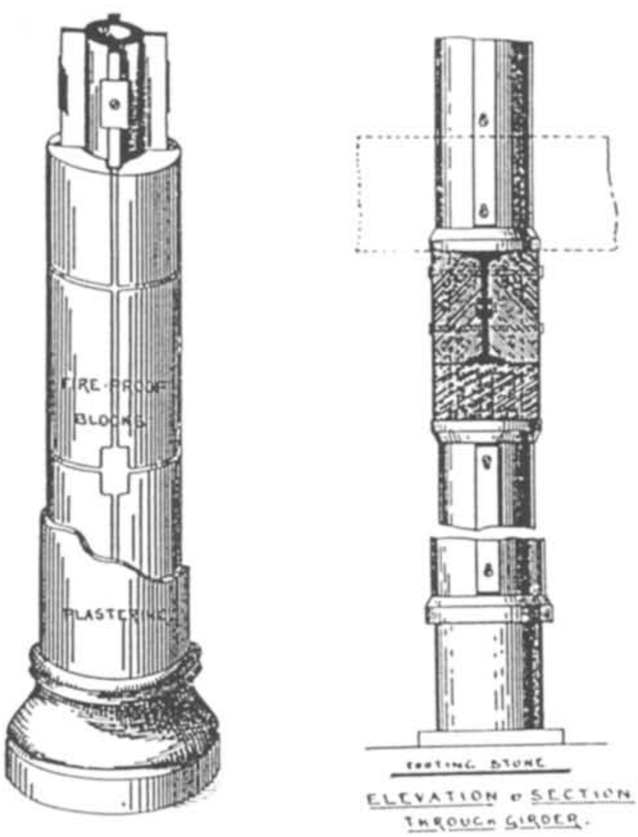
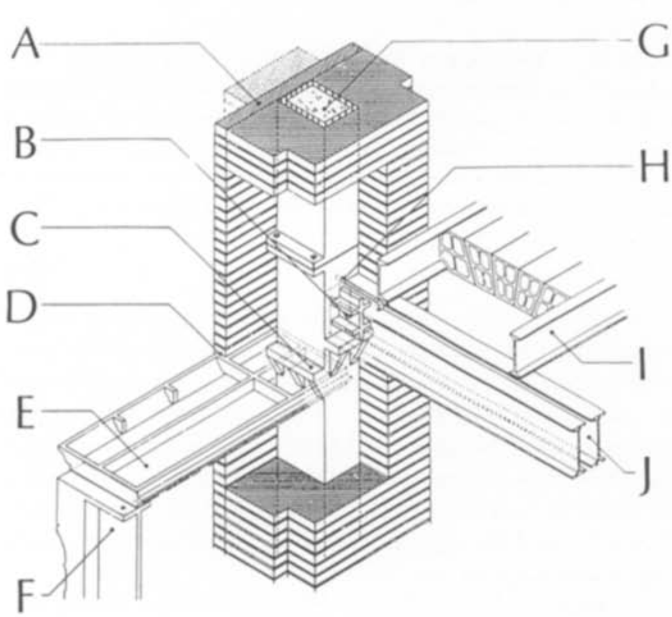



FIGURE 3: *(below, left) Designed Encasement of Iron Framing For Fire Protection (1874).* In response to the catastrophic failure of unprotected iron frames during the great fires of Chicago and New York in the early 1870s, William H. Drake and Peter B. Wight proposed fireproofing iron columns with terra cotta blocks and plaster. The columns supported heavy timber beams to carry the floors. Masonry and heavy timber buildings were the only ones that survived the massive destruction of the fires.
Terra cotta was used as a substitute for conventional brick masonry because it was lighter weight and manufactured in larger units. The mass and noncombustibility of terra cotta insulated the metal from extreme temperatures that radically reduced its strength. Sketch from *Brickbuilder* 6, August 1897.

◀ FIGURE 5: *The Former Home Insurance Building from Chicago (1884).* The first phase of skyscrapers was functional in style. Designers concentrated on solving practical problems of building tall buildings. William LeBaron Jenney's acclaimed high-rise has been mistakenly accredited as the first skeletal building, and thus the first masonry "curtain" wall. This was found not to be true during its demolition in 1931. Not only were there earlier examples of relieved exterior walls, but the construction of the Home Insurance Building much more closely resembles previous bearing methods with metal elements added than truly relieved floor-to-floor walls. Twelve-inch thick masonry encased cast-iron tubular columns. Cast iron spandrel pans rested, with attachment, upon column haunches. The pans were notched around the exterior facing, allowing its weight to be continuous to the floors below. The iron skeleton reinforced the building's structural system, but the masonry mass stabilized it. The masonry was not independent of the frame. The ten-story structure's massive appearance did not appear much different than its genuine bearing wall predecessors. Thick granite panels having superior strength faced the almost three-feet thick masonry and iron piers at the bottom two floors to support the accumulated weight of the top floors of brick above.




▼ FIGURE 6: *Construction Detail of Home Insurance Building.* Isometric drawing shows how the framing components of the Home Insurance Building's perimeter walls were assembled. Drawing and notes by Deborah Cohen and Maxwell Merriman of the University of Cincinnati.

*A:* 4" brick facing , which at corner and central piers projected to 12" (*dashed-in*).
*B:* Single bolt smaller than hole loosely connects both floor girders to double-cross separator bracket that is cast with column.
*C:* Spandrel pan bears on shelf cast with column. Apparently no bolts were used in the connection in order to permit potential rotation due to differential settlement of the piers.
*D:* Spandrel pan notched 4" back at this point to allow the brick facing to be independent of the spandrel. This was perhaps intended to minimize potential cracking from differential settlement of the piers.
*E:* 4" deep cast iron spandrel pan that spanned from column shelf to mullion, to be filled level with concrete to erect brick spandrel wall. Width varied with respect to height in accordance with 1884 Building Code. Code specified thicker walls at the base of bearing walls for stability and capacity. Only mechanical connection in evidence may have been a single bolt at the back of the mullion.
*F:* Cast iron structural mullion.
*G:* One story high concrete-filled cast iron column "built into" the masonry pier. Size decreases with the building height in accordance with 1884 Building Code. The Code recognized that accumulating floors required larger columns.
*H:* 1" diameter iron rod bent into notch in top flanges of both floor girders and secured to inside face of column, pulling girders tight to column.
*I:* 8" wrought iron floor joists at 5'-0" on centers which support hollow tile segmental flat arched floors.
*J:* Two 12" wrought iron floor girders span from interior column to shelf cast with exterior column.

◀ FIGURE 4: *Terra Cotta and Plaster Encasement of Cast Iron Columns (1897).* Construction of iron skeleton with terra cotta blocks and plaster casings in the Alms and Doepke Building on Central parkway in Cincinnati designed by Daniel H. Burnham. The building also uses terra cotta structural tile as flat arches between floor girders as the floor structure to further its fire resistivity. Exploratory excavations were performed to verify the integrity of the construction while planning for the structure's adaptive reuse.

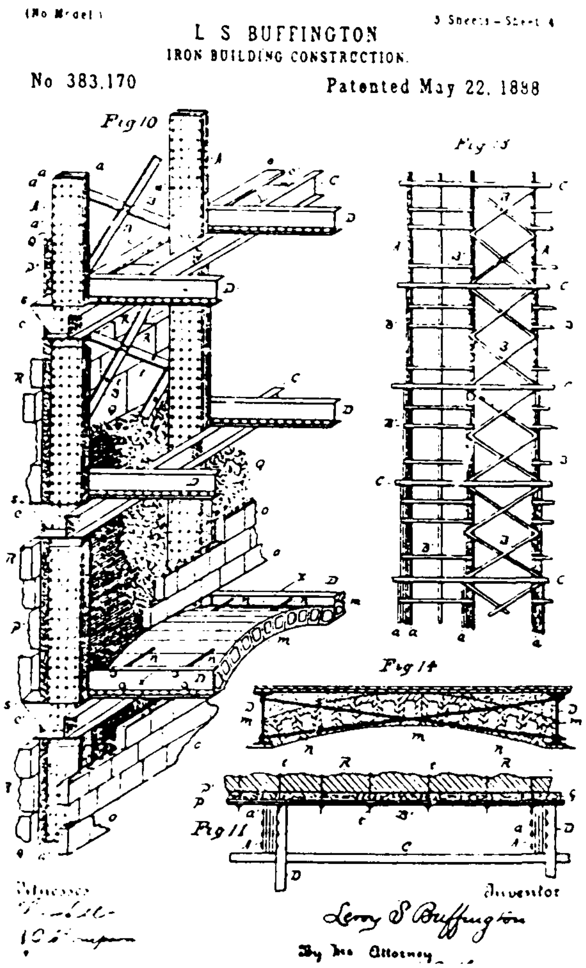



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FIGURE 7: *Iron Building Construction Patent (1888).* **Patented by Leroy Buffington in mid-1888, diagrams clearly show the riveted frame utilizing cross-bracing and rods for internal stability independent of the envelope. Columns use haunches at each floor as "ledges" to support that floor's masonry exterior wall, thus relieving its weight at each floor. Drawing from *Art Bulletin 26*, March 1944.**

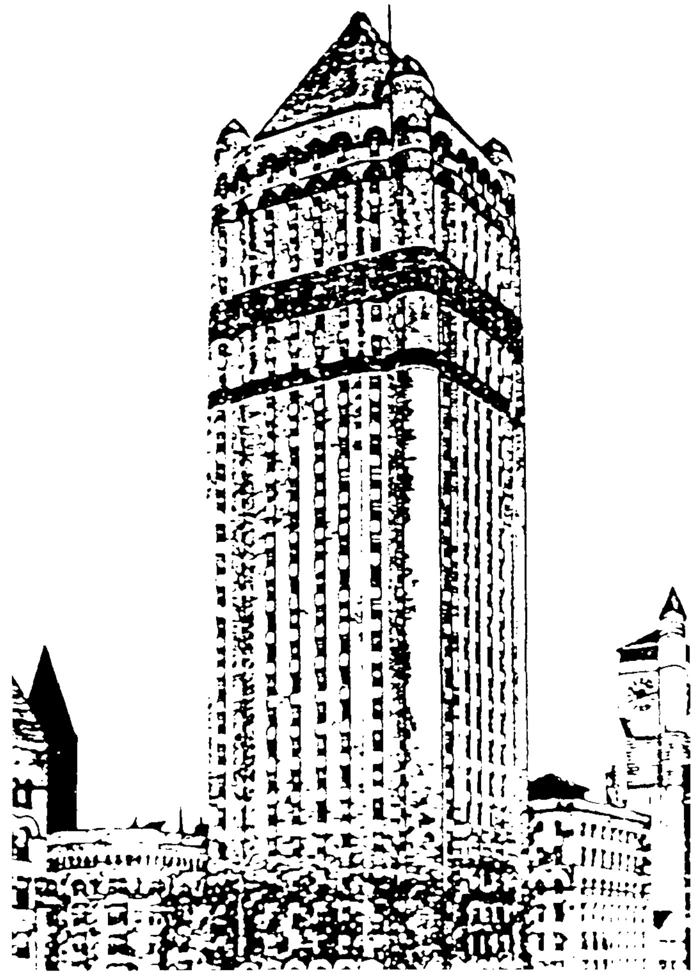

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FIGURE 8: *"Cloudscraper" Proposal (1887).* **Applying the patented iron framing system, Leroy Buffington proposed construction of a 28-story tower. Made possible in concept by using the iron construction techniques learned from Gustav Eiffel, this idea revolutionized the approach to structure for American high-rise buildings. Drawing from *Inland Architect and News Record 11* July 1888.**

and heavy stone from cladding upper floors. Stone was not slabbed thin because it was not used thin. Low-rise structures still used bearing walls od thick stone and brick.

America's commercial growth through the late nineteenth and early twentieth centuries ballooned the need for building space. By the 1870s, emerging giant enterprises wanted imaginative headquarters that advertised their commercial success. This desire for monumental architecture meant that unique permanent-appearing edifices would be coupled with new codes that required fireproof facades. This motivated more interest in stone. But stone's weight, its industry's slow and deliberate fabricating methods, and unimproved-medieval installation techniques handicapped its entrance onto high-rises. Terracotta maintained its dominance until its structural and finish problems became known.

Architects' urge for light and ornament quickly draped building fronts in glass and cast iron. Masonry fronts fell from favor to iron fronts through the 1850s in European and American cities. Susceptibility to fire kept them from being universally popular. Chicago's great fires ruined buildings with unprotected iron frames and ended unprotected construction. Catastrophic failures caused codes to require proper resistivity. This motivated the ingenuity to encase the frames in masonry, whose structures survived the fires.

Solving the fireproofing problem required the fireproofing to be supported directly on the building frame. Masonry re-emerged. Now it provided the necessary fire protection for the iron frame that once replaced it. It adapted from being load-bearing to being hung on the frame. George Johnson invented a system of interlocking clay tile to encase framing members . Chicago architect Peter Wight and terracotta producer Sanford

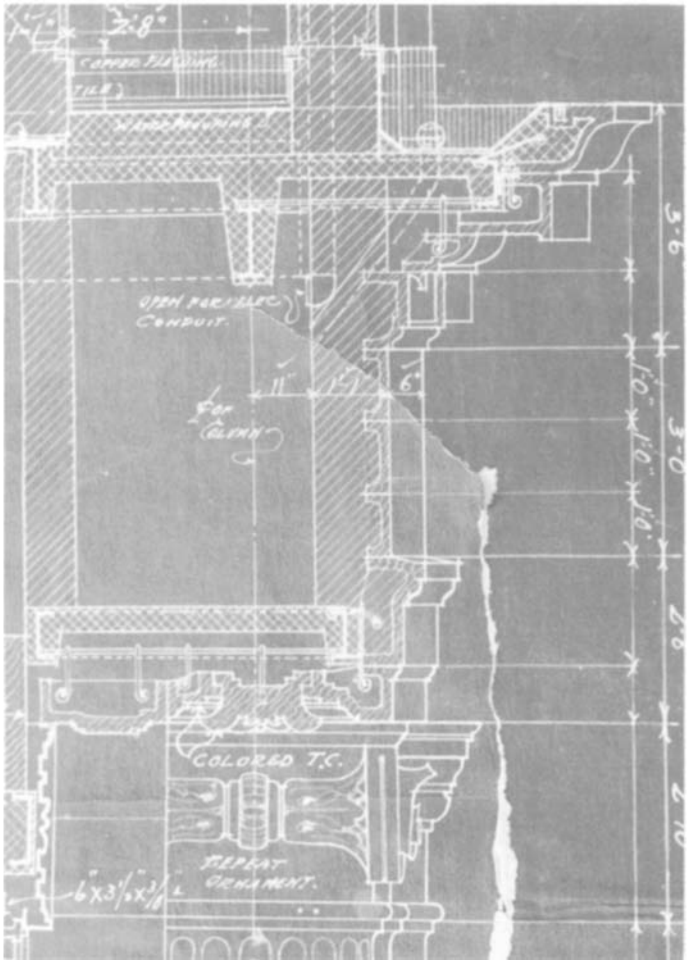


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FIGURE 9: ***Partial Cornice Section of Union Central Life Building.*** **Cass Gilbert's 32-story tower in Cincinnati was the tallest building outside New York upon its completion. Using built-up rolled shapes riveted together for its skeleton, the building's steel columns and beams were encased in thick masonry similar to LeBaron Jenney's Home Insurance Building from 25 years before. Instead of brick facing, Gilbert's design used marble for the first five floors and terra cotta above. While outrigger beams existed in the exterior masonry wall mass, the masonry was solid to the ground identical to bearing wall construction. In reality, the five-floor solid stone base supports the cladding above. The steel frame provided compressive capacity that gave greater strength to carry more floors. Masonry provided interstory stiffness, fireproofing, and backup for the integral terra cotta cladding. The effects of building movement and natural weathering have deteriorated and scarred the terra cotta cladding. Avoiding this damage required separation of the skin and skeleton to isolate their movements. Like most other buildings of that era of similar construction, cladding became distressed from differential movements in the frame behind and accumulated loads from above floors.**

Lovis followed in 1874 with a patented wrapping system, the first where a skeleton independently supported its own masonry encasement. This technique reversed the previous roles of metal and masonry and oriented the mindset towards full masonry separation at the building's exterior wall (Figs. 3 and 4).

## THE MASONRY CURTAINWALL IS BORN FROM FIRE

William LeBaron Jenney's 1884 Home Insurance Building in Chicago encased its steel columns in masonry. Encasement insulated the metal from the heat of fire that reduced its strength to a fraction of its original yield. Twelve inches of brick surrounded its metal members. Its exterior masonry spandrels rested on the piers, not the floor, which transferred to granite at the ground two floors. Mistakenly accredited as the first masonry curtainwall, the conventional brick wrapping the columns was not relieved, and was not a true veneer (Figs. 5 and 6). Masonry-supporting shelf angles first appeared in Leroy Buffington's 1880 proposed Cloudscraper using Gustav Eiffel's riveted connections (Figs. 7 and 8). The concept was realized in Burnham and Root's 1890 twenty-one story Masonic Temple in Chicago. Thirty-five years after Bogardus' stack, architecture finally adopted the original authentic masonry curtainwall.

With the skin structurally supported on the columns behind and the floor beneath, architects experimented with cladding materials. Terracotta "stone" spread because of its light weight, sculptural abilities, and fire-resistive characteristics. Its porous bisque and irregular glazing quality weathered poorly though, was damaged easily, and was difficult to properly repair. Harsh climate cycle extremes in the northeast and midwest aged, cracked, and spalled panels prematurely though. Water leaking in disintegrated anchor straps, corroded support steel, and split the manmade "stone" faces. Failure to understand these mechanics and then provide accommodations for facade movements aggravated the weathering deterioration (Fig. 9).

## COMMERCIAL MOMENTUM OUTPACES MASONRY'S CONVENTIONAL LIMITS

Life-safety concerns moved masonry onto framed buildings and back into commercial construction. Weathering concerns reevaluated use of brick and terracotta, while natural stone's durability remained attractive. Through engineering experience and improved construction methods, stronger frames reduced restrictions on wall weights. Now only the stone industry itself needed to change from its standard production protocol for institutional and monumental "thick" stone to produce thin stone for the new application.

Stone had stayed thick to preserve its preference as building-bottom facing. Typical to Jenney's application, storefront-level stone supported bearing walls above, requiring thickness consistent with normal stone production output. Stone installation science remained almost synonymous with mostly medieval unit masonry techniques. Stone indeed needed to get thin to move up the building. Market demand had to expand to influence the fabrication industry to adapt to new "structural" applications from past "monumental" practices. Corporate American economy provided the momentum to slowly transform century-old practices for the modern application of the

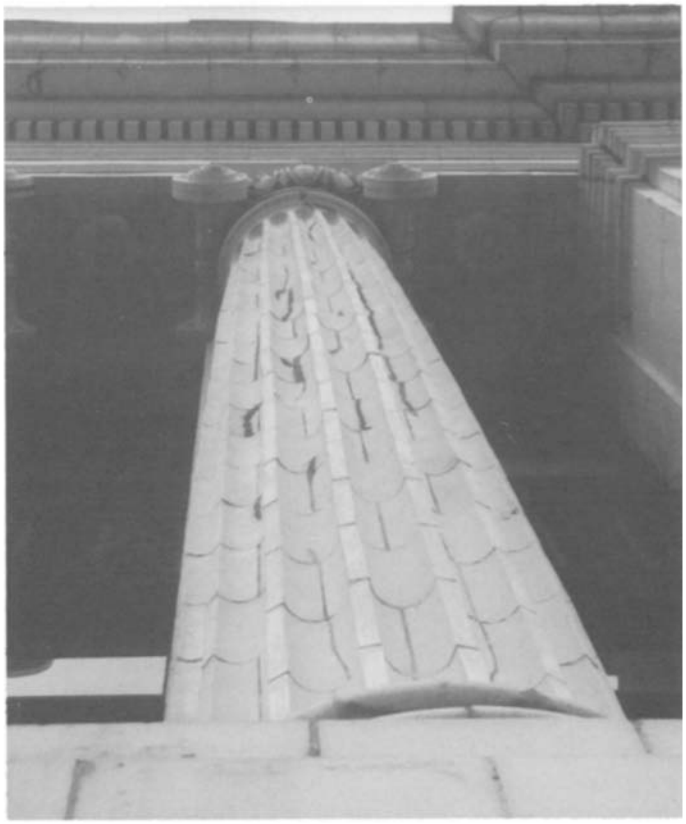

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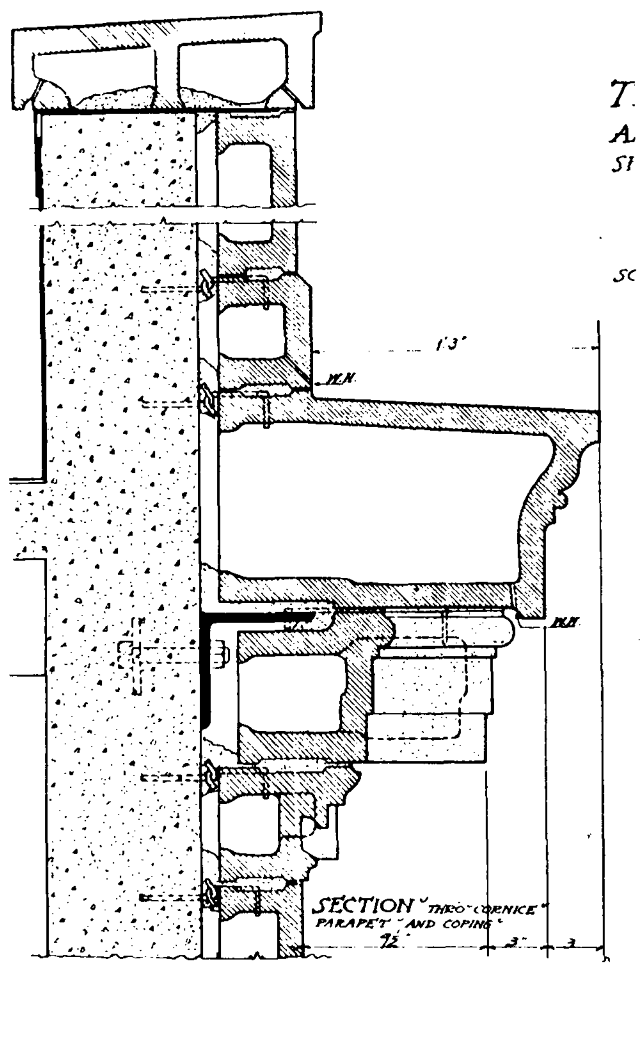
Figure 10: *Terra Cotta Cladding Damage.* Cracking and infiltration occurred primarily as a result of restricted movement and excessive cumulative compressive stress in the outer cladding. Because no movement allowance was provided beneath the outrigger beams, masonry weight on the beam became transferred directly onto the top-of-the-wall below. In the worst cases, where steel beams' cavities were poorly infilled, loads from above were transferred only into the facing. The facing, which was intended to be a non-structural cladding, supported the loads from the exterior walls of the stories above.

Figure 11: *(right) Early Design of a Relief Angle (1927).* Documented by the National Terra Cotta Society's Manual, this detail shows how structural separation should occur between the structural member carrying the floor's weight and the top of the wall below to prevent cumulative loads. While the concept of masonry "curtainwalls" existed since the 1800's, the evolution of details to truly accomplish the concept took decades. The Manual also recognized the influence of moisture in the wall by adding flashing and weeps to evacuate it back to the outside and help prevent corrosion of the metal components. Corrosion protection remains an issue receiving serious study even today.

material. When frame engineering provided the opportunity, architects lifted their designs with it.

Real estate developed with escalating commerce. Architecture responded to its unique demand for economical space by perfecting the uniquely American skyscraper. More floors on the same footprint required lighter building skins. Added repetitive floors meant taller building shafts. Larger windows gave better views and ventilation, which created more desirable tennant lease space. Unique architecture peaked interest. Better space commanded and yielded higher rent.

In these first forty years of curtainwall development, builders constructed brick and terracotta walls using virtually unchanged traditional multiple-wythe tied techniques. Masonry curtainwalls simply adapted to this approach by relieving its weight at each floor. Craftsmen did not change their techniques. In the past, buildings' bases commonly used loadbearing stone to support tall spans of brick above. Its superior compressive strength resisted the crushing weight of the multistory bearing walls above. Load relief at each floor reduced the bottom wall mass. Still, architects continued using stone like in their previous designs because it was durable. The familiar style of ornament continued without altering wall construction or manufacturing methods until well into the twentieth century.



## CONSEQUENCES LEARNED FROM FREEING THE FAÇADE FROM THE FRAME

Because a curtainwall is not as structurally integrated with the skeleton behind as was its preceding multistory bearing wall, its appearance also could become independent of that frame. This novel construction development separated aesthetics from structure to designers' delight. Still, the skin remained functionally dependent upon the behavior of the primary frame behind it. Lighter iron and steel building frames in taller, thinner building profiles move more. Substantial lateral bracing must stiffen the towers to keep these dynamic movements within the capabilities of the skin to accept them.

Curtainwall's transfer of facade weights to the skeleton at separate floors assisted stone onto high-rises similar to its approach that assisted masonry. Lighter and thinner walls on higher wind load locations required intermediate structural framework between that skin and its building structure for reinforcement. Stone panels larger than unit masonry offered potential installation economy. True curtainwall philosophy uses metal's tensile and flexural strength in the intermediate frame to reinforce those properties that masonry lacks. Theoretical masonry curtainwall concepts combine stone and masonry on a substructure to the building frame. This secondary sub-frame could hang like a curtain from each floor's edge.

Using materials inherently strong in the components needing that strength minimized exterior wall thickness and the amount of materials composing it. Weight and cost decreased. With building height increases, the wind loads they experienced increased exponentially. Movements grew with those loads. Weather effects infiltrated more readily. These problems were diagnosed later once deterioration exceeded the usual condition of the walls. Lateral strength, stiffness, and weather-tightness performance criteria began to evolve. Stones could be added to exterior walls on these frames if its rigid body was effectively isolated from the dynamics of the interfacing components. However, construction habit and engineering ignorance still used stone and large masonry units in traditionally assembled rigid bays that failed in flexure. Comprehending the behaviors between interconnected cladding and framing parts became the hidden formula to properly designing lasting curtainwalls with stone and masonry.

Only fewer connections between the facade and frame promised to resolve the differential movement problem. While this movement issue did not solely direct skin-and-frame separation, early designers soon learned that it controlled exterior wall mass. Severe deterioration in masonry-backed terracotta occured unexpectedly in young buildings (Fig. 10). Owners and architects especially objected to the visual damage. Owners and engineers feared the corrosion and infiltration following that destroyed the wall's structural and weathertight integrity.

To remedy these faults, masonry curtainwalls eventually added movement joints at each floor. This was a "soft" joint to absorb shear from interstory lateral sway, vertical column length changes, and slab-edge deflection. Exposed frames experienced increased column changes due to thermal effects. Occupants and dead-loading increased thin floor deflections. Sealant filled this soft joint located beneath each floor's relief angle. Eventually, behavior between bays was discovered and conquered with vertical movement joints when problems persisted (Fig. 11).

## ARCHITECTURAL FASHION EXPLOITS A SKIN SEPARATE FROM SKELETON

Lighter skins allowed lighter superstructure perimeters. Inflexible masonry did not accept movement without cracking. Glass and aluminum systems using movement joints could. Greed for bigger windows, more light and view from the higher vantage, and maximum rentable area made skins thinner. The European-envisioned International Style exploited this advantage. Still maturing steel skeletons with limber connections discouraged carrying massive materials. Stone was still slabbed thick in its loadbearing tradition. Interstory drift and column length displacements with spandrel deflections already far exceeded traditional cladding capabilities. The skeletal frame's floor-to-floor behavior was incompatible with monolithic masonry. More mass aggravated this.

The metal and glass aesthetic mounted popularity through this century's second quarter, especially in Europe. The "functionalistic" fashion captured the spirit of higher technology. Lead by visionaries like Le Corbusier, the appeal of the machine made it an architectural icon. More durable equipment manufactured building and cladding components. Designs such as New York City's Chrysler Building created sleek machinelike enclosures made of metal and glass mixed with narrow masonry piers. Architectural style sought to exalt industry as the power driving modern culture.

## RELUCTANT REJECTION OF TRADITIONAL STYLES

An opposite opinion on architectural style continued to borrow historic elements. That eclectic approach of borrowing familiar forms faded slowly. Some designers reconfigured past Gothic and Classical parts built of new materials such as the Chrysler Building's metal gargoyles. This ecclectic attitude culminated during the Chicago Tribune Tower Competition. Its global entries and its award both exhausted the final uniquely American era and initiated the Modern movement.

Corporations wanted new images. Copied old designs connected companies with the past losing them commercial notoriety. Rather than experimenting with other classical recipes, Europe progressed with modern architecture clad in metal and glass. Their cities had the "originals" that the revivalists had copied, there was little interest in more. To get current, once again Americans simply applied the International Style vocabulary in their exclusive height-obsessed capitalistic context.

High-tech aesthetics emphasized capitalistic individuality. The International Style in America borrowed the European-professed intellectual forms and surfaces and extruded them into taller and taller versions. This eliminated the inherent scale of masonry facades that pedestrians felt comfortable with. Without precedents or formal references, the oversized style lost human relationships. Expansive smooth shiny surfaces offended human senses. Buildingscapes became increasingly glaring and noisy. Streets became alienating caverns of characterless reflections.

## UNEXPECTED ENVIRONMENTAL PROBLEMS WITH EARLY "THIN" WALLS

Even revivalist purists, once only comfortable constructing with the texture and irregular appearance of brick and stone, pursued the crisp lines of the new architecture. The alien style appealed to the intrigue of their intellect rather than the familiarity of their past. The new exposed materials were divorced from pedestrian experiences. Sensory stress in that environment escalated. The International Style interpreted anthropometric scale, color, and texture in a completely unknown language.

Curtainwall envelope performance was unproven and undeveloped, though. Lacking the typical two-foot thickness of loadbearing walls, early curtainwalls functioned quite differently. Rapid realization of physical and experiential dysfunction slowed the race to the new exterior wall method. Expectations caused this concern as much as undeveloped technical means. New problems required advanced technological responses. Sealant formulations lagged behind movement and modulus needs of the glazed wall's joinery. Higher altitudes caused greater weather extremes due to a reduction in surrounding protection. Faulty seals allowed enormous air and water infiltration after only short lives. Light metal conducted cold quicker than wood, masonry or cavities, thus lack of thermal mass or thermal break caused frost and sweating on interiors. Different metals contacting other building materials corroded profoundly. The high-tech facades soon looked ruined.

Traditional masonry buildings seemed to grow increasingly endearing as they collected dirt and weathered, even if they looked commonly familiar. Eroding enthusiasm for deteriorating buildings built to model the modern machine, together with their disappointing performance, motivated a gradual resurgence of the conservative masonry traditionalists.

Ironically, preoccupation with architectural fashion was the same disposition that discounted terracotta and ignited acceptance of slick curtainwalls around 1925. The best of both blended in the exterior wall developments that followed. Erection methods invented for glass and metal were applied to primarily masonry practices. Unit masonry became modularized to curtainwall criteria. Support connections became reconfigured to attach to the steel skeleton. Thinner stone became increasingly available as fabricators modernized slabbing machinery. Foreign producers welcomed thinner stone, for less weight per unit area was cheaper to ship overseas. This made their stone more competitive and brought new sources to the market. Re-introducing the familiar materials that were compatible with human experience made both the building and the sidewalk more inhabitable.

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**FIGURE 12:** ***Mountainside Quarrying of Marble in Carrara (1993).*** **The first step in producing stone is mining it from the earth. Removing the "raw" stone material from its source in the earth is called quarrying. The Italian moutainsides near Carrara have been supplying stone for building and industrial uses for centuries. Michelangelo chose his blocks from one of the quarries. Excavations into the mountain yield blocks that are lifted by derrick to the rim and then trucked to the fabricating yard. All quarries are different, due to their natural geological formation of the deposit in the earth and the structure of the landforms around it. The process of removing the blocks from the ground and "seasoning" them is just as important to the stone's eventual durability as the many fabricating, design, and installation procedures that follow. Millions of years of encapsulation, pressure, and moisture are released when the block is quarried. The stone should be allowed to gain environmental equilibrium out of the ground before fabrication begins.**

FIGURE 14: ***Transporting Blocks from the Quarry to Fabrication*** *(1990).* Blocks removed from the quarry are typically limited to the size that the fabricating equipment will accommodate, unless the end use is special. Gang saws, jointing, and finishing equipment may accept blocks up to seven feet tall unless the end use is special. To optimize yield, that is, minimize waste, blocks are usually quarried slightly larger than piece sizes needed for the project. The amount of oversizing depends on how irregular the quarried block shapes are, the aesthetic characteristics of the stone such as veining or color concentrations, and potential rift planes in the blocks. Blocks are inventoried and moved to the fabrication site using heavy-duty equipment. It is a mining operation. Blocks removed from the quarry are all numbered and recorded so that their relative locations are known. This maintains a history of material performance, yields, and aesthetic consistency for material from that quarry.

FIGURE 15: *(below) Diamond Wire Saws in the Danby Quarry (1993).* The advent of industrial diamond production and then their introduction to the operations of the stone industry has revolutionized nearly every aspect of natural stone production. Wire loops with diamond wire segments are being used in the quarries to remove layers and blocks from the beds in the famous underground Danby Quarry in Vermont, once the pride of the Vermont Marble Company and now owned by the Italian firm R.E.D. Graniti through Vermont Quarries. Diamond wire loops not only expedite removal of material from the tight confines of the underground chambers, but also impose less physical stress on the stone without blasting or wedging. Because sawn block sizes can be much more closely controlled, waste is dramatically reduced. Quicker extraction and less waste can result in lower costs.

## ENGINEERING ANALYSIS EVOLVES WITH CONSTRUCTION INGENUITY

When bearing walls were both structure and facade, the single-entity behavior was predictable. The masonry system had been perfected over centuries of experiment and intuitive refinement. Behaviors of separate skeleton structures with multiple-component sophisticated skins could not be easily predicted. Experience with multistory frames was in its infancy.

The exterior wall's design depends upon the frame's stability. Likewise, the frame responds differently to potential loading combinations on the cladding. Only advent of matrix methods

FIGURE 13: ***Pneumatic Diamond Drilling in the Quarry in North*** *Carolina (1990).* This drilling rig uses compressed air to pneumatically drive a diamond drill bit with a hammering action into the granite bed of North Carolina Granite Corporation's Mt. Airy Quarry. Notice from the background that this quarry is flat and open, nearly the opposite of the Carrara site. Holes are drilled to a depth where a rift or shelf plane occurs, or to a depth corresponding to a block size. They are spaced close enough to either hydraulically wedge or blast free layers of stone from the quarry bed with powder charges. The layers are then sawn or broken into blocks to be transported to the fabrication facility. This material is being quarried for the Amoco recladding in Chicago.

compiled by electronic computers could these complicated interactive relationships be analyzed. Reduced-mass skins move more. Variable occupancies and more volatile environmental exposures expanded loading conditions. These greater complexities advanced structural engineering. Proper analysis and more comprehensive models increased understanding of the combined systems. Each component could be coherently designed to fulfill its specific function. Accurate engineering defined those functions. Expanded engineering awareness accelerated building accomplishments.

## ADAPTING STONE TO FIT INTO METAL CURTAINWALLS

Industrialization brought more than mechanization. Factories producing standardized fabricated building parts for quicker assembly in the field caused radical realignment of the human portion of construction...labor. New skills, new equipment, and accelerated schedules dramatically changed how labor was used. Traditional methods of masons carving on site were obsolete because there was no longer that time on large urban projects. The stone industry had to adapt or abstain from the fortunes in commercial building.

Stone's cultural appeal enticed it into new curtainwalls. The mid-twentieth century manufacturing mindset discouraged inefficient on-site piecemeal methods. It favored the more expedient approach that used factory-fabricated building components that could then be assembled in place. Stone, unlike brick and terracotta, could be made in large, structurally sound pieces to cover more wall at once. The early twentieth century automobile industry proved that prefabrication saved time and money while raising production. Assembling these finished parts, especially larger ones, minimized the inflating cost of organized labor. Completion quickened, standardization maximized interchangability, and quality increased at lower costs. Ideas applied from consumable goods production like the automobile industry revolutionized the previous cut-to-fit-in-place construction culture.

Constricted urban sites and short schedules prohibited the old approach from continuing. Field-fabricated stones fitted and installed individually became extinct in high-rises. The process consumed too much time and capital to support an appreciable workforce that anxious prospective occupants just could not afford to wait for. Stone manufacturers began to realize what other industries had learned, that replication and part interchangeability answered new market demands and also promoted consistent quality and accuracy.

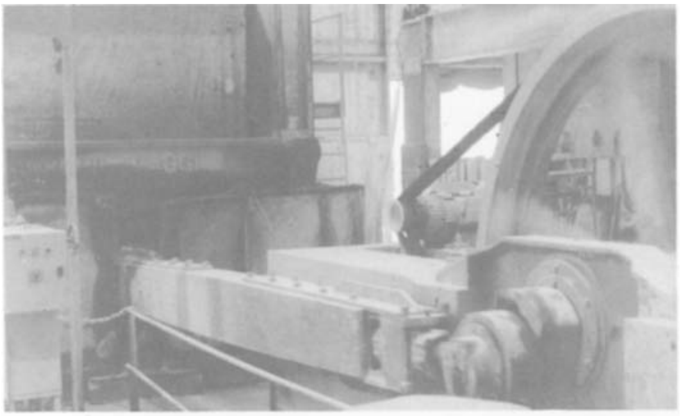
Curtainwall construction required stone to integrally fit within the exterior wall framing system. Large walls used many similar pieces. For output to increase both quantity and precision, mechanized stone production replaced antiquated quarrying, sawing, and finishing methods. Metal curtainwall substructural frames adapted to include stone. Conventional stone panels tied back to masonry wythes that encased the frame of the building. Their large size and extremely heavy weight required direct support and anchorage. Evolving architectural styles articulated facades that allowed smaller and thinner panels.

Buildings having homogenous, massive masonry walls with small windows hid their leaks by absorbing infiltration. Thin walls built of heterogeneous factory-manufactured components could not hide their leaks. The diversity of materials having different expansion properties in large panels dramatically increased differential movements between their unit boundaries. Early oil-based sealants did not accept those amounts of movement and thus failed to keep the joints closed. Polysulfide rubber formulated in the thirties started to accommodate the large movements and could keep the joints tight, even between dissimilar cladding materials. By mid-century, performance standards raised sealant's quality, and with it raised the environmental integrity of the mixed-material curtainwall envelope.

## MODERNIZED DIMENSION STONE MANUFACTURING

Eventually, "structural" stone production specialized to meet the competitive demands of the construction economy. Blocks extracted from the quarry became more regular due to new drills, saws, and handling equipment (Figs. 12, 13, 14, and 15). Yields improved. More-regular blocks slabbed better in the multiple-wire loop saws, and later the gang saws that replaced them. Gang saws divide the quarried blocks into thin slabs with groups (or "gangs") of vertical, parallel metal bands that are sawed back-and-forth over a slurry matrix. The solution of water, lime, and an abrasive cutting ingredient moved by the blade grinds through the block. The evenly-spaced blades cut the block into vertical "rough" slabs of relatively equal thickness (Figs. 16, 17 and 18). Compared to wire, more dependable and durable gang-saw set-up and maintenance reduced. Blades tended to wander less than wires, increasing thickness consistency. The slurry is recycled and replenished with fresh abrasive such as carborundum to maximize efficiency. Later, industrial diamonds were added to the plain blades. Water lubricates and cools the diamonds and flushes the saw grooves clean.

Mechanical finishing and "jointing" beds quickened production. Rolling tables move the slabs from their vertical orientation in the gang saw to horizontal on the finishing line. In a polishing line, rough slabs pass through a line of spinning heads fitted with progressively finer abrasive pads. Again flushed with water to cool and clean the surfaces, the machines apply up to 3500 rpm under perhaps 2000 psi to smooth the sawn face to a glasslike polish (Fig. 18). Other lines may sandblast or "flame" the surface for rough textures. To cut the slab to finished dimensions, conveyored beds align it beneath movable saw heads suspended on beams overhead (Fig. 18). Similar adaptations of this equipment can cut edge kerfs, quirk miters, or drill anchor holes in the edge or back of the cut-to-size panel. Computerized drives now syncronize positioning of the bed and saws or drills needing only a few minutes and the strength of one programmer on a keyboard. These tasks used to consume hours for hundreds of men and required all



▲
FIGURES 16 and 17: *Gang Saw Cuts Blocks Into Slabs (1992).* When producing stone panels, the first stage of fabrication involves slabbing, or "slicing" the quarried blocks into slabs. Usually several blocks are cribbed into the chamber beneath the "gang" of parallel blades. A large flywheel strokes the blades back-and-forth through the block and cuts grooves through the block until parallel, vertical slabs remain. Spacing between the blades is set according to the required panel thickness, allowing for tolerances of sway and wander. Gang saws used to use smooth metal blades which moved a cutting medium such as water with sand or carborundum through the grooves to remove the stone. Placing diamonds on the blade edges eliminated those cutting media while quickening cutting and improving accuracy. Water is still flushed over the block to clear the grooves, then filtered, and recycled to be flushed over the block again.

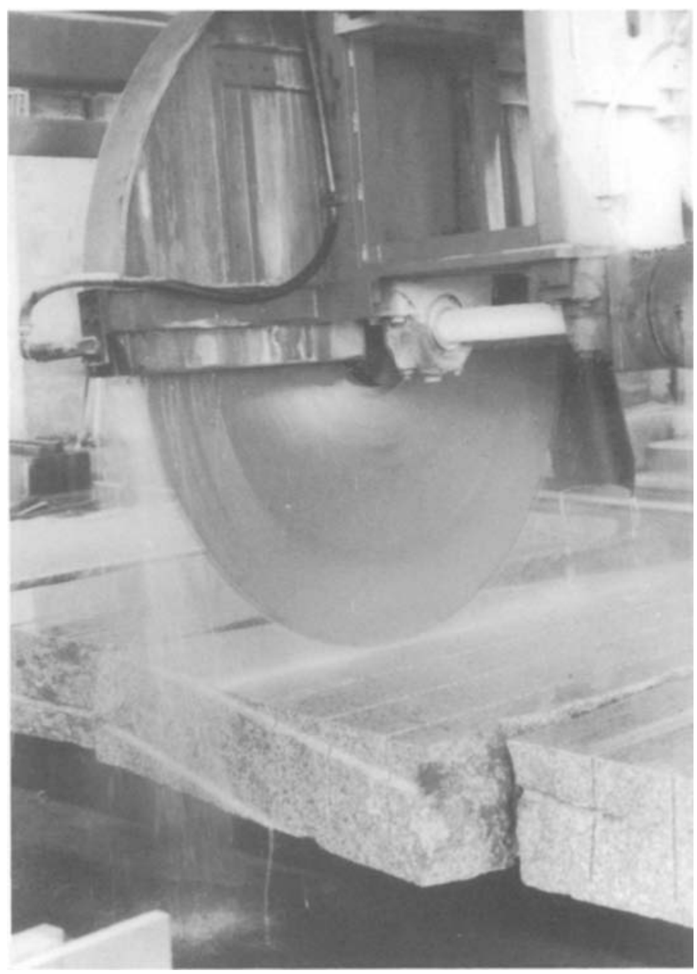
FIGURE 18: *(right) Automated Polishing Line (1992).* Spinning heads of abrasive pass over the sawn labs under up to 2000 psi pressure in an automated bed to put a finish on the slab faces. Typically, the line smoothes the surface incrementally, with the first heads being course grit to remove gang-saw grooves, and the final heads being fine grit to produce the final finish. Diamond matrix heads specially formulated for the type of stone are now almost exclusively used in gradually increasing fineness to finish the surfaces. The bed of the line is actually a conveyor that moves the slabs through the line of progressively smoother heads. Different finishes such as honed, high honed, polished, and "mirror" are possible on the lines if the stone material is polishable. Rate of speed, pressure, and "grit" vary according to the stone type. Flaming, or thermal finishing, sandblasting, bush hammering, cleaved, and other finishes may be available depending upon the type of stone considered.

their combined strength and endurance. A more consistent and abundant product results (Figs. 19-22).

Shaped-stone also benefitted from mechanized technology. "Cubic," or thick-profiled configurations that emphasized relief and depth once prolific in terracotta are again popular. They are commonly combined with flat stock to articulate stone facades. Where gang saws divide flat slabs, automated rotary disc saws on rotatable heads carve almost any shape imaginable. Irregular shapes are fabricated by following a full-size template with sequential passes of the rotary blades.

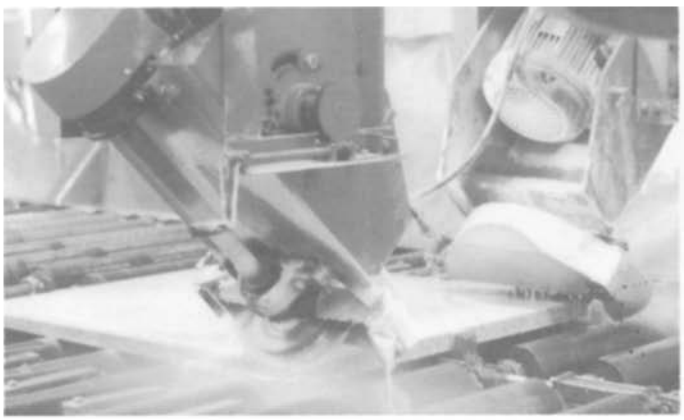
Thinner stone caused new approaches to anchorages and stone fabrication. Some designs desire thicker appearances or solid corners like the "old" buildings, but need to avoid the mass. Single-piece orthogonal profiles can be built by gluing and pinning flat-slab sections of stone together. Newly formulated chemical adhesives adhere to the densest stones and all different minerals. Correct preparation and cure maintains lamination in extreme exterior environments. Liner blocks and pins reinforce these joints to make tight structural edge seams. Interlocking pins or plates engage both the liner block and the face pieces to mechanically lock the two together without disengaging. Pins are independent of the epoxy. Epoxy can be stronger than the stone itself.

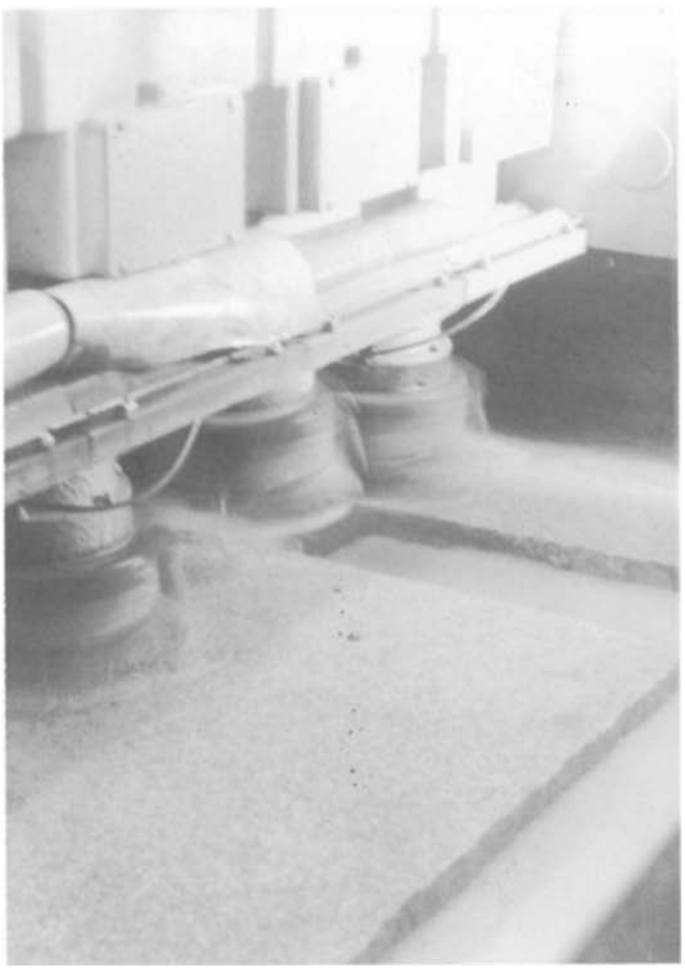
Possible assembled-shape profiles and configurations are endless with quality fabrication. But while prefabricated stone



assemblies create the shapes that architects imagine, they have also been the most vulnerable to deterioration. Adhesive can theoretically transfer forces between laminated stone pieces. However, durable and lasting chemical bond is feasibly unverifiable. Pins complete the connection. Adhesive alone is not dependable because too many uncontrollable variables influence its performance. Improper stone surface preparation, different materials, adhesive mixing, movement during curing and improperly controlled curing climate compromises the joint's integrity. Past problems also exist with pins when quality assurance measures are not fully employed. Pins must engage both laminated pieces. Set up drills and hardware to verify engagement. Design pin sizes and frequency to mechanically transfer the whole load. Ignore the contribution of the adhesive.

But superabrasives and diamonds have transformed stone use the most. Successful production of high-quality synthetic industrial diamonds and carbides revolutionized the fabricating machinery used in quarrying, slabbing, shaping, cutting, and finishing. In 1955, General Electric's H. Tracy Hall's scientific team invented the belt that simultaneously encapsulated the one-million lbs./in.$^2$ pressure and 3300 degrees Fahrenheit to convert graphite to diamond. By the mid-1960s when commercial diamonds arrived, the modern building boom moved the stone production industry to apply them to stone cladding.





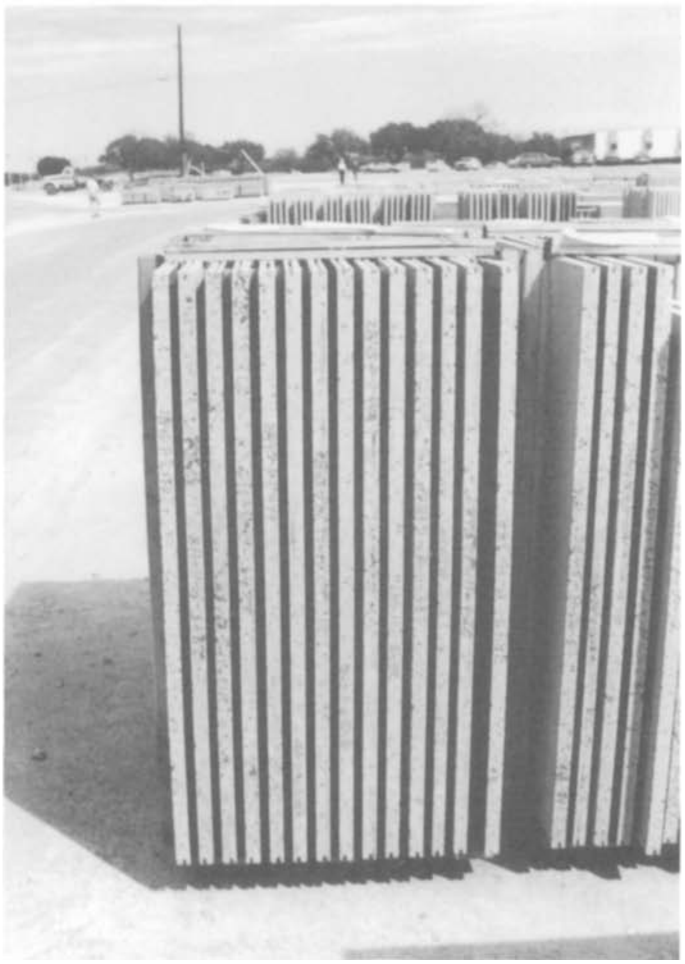
## STONE'S POTENTIAL IN ARCHITECTURE IN THE FUTURE

Over the last two decades, architectural projects increasingly adopt natural stone as their preferred building cladding. The offensive effects of the mid-century's glass and metal aesthetic grew less acceptable. Ecclectic classical styles and re-interpreted elements are again fashionable in the Post-Modern climate. Manufacturing technology advanced. Thinner, lighter, less expensive, and easier-to-obtain stone fits economically into metal-framed curtainwalls. Conversely, curtainwalls developed to more easily accommodate stone. Often, cladding systems are unitized. Stone's mass provides more thermal and

▲ FIGURE 19: ***Automated Jointing, or Cutting-To-Size (1992).*** **Usually after surface finishing, the finished slab is cut, or "joined" to specific sizes or dimensions (hence the name *dimension stone*) also on automated lines. Sequenced operations cut length, widths, and even, as in this figure, quirk miters. Cuts are completed automatically by small diameter diamond radial blades each on its own rotatable head without handling the stone. Water is always used to cool and lubricate the blades as well as flush the groove clear of abrasive stone dust. After sizing, similar equipment cuts kerfs in edges and other types drill holes for anchorage devices to mechanically engage the stone.**

◀ FIGURE 20: ***Radial Diamond Saw For Cubic Shapes (1992).*** **Stones that are flat panels, but thick, blocky, or with a spacial cross-section profile are called " cubic" because they are not fabricated from thin slabs. In the fabrication of cubic shapes, large diameter saws mounted on traversing beams cut the blocks into thick cross-sections. The saw blade passes across the slab multiple times in the same groove, each time dropping slightly until the depth is complete. A narrow strip is left uncut, which will split. Blade drives are computerized to maximize accuracy and minimize labor.**

◀ FIGURE 21: *Sorting for Packaging* (1987). Once the dimension stone fabrication is complete, including anchor preps, each piece is to be checked for accuracy against its shop fabrication ticket. To control quality, any variances in the stone's configuration from the designed shape must be within the specified tolerance. The tolerance is the maximum acceptable variation from the theoretical shape. The stone's individual mark number is indelibly marked on an edge, or several edges and the back, which will not be exposed in the final installation. The identification marks should be easily viewed by those handling the stone during shipping and installation. Here, sunset beige granite quarried and fabricated at Marble Falls, Texas for the AT&T Corporate Center in Chicago is sorted before being crated outside the fabricating facility.



◀ FIGURE 22: *Crating Finished Stone Panels for Shipment (1987).* Stone panels are crated and await loading onto air-ride (cushioned) semi-truck flatbeds. Some crates, especially of small or irregularly-shaped pieces, may be closed. All panels are marked on their edge with the job number, piece mark, and which one of the sequence of duplicated configuration that panel is. A separate packing list should accompany every crate. This must be verified by the recipient prior to acceptance. Production, crating, and shipping should be sequenced in the same order as the installer's erection sequence to avoid rehandling, damage, and replacement pieces which both delay erection and likely will not match the surrounding stones. Crate sizes and weights need to be limited to the capacities of the equipment handling the packages. Weight of the crate plus equipment cannot exceed the capacity of the building under construction. Orienting panels vertically in the crates at the fabrication facility, which is typically the final orientation for the panel on the facade, minimizes the potential of edge damage caused by rolling the panel upright from the horizontal position. Vertical crating also minimizes extra handling if the panels need to be removed from a crate out-of-sequence. coordinating this engineering, fabricating, crating, and delivery process to match production in the field is a monumental task which can have considerable impact on productivity and quality.

sonic isolation than metal or glass. Natural stone's low maintenance and weatherability make it durable. Its unique natural character and endearing beauty with age makes it the skin of stature. Stone covers building facades whose aim is to present a distinguished architectural statement.

Stone suggests permanence and richness. Architects recognizing this are attracted to it. Where mass, gravity, friction, and stacking were the construction methods of the past, multiple versions of dowels, grooves, kerfs, and epoxies are the modern methods attaching thin stone panels to its backup. Stone now should not support any loads other than its own weight. Recent designs metaphorically imply massive appearance by assembling thin slabs into built-up shapes that disguise their thin section. Stone can now be manufactured so thin and in face sizes so large that panels lack the capacity to support their own weight. Adding a superimposed wind load requires reinforcement or special anchorages.

Engineering practice needs to coherently consider the many factors that influence each stone. It must check each stone's function as an independent structural component and also its functions in combination with the other interfacing systems composing a building's skin. Organized, sequenced study of these behavioral considerations elevates reliability, performance, and economy of exterior walls clad in natural dimension stone.

Stone is again a fashionable building cladding. Unlike its previous periods of popularity, stone is now skin alone instead of also being part of the building structure. A rational, sequential analysis of aspects influencing stone performance will substantiate a design's validity. Reproducible, objective engineering designs have consistent quality and safety results.

This text presents an overall approach that directs selection, design, and installation of stone in the context of modern construction. Effective design is only possible when following a comprehensive and uniform process. Applying the process improves both economy and safety. Ultimately, this reliability enhances natural stone's visual appeal.

Chapter 3 *The Future of Stone Cladding* outlines the considerations important in selecting, designing, and installing dimension stone and its anchorages. It will assist further development of the exterior stone industry, free designers and architects, and improve public well-being.

3

# THE FUTURE OF STONE CLADDING:

## Toward Load-and-Resistance Factor Design

CONTEMPORARY architecture continues to present increasing opportunities to use stone as cladding systems. Dimension stone is more available because fabrication and installation are more economical. Expanded engineering and construction experience need to be included in approach that objectively addresses influencing considerations. The modern methods of design and construction need to be applied to stone cladding.

Chapter 4, *Developing Responsible Design Values* suggests how material, system, and application considerations fit load-and-resistance factor engineering design.

Chapter 5, *Guide Specification for Stone Systems* applies the considerations to the design together with experience gained from past installations. The specifications outline principles consistent with this comprehensive "new with old" new engineering approach. The objective, thorough approach improves cladding quality.

Reliability must be maximized to ensure public safety. Public confidence in stone construction can be improved by individually evaluating multiple considerations to establish a "safety factor". Load-and-resistance design takes this approach. Traditional, and sometimes considered arbitrarily assumed safety factors do not adequately address the stone and building industry's knowledge of natural stone materials, its anchorages, its support systems, or building behaviors in modern applications.

Economy must be maximized to increase the value of stone facades. Lessons of experience applied to similar conditions reduces initial costs by improving known practices and reduces end costs by improving durability. The financial advantages improve the public's innate cultural appreciation for stone.

Load-and-resistance factor design (LRFD) enlists this knowledge and can achieve these goals. This approach should be developed as the engineering standard for establishing the structural integrity of stone systems.

Correct organization of a design process enables us to apply it appropriately. Applying the stone design process includes deriving variables (or considerations) that influence performance. Evaluating these considerations rationally allows them to become proper engineering and construction criteria. A modern approach to exterior stone cladding must be increasingly more objective and less arbitrary to improve wall quality in the future.

This manual outlines a process to conceive and construct stone cladding in the modern engineering and construction context. The slim legitimacy of usual practices by tradition and habit are only supported by "conservative" design. But the results indeed are not always "conservative," or even lasting. The presented process begins with principles learned practicing safety-factor based "allowable-stress" approach. This knowledge founds load-resistance factor design. The most obstinate obstacle to instituting this more rational approach is architects' and engineers' roots in their old routines.

This manual reviews many aspects that influence exterior dimension stone, its anchorages, and support systems. It analyzes the aspects considered by an architect who desires an aesthetically appealing and quality-finished product. It analyzes the aspects considered by an engineer who conceives a functional exterior wall assembly that is compatible with other interfacing systems over the structure's life. It analyzes the aspects considered by a contractor who expects to install a reasonable system that can be constructed economically. Combining all these satisfies the owner who expects the cladding to work, be durable and be easily maintained.

Any solution must protect public safety. The appearance of the stone facade, its durability, and the economy of its construction are secondary to safety. Considering all the aspects from each point-of-view create a dimension stone-clad exterior wall that will confidently perform safely.

This manual dissects issues into a design. Issues are organized to establish sequenced objective criteria for selection, design, and installation. They are founded upon proven practice and past exemplars. These criteria are parameters for conceptual design and must continually be applied through that design's construction. Some already know and perhaps inconsistently consider these criteria. This disassociated approach makes the industry appear disorganized and less genuine. The approach lacks the significance resulting from applying the influential parameters in sequence. Compiling these segments at the right time in the process makes the conclusion understandable and logical. Linking testing procedures into this process adds measurable objectivity to the procedures. Reviewing similarly built stone skins throughout the process gives engineering and construction decisions validity. Thus, the rational approach substantiates its results with a coherent process. Stone cladding must follow this approach toward LRFD.

Only recently has exterior wall stone cladding construction been intently assessed. Forensic investigations of problem facades motivated new engineering roles. Actual engineering now begins with stone selection and continues through final installation. Responsible engineering includes more subjective analysis of similar real-world exemplars than it does "objective" theoretical laboratory tests. Existing stonework proves its performance by its endurance, or lack of endurance, in actual exposures through real time. Any test presupposes conditions due to procedural assumptions whose correlation to actual environments is not always known. Review of stone's precedents reveals how architectural styles and traditional technology failed to move the stone industry from centuries of bearing-wall techniques. It took the invention of the skeleton frame and the troubles of terracotta to convince stone producers to begin developing new methods. Not until the last two decades have those expanded fabrication and attachment techniques adapted stone to fit within lightweight curtainwall construction.

Fabrication means and aesthetic appetite evolved to promote that adaptation. New structural engineering computer analysis predicted dynamic behavior of skeletal frames. Failed weather tightness in metal-and-glass curtainwalls spawned new sealants and advancements in assembly that also accommodate stone. These discoveries blended with existing stone practices to clad tall buildings with stone veneer.

Skyscrapers significantly accelerated thin-stone veneer's development. Without increasing height demands, stone could remain massive, thick, and similar to its medieval uses. Extensive material and assembly testing along with forensic study of early-generation "high-rise" facades continue to steer stone science. Newer, bigger buildings employ those lessons learned. This manual includes the considerations learned from those lessons. Together, investigation and replacement of old stone facades provide the most significant momentum for modern stone design and construction that evolves today.

The true challenge for dimension stone designers is to transform typical stone engineering practices into a modern format accepted by other structural disciplines. Even though the "arbitrary" approach has avoided catastrophic failures so far, most stone-clad exterior curtainwalls function poorly and are experiencing hidden deterioration. The responsible approach pursues rational, objective evaluation that recognizes the nature of stone as a material and also its behavior in its intended application. Stone material is variable and its applications vary. Irrationally determined or underived safety factors that do not objectively consider these variables can not guarantee safety.

Load-and-resistance factor design suggests that several primary uncertainties that strongly influence stone panel and anchorage performance can be categorized. Each element can be eventually figured, then formulated into a whole "equation." Testing methods correlated to exemplars evaluate those uncertainties objectively. Statistical methods can be used to reference actual stone construction and translate their results into terms of probability. Probability defines engineering risk and reliability. The overall stone selection, design, and installation process must render an exterior wall product that is consistently reliable by society-accepted standards. The present method using single safety factors results in fluctuating reliability because coefficients are relatively constant while uncertainties are not. Deliberately accepting changing reliability is not responsible. Load-and-resistance factor design promotes interpretation of pertinent uncertainties individually to enable consistent reliability.

Most other structural engineering disciplines adopted the load-and-resistance factor design philosophy as their approach to design. That format best includes variable uncertainties and variable applications. Modern stone cladding involves many influences that can follow that same process. Load-and-resistance factor design comfortably integrates and applies those many influential considerations. New considerations not yet contemplated can be added as future research requires.

Stone cladding designers need to complete more research to fully develop load-and-resistance factor design for dimension stone cladding. The research can follow this manual's framework. Further testing of the known predominant influences correlated to existing safe skins will give each aspect legitimate objectivity, and will be the fundamental process for establishing LRFD values. The LRFD equation predicts interactive behavior. Once compiled into an evaluation equation, combined individually derived reliabilities for uncertainties establish "true" reliability for the cladding system.

Uniform practice of load-and-resistance factor design will expedite the gathering of correlating data. Architects, engineers, consultants, contractors, and owners on behalf of their own liability and their ethical responsibility to the public must work toward consistency reliability. Partnering early in a project joins the necessary expertise to accomplish this goal. Pursuing this goal together will advance the stone industry to all our benefit by providing safer, richer facades clad in stone.

4

# DETERMINING RESPONSIBLE DESIGN VALUES:

## Formulating Load-and-Resistance Factor Design for Exterior Stone Cladding

EXPERIENCED engineers make design judgements based upon the information pertinent to the project. Responsible decisions consider objective testing of novel portions of a system with subjective comparisons to similar existing work. Regardless of the size of the project, using stone mandates consultation with a qualified designer and experienced installer to determine which information applies to that project.

Once the appropriate information and previous examples are gathered, interpret test values and balance these presumptions with anticipated exposures. Conceive an exterior wall system that maximizes economy and performance without compromising safety or durability.

Fine-tuning the concept involves individually evaluating "uncertainties," which are variables that effect reliability. Reducing risk of failure increases a system's reliability. The process of refining risk begins during initial stone selection and continues through the completion of construction. Even after completion, assure proper cladding performance with maintenance inspections and required intermittent repairs. This identifies any conditions that were not properly predicted and upkeeps shorter-life components.

Stone became thinner as modern exterior walls evolved to become lighter. Stone safety factors did not develop with this change from their masonry heritage. The many aspects that vary between stone applications are now usually lumped ambiguously under a single safety factor depending only on the type of stone. These empirical safety factors arbitrarily hide true reliability behind seemingly large coefficients. They ignore individual uncertainties by remaining constant without regard to the application or backup for the cladding. Their true reliability is unknown. Economy is sacrificed when a safety factor overestimates risk. Safety is sacrificed when a safety factor underestimates risk.

The empirical safety factor approach "designs" stone with unknown margins of safety. Thin-stone failures are surfacing after only a decade of existence. Seemingly inflated factors do not assure safety or durability. Simply oversizing support or thickening stone panels do not necessarily forgive the failure to design and build cladding to work within the dynamics of the exterior wall system. Interactive behaviors must be deciphered, analyzed, engineered, and constructed properly.

Unique cladding applications require new evaluation techniques. Evaluate concept, testing, exemplar, engineering, depiction, specification, construction, and inspection techniques.

The different aspects of stone and support materials, systems, structures, and environments, in a project can be segregated into performance variables. These can be individually evaluated as engineering "uncertainties" to be more responsive to modern cladding construction.

Professional designers and installers can achieve more economical and reliable exterior walls by analyzing different uncertainties separately and then controlling them individually during construction. Experience from both successful and troubled facades suggest how to treat and prioritize cladding variables. Real risk can be measured. Performance and safety can therefore be improved.

The chapter on Determining Responsible Design Values outlines an approach toward load-and-resistance-factor design for exterior stone cladding systems. It is based upon a rational limit-state philosophy. Load-and-resistance-factor design formulates individual variables that may be pertinent to cladding a structure in stone. This method should replace traditional safety factors to improve evaluation methods to the same technological levels as available construction techniques. This will again raise stone cladding dependability to the revered cultural respect it held throughout its masonry tradition.

The following sections explain the foundation of load-resistance factor design: Failure Means Fracture; Risks Compared with Their Consequences; Reliability with Changing Variables; Load Derivation and Design Applications; Consolidated Uncertainties in Current Stone Engineering; Segregated Uncertainties in a Limit-State Approach; Factors for Loads and Resistances.

## FAILURE MEANS "FRACTURE"

**The engineering definition of failure is: *"those conditions of a structure at which it ceases to fulfill its intended function."***

Think about material strength when defining failure for stone used as exterior wall cladding. Stone is a brittle material. If overstressed, it fractures or ruptures and breaks apart. This is obviously not part of its intended function. Fracture is the failure state for stone. Failure from material overstress must be prevented.

Think about anchorage and framing function when defining failure for stone used as exterior wall cladding. Proper function of stone systems during the structure's life avoids "forced" deformation or resisted movement that overstresses stone. Unplanned movement or confinement may threaten cracking or dislodging the stone from its original position. Stone breakage is almost always the consequence of the support framing or anchorage behaving in a way that causes unplanned concentrated stress somewhere in the stone panel. Other forces may be distributed within the panel well within safe capacity, but the concentrated "hot spot" causes failure. Rarely is it the panel itself without deleterious influence from its backup. Failure from support "malfunction" must be prevented to allow the cladding to attain its potential strength.

"Yield" is essentially rupture. Yield also fails stone because stone does not deform plastically. No post-yield reserve exists as it does for other materials like metals, which bend without breaking. Thus ductility after overstress in metals presents detectable warning before breakage. Initial overstress in stone results in breakage without visual warning. Those forces that fail stone act invisibly. Once those critical conditions occur, failure already happened, for the stone has fractured.

Evaluating risk involves combining the forces that may fail the stone system and predicting their probabilities.

## RISKS COMPARED WITH THEIR CONSEQUENCES

Risk represents the possibility of stone failure. Because stone fails by fracture, which can occur suddenly without warning or detection, the effects of its variables are invisible until the stone fails. Proper design permits only an extremely small chance of failure. The risk of stone failure and thus the chance that its consequences would occur should be almost none. Achieving this scant remoteness when indicators from many variables and combinations are hidden is an engineering challenge that load-resistance factor design objectively addresses.

Mild steel "fails" gradually by plastic bending because it has ductile reserve that stone does not have. Since steel structures are not designed to "bend" in service, if this undesirable condition occurs due to whatever cause, it usually can be corrected before collapse. Thus the consequence for yield in steel is not usually catastrophic.

Differences in material properties do not change how much risk is acceptable. They do affect how uncertainties that cause risk are evaluated. The combined effect of these uncertainties cannot exceed the acceptable risk of failure.

Investigate each aspect that influences risk to establish the failure state or the condition at which failure is expected to occur for the cladding system. Compare their effects as the failure limit is approached. Compare their consequences if failure occurs. Compare each influence with each other. Predict the probability of simultaneous occurrences. Determine the consequences of combined effects that may approach the limit state. Any consequences of effects or their combination must not risk exceeding the limit state of the cladding system.

Breakage is failure. It is a severe consequence in comparison to a limit-state based on plastic yield for metal. Extensively investigate influences that threaten failure. Maintain the appropriate margin-of-safety by limiting their probabilities to levels of acceptance shared by other primary structural disciplines.

Each force acting on the stone causes effects that approach failure as the force intensifies. As the force increases, the stress or effect increases also. As the force's effect nears the failure limit, the risk of that influence causing failure also increases. Thus, to limit risk, design quantifies the occurrences of the effects to limit the risk of failure.

### Limiting Risk of Failure Means Limiting the Probability of the Consequences Occurring

Engineering structures under other building disciplines allow for one percent failure under the worst conditions. Practice indicates that this definition does not mean that one-of-one-hundred structures fails, but that one-of-one-hundred structures exposed to *both* the highest load influences contemplated with the lowest capacity influences actually fail. This results in an actual failure rate of 1:3500 to 1:4000 according to Andrszej Nowak and Ted Galantos in *Making Buildings Safer for People.*

## RELIABILITY WITH CHANGING VARIABLES

Absolute safety is not possible. Attempting to provide a design with adequate strength that is flawlessly constructed and will survive any loading and environmental imposition is unrealistic. Accurately predicting all potential load effects and movements is not possible. Avoiding absolutely all potential weaknesses or inefficiencies is also not possible. What is possible is to balance realistic and attainable quality with historically expected exposures to achieve a feasible design. To attain this balance, effects (or parameters) to both sides must be measured individually and in relation to each other.

Risk of failure is the risk that quality of the system will be less than the forces upon it at some time. The degree of risk is intuitively involved with direct cause (a superimposed load) or indirect cause (movement from those loads), property or people affected (what is their exposure if failure occurs), and cost (of replacement, repair, or damages from that consequence). One must analyze risk within each of the parameters to establish an appropriate safety level, or reliability.

In building, the reliability is associated with the risk due to uncertainties in loads, affects, structural material performance, durability, and compatibility. Uncertainties result from natural material and force variations, approximated engineering design (engineering is not a precise science), variations in construction techniques, and unpredicted behaviors.

Reliability increases when risk is controlled. Minimize risk by increasing control over both causes and consequences.

Causes are controlled by either:

1. Eliminating the source (a load or restraint), or
2. Reducing the exposure or magnitude of the source (appropriate capability, piece size, thickness, material consistency, or anchorage).

Consequences are controlled by:

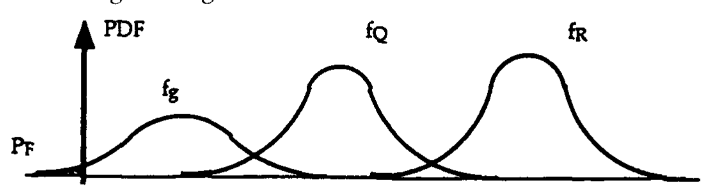
1. Increasing warning provisions (such as support redundancy or controlled restraint which retains a fractured stone with out releasing it from the facade),
2. Failure isolation (prevent progressive failures, meaning the failure of one stone caused solely by the failure of another),
3. Fail-safe design that directs the affects of the causes to a less-significant but perhaps more reliable component.

Determining the means to control risk is an economic evaluation. As a commodity, the optimum safety that can be realistically afforded is a result of quantifying the probability of failure. That quantification under the theory of reliability provides that the performance of a structural member can be measured by its probability to fail. A safe state is the condition where the probability to fail is less than that "threshold" allowed. The safe state is evaluated by predicting where load effects (Q) are less than resistance capacity (R), both of which can be expressed in limit-state parameters such as load components, material properties, and time or exposure considerations.

Again citing *Making Buildings Safer for People,* where "g" is the limit-state function:

$$g = R - Q$$

and a negative "g" is failure.




This probability of failure is calculated using the reliability index (B):

$$B = -F_N^{-1}(P_F)$$

where "$F_N$" equals the standard normal probability function, and "$P_F$" is the probability of failure.

| RELIABILITY INDEX | PROBABILITY OF FAILURE (PF) |
|---|---|
| 0 | 0.5 |
| 1 | 0.159 |
| 2 | 0.0228 |
| 3 | 0.00135 |
| 4 | 0.0000317 |

**Table 1. Resulting reliability indexes and their probabilities of failure.**

Actual failure occurs where load effects (Q) exceed resistance capacity (R) as depicted where their respective probability functions $f_Q$ and $f_R$ overlap. The limit-state, or "safety margin" $f_g$ is the difference between resistance and loads, and that area within the distribution below the probability function (PDF). And because each of the load and resistance effects are composed of multiple components, the limit-state (failure-state) functions are defined by the sum of the effects from these individual parameters. These parameters include serviceability as well as strength considerations and evaluate not only their magnitudes, but also their frequencies. Established figures in practice for other materials used in building construction are:

| TYPE OF MATERIAL | LIMIT STATE | RELIABILITY INDEX | PROBABILITY OF FAILURE |
|---|---|---|---|
| cold-formed steel | all | 2.5 | 0.0062 |
| hot-rolled steel | tension | 3.4 | 0.0034 |
| reinforced concrete | flexure | 2.7 to 3.5 | 0.0014 to 0.00095 |
| reinforced concrete | compression | 2.5 to 4.5 | 0.0062 to 0.0000034 |
| wood | flexure | 2.0 | 0.023 |

**Table 2. Comparative reliabilities and failure probabilities for certain states of common building structural material.**

While the safety of all structural types defined by their probabilities of failure are not uniform, it should be suggested that for stone itself, they could be.

The total cost of structural reliability ($C_T$) includes the initial cost of construction ($C_I$) with the total cost of failure ($C_F$):

$$C_T = C_I + P_F \times C_F$$

The approach to developing rational safety criteria includes:

1. Identifying the influencing conditions;
2. Formulating the limit-state functions for these influences;
3. Determining the required safety level;
4. Evaluating current practice and compare it to the determined limits of (3);
5. Calibrating the design load and resistance factors based on these compilations and known existing exemplars.

## LOAD DERIVATION AND DESIGN APPLICATIONS

Establish design loads that the stone, its anchorages, and eventually the backup support for the anchorages, are to resist. Primary loads on vertically oriented stone panels within exterior walls are lateral loads induced by wind along with the panel's dead load induced by gravity.

As wind velocity increases with its wind-borne precipitation or other elements, so does the pressure it exerts on any obstruction such as a building wall, and that increase in pressure is exponentially proportional to the increase in velocity. It is this pressure that is the lateral load on a vertically oriented panel. Further, "microgeographic" influences such as other nearby buildings or land features or its own surface shapes can cause vortices that amplify pressure above the physical increase in velocity. These are commonly called "hot spots" where stress concentrations are likely. This resultant load, which is resisted by the stone by its flexural strength, must be transferred to the supporting backup, usually through compression (from positive-inward pressure) or tension (from negative-outward pressure, or "suction") in an anchorage device.

As the size and density of the actual stone material increase, so does the panel's dead weight (or gravity load). As a function of the stone's solid volume and density, this resultant load must be transferred to the supporting backup, usually through shear in an anchorage device at the stone's bottom edge, a pocket in the stone's hidden face, or through a mechanically attached liner block on the stone's back face.

The complex stresses occurring at the anchorage-to-stone interface must be kept as predictable as possible. The behavior cannot be complicated by combining lateral and vertical support at the same point-of-contact. Remember increasing reliability means controlling risk.

*Guide Specification for Stone Cladding Systems* explains these principles in *Anchorage Device Mechanics*.

Building codes having jurisdiction in the building's location or the project-specific wind tunnel studies are usual sources for lateral wind loads that are to be superimposed onto the exterior wall cladding. Building codes or seismic testing will identify lateral differential movements, which are resultant effects of lateral forces, which must be accommodated within the exterior wall cladding system. Stone density derived from ASTM C 97 *Test Methods for Absorption and Bulk Specific Gravity of Dimension Stone,* with panel size determine the stone's dead load. With exterior stone cladding, this book emphatically recommends that loads due to displacements in backup are to be eliminated, or isolated from influencing the stone, and thus are not needed to be considered *if* this principle is satisfied.

For discussion purposes, an example will be reviewed in the following Commentary paragraphs to help show how to apply these principles.

COMMENTARY: Design loads for this example are extracted from wind tunnel testing for the wind loads and ASTM C 97 for stone density. Typical building mid-shaft maximum loads are +/- 60 lbs./ft$^2$, thus for a 4'-7" by 5'-0" stone that is supported only at the corners, each of the four corner anchorages resists equal lateral-load reactions from the panel (tributary areas should be determined differently depending upon the support layout) and must resist one-fourth of the panel's total wind load:

4.63' x 5.0' = 23.125 ft$^2$/stone x 60 lbs./ft$^2$ = 1,388 lbs. per panel
1,388 lbs. per panel / 4 supports per panel = 347 lbs./anchor

For simplicity, and because all three prospective stones are nearly identical in density, use 168.5 lbs./ft$^3$. Given that only the bottom two anchorages support the stone's weight, each of the two bottom corner anchorages resists equal gravity-load reactions from the panel (tributary areas should be determined differently depending upon the support layout) and must resist one-half of the panel's total (dead) gravity load:

23.125 ft$^2$/stone x 1.25" nominal thkns (+1/8" max. tolc) = 2.65 ft$^3$
2.65 ft$^3$ x 168.5 lbs./ft$^3$ = 447 lbs. per panel
447 lbs. per panel / 2 supports per panel = 224 lbs./anchor

## CONSOLIDATED UNCERTAINTIES IN CURRENT STONE ENGINEERING

Traditional dimension stone engineering practices a kind of informal allowable-stress design (ASD) philosophy. This approach uses a safety factor to account for all uncertainties by discounting tested stone strength values. It has yet to be discerned which influencing uncertainties, whether tangible or not, are part of this safety factor. Application, exposure, backup, durability, anchorage, or specific material variabilities, all of which are valid discriminating concerns and can be dissected from the overall "blanket" safety factor, presently are not independently considered.

This safety-factor (SF), which is usually and almost solely applied depending upon the stone's geologic type (granite, limestone, marble, or slate), discounts the nominal strength, or resistance ($R_n$), which must exceed the unfactored service load ($Q_s$):

$$R_n / SF \geq Q_s$$

The allowable stress philosophy implies an elastic stress calculation, that is, that all loads and anticipated overloads occur within the elastic range of a material's, in this case, the stone's behavior. Ignoring hysteresis and some characteristic behavior depending upon moisture, natural stone's behavior under load is assumed to be elastic. Even though elastic, stone's stress-strain relationship is not linear, and more importantly, is not consistent across the body's section (nonisotropic) because the material is variably heterogeneous.

This philosophy consequently assumes, however, that all loads and all strengths (resistances) have the same average variabilities. For instance, use of a safety factor of 2.5 for granite panel flexure and 3.0 for its anchorages arbitrarily assumes that the influences of environment, material, support, installation, and any combination of uncertainties, known or unknown, will likely not exceed that permitted risk of failure (thus "reliability") assigned by the safety factor.

Designers and installers are aware of the phenomena that influence stone and anchorage performance. They are not specifically provided for in this allowable stress design approach. A more disciplined investigation of the likely uncertainties is prudent to assure a safe and economical exterior wall.

## SEGREGATED UNCERTAINTIES IN A LIMIT-STATE APPROACH

Limit-state design, or load-resistance factor design (LRFD) is the now predominantly practiced philosophy for the structural design of steel and reinforced concrete, and most recently, masonry and wood. A more rational approach than allowable-stress design (ASD), it is a probability-based procedure that provides both for the possibility of overload and underdesign, but treats both independently. In limit-state design, each influencing aspect of either overload or underdesign could also be considered independently before final compilation of the overall load, or resistance factors. This approach directly addresses the concern with the somewhat arbitrary approach of ASD.

From the ASD equation, the limit-state philosophy generates factors for both loads and resistances. The nominal resistance ($R_n$) becomes actually a composite of the factored uncertainties ($W_i$) influencing the resistance:

$$\text{factored resistance} = \text{sum of } W_i R_n$$

The factored service load ($Q_s$) becomes actually a composite of the factored uncertainties ($Y_i$) influencing the loads:

$$\text{factored load} = \text{sum of } Y_i\, Q_s$$

Having considered the different variabilities of the primary uncertainties separately as well as their frequencies and chance of simultaneous occurrences that are pertinent for that particular condition, load and resistance factor design structures its equation thus:

$$\text{sum of } W_i\, R_n \geq \text{sum of } Y_i\, Q_s$$

COMMENTARY: The definition of "safety factor" in comparison to load-resistance factor design is that "safety factor" was a method of structural design that established the usable fraction of a material's ultimate strength that could not be exceeded by the effects from the actual design loads. The material's ultimate strength in a particular stress state was divided by the appropriate safety factor for that considered condition to render its working stress.

Different conditions and different stress states could require different safety factors. ASD as practiced in stone engineering does not address this. Traditionally, a uniform, one-safety factor-fits-all-conditions has been practiced by most stone engineering professionals. Some discrimination has been practiced between stone types and their strength variabilities, meaning that limestone and granite, for instance, have been designed with different factors, but little or no objective engineering-judgement structure has been established to address the remaining genuinely different ingredients to the design of stone, stone anchorages, and exterior wall cladding systems.

The subjective, and arguably arbitrary safety factor approach does not necessarily mean, that with a higher "reduction factor" applied to the ultimate stress derived from ASTM standardized test methods, that a greater margin of security results. Obviously, establishing the appropriately low probability that a failure would occur requires the objective evaluation of *all* the qualitative influences upon the condition, not just stone type and flexural strength test variability. Material properties are only one of these types of uncertainties. In fact, there are several more considerations to evaluate within the "material" realm alone, as previously discussed.

Allowable strengths or working stresses, were determined by reducing the test-established ultimate strength by that safety factor believed to assess the overdesign required to avoid failure. ASD's failure to objectively evaluate, and instead ignorantly attempt to "cover" the other factors is the reason load-and-resistance approach is especially warranted for dimensional stone cladding design and construction.

Using statistical methods, in the general concept within the context of stone cladding engineering, load-and-resistance factor design compares actual loads that are increased by a factor that is proportioned to their probability of being exceeded to actual resistances that are decreased by a factor that is proportioned to their probability of not being attained. Frequency of occurrence, variabilities, and consistencies imply probability, an approach that has only been addressed in qualitative, not quantitative means by the safety-factor approach to allowable stress design. Equating factored loads with factored resistances results in a statistically and calculated engineering response that has a predetermined probability to assure safety. Furthermore, because the combination of factors has been selected according to the particular material, condition, environment, stress, and other considerations unique to the project at hand, rather than being dependent upon a blanket safety factor, our confidence of relative durability and reliability is high. It is also replicatable by other professionals evaluating the same condition, but with perhaps different experiences or preconceptions. Subjective inference has been the most difficult hurdle to attaining uniformity in the stone engineering industry. Engineering judgement should be founded upon discernible and explainable reasonings, not simply a certain professional's reputation or marketed profile.

## How Overloads Can Arise

Overloads can arise from underestimation of the effects of loads by oversimplifications in structural analysis; or variations in construction installation procedures, either planned or by human error; or variations in the assumed boundary conditions founding the analysis.

Violation of these conditions in stone cladding is likely caused by some of these first-order overload uncertainties:

## Overload Uncertainties

- Failure to structurally isolate the panel from influences by other stones, or
- Failure to maintain the anchorage's designed engagement mechanics within the stone, or
- Nonplanar support caused by the differential displacements of the backup support framing, or
- Alteration of the designed anchorage or human error or injury to the components during the installation, or
- Magnitude of the applied load exceeds what was designed for, or
- Magnitude of the applied load's variations exceeds what was designed for, thus fatigue is accelerated, or
- Frequency of load variations and load reversals (positive and negative lateral loads) exceeds what was designed for.

## How Understrength Can Arise

Understrength can arise from:

- Overestimation of the nominal resistance of the stone material, or
- Overestimation of the nominal resistance of the anchor device, or
- Underestimation of the effects of weathering or climate.

Violation of these conditions in stone cladding is likely caused by some of these first-order understrength uncertainties.

## Understrength Uncertainties

- Failure to control the panel size or thickness to maintain minimum section properties, or
- Failure to maintain the anchorage's fabricated preparation to maintain an engagement within designed maximum and minimum limits, or
- Failure to control the location on the building or locational-dependent properties of individual stones, or
- Underestimation of the moisture-dependent properties, and thus the relative variability of wet and dry strength, or
- Underestimation of the directional properties, and thus the relative variability between parallel or perpendicular to rift or vein, or
- Failure to control the frequency of inclusions or faults, or
- Failure to recognize the influence of mineral crystal size relative to both the overall panel thickness and the local properties at anchorages, or
- Underestimation of the deleterious effects of weathering, which include precipitation (rain, sleet, hail, snow, and ice), temperature (repeated cyclical warming and cooling), freeze-thaw (separate from temperature, this includes extreme cold in the presence of moisture), and atmospheric agents (water vapor and airborne pollutants such as acid that dissolve or weaken some mineral constituents and bonds).

## Probabilistic Evaluation

Probabilistic evaluation of the possibility of these conditions occurring (even if empirical, to begin), and to what degree they influence the stone's and its anchorages' performance are suggested to be considered and researched to be able to quantify the potential for overloads.

Probabilistic evaluation of the possibility of understrength conditions occurring, and to what degree they influence the stone's and its anchorages' performance are suggested to be considered and researched to be able to quantify the potential for understrength.

Because stone is a natural material, many uncertainties influence its resistance. Because these uncertainties also have differing variabilities, extensive investigation will be required to discriminate rationally between them.

Material strength variabilities, material fabrication tolerances (especially at anchorage preparations), the effects of weathering in different exposures, and the variabilities of the anchorage components and support systems themselves contribute independently to uncertainties that influence resistance. A more rational approach than that now practiced could benefit the "uncalculated" confidence for safety we now expect, but don't necessarily receive from a single safety factor.

## FACTORS FOR LOADS AND RESISTANCES

Load and resistance factors are implemented to establish a margin of safety. This margin is the system's reliability that considers all aspects of the use of the stone, its material property consistency, rift or vein orientation, types of loads, fabrication and installation tolerances, and potential support configurations, influences such as installation methods, anchor types and redundancy, risk and consequence of a potential failure, climate, and movements of the skin and backup structural framework, which when considered together in whole provides for an appropriately small opportunity for failure over the entire expected service life of the building. The result is a reliability that is rationally derived and specific to the conditions and exposures considered for that project.

### Load Factors

Load factors should be proportional to the predictability of the magnitudes used in the engineering design, their frequency in approaching those maximum magnitudes, their "sense," whether gravity or lateral, and whether they are cyclical and repeating or not, and how frequently those alternating load senses approach the maximum load capacities.

### Gravity Loads

Gravity loads should be factored according to the average variabilities of uncertainties that affect the stone's weight. Include wet-weight for stones affected by saturation. Use ASTM C 97 average densities, nominal panel face size, and maximum panel thickness (standard design module thickness plus either allowable or expected tolerance in slabbing, whichever is the largest). Keep in mind that this approach assumes that the stone is isolated from gravity loads from other facade elements including other stones. Potential for this assumption to be untrue are to be considered with the anchorage type or weatherproof joint considerations.

COMMENTARY: Gravity loads are a function of density, which is nearly exactly quantifiable in ASTM C 97, *Absorption and Bulk Specific Gravity of Dimension Stone,* as are the panel size and thickness, which are functions of fabrication tolerances and quality control procedures in production. Using the maximum module thicknesses produced by the fabricator with very small allowable deviations in face size, volume of the stone is known, thus justifying a small load factor.

Gravity loads are "single-sense" or single directional loads. Test methods placing the sample in a stress state simulating a real load that will be occurring in this single sense should exert the load slow enough to not impact or shock the material, yet its rate of application should be representative of how the load increase could normally be experienced in the material's in-place or during placement.

### Lateral Wind Loads

Lateral wind loads should be factored according to the average variabilities of uncertainties that affect the wind or other potential lateral loads on the stone. Use the project-specified source for wind-pressure magnitudes. This factor should account for the effect of stress reversals, the magnitudes of the stresses occurring through those cycles relative to the overall capacity, and cyclical loading resulting from shifting wind directions and mild "impact" caused by wind gusts.

COMMENTARY: Wind load derivations are usually calculated from the local building code that likely cites ANSI A58.1 (now adopted by ASCE code) or a separate wind tunnel study if the project is of considerable size or in a congested context where affects of local building geography may influence wind pressure magnitudes more than local climatology. Using the velocity or pressure magnitudes based on historical maximums expected over a twenty year occurrence, the likelihood of exceeding such a magnitude, even for a 100-year building life expectancy, is small, thus justifying a conservative load factor. Should loads that approach 80 percent of those 20-year magnitudes every-year or more (is the annual maximum within eight-tenths of the twenty-year maximum?), then this load factor might be increased to the designer's judgement.

Lateral loads are "alternating-sense" or dual-directional loads. Alternating load senses such as positive and negative wind loads reverse the material's stress states, and result in stresses that can fatigue and thus lower the material's ultimate strength after some number of cycles. The proportion of ultimate strength reached at each of the cycles with the number of reversals that occur both affect how rapidly the original ultimate strength is depleted. While flexure and shear-flexure stresses typically can be cyclical as caused by wind, ASTM C 170 *Test Method for Compressive Strength of Dimension Stone,* compressive strength, C 99 *Test Method for Modulus of Rupture of Dimension Stone,* modulus of rupture, and C 880 *Test Method for Flexural Strength of Dimensional Stone,* flexural strength standard methods only test one direction "sense" by gradually and continuously increasing the load until the stone fails. Test methods evaluating individual anchorage assemblies with custom apparatus can simulate alternating stress states if set up to do so by reversing load inductions.

### Resistance Factors

Resistance factors should be proportional to the predictability of the condition resisting the loads, whether related to material, fabrication, installation, or structural stiffness.

### Stone Material Strengths

Stone material strengths are factored in relation to the variability of the material strength properties. Ranges in predictability parallel the consistency of the material, which is deducible from the standardized ASTM C 99 modulus of rupture and

C 880 flexural strength test methods. The designer should also include considerations for wet or dry, parallel or perpendicular to the rift, and other variables which will affect the stones performance in the project. It could be suggested that the Material Strength Factor is to be proportional to the standard deviation's fraction of the overall average strength. A larger standard deviation fraction of the overall strength means greater variability, thus less predictability, which then requires a higher material strength factor. This factor should relate separately to C 99's and C 880's test results' coefficients of variation, which is the ratio of standard deviation to average strength.

Research and statistical analysis will need to evaluate how the coefficient of variation fits within the allowable risk envelope to derive what a material strength factor might be for stones of different variabilities.

COMMENTARY: Common present practice is to assign safety factors according to what type of natural stone is being used. This safety factor is intended to encompass the many considerations together without being specific about any. Granites are sometimes designed for 2.5 to 4.0; limestones between 4.0 and 7.0; and marbles and travertines, if used on an exterior, between 6.0 and 12. These safety factors have been typically applied as strength-reduction factors, which are compared with actual design loads and the stresses that result from their application, which is the traditional allowable-stress method.

This presented approach recognizes that the primary aspect of the materials' differences in their ability to resist structural loadings with predictable assurance is to consider their variability. The different stone types naturally possess different consistencies because of to their geologic formation. Granites may tend to be more stable and consistent due to their igneous formation and predominantly siliceous content. Limestones could be less stable and more directionally variable due to their sedimentary formation and predominantly calcareous content. Marbles are usually the least predictable because of their metamorphic transformation from other, usually sedimentary mixtures. The affects of pressure, time, and heat vary considerably within the same mass which results in greater variability. The nature of stone is simply that there is some level of inconsistency, regardless of the content or the mode of its formation. Whether the formation and content compose a strong or weak material cannot influence the consideration of its consistency. Because the ASTM standard test methods include specific provisions to evaluate strength directionality and wetness, these test results can be incorporated into the statistical measure of the material's consistency, which is the basis for the material strength factor.

Material strengths are typically evaluated and analyzed by using test values and ultimate strengths. Ultimate strengths are determined identically for both safety factor (allowable stress) and load-resistance factor design, with repetitive representative samplings evaluated with standardized test methods that are recognized to quantify the limits of certain fundamental stress states such as compression, flexure, and shear-flexure. Special tests for anchorages or assemblies establish capacities not specifically addressed by the standardized unit-stress test methods.

Rupture, or stone breakage, is understood and defined to be stone failure. The load magnitude at the instant the stone fails is the ultimate capacity of that sample under that stress state the test procedure was designed to measure. The average value of a set of samples' ultimate strength values from their individual tests that were prepared properly for that test method and thus tested for that certain stress condition is recognized to be the ultimate strength in that stress condition to be used for engineering evaluation with the resistance factor.

### Anchorage and Support Framing

Applications are factored in relation to the intended use of the material strength properties to either the panel's support or its span. They could also account for the redundancy of the anchorages in the overall system. Because anchorages are subject to the installation and handling variables caused by humans which the "spanning" of the stone is not affected by once in place, and because anchorage failures risk worse consequences than spans due to reduced redundancy once an anchorage fails, design factors for anchorages are reduced. Thus linear-engagement-type anchors (such as kerfs) are considered differently than point-engagement-type anchors (such as pins, tooled-rods, rod-and plugs) because of relative disengagement redundancy.

Because the location of the anchorages within the panel affect the distribution of the flexural stresses, and a panel behaving as a two-way plate inherently has a stress distribution and redundancy not shared by a panel behaving primarily in one-way bending, which tends to concentrate the stresses along a definitive line, design application factors are suggested to relate to the aspect ratio of the panel between its supports. Where height-to-width spans between supports are approximately equal, plate action will better distribute stresses and could justify a smaller application factor. Biaxial bending increases economy.

COMMENTARY: Knowing flexural stress distribution within the panel is extremely important in the design of a brittle, nonplastic material. Stone is highly sensitive and unforgiving to stress concentrations that are the result of one-way bending. Force flow within the panel is to one area, and if the panel has any faults or weaknesses at that location, early failures become much more likely. Some benefit is to be offered to the retention systems that recognize the inherent redundancies of the types of anchorages used and their layouts, which respond more favorably to the nature of stone as a structural material.

With a linear-type anchorage such as a kerf bar, which may only be effectively supporting the panel at its corners, for instance, is still engaging the stone panel across its entire width. Failure of the stone at the original support location does not cause the stone to be released from the facade because the stone panel is still "captured" by the kerf bar within the remaining portion of the stone's kerfed edge. Instead, the stone panel's reactions are transferred along the length of the kerf bar where the stone is still intact. Intermittent visual inspection of the facade will discover these conditions, which can be repaired or otherwise correctly restored, without failure, injury, or damage.

## Climate Factors

Climate factors consider the severity of the deleterious effects of weathering that the stone will be exposed to. Because freeze-thaw cycling, out of all the weathering effects, can be the most traumatic cause of stone strength reduction and thus panel and anchorage capacity, the climate factor could be proportioned to the expected number of cycles the facade might be required to withstand over the life of the building. For climatic weathering caused by acidic precipitation from industrial areas, the affects of weathering increase, requiring the climatic factor to decrease. In these conditions, the solution will be acidic according to the average pH of that region's rainfall. Cycle frequency and total quantity should be based upon a fifty-year building lifespan or the expected service life of the building, whichever is greater.

ASTM is developing a standard test method that plots the loss of flexural capacity with intermittent cycles of freeze-thaw exposure with a slightly acidic solution. The test is calibrated with the C 880 method. Modifications to this test to exclude the acid solution for climatic regions that do not experience it, or keeping the acid but excluding the freezing (but keeping the thermal cycling) for climates that endure those exposures would be appropriate measuring methods for evaluating the climatic effect on understrength.

## Finish Factors

Finish factors consider the vulnerability of the stone's surface to absorb moisture or propagate microcracking. Processed finishes that violently treat the stone's surface like flaming, cleaving, bushhammering, and some high-pressure sand or water-blasting cause the inherent microcracking structure of the stone to propagate. Cracks and faults destroy some bond between the minerals, and thus the strength of the material affected by this finishing method. Increased microcracking also increases the absorption of moisture by capillary tension, and thus accelerate the effects of all moisture-dependent behaviors. While some of the finish's influence might be evaluated by actually testing that finish in such tests as C 880, it is not yet concluded that the finish effect is adequately measured unless researched independently.

## Cladding System Originality

Originality factors consider the novelty of the cladding system and the relative significance of the performance of similar precedents. Original designs not having a prototype existing likely is less predictable than cladding systems having time-tested exemplars whose performance is known. If a cladding system uses components and techniques already existing in a well-performing wall, using that system may be more reliable. Conversely, new advancements, even if extensively tested in a laboratory, cannot replicate the effects of time and exposure in nature.

This factor may be the most subjective, yet also the most important. Relating to an exemplar requires that knowledge of that exemplar's performance is objectively quantifiable. Its validity as a model is only as valuable as its age, its degree of duplication, and the similarity of its boundary conditions to those of the contemplated project. Yet many stone projects copy existing support and anchorage systems. These designs should benefit from the reliability of their past success. Likewise, truly new designs should be considered more cautiously until their real-life in real-time performance is established.

5

# GUIDE SPECIFICATION FOR STONE CLADDING SYSTEMS

The *Guide Specification for Stone Cladding Systems* includes two parts of study. Sections 1, 2, 3, and 4 review variables in work that interface, or "surround" stone panels and their anchorages. These are boundary conditions that define the support system's intended behavior:

1. Scope and Applicability of This Guide Specification
2. Expected Performance Standards for Depicting and Specifying Stonework
3. Materials Used to Construct Interfacing Systems in Exterior Walls
4. How to Keep Exterior Stone Joints Weathertight

Once the design professional knows how the interfacing systems are supposed to behave, specific stone engineering begins. Sections 5, 6, and 7 review material and anchorage variables:

5. Testing Used to Design Stone and Its Anchors
6. Anchorage Device Mechanics
7. Case Study Testing Applied to the Design Process

The Specification outlines the load-and-resistance design principles introduced in Chapter 4, *Determining Responsible Design Values* for stone and anchorages, and can be developed specifically toward that engineering philosophy.

## SECTION 1 SCOPE AND APPLICABILITY OF THIS GUIDE SPECIFICATION

Natural dimension stone is a desirable building skin. Stone material that is well matched with its use provides a durable and genuinely beautiful facade. A properly conceived system requires relatively little maintenance. Consistent quality is achieved by choosing appropriate materials, properly selecting anchorages, and carefully installing all parts of the system. These features raise stone's aesthetic potential above any other facing material. Inappropriate applications that might combine incompatible stone and support conditions diminish stone's unique qualities.

This *Guide Specification* reviews performance variables that should be considered during the construction process. The process begins with material selections and continues through their installation. Stone anchorages, loading superimpositions, exterior wall framing behavior, structural frame behavior, adjacent material compatibility, and stone material structural properties require intent evaluation.

This chapter organizes the process in a rational sequence. It is consistent with currently respected exterior dimension stone practice. Several new concepts are introduced to enhance system performance. Primary variables are presented by category of influence. Each variable is considered individually to best match stone and anchor with their intended application. Optimum performance is based upon conditions of the particular project. Following the progressive analysis leads to a sound functional building wall that will maintain its original appeal. There can be no "cook book" approach because each set of project circumstances, environment and design is different.

## Why Stone As a Natural Material Requires a Unique Engineering Practice

Because stone is a natural, not manufactured, product, the many varying properties it posesses influence each use a designer contemplates. The techniques used to design a stone retention system with its cladding must be coordinated with the procedures used to place the stones into or onto them. The process of designing the actual attachment devices must also be sensitive and compatible with the surrounding components.

These processes and techniques are also subject to all aspects influencing the stone system itself, which generally include primary building frame dynamics, the corresponding support substructure interaction with these movements, anchorage device interaction with both of these behaviors and the stone itself. Several separate considerations also include the fundamental strength characteristics of the stone as a material and then separately, the stone as a "panel" in its intended, supported configuration. Thus, the particular stone is selected and its shapes are articulated within the architectural scheme to form a skin. The skin must adapt to the overall building movements in the changing environment throughout its life, from the inception of construction until it ends service. These different stages demand different performance abilities. The building skin is supposed to easily accommodate the structure's reactions with its own reactions to both climatic and structural forces. Construction forces are also vital early in the structure's life.

The *principal design goal* is to attain a properly designed skin that symbiotically coexists with the building frame and allows the *natural* beauty of the cladding to endure and increase in character with age.

## The Structure of the Engineering Process

The process of evaluating stone materials and systems is elaborate, but when organized, can render an objective solution which has high reliability. This process is presented in this guide specification.

## A Stone System's Boundary Conditions

The evaluation process is presented as the *Guide Specification for Stone Systems.* This specification prescribes the system's boundary conditions and the engineering of the stone and its anchorages within those boundary conditions. Verifying the behavior of the building structure, the stone retention system, the anchorages, and the environment through the wall assures the best performance compatibility possible.

Sections two, three, and four of the *Specification* present and discuss compatibility design factors. Consider these while defining the boundary conditions for the building structural frame or the exterior wall structure. Simplified (for engineering structural analysis) these aspects are the behavioral conditions of systems that interface the stone cladding. They define the engineering "boundaries" for exterior wall and stone.

Sections five and six of *Specification* present and discuss direct engineering design factors directly relating to the stone cladding. Consider these while evaluating the fitness of the stone cladding itself. These include properly matching material characteristics and anchor device mechanics with the boundary conditions. Section seven shows how these are tested.

Evaluation of the variables that influence stone design offer many insights into the overall design of the stone anchorages. Stone anchorages attach the stone panel to its retention system and support the exterior cladding. Suggestions in these evaluations advise the designer about practical analysis and installation techniques for many exterior stone cladding conditions. They require an experienced stone expert to exercise professional judgement in their application. The *Specification Guide* compiles the industry's currently recognized practices into a deductive sequence and also recognizes the value of comparing prospective work with existing stone buildings. The *Specification* suggests interpretative methods to better apply these known practices. Improved durability, higher performance, and better economy is the goal. The entire exterior wall system design, including skeleton, skin structure, and stone is to be conceptualized while selecting the stone material. Re-verify each part's structural capacity through the engineering process of the individual anchorage devices. Consider constructional compatibility and realistic installation workmanship of the stonework and interfacing systems. How each of the interfacing systems' behavior interacts through these stages, if rationally anticipated, can be used to enable the final design to perform as specified. The product of the process is a rationally designed and built assembly of systems that fits together and performs well in its environment.

The *Specification Guide* presents a method of determining the principle boundary conditions to attain expected cladding performance. The method requires correct interpretation at each step. The engineering approach cannot be proven without developed boundary conditions. Understanding conditions such as building frame's dynamic influence on the skin structure, and correlation of existing building durability with stone testing allow proper engineering evaluation. Many such items are listed for the designer's thorough and accurate review during actual stone and anchorage engineering. For the best benefits, they should first be closely accounted during building structural frame design and detailing.

## The Engineering Sequence

This engineering sequence outlines specific quantitative and qualitative engineering considerations which enhance traditional methods. Chapter 4 *"Determining Responsible Design Strength Values"* suggested that a limit-state design approach such as load-resistance factor design is the more appropriate design method because there are many variables involved that influence exterior wall system and stone material performance. Chapter 4's outline requires research data related to this philosophy to derive the formulas that correlate to known successful stone installations. *Determining Responsible Design*

*Strength Values* dissects the known concerns into individual issues that substantially influence overall system performance. This *Specification Guide* applies them. Considering each issue by itself relative to the particular project application renders a better prediction of system durability. This increased reliability means longer lasting and safer stone skins.

## Applying the Sequence in a Case Study

Five examples of actual stone cladding design evaluation are presented in section seven with commentary to align with earlier presented design principles.

First, review standard unit test methods. Begin initial material property interpretations by comparing test results with the performance of the same stone on existing buildings and similar stones in the same environment.

Second, theoretically test the proposed panels by mathematical finite-element analysis. Generate support and dimensional configurations by computing what the stress magnitudes are and how they are distributed.

Third, load a sample panel. Preliminary panel capacity load tests calibrate the theoretical structural analysis' accuracy.

Fourth, test individual anchors. Individual anchor tests isolate the capacity and the mechanics of the device intended to hold the stone in place, all according to the magnitudes concluded from the sample panel and finite-element results. Compare these results with existing similar stone system anchor performance in the same or similar stones or architectural facade configuration.

Finally, prove the assembly's capacity in chamber. A chamber test of the panel with those anchorages are loaded together to prove the integrity of the overall assembly, which was developed given the observations from the previous tests. Compare these results with the performance of similar assemblies in similar environments on existing buildings.

While the criteria for this case study is based upon a single safety factor coefficient, the techniques remain perfectly valid under a limit-state LRFD scheme.

## The Approach Related to Existing Practices

A designer, whether architect, engineer, or contractor, must exercise prudent professional judgement in implementing this *Guide Specification* advice. Its instruction suggests which standards ought to be specified for component compatibility, which principles ought to be observed during design, and which approaches ought to be followed to assure proper installation. These guidelines for exterior stone anchorages are explicitly for experienced designers. A prerequisite knowledge of both the performance characteristics of stone materials, and stone applications in construction are necessary to be able to comprehend, appreciate, and thus fairly evaluate how the offered considerations influence the safe design of the stone cladding.

Most of these ideas have been practiced at least in part by the respected designing professionals and installing craftsmen while being endorsed by project contracts in the past. However, their approach has not been consistent, thorough, or complete. Justifications for their logic did not exist because inconsistency was common through the entire design process. The separate parts of the process are sequenced to assist experienced professionals to be able to compile, both responsibly and meaningfully, those aspects which require evaluation. Creativity for artistic ends cannot be allowed to discount the engineering principles that are uniquely inherent to stone.

A new design philosophy had not yet evolved because architects and builders had not changed how they thought about stone. Misunderstanding the "nature" of stone has kept stone from developing to its full potential.

Stone is brittle, inconsistent, difficult to work, but uniquely beautiful. The *Guide Specification* defines characteristics that can bring stone systems safely into modern exterior wall function. Few constructed skins have anticipated the spectrum of influences that require response for lasting durability.

The next three sections describe the performance variables that establish correct boundary conditions for stone and its anchorages in the interfacing exterior wall structure and also the building frame.

# SECTION 2
# EXPECTED PERFORMANCE STANDARDS FOR DEPICTING AND SPECIFYING STONEWORK

Presentation, interfacing materials, and surrounding joints are three variables that influence stone support performance. While they are not mathematically calculated, their accuracy and match with the cladding requirements are critical to proper system function.

Architectural conformance and behavioral compatibility depend upon decisions regarding these parameters. Subjective expectations and objective reality must be consistent.

*Standards for Depicting and Specifying Stonework* simplifies presentation by preserving expected system boundary conditions. Presentation translates into installation. Study these expectations at the project's inception to assure that they can be legitimately built and structurally analyzed by practical methods.

This quality of placement is determined by specifying realistic installation standards for systems surrounding stonework. This quality is an important boundary condition.

Interfacing work should be installed where drawn, on contrast documents and drawn where the cladding requires it. Qualified professionals understand these relationships and account for them during design. The multiple systems in the exterior wall must be married to comprise a symbiotic, and economic whole.

These are presented by

- Standards for Presenting Stonework in Contract Documents

- Limits and Dependencies on Interfacing Work
- Qualified Stone Designer Expertise

## Standards for Presenting Stonework in Contract Documents

Standards are established in several different parts of the project documents; they describe, regulate, or control the quality or how realistically "perfect" the work constructed is expected to be. Architects, engineers, design-build authorities, contractors, or other specified authorities such as consultants could issue these documents as the performance code. These standards are recognized by the established stone and marble trade industry as hereby prescribed.

### *Drawings*

The architectural drawings define pictorially the scope of the work included in the project by illustrating the arrangement of stones, what the stone types are, their finishes, their thicknesses, and face sizes and shapes that fit together. The drawings detail the general relationships to other interfacing construction elements including the building structure. Usually those relationships to the adjacent components, featuring the stone, its anchorage, and support's element sizes and shapes are coordinated to fit within the exterior wall upon its final design, including all its specific componentry. Consider submitting the first stone separately to obtain approval on architectural configuration, then the entire system, to expedite engineering.

### *Specifications*

The architectural specifications define verbally the scope of the work included in the project by describing the rules the building has been designed by and is to be installed by while including very specifically all materials, activities, and procedures required to accomplish its completion. The specifications should include:

Performance criteria that define the minimum performance levels that the system is intended to perform at in resisting external effects including:

Established industry standards such as:

- ASTM (American Society for Testing and Materials) primarily for test methods and material specifications
- ANSI (American National Standards Institute) for standard building performances
- ASCE (American Society of Civil Engineers) for standard building loads
- AAMA (American Architectural Metal Association) for architectural curtainwalls and their infills
- AA (Aluminum Association) for aluminum
- ACI (American Concrete Institute) for reinforced concrete
- PCI (Precast Concrete Institute) for precast concrete
- AISC (American Instutute of Steel Construction) for structural steel
- AISI (American Iron and Steel Institute) for stainless steel.

### *Local Codes*

Local codes having jurisdiction over the project's location advise of special requirements for the exterior wall construction usually beyond those explicitly standardized.

### *Wind Load Resistance and Performance Criteria*

Wind Load Resistance and Performance Criteria define the superimposed wind or other loads and their source of derivation. Use wind tunnel test conclusions or The Bureau of Standard's maps to establish the magnitudes of the wind loads. These loads are used to design the lateral strength of the wall structure and its cladding. If the system's materials are governed by established structural standards, the limit-states for designing that system should be cited from those respective references. Identify them specifically. Specific limit states are most likely described for the stone and its anchorage separately within the project specification. Specific performance limits on maximum allowable wall displacements caused by these loads should be included in the specifications. These limits must be proven to be compatible with the criteria followed to engineer the anchorage and its engagement in the stone, whose necessary stability may require more stringent limits.

### *Dynamic Movement Criteria*

Dynamic movement criteria include the displacements derived from the building's structural frame analysis which are predicted reactions to the same wind loads derived in *Wind Load Resistance and Performance Criteria.* These displacements are moving supports for the skin, which must be designed to safely and comfortably accommodate them without distressing any components.

### *Thermal Resistance and Performance Criteria*

Thermal resistance and performance criteria define the temperature extremes the wall should be exposed to based upon the project's geographic location. An average "U-factor," or thermal resistance value, along with allowable vapor transmissivity, are defined to establish insulation requirements and size expansion-contraction movement accommodations.

### *Climatic Performance Criteria*

Climatic performance criteria define the air and water infiltration extremes the wall should allow given a specified fraction of the wind on an extreme exposure with a rate of rainfall usually equivalent to perhaps the fifty-year worst based upon the project's geographic location. An established volume of air-per-area-per-hour also governs the pressure for *controlled* water leakage with re-dissipation to the exterior.

### *Stone Material Limit State*

Stone material limit state defines the method of establishing a safe margin of overdesign given the factored wind load expo-

◀ FIGURE 23: ***Preliminary Typical Facade Visual Mock-Up.*** **For the purpose of approving the stone selections for the typical floor and lower register cornice, this floor-high prototype combines the architect's preliminary design selection of stone type, color, finish, and configuration. These mock-ups are vital to visual confirmation of the design and to comprehend the stone's natural variations within the overall arrangement. These variations are difficult to represent and to visualize on normal 12" x 12" samples. For A T & T Chicago, Cold Spring Granite featured their sunset beige thermal-jet(thermal-finished with a water-jet rinse) granite with polished edge bands in the typical floor areas. Sunset red polished and honed granite was used in the lower register, which later was changed to thermal finish due to this mock-up. Spandrel area represented at the top of the mock-up are mountain green thermal and polished, with one panel patterned with a sandblast design. The gentlemen in the foreground show samples of the darkest ranges of streaking or veining that would be part of the sunset red material.**

◀ FIGURE 24: ***Preliminary Storefront Column Visual Mock-Up.*** **For the purpose of approving the stone selections for the storefront areas, this partial column prototype of a sidewalk-level column's stone cladding built the different stones, shapes, and finishes initially designed by the architect. Featuring Cold Spring black polished water table, base, and window surrounds, all the corners were solid and 'L'-shaped in cross-section. Face stones were sunset red polished, with honed rustications. The midwidth medallions were polished mountain green. A similar sample of sunset red showing the expected dark concentrations was again viewed with the this mock-up to begin to determine the acceptable range. Due to the high-visibility and close vicinity of observers of this area, more restricted color ranges, and thus more uniform appearance was provided.**

sure from wind load resistance and performance criteria, material unit-strength statistical variability from established industry standards such as ASTM standards, the environmental exposures, and consideration of the anchorage design type and the conditions of its support and installation. Commonly there are different limits established for panel flexural performance and stone stresses interacting directly with the anchorage device.

### *Anchorage Device Limit State*

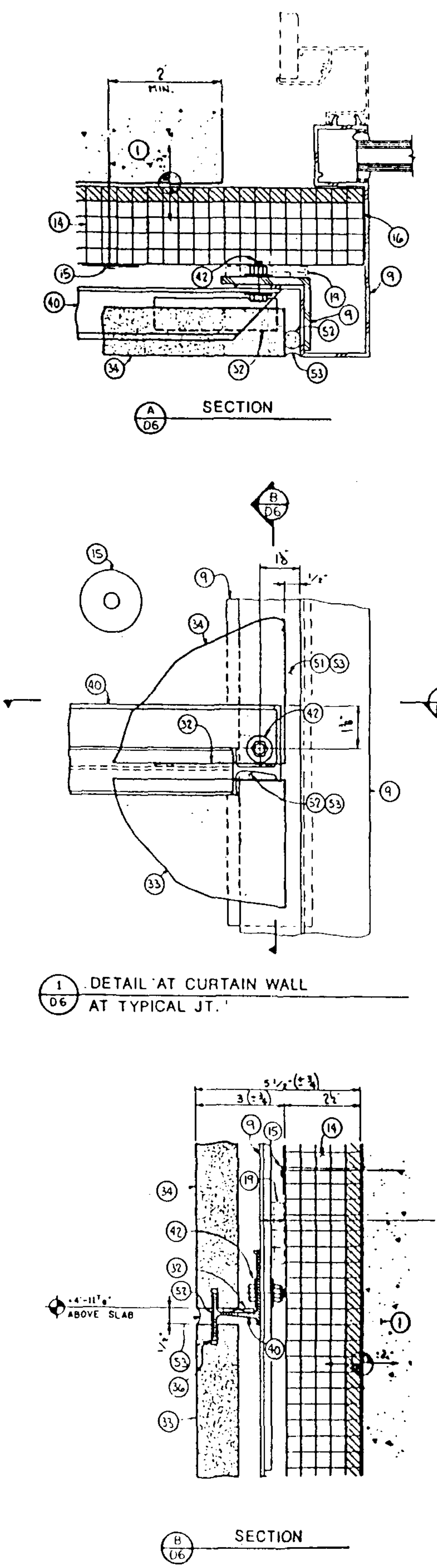
Anchorage device limit state defines the method of establishing a safe margin of overdesign given the factored reactions from the stone derived from the stone material limit state, the device's configuration stability, influencing factors of the supporting frame's dynamics resulting from wind load resistance and performance criteria, the environmental exposures, and some consideration of the stone material characteristics local to the device's engagement into the stone and its expected behavior under the stresses induced by that engagement.

### *Performance Proofs*

Performance proofs are the actual physical methods executed to prove that the system's performances are in conformance with the prescribed standards.

### *Initial Evaluation*

These methods involve test methods that simulate loadings or environmental conditions that are conducted initially, as a prerequisite to actual on-site construction, on a prototype mockup (Figs. 23 and 24) or actual material samples to measure the assembly's or material's performance. Pressurized chamber




◀ FIGURE 25: *Excerpt From Detail Assembly Shop Drawing At A Panel Anchorage.* **A typical condition at a stone anchorage attachment to a curtainwall mullion for A T & T Chicago. The window mullions supported the stone installation were shown and labeled, even those not by the stone contractor, including the main building frame (and its tolerances), insulation, curtainwall, glass, and all fasteners. All components are identified by a numbered bullet, which fully specifies the part, by whom, its sequence, (identified by order of number), the part's size, material, finish, and any other installation instructions particular to that component.**

and apparatus-pull procedures are customized to evaluate capacities of assemblies such as sample walls or anchorage-engagement interactions within the stone. All test methods are intended to model real-world forces, whether environmental or structural. Once conducted upon the original sample construction that duplicates the componentry and construction methods to be used on the remaining project work, results are compared to the performance criteria to establish the design's compliance with AAMA, ASTM, or any other designated project-specific requirements.

### *Consistency Evaluation*

Following initial proof-of-performance, consistency in the methods of construction, techniques used to place the work, and material strength is assured with intermittent quality control tests. These test methods may be simplified or abbreviated versions of the original tests, for the comprehensive nature of the initial methods are formulated to enable investigation into system performance as well as simply prove fundamental conformance. For tests determining the material's properties initially, enough samples must be tested to establish statistical assurance upon which to base the engineering judgements of the project. Quality control sample testing often have fewer samples, enough only to suggest consistency with the original test results, and thus those engineering conclusions deduced from those tests.

### *Architectural Aesthetic Intent*

Architectural aesthetic intent prescribes details of material and systems to satisfy the project's aesthetic design intent as conceived by the architect. High emphasis is usually placed on the profiles, planes, and configurations that are exposed-to-view. Value engineering commonly alters how these shapes are achieved to allow the contractor to optimize fabrication, assembly, and installation to perhaps lower costs or avoid durability problems inherent in the architecturally proposed scheme. Responsibility for the system that is the result of this improving process is shared between the builder and the original designer, for the process involving the presentation of actual shop drawings illustrating these concepts and their review by the architect is necessarily an interactive one.

### *Allowable Stone Fabrication and Installation Tolerances*

Allowable stone fabrication and installation tolerances enumerate allowable deviations from theoretically perfect lines, and are required to accommodate reasonably the expected limited perfection of manufacturing and placement of primary and secondary structure, anchorages, and stone in construction. These tolerances should be compatible both with manufacturing limits attainable with that type of stone, its shape and its finish while considering the fabrication technique used to produce it as well as the intended fit of the designed parts.

### *Interfacing Work*

Variability of interfacing work by other trades must be contemplated also, for realistic, recognized tolerances for that trade's work should be allowed where that work interfaces the stone. Imposing finished stone's tight tolerances upon a substrate or adjacent work that may not typically be completed to that exactness promises that interferences will occur. Limits stated at each stage are not cumulative.

### *Shop Drawings*

Shop drawings are the installer's or fabricator's (or both's) drawings of their detailed presentation of the work (Fig. 25). In general, this includes all information for the fabrication and installation of each individual stone relative to construction that precedes its placement and all construction that succeeds that stone's placement. This sequence-focused mindset motivates a thorough presentation that avoids errors and field interferences that improves satisfaction, productivity, and profit.

Indicate the overall stone layout with reference locations from base building grid lines. Show key workpoints that key the organization of the stone arrangement and locate the starting points for installation where sequence is pertinent.

Within the layouts, identify individual stone types, finishes, thicknesses with thickness tolerances, joint sizes with width tolerances, joint infill materials, and movement joints. All pieces should have a piece mark for identification that allows tracking through fabrication, shipment, and installation.

Show fabrication information for the anchorage preparations in the stone, their sizes and locations. Acceptable tolerance ranges for guaging sawcuts and holes from the stone's adjacent faces are critical to maintaining the structural adequacy of the stone's attachments and in-plane alignments after installation.

In detailed sections, show the anchorage device and how it is engaged into the stone. Indicate by written notation what type it is and what its size is, especially if sizes or types vary in the project. Define how the anchor accepts variances in location of the support it attaches to, and state the tolerance limits of the supporting substrate that it is designed to accommodate. The drawings should show contiguous construction provided by others, note that those components are by others, and suggest what the installation criteria for that work is if it influences the stability or durability of the anchor and supported stone.

In depicting the interfacing or adjacent work by others, identify its sequence of installation as being before or after the stonework. Define the limits (tolerances) of that work's final location so that it does not interfere with stonework, its anchorages, or support. These tolerances should be consistent with that trade's generally accepted practice. Should the placement limits required of this interfacing work need to be less than the project specifications allow for that work, or those generally recognized by that trade, the condition should be reconciled with the architect prior to commencing installation. Do this preferably during shop drawing preparation. The objective of this reconciliation should be to adjust the location of the dependent construction to create enough room for the anchorages to accept the stonework while accommodating the variable tolerances between the two trades' work. This assures that the completed stonework is also permitted to be placed within its acceptable limits as specified.

## Limits and Dependencies on Interfacing Work

The placement accuracy of interfacing work controls the adjustability parameters for the stonework installation. As allowable placement ranges widen, increased required adjustability causes anchorage components to become larger to accept the most open extreme, and sometimes more complex to also accept the most closed extreme. When substrate construction deviates from its designed position and cannot be accommodated by anchor adjustability, stone finish planes are not true and anchorages may be beyond their structural limit. Specified erection tolerances and procedural requirements obligate the installer by contract to perform the work within those parameters. The intent of those limits is to control quality and consistency of completed work to result in a uniform, clean, and accurate completed appearance.

## Conformance of Preceding Work

It is suggested that the stonework installer should progressively inspect, ahead of its own progress, other contractors' work that will adjoin the stone to assure conformance to limits defined on the shop drawings. Inconsistencies need to be reported to the authorities directing the work. While it should not be construed to be the stone installer's responsibility to establish, prove, or disprove the conformance of work by others, any inconsistencies discovered early might offer an opportunity for correction without impacting stonework progress. The procedures and workmanship of finished stonework are often not anticipated by other trades. The general contractor's or construction manager's responsibility for coordination should carry the burden of investigation to find problems by themselves proving the conformance of their subcontractors' work before any deficiencies impact project cost and time.

### *Correctable Conditions*

Any boundary condition that may compromise the anchorage's contemplated engineering design and its initial assumptions must be corrected to restore expected conditions. Anchorages or support framework could also be re-engineered to accommodate those existing deviations if the boundary condition is irreconcilable. The correction must preserve the standards and criteria established for the project.

### *Suggested Typical Tolerances for Finished Exterior Stonework*

Highly or closely visible areas might be "tighter." Areas only seen from a distance may be "looser." Either option must be compatible with the panel size, anchorages, retention system, all boundary conditions, and recognized installation methods.

### Variation from Plumb

Wall surfaces, rises, external corners, vertical joints or other conspicuous vertical linear features should not exceed 1/4 inch (6.4 mm) in any story or 15 feet (4.5 m) maximum. Lines must be true.

### Variation in Level

Horizontal grades and other conspicuous horizontal or flat linear features should not exceed 1/4 inch (6.4 mm) in any window bay or 15 feet (4.5 m) maximum, nor 3/4 inch (19.1 mm) cumulatively in 40 feet (12.2 m) or more. Keep lines true.

### Variation in Linear Building Lines

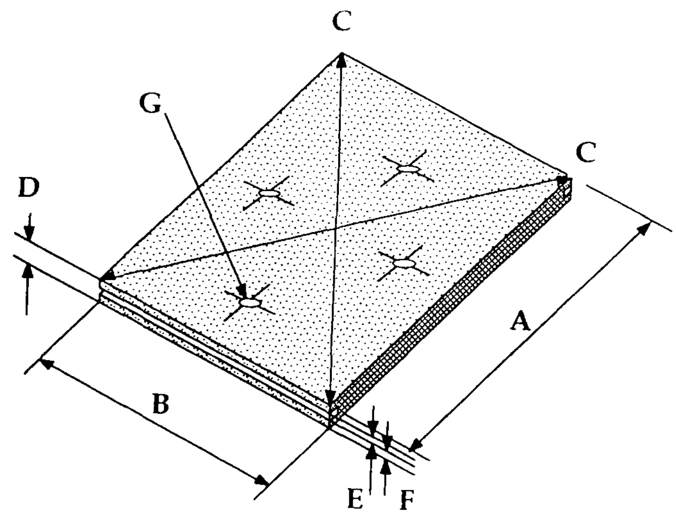
Theoretical positions shown on drawings, and the portion of wall facing relative to those positions should not exceed 1/2 inch (12.7 mm) in any window bay or 20 feet (6.1 m) maximum, nor 3/4 inch (19.1 mm) cumulatively in 40 feet (12.2 m) or more.

### Variation in Face Plane

Adjacent pieces (lippage) for polished, honed, or other smooth-finished stones should typically not exceed 1/4 of the width of the joint between the pieces (for example, 1/16th inch is the maximum acceptable at 1/2 inch joints between adjacent stones). In high visibility areas where the stonework is within ten feet of the common pedestrians' view or their immediate touch, this typical variation could be limited to 1/32nd inch between the pieces regardless of the joint width. If individual stone panels are relatively large, this small tolerance may be unrealistic. When the stone's finish is flamed, bushhammered, cleaved, or otherwise rough and creates an irregular face, the acceptable lippage shall be the sum of those named above added to the maximum deviation in stone panel thickness or face plane.

### Individual Stone Fabrication Tolerances

Tolerances recognized by the industry stone fabricators are those recommended by the National Building Granite Quarrier's Association (NBGQA) (Fig. 26).




▲

FIGURE 26: ***Individual Stone Fabrication Tolerances.*** **Practical minimum tolerances which should be expected on most normally-produced dimension stone. While many fabricators may boast better, design engineering and expectations should accommodate these variances at a minimum. As panels increase in size and thickness, these tolerances may not be sufficient:**
**A: length = +/- 1/4 inch.**
**B: width = +/- 1/4 inch.**
**C: diagonal out-of-square = +/- 3/8 inch.**
**D: thickness(up to 2in.)= +/- 1/8 inch,(over 2in.) = +/- 1/4 inch.**
**E: kerf fin width = +/- 1/16 inch.**
**F: kerf slot width and depth = +/- 1/16 inch.**
**G: dimple and hole location = +/- 3/8 inch.**

## Qualified Stone Designer Expertise

A designer experienced with stone mechanics, structural mechanics, and stone construction may be required to properly design the stone, anchorage, backup, and coordinate the overall retention system with the base building structure. If the application duplicates the manufacturer's typically suggested details, and the stone or anchorage is not structurally challenged, perhaps only an experienced stone installer may be required. Whether the professional is an architect, engineer, consultant, or contractor, this specialist's expertise and previous experience should be commensurate with the complexity and size of system predicted to be necessary for the project.

### Specialist Qualifications

A stone design specialist is recommended where stone cladding conditions are not ordinary or typical. Complex connections, unusual loadings, composite systems or materials, or uncommon installation or manufacturing methods require construction ingenuity and structural insight to resolve conditions correctly. Other criteria needing expert review are atypical performance requirements, untested constructions, dynamic backups, or a unique architectural design. Experience with architecture, engineering, stone materials, and cladding rehabilitation are combined to resolve a constructible and lasting solution. Select a specialist by matching the person's understanding to the project by the following considerations.

### Retention System Performance Record

The specified or developed cladding or stone retention systems should be comparable to the professional's design and construction expertise.

Stone-anchor interactive mechanics and the knowledge of how to deduce its performance capability are vital to predicting retention capacity and stability. Comprehend the internal behavior of the stone where it is engaged by the anchorage as well as the behavior of the anchorage where it contacts and engages the stone. Know how to evaluate and apply it.

Cladding system complexity within the overall exterior wall that potentially could develop should be comparable to the professional's expertise. The designer's sophistication should be comparable to the nature of the system to be engineered; Load conditions that generate the critical stress state and in particular, the path the load travels from surface to structure must be traced. Both panel weight and superimposed loads at the stone panel concentrate and collect at the panel support anchorages, and transfer to the framing and the structure. Knowledge of this path, the different materials, behaviors, and their design codes should be comparable to the professional's expertise.

Type of base building structural frame construction, its dynamic movements and its influence on the cladding systems should be comparable to the professional's expertise.

Building codes and authorities having jurisdiction over the prospective project as well as those codes' trend of interpretations should be familiar to the professional's experience.

SECTION 3

# MATERIALS USED TO CONSTRUCT INTERFACING SYSTEMS IN EXTERIOR WALLS

Presentation, interfacing materials, and surrounding joints are three variables that influence stone support performance. While they are not mathematically calculated, their accuracy and match with the cladding requirements are critical to proper system function.

Architectural conformance and behavioral compatibility depend upon decisions regarding these parameters. Subjective expectations and objective reality must be consistent.

Section 3 *Materials Used to Construct Interfacing Systems in Exterior Walls* reviews structural and environmental compatibility parameters for adjacent systems. Framing and anchor devices made of metal not corrode, deteriorate, or react with other components. High-performance weatherproofing products must maintain their integrity. Both must remain intact in environments that alternate in extremes.

Deficiencies in these systems forfeit the boundary conditions for the stone. This compromises durability and may cause failure. This section includes Metal Integrity and Compatibility; and Joint Filler Function and Capability.

## Metal Integrity and Compatibility

Both ferrous and nonferrous metals are used in anchorages and exterior wall framing. When used against stone or together in the wall system, moisture, mineral and metal contact must be controlled. Prevent direct deleterious contact and indirect potential contact caused by moisture migration in the hidden internal wall environment. Metals must maintain their original strength, form, and finish through the building's expected service life.

### *Considerations for Use*

Select metals used for anchors or anchorage system components to be suitable for their intended use. Avoid galvanic activity by isolating dissimilar materials with stable, inert separators. Investigate nobility relationships with adjacent metallic, ferrous, or stone materials that may come in contact with the anchor, fasteners or framing. Prevent soluble ions or chemical precipitate from being collected by migrating internal moisture and do not allow "bridging" by moisture transferred across dissimilar metal surfaces. For a metal component that penetrates several "layers" of the wall, design it for the most severe exposure, or "layer."

### *Metals in Contact with Stone*

Metal components in contact with calcareous stone for anchorages such as limestone and marble shall be noncorrosive in the atmospheric environment while in contact with the stone and any potential precipitates.

Properly protect metal components with a coating or a sheathing compound that does not react or stain adjacent stone or other wall materials. The severity of exposure will vary with the secondary minerals in the stone, the environment of the project, surrounding materials in the wall, and even locations within the same wall. Any compound considered must be compatible with the perimeter sealant.

Nonmagnetic austenitic types 302 and 304 stainless steel may typically be used without protection.

Do not use aluminum alloys when the potential of moisture exists unless the metal surfaces will not become exposed beneath its protection. Anodizing is not appropriate protection in the presence of a calcareous or calcitic stone.

Metal components in contact with siliceous stone for anchorages such as granite shall be noncorrosive in the atmospheric environment while in contact with the stone and any potential precipitates.

Properly protect ferrous metal components with a coating or a sheathing compound that does not react or stain adjacent stone or other wall materials. The severity of exposure will vary with the minerals in the stone, the environment of the project, surrounding materials in the wall, and even locations within the same wall. Any compound considered must be compatible with the perimeter sealant.

Nonmagnetic austenitic types 302 and 304 stainless steel may be typically used without protection in nonacidic or noncoastline conditions.

Use flouropolymer coatings or class-I 215-R1 (0.7 mil thickness) clear anodizing on aluminum in potentially reactive exposures. For nonindustrial climates (without acid precipitation) and stable mineral compositions, mill aluminum may be used. Mill aluminum will form its own protective, nondestructive oxidation naturally. Remove any press, saw, mill, or other fabrication lubricants in an alodyne bath to prevent stone staining before use.

◀ FIGURE 27: *Stone Cladding Framing Above The Roof.* **Similar to the complicated backup metal framing behind the base stone, the top of A T & T is also well articulated to give a unique identity to the building in Chicago's skyline. Where the base framing attached to the concrete, the top framing attached to the structural steel building frame. Full-size plastic shins completely separated the aluminum work from the dissimilar steel. Large slots allowed for tolerances between the building frame, the intermediate aluminum facade framing and the angles welded to the steel that provided for the bolted attachment.**

### *Metals Not in Contact with Stone*

Metal components not in contact with stone (Figs. 27, 28) will be exposed to weathering elements if these components are exterior of the vapor barrier. They require the same protective considerations noted previously. Select metals and their finishes to be compatible with the combined criteria of Considerations for Use. Stainless steel, galvanized steel, zinc-rich painted steel, epoxy-coated or painted steel, or aluminum, are options given that the metal and finish meet the compatibility conditions of the stone anchorages and the vapor barrier or second-line-of-defense. Prevent galvanic action potential.

Metal fabrication that exposes base metals that require protection need to have the protection replaced over these exposed edges. Coatings should cover these exposed cut-ends of steel if its exposure to moisture is substantial enough that structural deterioration may occur within the building's expected life. Drilled or punched holes for fasteners could require the same considerations, especially with self-drilling screws, which expose the base fastener metal. Further, if the opportunity exists that any corrosion product may leach onto the stone to cause staining, protection must be provided to prevent such corrosion. Aluminum naturally oxidizes to form its own protective coating. Concealed aluminum components will protect themselves by natural weathering, or oxidizing, if it is adequately separated from dissimilar metals and isolated from any calcium carbonate or its precipitate.

### *Metal Separations*

Metal components potentially in contact with other components made of a dissimilar metal should be isolated to prevent direct contact, which would result in galvanic deterioration. Adequate separation must be further provided to prevent moisture or condensation to collect between the two components and bridge their intended separation, thus acting as an electrolyte.

### *Metal Nobility*

Metal nobility is the metal's molecular makeup which, upon contact with other elements of another dissimilar metal in the presence of any moisture (even humidity). The less-noble metal (lower in the electromotive-force series) will disintegrate as it "sheds" its material through electrical conductivity to the more-noble metal (higher in the electromotive-force series). The series, in decreasing order, include:

1. stainless steel
2. aluminum
3. zinc (galvanizing)
4. iron (steel)
5. tin-nickel
6. lead
7. copper

Aluminum, a material of high relative nobility, may attack other materials. Evaluate aluminum parts specifically with the individual stone and the anchor's environment to determine if additional isolation might be required, and thus exactly what type (anodizing or coating) is prudent to attain that protection.

## Joint Filler Functions and Capability

Match joint filler product and performance with the conditions of the joint. Test the products on the actual substrates to prove they meet the requirements. Anchor design philosophy emphasizes correct boundary condition construction, which directly involves the joint filler. While the filler is a necessary component in the weather-protection function of the exterior skin, the performance of this function cannot compromise the structural performance required by the stone retention system. Testing of the joint filler both for material compatibility with its adjacent components as well as scrutiny of the performances during other structural testing should preserve the boundary conditions particular to that assembly, whether elastomeric or rigid.

Use Sealants to weatherproof joints between stones and adjacent components to form a "wet"-applied but "soft" filler. Sealants form in-place gaskets that insulate movement and prevent forces from being transferred between cladding parts. Sealants in contact with stone must be applied in the conditions and upon the substrates and installed with the recommended methods as intended for the project. The manufacturer, installer, and facade designer should specifically consider performance characteristics of the sealant product in relation to the conditions of the project. The elastomeric capabilities of the sealant must be compatible with the performance requirements of the joint. Compare adhesion strength with the full movement range expected at the joint.

Sealant product tear and peel strength should be evaluated on each of the substrates to determine the cleaning and primer requirements that will be required during its installation.

Movements between the bordering substrates will cause changes in the joint size. Sealant elasticity and its compressibility modulus, and its durometer must be matched to accommodate those movements.

Potential contact with other sealants, or the same sealant but in a cured or curing state requires consideration to assure adhesion and compatibility.

Maintaining aesthetic appeal requires consideration of the resistance of the sealant to soiling, fading, or its propensity for attracting contaminants. The sealant's compatibility to the stone material is established by testing the stone's tendency to absorb curing compounds and as a result, discolor, or stain.

Adhesion testing is appropriate for determining the peel strength of the sealant to the substrate. Most manufacturers offer this testing to benefit the project, and most require it to validate their performance warranty. Some manufacturers require periodic tests during installation to assure that proper substrate surface preparation is occuring routinely and that product quality is good.

Application conditions are prescribed by the manufacturer of the specific product to be used as printed in the "manufacturer's installation instructions." Temperature range, substrate condition, and necessity for a primer will be discussed as recommendations in the original compatibility and adhesion tests performed by the manufacturer for those conditions.

Stain testing is appropriate for determining whether the sealant's curing agents or sealant compounds will be absorbed into the stone and discolor or visually degrade the substrate in any objectionable, but obviously noticeable manner. Most manufacturers offer this testing to benefit the project and to prove their product as nonstaining. Most require the tests to validate their nonstaining warranty.

Use gaskets to weatherproof joints between stones and adjacent components to form a "dry" but "soft" filler. A gasket can seal the joint itself, or a gasket can be a backer for "wet" sealants to insulate movement and prevent forces from being transferred between cladding parts. Gaskets normally rely upon compression, not adhesion, to maintain their seal. Apply gaskets onto their substrates and install them by methods recommended by the manufacturer from tests conducted for the project. Select gasket materials to accommodate cumulative construction tolerances between the stone and those

◀ FIGURE 28: *Coping Bracket At Setbacks.* **Besides supporting the cubic cornice stones composed of two-piece profile, the bracket also had to support the top of the facade panel below and provide interface for the roofing and the flashing. The roof floors and parapet curbs were waterproofed with a membrane in this view. Plastic shims separated the aluminum assembly from any potential contact with dissimilar materials, and leveled the bracket to the correct elevation. Stainless-steel sheet metal through-wall flashing wrapped over the bracket, and was separated from the aluminum with cloth-reinforced vinyl tape. The back leg of the bracket provided for mechanical attachment of the roofer's stainless-steel counterflashing, which closed the wall cavity and terminated the roof membrane.**

tolerances between its adjoining components while still maintaining adequate precompression to hold the seal against pressure differentials in the wall. Design gasket material and joint size to accommodate movement that accumulates where stone interfaces adjoining cladding parts.

### *Materials*

Gasket material is selected to be compatible with both the stone and also adjoining materials while remaining stable and pliable in the extremes of the environments it is exposed to.

Extruded gaskets are typically neoprene, vinyl, or silicone. These are typically solid and of higher durometer, and thus lower compressibility.

Cellular gaskets are typically foamed butyl, polyethylene, or polyurethane. These are typically spongelike and of lower durometer, and thus higher compressibility.

### *Staining*

Some gasket materials may bleed their curing compounds and color resins, which could cause staining of the stone either directly by absorption into the stone or could indirectly stain the stone by precipitation runoff onto the stone and across other facade elements. Verify with the manufacturer the compatibility and stability of the gasket material and its chemical compounds with the contacted or surrounding adjacent materials. Where pertinent manufacturer's information is unavailable, absorptive-type weathering testing with the stone for the project might be appropriate to assure that the stone's aesthetic integrity is maintained.

### *Use of Mortar*

Use mortar to weatherproof joints between stones and adjacent components to form a hard filler. When the condition does not require an elastomeric filler to isolate movements and forces, mortar joints create structural bearing between stones and other cladding parts. Use mortar materials to set stones into position and for pointing joints between the stones.

Mortar-filled joints accept only infinitesimal micro-movement. Mortar joints are rigid and depend upon chemical bond and compression for attachment. Mortar in typical masonry construction forgives differential movements from moisture swelling and temperature changes because typical unit masonry units are small. Larger stone units cause greater differential movements in the joints, which are then less likely to maintain chemical bond, and thus, weathertight integrity.

### *Staining and Mortar*

Portland cement, masonry cement, latex cement, and lime used in preparing cement and lime mortar should be non-staining. Dyes or other coloring compounds can migrate into the stone and cause staining. Admixtures added to cements and mortars should be verified not to absorb into the stone to cause discoloration or degradation.

### *Non-shrink Grout*

Non-shrink grout should not be used. Many non-shrink grouts actually expand to not allow cracking, which can induce loads into the stone. Avoid using non-shrink grouts that include ferrous ingredients that could rust in contact with moisture, and could leach and discolor the adjacent stone.

SECTION 4

# HOW TO KEEP EXTERIOR STONE JOINTS WEATHERTIGHT

Presentation, interfacing materials, and surrounding joints are three variables that influence stone support performance. While they are not mathematically calculated, their accuracy and match with the cladding requirements are critical to proper system function.

Architectural conformance and behavioral compatibility depend upon decisions regarding these parameters. Subjective expectations and objective reality must be consistent.

Part 4 *How to Keep Exterior Joints Weathertight* reviews the complicated relationships between skin components. Leaks and binds are avoided when joints are sized and filled right. These are summarized by:

- Avoid Restraining Stone Panel Movement
- The Environmental and Structural-Proof Function of the Joint
- Isolate Components That Occupy the Joint
- Static Effects That Influence Joint Sizing
- Dynamic Effects That Influence Joint Sizing
- Effects That Change Horizontal Joint Widths
- Effects That Change Vertical Joint Widths

Proper design of the joints must include the behavior of all the effects that change the joint widths to be sure the joint filler material is compatible.

Movement range, adhesion, stress, and elasticity must all match to preserve a weathertight joint.

## Avoid Restraining Stone Panel Movement

If the structural philosophy of the exterior wall is to treat the stone panels as infill, the joints surrounding the panel must allow freedom of movement. Restraint results in added stress to stones where freedom is not allowed. Because restraint, or "confinement" is usually unintended, magnitudes of resulting stress are difficult to predict. Therefore, avoid creating conditions where potential restraint may occur. Restraint often concentrates at corners and can quickly escalate the stone's experienced stress above its capacity, which could cause failure or even collapse. Avoid unintended restraint by simply independently supporting each individual stone or unitized group of stones. Then also allow for each panel or group's unrestrained movement. If stones are grouped, the larger areas accumulate larger movements at the group's boundaries.

### *Achieve Freedom-of-Movement*

Avoid restraint by surrounding each stone with "soft," structurally open joint that avoids confinement stresses. With "soft" joints and independent "retention" (treating the stone panels as infill), there is no rigid continuity between the separate stones or stone groups and their adjacent components. Rigid structural continuity caused by contact or overcompression causes forces from one stone or its boundary components to be transferred unintentionally into the edge of another stone.

### *Construct Soft Joints*

Isolate the stone or stone group from load influences from any adjacent stone panel or wall component that is not specifically designed to support that individual stone. Fill joints with a material that maintains weathertight continuity across the joint without conducting forces between adjacent construction. Weatherproof the joint with a sealant as a "wet"-applied seal, or a gasket as a "dry"-applied seal. Provide these soft joints where movement freedom is required between stones or between a stone anchor and another stone or group. Locate either a "soft", or moving joint often enough to alleviate the anticipated skin movement and still maintain the maximum aesthetically acceptable joint width. To independently support stones, their surrounding joints are structurally open to allow the stone to "float" as infill.

### *Assure Consistent Workmanship*

Consist workmanship provides the same quality in the field that exists in design. Assure that no element such as shims, anchors, concrete splatter, flashing or gutters, insulation, excess joint backer or bond-breaker tape, or other debris becomes included anywhere within the depth (thickness) of the intended soft joint. These might cause "confinement," "pinching," or "prying" on stones, which unintentionally transfer forces.

Clear the full depth of the joint of any debris. The first step required for proper sealant or gasket installation is clearing the joint. This assures that the designed range of movement is allowed.

The second step required for proper sealant or gasket installation is cleaning and preparing substrate surfaces to receive the joint filler. Clean the surfaces of the substrates to promote proper adhesion. Accomplished this surface preparation in accordance with the manufacturer's published recommendations. This assures proper material performance which will then accommodate the recognized movement within the joint.

The third step required for proper sealant or gasket installation is keeping the substrates clean and dry once prepared until the joint filler is placed. Protect the prepared surfaces. Confirm that any compounds such as cleaners or primers are compatible with the substrates. Where dissimilar materials such as aluminum and stone form a joint, different preparations may be required for each side of the joint. Keep these separate. Keep dry. Keeping a joint filler functional while the skin is in service also keeps debris from entering a joint and confining movement.

The Environmental and Structural-Proof Function of the Joint Assure that the stone panel remains isolated from loads from other facade components. Keep the stone structurally independent. Make joints between individual stone pieces and their surrounding and adjacent exterior wall elements "soft" joints filled with an elastomeric sealant or gasket material.

Aesthetic desires predominantly request minimum joint widths. To keep environmental elements from penetrating that joint, the joint width needs to be sized to accommodate several different affects that occur simultaneously. Through these effects, remain within the working range of the joint filler's elasticity. This maintains adhesion. Proper joint filler performance depends upon preparation and placement in accordance with the manufacturer's recommendations.

## Isolate Components That Occupy the Joint

Do not compromise joint design by incriminating on the designed open joint width with anchor clips, fasteners, or other hardware. Maintain freedom-of-movement and proper movement ranges. The joint range is only preserved if their is *no* confinement of the stone panel. Items occupying the void, or joint, between the stone cannot restrain either the stone itself or the performance of the joint filler. Even if structural freedom is not violated, weathertightness may be if movements exceed the filler's capabilities. Allow movement and proper installation techniques for all components at that joint.

### *Joint Design at Anchors*

Keep anchors out of the weatherseal. Anchors that engage the stone in its edge or at its face and pass within the joint could occupy part of the joint width. Unless the sealant or gasket occurs in front of the anchor, the effective joint width at the anchor is reduced by the anchor thickness plus any setting shim or space. Independently analyze joint conditions where anchors or other hardware occur at the joint. Assure that proper clearance exists.

### *Place a Backup for Sealants*

A backup provides a proper sealant cross-sectional profile and prevents three-sided bond. Place a backer rod in the joint to allow consistent filling of the joint with sealant material. Guage its depth according to the sealant substance and joint width requirements. By forcing the "wet" sealant against the rod, sealant is pushed against the surfaces to be adhered. Full contact improves bond. Tooling of the surface skin for a smooth even finish. The filled joint gives good surface contact between the sealant to its substrate to achieve a proper profile. The "hourglass-shaped" width-to-thickness sealant profile allows flexibility by accepting movement at its "neck" while minimizing adhesion stresses at the wide surface of contact.

Select a backer rod material that is also a bond-breaker. This prevents adhesion of itself to the sealant. Located at the back of the joint, the bond breaker prevents the sealant from adhering to three-sides within the joint. Three-sided adhesion restricts elasticity and eventually causes adhesion loss to the intended substrates. Consult the sealant manufacturer's recommendations to properly match the type of polyethylene, polyurethane, open, or closed cell products to the sealant to be used. While open-cell backer rod has been criticized for holding moisture, which can be avoided by only installing what one would caulk during that "dry" session, closed-cell backer rod, when punctured, can breathe and blister the curing and skinning sealant, causing pinhole leaks. Review both sealant and backer rod material manufacturers' information to assure chemical and functional compatibility.

Select backer tape instead of a rod where face depth is limited. Where the available joint depth does not permit room for a full backer rod and the sealant, a bond-breaker tape prevents three-sided adhesion. Review compatibility issues identical to those for the backer rod. Review the manufacturer's installation instructions to assure that the application and installation techniques allow the expected ultimate sealant performance.

Complete joint preparation prior to placing backer components into the open joint. This helps avoid puncturing closed-cell tape or rod and destroying them with solvents. Apply chemical cleaners (such as methyl-ethyl-ketone) and substrate primers with proper applicators and in proper amounts in strict conformance with manufacturer's recommendations. Take care to have all these solutions either dried or evaporated before placing backers. Solvents or primers could be absorbed or trapped if not dried. Contact with the sealant would likely result in adhesion failure and a leak. Petroleum distillate solutions used to prepare adjacent metal surfaces could stain the stone. Devote appropriate care to protect the stone and apply onto the metal only. Keep contact off the stone. Even if the solvent does not cause a discoloration, the absorbent stone could retain the solution and prevent sealant adhesion onto the stone edge. This failed adhesion causes leaks.

### *Match Sealant Modulus to the Project Application*

Sealant modulus is the usable flexibility, or elongation and compression capability of the sealant compound. The flexibility cannot threaten adhesion by high pulling stresses. The sealant body integrity cannot be jeopardized by tearing. Modulus varies with product type and sometimes between manufacturers of the same sealant type.

### *Maximum Material Capabilities*

Low modulus sealants have the greatest elasticity and greatest ability to accommodate wide movement ranges within a given nominal joint width. An ultra-low modulus silicone can accommodate +100% and –50% movement. These can accommodate elongation up to its original joint width (expansion equal to the original joint width, which is +100%) and compression up to half its original width (which is –50%). All silicones are not compatible with all stones, however.

### *Confirm the Movement Range*

Select a sealant that has a usable range of joint-size change that meets the project requirements. Compare the modulus with the expected movement range. Do not exceed what is recommended and warrantable by its manufacturer. Verify that this range includes the movement extremes predicted to occur after placement of the sealant during construction (in a new building). Include construction phase effects that may not typically be an increment in the frame engineer's predicted movements such as equipment or material live-load displacements or axial shortening due to the building's own weight. Match the sealant's cross-section aspect ratios and sealant's modulus to the project's conditions according to manufacturer's recommended specifications.

### *Determine Proper Design Joint Widths*

To determine what the designed joint widths should be, multiply the sum of the projected movements at the condition by the sealant's acceptable range. Size the joint according to the sealant's performance criteria both for expandability and compression.

Compute maximum joint width, for example, as the "sum x 2", where the sum = addition of all effects that will stretch the joint open for a sealant, and x 2 is the ratio for a sealant that accepts +100% elongation. This ratio changes proportionally when the sealant's elongation performance is different from the +100% range of the example.

Compute minimum joint width, for example, as the "sum x 2", where the sum = addition of all effects that will compress the joint closed for a sealant, and x 2 is the ratio for a sealant that accepts –50% compressibility. This ratio changes proportionally when the sealant's compression performance is different from the –50% range of the example.

The nominal joint size designed for that condition of the project should fall between the maximum and minimum computed.

### *Gasket Durometer*

Gasket durometer is the relative softness and compressibility of the rubber, silicone, or neoprene gasket material. Durometer for the solid is similar to modulus of the cured sealant. Select a gasket with usable compressibility that matches the extremes of movement predicted to occur at the joint after the gasket is placed within the joint. Include the movements that follow initial compression of the joint that closes the joint in the construction phase. Most gaskets do not depend upon adhesion like sealants, but instead rely upon contact and precompression. Initial compression must exceed the expected elongation that may occur from the sum of effects causing the joint to open. Size the material using the same approach advised for sealants.

## Static Effects That Influence Joint Sizing

Consider "static" influences that change joint widths. Static effects are conditions of construction or manufacturing that typically do not vary over time after the manufacturing or construction phase is completed.

### *Stone Dimensional Tolerances*

Expected size variances in the fabricated piece size from the intended designed piece size are dimensional tolerances. These variances should be minimal, but realistically controllable within the production and manufacturing processing of the stone. Tolerances vary among fabricators due to their equipment and maintenance. Tolerances vary with the type of stone material, its heterogeneity, and the size of the panel. These differences include altered configuration, face size, thickness, flatness, straightness, or squareness of not only the finished surfaces but the entire piece overall, including the fabricated preparations for the stone anchorages and "backsides". Quality assurance programs with effective equipment and operation control fabrication accuracy. Excess variation outside designed (expected) tolerances prescribed by the fabricator must be corrected, accommodated in the system, or the panel replaced.

### *Stone Installation Tolerances*

Expected locational variances in the placement position of the installed stone relative to its intended designed position are installation tolerances. These tolerances should be minimal, but realistically achievable within the setting and handling techniques used to install the stone. Consider that unrealistic expectations for accuracy dramatically impact manufacturing and installation costs. Balance the highest visual quality practical while still allowing for what is customarily workable for the craftsman.

Correctly designed anchorages adjust for anticipated (expected) support framing placement deviations. Allowable placement deviations should be the final difference between actual and theoretical placement in the completed stonework.

The stringency required for accuracy should be relative to the proximity of the work to an observer's view, whether occupant, pedestrian, or neighbor. Tighter tolerances may be expected where the work is viewable at close range, where deviations are more perceivable. Some moderation from tight tolerances required in closely viewable work such as sidewalk-level storefront work could be afforded to the installation regulations governing finished work some distance from any viewer. That level of "quality" can not be perceived at a greater distance. Excess variation outside designed (expected) tolerances, prescribed by the building designer with a prospective stone installer, must be corrected to maintain aesthetic quality and structural integrity.

## Adjacent Work Installation Tolerances

Expected locational variances from any interfacing component's intended designed position are adjacent work installation tolerances. These affect any fixed construction that adjoins the stone, including its support framing. When other work is placed adjacent to finished stonework either as support or infill that work must conform to the same stringent installation accuracy as the stone. The tolerances for that adjacent work should be equivalent to the specified tolerances for the stone. Otherwise, accommodate differences in tolerances with adjustability in the attachments.

Typical concrete, lightgage metal framing, miscellaneous metal, or structural steel, which often interface or support finished stonework are not typically built to stone's accuracy. Structural carpenters and ironworkers don't work to the specified placement limitations of the finished skin.

Variations in placement of adjacent work require clearances. Given that sufficient "correction" room exists, such as in a wall cavity, and that attachments adjust, "errors" in placement can be accommodated. Adapting separate systems with different tolerances is critical to maintaining intended weatherproof joint widths at these interfaces. Desirable, or allowable finished stone placement tolerances based on aesthetic criteria cannot be imposed on other trades' work. Maintain traditional industry installation "inconsistencies" for interfacing work. Accommodate these in framing and anchorage adjustability.

## Dynamic Effects That Influence Joint Sizing

The building's structural skeleton frame's dimensional changes can be caused by:

1. column elastic shortening from building weight and occupancy
2. member length changes and twisting from flexure caused by lateral loads
3. member length changes and twisting from thermal differences
4. member position changes from column differential settlement
5. member size changes from shrinkage and creep
6. beam displacements from floor loads

These effects must be quantified by the engineer of record for the building's structural skeleton. Cladding joinery and anchorages must accommodate these changes. Isolate these movements to be accepted at moving or control joints. Prevent restraints from occuring between cladding parts by accommodating movement at the anchorages. Some are appropriately resolved where the exterior wall stone retention framing attaches to the building frame. Others should be resolved at the stone panels' anchorages to the exterior wall framing.

Accommodate Movement in Anchorages to Frame-Supported Stone (Fig. 29) Satisfy criteria for avoiding confinement and freeing the stone panel of restraint by allowing differential movement to occur between the stone panel and its

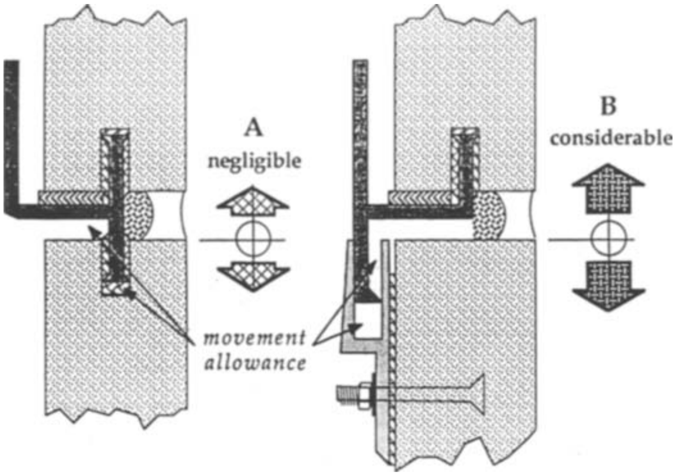



FIGURE 29: *Acceptance Of Exterior Wall Dynamics Within Stone Anchorages.*

**Anchorage A can accommodate negligible movements that can occur between stone courses caused by skin thermal or moisture differential size changes and slight lateral racking. Cushioning materials that fill the kerfs can accept some tolerances and negligible movements between stones. Magnitude of the combined theoretical movements cannot exceed the compressibility or expandability of the kerf fill, nor cause consequential stress on the stone kerf fin.**

**Anchorage B can accommodate movements that can occur between floors, frames, or unitized panels caused by dynamic live loadings and multiple-story bending and racking between levels. An open "track" within anchorage hardware outside the stone can accept larger tolerances and considerable dynamic and longterm vertical or sideway movements. Theoretical movements with tolerances must be less than the slot range plus minimum hardware engagement.**

supporting frame. Accept this movement within the stone anchorage. The anchorage of the stone to its supporting frame accepts movements that result from the exterior wall and structural frame dynamics without transferring movements or forces from these boundary conditions to the adjacent stone. Whether the amounts of movement are "negligible" or "considerable" will depend upon the mechanical sensitivity of the device between the stone engagement and attachment to the frame. Some anchorage types may be more tolerant than others to the same movement magnitudes, depending upon where the conditions and the environments that the anchorages exist. Match the movement to be accepted with the capability of the anchor device to accept the movement.

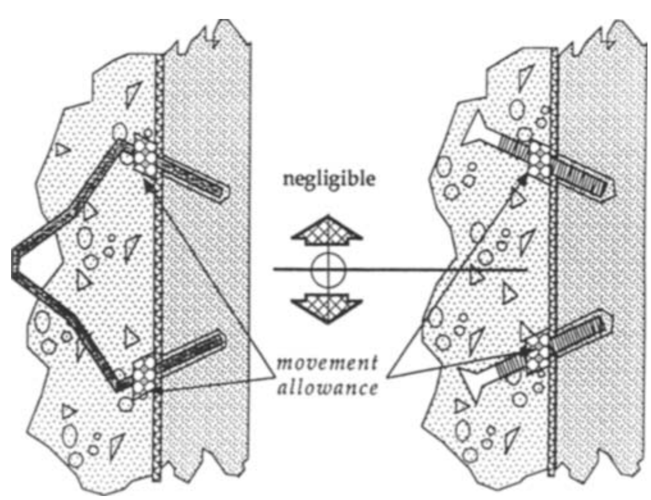
## *Accommodate Movement in Anchorages to Precast-Supported Stone (Fig. 30)*

Satisfy criteria for avoiding confinement and freeing the stone panel of restraint by allowing differential movements to occur between the stone panel and its supporting precast panel. The anchorage of the stone to it supporting precast panel accepts movements that result from differential thermal expansion and contraction within the panel without binding the stone. While these amounts of movement seem "negligible," because the stone and panel are sandwiched together, even in small distances between anchors, threatening stresses can arise. To avoid restraint caused by chemical bond of the portland cement of the precast concrete to the stone, eliminates bond between the stone and panel by placing a bond breaker between them. This suspends the stone only on the pins, whose capacity is determinate. Use rubber grommets to allow infinitesimal flexure in the pins between stone panel and concrete embedment. Size pin diameter to flex while still maintaining sufficient shear capacity. Provide means to accept overall facade interstory movements between the individual precast panels.

## *Cumulative Movement Affects*

Dynamic movements occuring in the building frame and exterior wall framing (curtainwall, strongback, trusses for example) due to superimposed loads cause movement between individual cladding units in the building's skin. Consider these cumulative effects that occur through the construction phases as well as the building's occupancy. Combine both short-term and long-term components of each of these potential influ-

FIGURE 30: *Acceptance Of Precast Panel Dynamics Within Stone Anchorages.* **Two different examples of how to accommodate negligible movements that occur between the stone cladding and its supporting precast panel(or stone panels pinned onto grout pockets on a truss)caused by skin thermal or moisture differential size changes within the panel. A bond breaking "slip sheet" separates the stone from the panel behind to prevent bond. Rubber grommets on the pins or bolts allow the pins to flex between their engagement to the stone and in the concrete panel. Tolerances are compensated for during grout or concrete casting around the pins. Anchorage engagements are inclined and in opposing directions to provide secure mechanical attachment. Magnitude of the combined theoretical movements must be limited to the capabilities of the effective pin flexibility and spacing between them. Prying between pin and stone must be prevented.**



ences. Add effects that could happen simultaneously to determine movement amounts to be accommodated around the panel, not within the panel. If these cumulative dynamics are controlled and isolated within the joinery, stresses within the stone are not affected and structural reliability is maximized. Cumulative movement factors should be involved only at the joints between the panels, not within the panel itself.

### *Joint Sizing*

Size joint width to be compatible with sealant performance and the movement range extremes. Given that loads associated with an individual stone are kept isolated from boundary effects, in-plane movement should not influence internal stone panel stresses.

### *Alignment of Moving Joints*

Horizontally align movement joints completely through the wall in contiguous and adjacent substructures such as strongback, trusses, windows, curtainwall, or panels to accommodate movements so not to confine or restrict components where offsets would occur.

### *Consider Joint Shear*

Adjacent, parallel panel or mullion edges that border sealant joints that move in parallel but opposite directions cause shear in the sealant. Frame racking causes shear in horizontal moving joints. Axial length changes can cause shear in vertical moving joints. Sealant manufacturer's recommended allowances for shear displacements are typically different than perpendicular elongation and compression, especially when all three movements tend to occur at the same time at panel corners (Fig. 31). Compare these allowable shear capabilities with anticipated joint conditions and proportion joint widths accordingly.

### *In-Plane Joint Shear*

In-plane movements occuring in parallel but opposite directions in a joint cause shear in the sealant. Shear stresses in the sealant body become interactively additive to the perpendicularly oriented tension and compression. If not figured separately, shear alone or in combination with the perpendicular movement components could consume the sealant's acceptable movement capability before expected. Amount of shear acceptability could typically be one-half of the compression width. Add transverse and longitudinal movement effects in their correct "sense" to verify actual joint activity. Avoid designs that orient movement joints parallel with primary movements.

### *Concentrated Movements at Panel Corners*

At panel corners, building corners, or corner features, joint intersections experience almost instantaneous and extreme stress changes. The tension or compression at the horizontal joint becomes shear-with-tension within the vertical joint in an interstory racking situation. Interior and exterior corners in "plan" create movement in-and-out-of-plane further increase these potential movement-caused stresses. Because three-dimensional joint activity is complex, these extreme conditions are prime trouble locations. Designers prefer sharp corners and thus small joints when physically larger joints are required. Shear accompanies tension and compression at the corner cladding because the opposing facade planes change positions.

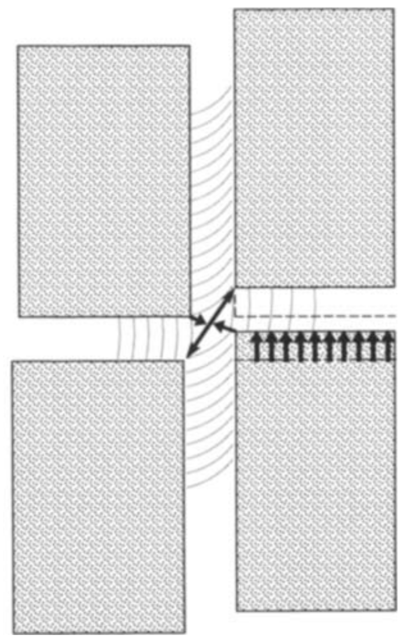


**FIGURE 31: *Acceptance Of Shear In Sealant Joints Between Panels.* Sealant joint shear is caused by movement parallel to the joint. It results in combined diagonal tension and compression stresses across diagonals of the joint. This is most severe where adhesion stress concentrations occur on the sharp corners of the stone, where direct compression or tension may be occurring. Because sealant capabilities to accept shear are often less than normal (perpendicular) compression and elongation, and diagonal movements caused by shear result in greater displacements than normal movements, shear can cause stress that fail sealants early if not accommodated or altogether avoided.**

### *Other Facade Elements*

Accommodate, but isolate contiguous facade items to avoid confinement. Accommodate substructures and features such as windows, doors, their supports, thermal and moisture protection materials, condensation evacuation systems, lighting fixtures, flagpoles, and window-washing retention devices, and other components mounted to, interface, or penetrate facades. Observe the same "freedom-of-movement" criteria defined for stone anchorages. Establish how these elements are supported, whether at the base building frame or by the wall framing. This will then define where differential movements and thus structural isolation can occur. Avoid penetrating the stone where the opening would compromise the panel's or its anchorage's integrity.

### *Structural Isolation*

Isolate elements that could induce movement or forces into the stone panel or its anchorage. Where they interface the stone, structurally separate the component's anchorage and its body from the stone panel by preventing movement from contacting or stressing the stone panel. Separation avoids unpredictable affects on the stone.

### *Moisture Control*

Control moisture infiltration by properly sizing joints to be compatible with the sealant's and gasket's elastomeric performance. Predict the capacity and "route" the path of the internal moisture collection and evacuation system to preserve weatherseal integrity.

### *Internal Moisture Collection*

In a cavity wall, provide a "flashing" or "gutter" system to gather internal moisture and re-direct it to the exterior. Moisture occurs inside the wall because of infiltrated precipitation or condensation. Bridge the cavity void continuously at the building perimeter in a horizontal band between the layer of the wall functioning as the internal moisture barrier and the exterior cladding surface. The barrier prevents uncontrolled moisture accumulation from descending through the entire vertical wall cavity. It avoids buildup of collected moisture at the bottom of a wall and thus the continuously wet lower floor stone. The greater the potential amount of internal moisture in the wall, the more frequent this protective band of moisture-diverting bridge should occur. This moisture evacuation system is commonly coordinated with the window condensation weepage system. Consider combining the two by designing the internal gutter to direct the collected moisture toward the windows without penetrating the cladding or its weatherseals. Weep moisture through baffled holes in window heads or sills.

### *Cavity Compartmentalization*

Continuous horizontal separation of the cavity by a moisture-diverting system also "compartmentalizes" the "dead" air space in the wall cavity. This prevents the "stack" or "chimney" effect, which diminishes the insulative value of the "dead" air space. The enclosed air pockets in the cavity also reduce immediate shifts in interior-exterior pressure differentials. This moderates air infiltration and also structural lateral loads.

### *Cavity Ventilation*

Compartmentalizing the wall cavity may require some ventilation provisions to allow that "dead" air space to breathe. Besides the weeps, vents allow slow pressure equalization with the exterior environment. Air changes in the cavity relieve captured humidity. Conceive and construct these ventilation systems for internal wall cavities to prevent moisture from being directed behind the wall. To balance the weeps that are directly above the flashing at the bottom of the cavity to relieve the water, locate vents at the "top" of a wall cavity's compartment directly beneath the flashing or gutter.

### *Corrosion*

Eliminate any potential corrosion that could be caused by dissimilar metal galvanic reaction and atmospheric or adjacent material chemical contact. Accomplished this by physically separating the materials with a chemically inert barrier such as plastic, PVC, Teflon, polystyrene, or a protective and durable coating. These separators could effect bolt action if the connection includes a friction component for capacity. Provide protection between components commensurate with the potential moisture and corrosion exposure within that "layer" of the exterior wall. As one penetrates the "layers" of the wall construction from the exterior, the environment better get drier. Different exposures exist at each layer or surface of the construction based upon how the wall materials absorb or evacuate moisture. Protect components that will get wet or become moist directly or by contact or "moisture flow" from another surface. Increase protection in potentially "wetter" zones.

## Effects That Change Horizontal Joint Widths

Climate, occupancy, building age, and environmental loads such as wind and snow influences on the joint's horizontal size. Several factors as outlined in the following, cause the joint width to change after the joint filler is placed during the construction phase.

### *Stone Dimensional Changes*

The size and sometimes the shape of a fabricated stone panel changes over time. These changes are caused by thermal expansion and contraction, moisture absorption and evaporation, vapor drive through the stone panel, and hysteresis or warpage. Tendencies to warp, swell, and thermally change vary with stone material, panel size and thickness, environmental exposure, climatic cycles, confinement by the anchorages, and integrity of the remaining building envelope. Note that the stone installer controls very few of these.

### *Cyclical Effects*

Cyclical building dynamics are short-term effects that repeat and reverse causing components to change size. These cycles cause joint widths to expand and contract. Some are column elongation and shortening due to lateral drift; structural frame's torsional twist; spandrel or exterior wall framing deflection caused by wind loads; frame racking caused by seismic events; both internal and external live loads; thermal variations for exposed exterior wall support framing that is outside of the thermal envelope; and construction activity such as equipment and material movements that are different than the comparatively uniform loads of typical occupancy.

All contribute individually, and at times collectively, to change the positional relationships of building and cladding components, and thus the joints.

### *Evolutional Effects*

Evolutional building dynamics are long-term effects, generally single-directional that generally reduce joint widths. Creep or plastic flow tend to decrease column lengths by compression, thereby compressing joints. Tension members experience extension, also compressing joints if the tension member is part of a "hung" curtainwall. Foundation settlement which, if differential, could rack bays into non-rectilinear geometry that might "pinch," or possibly break panel corners if severe.

### Effects That Change Vertical Joint Widths

Climate, occupancy, building age, and environmental loads such as wind and snow also induce influences on the joint's vertical size. The same forces that cause changes in horizontal joint width also change vertical joint width. Dynamic effects are usually exceeded by horizontal joint dimensional change because the static compression components that occur from a young, virtually unloaded structure tend to occur vertically while the skin is being installed. Also, evaluate lateral loads that cause in-plane displacements (racking) to assure that skewed corners do not overstretch or overcompress to tear, the joint filler body or break its adhesion to the substrate.

## SECTION 5
## TESTING USED TO DESIGN STONE AND ITS ANCHORS

Tests establish numerical values for material properties and assembly performance. Designers use these values to match behavior to construction requirements. Values indicate material strengths, variability, and may suggest potential durability.

Stone evaluation involves several types of tests. Historical tests state the properties of past stone. Initial tests measure unit strength of project material by standard methods. Quality assurance tests duplicate initial methods through production to assure consistency.

Special tests are designed to simulate specific project conditions such as anchors and support framing assemblies. Special tests confirm if finished assemblies perform as predicted from previous tests. Testing is an incremental process summarized as follows.

- Factors That Influence Stone and Anchorage Performance
- An Approach to Objectively Evaluate These Influences
- Standards from the American Society for Testing and Materials
- Geological Compositions of Stones
- Properties That Affect Natural Stone Structural Performance
- Tests Sequenced to Quantify Stone System Characteristics
- Interpreting Test Values
- Tests Designed to Evaluate Anchorages
- Tests Designed to Confirm the Capacity of an Assembly

### Factors That Influence Stone and Anchorage Performance

Quantify certain factors that influence the stone's and its anchorages' performance. Evaluate them both prior to selecting the actual anchor type. Identify the stone's minimum structural properties. Re-evaluate material characteristics and the influencing factors during the design process to choose the most appropriate type of anchor for that application, Continue to verify that this preliminary selection of the stone and its anchorage is proper and adequate. Compare this evaluation of the new design with existing buildings. Existing structures are our best laboratory specimens for proving performance. Improving established methods remains the main avenue for raising stone technology.

### An Approach to Objectively Evaluate These Influences

Objective evaluation of the stone itself begins with the philosophy that each stone is an isolated structural element. The boundary conditions, if properly executed, assure this. Without these "givens," neither the loads nor resistances can be determined. These boundary conditions are reviewed in Sections 2, 3, and 4 of the *Guide Specification* for Stone Systems.

Testing proceeds in a sequence that incrementally establishes the capacities and behaviors beginning with the raw stone material itself, and concluding with the finished dimension stone with its engineered anchorages and supporting backup.

#### *The Significance of Exemplars*

Begin the process of engineering stone with a study of exemplars. Review the endurance of similar systems and materials in the same climate. For projects duplicating the construction already proven by its satisfactory performance, few if any tests may be required. A sequence of tests that follow standard ASTM methods establishes fundamental material properties. All test interpretations should include correlation to existing work relevant to the project. Results from panel and anchorage tests are interpreted to imply panel sizes, anchorage design and their layouts, which must be proven by their own separate tests. Results from these tests are then interpreted to imply design of an overall assembly, which must be proven separately by its own tests. To receive meaningful values, design the tests to include considerations that most influence the risk of failure.

#### *General Testing Sequence*

The need for any of these tests depends upon the reliability and performance of existing work and the relative risk of failure. Existing similar materials and systems in similarly challenging environments are the best tests. Some applications do not require tests. Most applications may only require a few, perhaps to identify particular stone strengths or anchorage capacities. Consult an experienced stone designer to outline a program to prove the system that is appropriate to the application. Huge unnecessary expenses and lead-time delays can often be avoided without sacrificing stone cladding system dependability.

#### *Arbitrary Traditional Design Compared to Analysis of Individual Uncertainties*

Traditional stone design lumped these "risks" into an arbitrary safety factors. Current structural design philosophy encourages dissection of that single value into its discrete components of "uncertainty." Load-and-resistance-factor design, as it relates to stone cladding, its anchorages, and support framework is a relevant approach because its process addresses the many variables associated with stone and exterior wall systems.

### *Example Situation*

Paralleling the description of the stone and anchorage design process is an example. It includes testing and interpretations of both the standardized and the special test setups, which were conducted under traditional (specified) ASD-type criteria for the example product.

### Further Research Necessary for Full LRFD Development

It is intended that this groundwork be used to establish a basis for load-and-resistance factor design for stone cladding systems. It should organize design objectives and installation practice that will eventually develop into a standardized method to stone panel, anchorage, and exterior wall support engineering and construction.

*Discussion:*

This example design problem will review support and the anchorage at that support for about a 4 1/2 foot-by-5 foot face size granite panel nominally 1 1/4 inch thick. The application is that the stone panel is a column cover within a mullion-supported curtainwall. Consistent review of these design considerations is presented in a process that could be a model that is followed by a designer to evaluate other exterior wall dimension stone anchorage conditions.

### Standard Methods from The American Society for Testing and Materials

Standard test methods prescribe precisely how information regarding the material can be accurately obtained. The tests themselves do not describe how to establish sampling, nor do they describe how to interpret or apply the test's results. While ASTM has compiled a useful base of information-gathering methods, "when" and "why" they are to be executed remains entirely at the user's discretion. Individual test methods also lack correlation with existing work, unless that work was evaluated by the same test method. Following is a list of dimension stone facade-related standards listed by alpha-numeric designation. The order in which they appear does not imply any significance or relative importance.

- C 97 Test Methods for Absorption and Bulk Specific Gravity of Dimension Stone.
- C 99 Test Method for Modulus of Rupture of Dimension Stone.
- C 119 Terminology Relating to Dimension Stone.
- C 170 Test Method for Compressive Strength of Dimension Stone.
- C 295 Guide for Petrographic Examination of Aggregates for Concrete (not under Committee C-18 administration).
- C 503 Specification for Marble Dimension Stone (Exterior).
- C 568 Specification for Limestone Dimension Stone.
- C 615 Specification for Granite Dimension Stone.
- C 616 Specification for Quartz-Based Dimension Stone.
- C 629 Specification for Slate Dimension Stone.
- C 880 Test Method for Flexural Strength of Dimension Stone.
- C 1201 Test Method for Structural Performance of Exterior Dimension Stone Cladding Systems by Uniform Static Air Pressure Difference.
- C 1242 Guide Specification for the Design, Selection, and Installation of Exterior Dimension Stone Anchors and Anchorage Systems.

### Geological Mineral Compositions of Stones

The mineral consistency and mechanical formation of their crystals or particles control the characteristics of the material. These characteristics determine the construction capabilities of the different types of stones. The characteristics of the individual minerals are equally as influential as how these minerals were combined during their geological formation.

### General Geological Formation

As a mixture of minerals, rock originated from geological activity long ago. The differences in these geological processes is the basis for categorizing the stone types.

### Igneous

Igneous stones were once molten and formed by the relatively slow consolidation of liquid magma. This magma from the earth's core occurs near the surface through extrusive bosses or controlled eruptive activity hundreds of thousands of years ago. The outermost crust cools most quickly, resulting in fine grained stones. Plutonic stone beneath this outer crust cools more slowly, resulting in coarse grained composition wherein the individual minerals can sometimes be discernible. Igneous stones are primarily silicates and are thus classified by their proportion of silicon oxide ($SiO_2$), known as quartz in its mineral form. Their interlocking crystalline construction constitutes a high mechanical strength.

Granites are typically visibly grained igneous stones containing more than 90% free quartz and feldspar, meaning fully formed whole crystals, with some mica, and horneblende or pyroxene. Colors vary from almost white to dark gray and is most recognizable by its almost homogeneous visual texture. A gneissic granite contains foliated layers, a porphyritic granite contains large grains of feldspar within a fine-grain matrix. Dark colored patches are called xenoliths or phenocrysts, which are fragments of earlier-formed rocks caught in the consolidating or extruding magma, usually do not affect the structural integrity of the stone. Their size or concentrations may become a basis for selection during aesthetic evaluation.

Diorites or basalt are sometimes known as very dark granites. Still predominantly siliceous stones, they contain proportionally more pyroxenes, biotites, and horneblendes than granite does, and little quartz. Relatively high content of iron and magnesium results in a darker color.

### *Sedimentary*

Sedimentary stones are cemented sediments once formed in waterbeds. Their deposits are primarily ancient sediments from crustaceous sea creatures or their inorganic remains. Precipitous runoff carries fines from weathering or erosion of other rocks from the land into a sea, lake, or river bed. They are compacted, as well as, to a varying degree, chemically cemented together or welded by compaction to form a mass. The transporting water, in its changing volumes and velocities, stratifies these contents into equally varying layers of similar-density particles. Continuing over literally thousands of years, the accumulating layers increase in weight and overburden, which compact the layers beneath. They vary widely in appearance, texture, and structure because of this origin.

Sedimentary stones are primarily carbonates of lime ($CaCO_3$), or calcium carbonate (mineral calcite) where the primary deposits have been from sea life remains. Infiltrations, or layers of "foreign" minerals give these stones their colors. Carbonates of magnesia, carbon, iron, silicas, mica, talc, horneblende, and pyroxenes are laid into stratified beds either as chemical deposits or consolidations. Where the deposits are from water flows and erosion, the contents will likely be silicates such as sandstone.

Limestones of the purest variety are nearly pure calcium carbonate and are almost white in color. Other limestones include small amounts of magnesium carbonate. Textures vary from very fine to rough and fossiliferous. Some limestones can be polished. The amount of "foreign" ingredients in the stone increases the color. The presence of these substances result in many colors. Yellows, pinks, and reds include iron oxides. Blues, grays, and blacks include carbonaceous derivatives of organic matter. Greens may be due to talc. Consider the potential chemical activity of these other ingredients when determining the compatibility of interfacing materials.

Dolomites are carbonates consisting predominantly of magnesium carbonate (MgCO) and range in color from almost white to yellowish in color. Dolomites are distinguished from other limestones also because they are more crystalline and granular. They are generally harder, more dense, and less soluble than limestone. Except for the proportions of primary carbonates, there is essentially no compositional difference between limestone, dolomite, and marble. Where limestone is distinctly sedimentary and marble is fully "metamorphosed," dolomite's crystalline structure is sometimes due to partial metamorphosis.

Travertines are porous, or cellularly layered, partly crystalline limestones of chemical origin precipitated from generally hot solutions of carbonated spring water, usually at the bottom of shallow pools. The open voids in the stone are the channels left by the water once its flow ceased or it evaporated. Because of its formation by directional flow, travertines typically demonstrate a strong directional veined appearance and corresponding directional strength variations.

Sandstone is a fragmental stone, composed of rounded or angular grains of sand (predominantly quartz, between 0.06 and 0.2 mm) cemented and compacted together to form a solid mass. Sandstone's toughness is due to the great pressure during consolidation. Its variety of colors is due to the mixture of minerals. The character of the cementing matter more than the grains themselves determine the appearance and durability of the stone. If silica, it is siliceous (light color, hardest and toughest to work). If carbonate of lime, it is calcareous (light gray color, easily worked, and susceptible due to solubility of lime). If iron oxide, it is ferroginous (reddish-brown color, also readily worked). If clay-ey matter, it is argillaceous (brown colors, absorptive, and susceptible to frost injury)

### *Metamorphic*

Metamorphic stones are the crystallization, or recrystallization of pre-existing stone by elevated temperatures, pressures, or both. Confined beneath the earth's surface, constituent materials of the original stone are re-arranged mechanically and chemically to develop new mineral structures. Simple consolidation that occurs with sedimentary deposits can become metamorphosed when the mineral structure, not just density, occurs. Heat from an igneous intrusion can result in thermal or contact metamorphism. The surface crust's movements during regional "mountainbuilding" results in deformations and pressures, which can result in metamorphism.

Marble is a calcareous metamorphic stone of sedimentary origin, composed originally and essentially of limestone. Its varieties in texture, colors, and structural characteristics vary almost endlessly due to its volatile formation. Thermal "baking under great pressure" recrystallizes the calcium carbonate of the limestone into the calcite of marble. Pure limestone would become pure white marble. The presence of the "foreign" impurities deposited with the limestone transform into the figuring and veining by chemical reactions. It is this veining and figuring that gives marble genuine beauty and alluringly unique appeal. The varieties of composition and formation also make marbles' structural properties as variable as their appearances.

Slate is an argillaceous metamorphic stone of sedimentary origin composed of microcrystalline fragments of indurated siliceous clay. Once originated as fine silts in sea bottoms, slate is most commonly derived from shale and quartz. Regional upheavals created pressures that re-oriented the micaceous crystals uniformly perpendicular to the stress, resulting in cleavage planes parallel to these crystal boundaries. This re-orientation is independent of the original bedding layers. Different colored layers will result in banding through the slate at random angles to the cleavage planes. Individual layers vary in texture and color caused by changing contents and turbulences of the tributary flows. These ribbons represent original bedding lines.

Quartzite is a siliceous metamorphic stone of sedimentary origin, composed originally and essentially of sandstone. It contains over 95% free silica and fractures in shell-like faces through the grains. Sand particles are recrystallized into an interlocking mass, resulting in a hard, strong, durable stone.

Serpentine is a siliceous metamorphic stone of igneous origin, composed originally and essentially of hydrated magnesium silicate. Veined with calcite or dolomite, it is usually green to greenish-black. Not a "true" geologic marble, its strength, chemical integrity and durability is among the highest of any natural stone.

Schist is a siliceous metamorphic stone of igneous origin, composed originally and essentially of mica. Foliated and laminated buildup of these mica flakes in discontinuous and uneven thicknesses give this abundantly produced and highly desired stone a pearllike reflectivity and sparkle. This is often marketed and produced as granite.

### *Mineral Structure*

Stone is a mineral aggregate and part of the earth's crust. The quantity of mineral species constituting any essential portion of a stone used in structural purposes is rarely over three to four. The architectural desirability of natural stones is dependent upon the mixture of the mineral ingredients. The arrangement of minerals within a stone are divided into four classes—essential, accessory, original, and secondary.

### *Essential Minerals*

An essential mineral in an aggregate forms the chief constituent of the stone, as quartz does for granite, and calcium carbonate does for limestone.

### *Accessory Minerals*

An accessory mineral in an aggregate is usually present, but for a minor, usually aesthetic or visually characterizing importance, as mica or horneblende is for granite. A characterizing accessory is the dominant of these minor minerals that gives the stone its name.

### *Original Minerals*

An original mineral in an aggregate was formed with the stone upon its first consolidation. Being an original mineral does not necessarily make the mineral an essential one.

### *Secondary Minerals*

A secondary mineral in an aggregate was formed from subsequent changes in the stone after its first consolidation, whether from chemical action, percolation from water, or other infiltration.

### *Mineral Compositions*

Individual minerals composing the aggregate within a stone, with the nature of their interfacing bonds, establish the visual characteristics of the stone. Primary minerals in dimension stone used for exterior applications include quartz, feldspar, mica, horneblende, calcite, dolomite, olivine, garnet, and pyrite.

### *Quartz*

Quartz is pure silica ($SiO_2$) having a hardness of 7 and is an essential component of granite, gneiss, mica schist, sandstone, and quartzite. Quartz is clear and colorless mineral which fractures and scratches like glass (silica is the essential ingredient to glass, also), is brittle and insoluble in acids. Microscopic fluidal cavities are believed to cause extreme scaling under intense heat such as flaming.

### *Feldspar*

Feldspar is silica with traces of alumina, potash, soda and lime having a hardness of 5 to 7 it is an essential component of granite and gneiss. Feldspar's toughness can exceed quartz because of its directional cleavage, and can add difficulty to polishing. Feldspar's weatherability relates directly to its contents of impurities, cavities, and cleavage flaws, which can be predicted by its porosity. Its color is dependent upon this purity also, for stone having hard, clear, transparent feldspathic content will absorb incident light and appear dark, where stone having softer, porous, and impure feldspathic content will reflect incident light and appear light.

### *Mica*

Mica is a complex composition of silicates of iron, alumina, magnesia, and potash having a hardness of 2.5 to 3 and is an accessory component of granite and gneiss. Feldspar's softness can greatly impact a stones fitness and weatherability depending upon its relative content in the aggregate. Appearing as small shining scales within the stone, mica is soft and fossil with an element of weakness, which also does not take or hold a polish well with exposure. If mica is prevalent and the folia lie in parallel layers, the stone will split relatively easily along these lamina and demonstrate directional strength characteristics relative to these layers.

### *Horneblende*

Horneblende is a silicate varying in alumina content having a hardness of 5 to 6. The nonaluminous type is a secondary component of crystalline metamorphic stones whose crystals are not easily separated, and are recognizable by their white, gray, or pale green color. Aluminous horneblende occurs primarily in igneous granite, gneiss, and diorite. Able to aquire an excellent and durable polish, its presence in stone is preferable.

### *Calcite*

Calcite is calcareous ($CaCO_3$) of 44% carbon dioxide and 56% lime having a hardness of 3 to 4 and is an essential, and original constituent of limestones, true marbles, and travertines. Pure calcite is pure white.

### *Dolomite*

Dolmite is calcareous ($CaMgCO_3$) calcium carbonate with magnesium carbonate having a hardness of 3 to 4 and is an essential and original constituent of partially metamorphosed limestones or "unpure" limestones, and occurs as compact, crystalline massive forms.

### *Olivine*

Olivine is a silicate of iron and magnesium having a hardness of 6 to 7 and is an essential constituent of basalt and other igneous stones.

### *Garnet*

Garnet is a silicate of alumina, iron and magnesium having a hardness of 6.5 to 7.5 and is an accessory constituent of granites, gneisses, and crystalline limestones.

### *Pyrite*

Pyrite is an iron disulphide ($FeS_2$) having a hardness of 6 to 6.5 and is a common accessory constituent of some limestones and dolomites, appearing as brassy yellow cubes. Because of its liability from staining and oxidation, pyritic stones should not be used where exposure to weather and moisture is possible.

## Physical Properties of Stone As an Aggregate of the Composing Minerals

Actual physical properties of a composite stone result from the manner in which the various constituent minerals are mixed or bonded together within the aggregate. Properties of the aggregate also depend upon the proportions of the different minerals that are mixed and the individual minerals' properties. This bonding of the different separate minerals becomes even more important than the properties of the individual minerals themselves in determining the eventual physical properties of the aggregate stone.

### *Density*

Density is quantified as a stone's weight per cubic foot as measured by ASTM C 97 and can be related to strength and absorptivity. This difference in density could be extremely slight, perhaps ranging between 155 $lbs/ft^3$ and 172 $lbs/ft^3$ in comparison to water, which is 62.4 $lbs/ft^3$ at sea level. With stones composed of the same mineral content, the densest stone is usually the heaviest, strongest, and least absorptive.

### *Hardness*

Hardness is quantified as a component's hardness or their state of aggregation in relative terms by the Brinnell Test, which compares the individual stone or its mineral to a diamond, which is 10. However hard a stone's individual minerals are, the stone will seem "soft" if the mineral particles adhere with only "slight ferocity." The measured hardness of the aggregate stone will be limited by the weakest of either the hardness of the individual mineral component or the particular bond between them.

### *Structure*

Structure is quantified by the aggregate's form, size, and arrangement of its component materials, and can be determined by visual inspection.

Macroscopic structure is distinguishable with the naked eye. The observer can characterize a stone as being granular, stratified, massive, foliated, porphyritic, or otherwise.

Microscopic structure requires aided, instrumental inspection to discern the aggregate's construction as either fine-grained or compact.

Crystalline characterizes tightly fitted structure where the individual fully formed crystals of the constituents are discernible, such as in granite, crystalline limestone, and true marbles.

Vitreous characterizes structure where the individual constituents are glassy and sometimes indiscernible, such as in stones like obsidian.

Fragmental characterizes structure where there are individual particles of one constituent cemented or welded together by another, with some degree of void between, such as in sandstone.

## Properties That Affect Natural Stone Structural Performance:

Different types of natural stones possess different degrees of heterogeneity, nonisotropicity, nonlinear elasticity, rift, and moisture susceptibility, which are all important variables to discriminate between during testing. Even with the same type of stone or same colors and grain from the same quarry, different degrees of these property variances exist. These properties are direct indicators of stone material's potential durability and strength if considered and designed for in a responsible manner.

### *Heterogeneous*

The composition of natural stone is an uneven mixture of many mineral and sometimes fossil ingredients. Each of these different geologic ingredients have different properties themselves, as do their boundaries where they interface other ingredients.

### *Nonisotropic*

Natural stone's composition differs along its length, width and throughout its mass resulting in location-dependent physical properties. Veins or rifts, are a predominant direction of crystal "flow" resulting in direction-dependent physical properties. How the different mineral ingredients are mixed cause the differences in properties due to being nonisotropic.

### *Brittle*

Natural stone's failure is characterized by a sudden rupture. Again, somewhat similar to unreinforced concrete, as unit stress increases in a generally convex curve, without change in curvature or apparent disproportionate increase in strain to stress, failure occurs. Because of the heterogeneity of the stone material with the varying elasticities, our ability to predict when rupture might occur also varies.

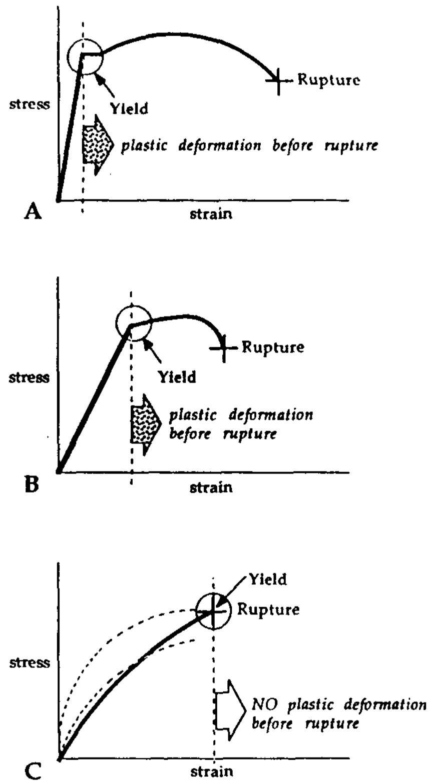



FIGURE 32: *Stress-Strain Behavior Of Natural Stone Compared To Other Common Facade Structural Materials.*

**Curve A is common mild structural steel with a modulus of elasticity of 29,000,000 psi. After a linear stress-strain relationship to its yield point, a considerable reserve of plastic (bending) and even strain-hardening behavior exists before mild steel ruptures. Bending before breaking is a potential visual warning that steel offers before it reaches catastrophic failure due to rupture.**

**Curve B is structural grade aluminum such as alloys 6063-T5 or 6061-T6 with moduli of elasticities of 10,100,000 psi. After a linear stress-strain relationship to its yield point, a short reserve of plastic (bending) exists before aluminum ruptures. Again, bending before breaking is a potential visual warning that aluminum offers before it reaches catastrophic failure due to rupture.**

**Curve(s) C represents natural stone used for building cladding, each stone with variable moduli of elasticity ranging from perhaps 1,700,000 to 6,500,000 psi depending upon the method used to calculate it and the type of stone. The metals above are homogenous and isotropic and behave almost identically in any direction regardless of the stress state. Being heterogeneous and nonisotropic, stone behaves differently in directions under different stresses. Behavior can vary within the same stone sample. Most important though, stone is not plastic beyond yield. Stone ruptures suddenly at failure without visual warning of overstress prior to it breaking. Stresses must be kept below the threshold capacity in the weakest direction and in the weakest stress state.**

### *Nonlinearly Elastic*

Natural stone's stress-strain relationship is slightly unproportional. Somewhat similar to unreinforced concrete, as unit stress increases, strain increases disproportionately more until rupture. Unlike concrete though, where the ingredient providing bonding is only Portland cement, with a relatively consistent strength, bond between the minerals in stone varies with the minerals that compose the stone and the pre-existing degree of microcracking. Because different mineral ingredients fracture differently due to their compositions and crystal sizes, the plot of the modulus of elasticity can actually "jump" or "step." Heterogeneous mineral mixture also causes different curves for the same stone material.

Stress-strain behaviors of natural stone compared with other primary exterior wall structural materials reveals the nonlinear elasticity and brittle failure (Fig. 32). Because stone is heterogeneous and thus its characteristics are both moisture- and direction-dependent, the elasticity curve will have different slopes for the same stone in these different conditions, thus yielding a variable modulus.

### *Moisture*

The presence of moisture alters stone's behavior, which results in wetness-dependent physical properties. These changes in properties are independent of freeze-thaw or corrosion and are simply strength differences that occur when the stone material absorbs moisture or vapor.

### *Finish*

Representing the project's stone finish on testing samples, whether the designed finish is flamed, polished, honed, water jet, split, or sawn and discriminating whether that finish is on the tension or compression face during testing is pertinent for some measurements of some material strengths. Correlation of strength values between the in-place finish condition and that surface's orientation to the stress should be consistent throughout the testing.

### *Sample Size*

Representing the project's stone thickness module by testing samples of the same thickness of the project's proposed panel is pertinent for some critical test values. Sample width may influence comparison between sample and panel depending upon grain size and rift relative to thickness. The wider the sample for ASTM C 880 tests, for example, the closer to the panel behavior may be experienced, up to a certain point. In that situation, it is also less likely that local imperfections in the stone will control its failure and test value.

## Tests Sequenced to Quantify Stone-Clad Wall Systems Performance

Dimension stone is a natural stone product used in building construction, which in its completed state, is fabricated to a specific size, thickness, and finished face as specified, usually for a particular project. The natural material maintains the same indigenous and varying physical characteristics that it did in the ground when it becomes a construction material. In fact, the extraction of in-situ deposits and then the transforming of those rock blocks into thin slabs commonly magnifies the material variabilities that result in inconsistent structural behaviors. It is those imperfections formed during the earth's "un-engineered" and "un-controlled" processes that give natural stone its beauty and its challenge to the designer.

### *Variable Behavior*

Unlike aluminum, glass, steel, concrete, rubber, and other familiar building materials that are products of refined mineral or material recipes combined under controlled processes to yield a distinctly predictable physical behavior, natural stone, even within the same quarried block, was created with varying "recipes" of minerals that include "impurities" under unknown, uncontrolled processes that yield nonisotropic and heterogeneous materials that demonstrate comparatively irregular and unpredictable structural and weatherability behaviors.

How Variability and Consistency Affect Margin-of-Safety According to Irwin Miller and John Freund in *Probability and Statistics for Engineers*, for any particular parameter or characteristic, the wider the distribution of the test values, the greater the variability, and the higher the probability for getting values further away from the mean. For instance, in evaluating the probability related to material strength, if the required margin-of-safety is, say 1%, meaning that there is 1% probability that the material is understrength, and the distribution is assumed to be normal, then the "exclusion value" corresponding to that 1% probability is 2.33 standard deviations less than the mean, or average strength. For the normal distribution, other random variables corresponding to probabilities are:

| RANDOM VARIABLE $z$ | | EXCLUSION PROBABILITY |
|---|---|---|
| 0 | for | 50% |
| 0.52 | for | 30% |
| 0.68 | for | 20% |
| 1.28 | for | 10% |
| 1.65 | for | 5% |
| 2.06 | for | 2% |

as represented by the equation:

*Exclusion Value =*
*Average Strenghth - (z Times the Standard Deviation)*

TABLE 3. ***Normal distribution probability.***

Conversely, to maintain that same margin-of-safety, the difference between the mean and the exclusion value decreases as the variability decreases, meaning that there will be a higher strength value representing the exclusion value for that same reliability. Consequently, lower variability increases the usable strength relative to the average strength values reported by the test methods.

This means, that for two materials having equal averages, or mean strengths, but different variabilities (standard deviations), in order to attain equal reliability, the stone having the larger variability will have a lower design strength value representing that exclusion value, which gives it the statistical reliability (Fig. 33).

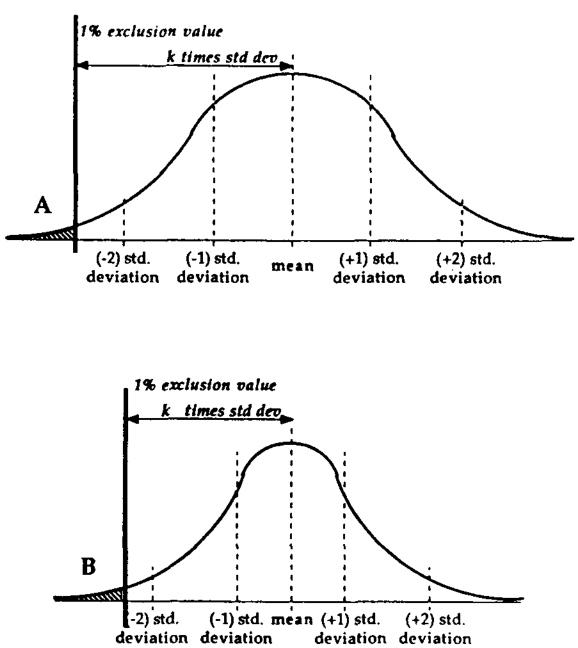



FIGURE 33: ***Material Strength Variability Influence On Margin-Of-Safety.*** **If a normal distribution is assumed to represent stone material strength, a statistical "exclusion value" can be determined that corresponds to a certain risk-of-failure. Correlating an exclusion value with an acceptable risk corresponds to a certain standard deviation, which calculates a certain minimum strength. Where risk and exclusion value are determined by general engineering practice, the corresponding minimum strength value changes with stone material average strengths and variability. Remember, the stone material strength itself is only one of many factors that influence cladding performance.**

**Curve A represents a wider distribution of tested strengths than curve B. It shows a higher variability, which increases the standard deviation, and thus lowers the exclusion (or minimum) value for less consistent material.**

**Curve B represents a narrower distribution of tested strengths than curve A. It shows a lower variability, which decreases the standard deviation, and thus raises the exclusion (or minimum) value for a more consistent material.**

### *Process for Evaluation*

Conduct tests by standardized test methods to measure physical strength properties. These quantify a material's behavior under stress. Physical strength and consistency are most important to the designer evaluating that material's permanence, durability, and safety. Use these tests to determine the stone material's minimum expected structural performance. The following test sequence is not appropriate for all projects. Where successful exemplars already exist and the project's applications do not challenge the known capacities of the stone material or retention system, only reference to previous testing may be necessary.

### *Historical Tests*

Tests of formerly produced material from the same quarry may be referenced at the inception of the stone selection process. Use them preliminarily to compare general stone types for their suitability and strength for the design. Consider how important these characteristics are for the particular project application. They may not all be pertinent.

If initial strength and consistency of strength is critical to the project, first review historical tests. These tests should be actual records of stone material extracted from the same quarry. They should be the most recent possible and best also to be from material taken from the same general area of the quarry. Values from these tests can be used for structural analysis for non-challenging applications where stone stress is not a limiting factor. They are helpful in establishing optimum panel sizes and thicknesses necessary in the preliminary architectural facade design. These tests may also indicate the propensity for the stone to absorb moisture, be affected by moisture, or may indicate its directional dependency, which are pertinent considerations for durability and retention system conceptualization. Where panel or anchor capacities are structurally significant, more tests may be required.

While historical or published strength values might be useful during conceptualization of the stone support design, they cannot be considered acceptable data for engineering projects of major size. However, most stone applications are not sized by structural limitations. Where existing structures using similar materials and support systems have performed well, and the material and systems are not structurally challenged, published strength values are satisfactory for engineering design.

### *Initial Tests*

Initial tests evaluate the new stone material suggested for use by the architect. Consider the guidelines discussed under Historical Tests to determine which tests are appropriate for a prospective project. Where required, complete these tests before beginning actual anchorage design. This enables the designer to use these tests to discover and define the basic strength values and their variability, which can be critical for panel flexural or anchor engagement capacity. These can be quantified with standard test methods established by ASTM for dimension stone as presented under Standard Methods from the American Society for Testing and Materials

When the project's use of the stone will be determined by its structural capability, initial testing is the basis for primary decisions regarding stone material structural performance. This initial body of tests should be comprehensive enough to statistically establish the variability of the material's strength properties. Derive average values and their statistical variance from these initial tests. Use them to mathematically engineer anchorages and stone panels. Employ later tests to confirm these early engineering presumptions by evaluating individual anchorage devices, full panels, and finally the complete assemblies. If the cladding design is not strength dependent, and sufficient satisfactory similar examples exist, these tests are likely to be unnecessary.

### *Quality Assurance Tests*

Quality assurance tests verify the material's strength consistency through production for the project. They are justifiable only where projects are of large size or monumental significance. Upon slabbing for production of the stone for the project, intermittently repeat-test small samplings using the same methods as the initial tests only for the critical strength values of the design. If panel flexure is the limiting (critical) criteria that controls the capacity, repeat the C 880 tests. If anchorage rupture is the limiting (critical) criteria that controls the capacity, repeat the C 99 tests. Compare values with those used in design. Quality assurance frequency depends upon the material variability and how critical the stress level is or how stone material strength relates to the cladding application.

The intent of this abbreviated program is to assure that the strength of the continuing production stone material conforms to the engineering values determined from the originally tested stone. Should basic strengths or variability change from the safe useful range established during the initial tests' conclusions, adjustments in anchorages or panels from that area of the quarry that the quality assurance samples came from may be necessary. Should lower strengths result that are not included within the statistical variances derived from the initial testing (in other words, if their risk is outside the established "envelope"), prior to installation, revised panel fabrication, revised anchorages, or perhaps simply delegation of that material to lower-stressed panels or facade regions may adapt the material to suitable use within the project.

### *Project-Representative Material*

For the stone testing prescribed for the project, obtain samples from different perimeter and core locations of separate slabs and different blocks extracted from the certified portion of the quarry that will supply the production stock for the project. Tests from the same material quarried and slabbed for the project, not other material, are required to confirm the stone material's structural properties' consistency.

## Test Value Interpretation

Because stone is a natural material, its structural properties vary between different stone types, between different colors in the same stone type, and throughout the medium of the same type and same color and even same bed of the same quarry. The magnitudes of these properties and the range of their variability can be quantified with standard test methods established by ASTM Committee C-18 on Dimension Stone. These established standard testing methods measure the physical properties of stone in unit-values that can be used as initial comparison criteria for evaluating stone strength. The published standard test method details the procedure, but does not interpret their values.

*Commentary:* Granite is the material preferred by the architect for this design example. A light, gray-tan is the color best fitting the exterior wall palate. Independent of cost, which is directly related to quarrying, extraction, and consistency of the quarry's horizons, initial strength and durability evaluations can be ascertained with the ASTM standard test methods. Duplicating the same method on successive samples will indicate the range of the natural material's structural properties' variability. Stones to be considered for this example because of their aesthetic qualities and domestic accessibility will be Nevada beige, Bismark pearl, and Texas pink.

### *C 97 Standard Test Methods for Absorption and Bulk Specific Gravity of Dimension Stone*

*Commentary:* C 97 results can be good preliminary indicators of the stone's durability. Even while differences in density (bulk specific gravity) are relatively small, a higher density could indicate less microscopic voids, faults, and perhaps a more intact crystalline structure. More indicative of a stone's potential resistance to weatherability, however.

Stones with higher absorptivity are composed of geologically crystalline structures with microcracks that conduct moisture into its medium. This porosity will not only allow (or "pull" due to capillary tension) moisture to pass into the stone and into the wall, or of more concern, to the anchorages, but it will also subject the stone to higher potential degradation. A higher absorptivity increases the presence of moisture, which can facilitate corrosion, chemical deterioration, and especially freeze-thaw weathering. It is important to remember that acids and other pollutants suspended in the atmosphere are absorbed with moisture into the stone, which can aggravate and accelerate the stone's and its anchorages' degradation. Stones with lower absorptivity usually perform better in extended freeze-thaw exposure and chemical degradation.

If the project's stone finish is to be thermal or flamed, which literally explodes the stone's surface, or even high-pressure water jet, which also violently fractures the surface skin of the panel, microcracking between and within the stone crystals is greatly proliferated. Thus the affects of moisture increase likewise due to increased absorption from capillary tension, which further propagates these cracks.

Considering absorption for the three material options:
Nevada Beige is least absorptive at 0.104% (SD=0.034)
Bismark Pearl is more absorptive at 0.133% (SD=0.034)
Texas Pink is most absorptive at 0.160% (SD=0.085)

TABLE 4. *Comparison of absorption.*

Considering bulk specific gravity for the three material options:
Nevada Beige is densest at 168.5 lb/ft$^3$ (SD=2.30)
Bismark Pearl is less dense at 167.1 lb/ft$^3$ (SD= 0.74)
Texas Pink is least dense at 166.4 lbs/ft$^3$ (SD= 1.65)

TABLE 5. *Comparison of bulk specific gravity.*

### *C 170 Test Method for Compressive Strength of Dimension Stone*

This standard measures the stone's crushing strength.

*Commentary:* Because compression strength is rarely a limiting factor in the design of dimension stone in modern curtainwall cladding applications. This test's results do not usually affect panel or anchorage engineering. In bearing wall construction, stone's high compressive strength was necessary at the wall's base and foundation to support the cumulative weight of the masonry wall above.

### *C 99 Test Method for Modulus of Rupture of Dimension Stone*

This standard measures the stone's combined shear strength with diagonal tension strength, which is most applicable in predicting the stone's capacities at its anchorage's engagements into the stone.

Considering modulus of rupture for the three material options (sawn finish):

Nevada Beige is strongest at 2300 $lbs/in^2$ (SD= 165)
Bismark Pearl is less strong at 1664 $lbs/in^2$ (SD= 145)
Texas Pink is least strong at 1570 $lbs/in^2$ (SD= 233)

TABLE 6. ***Comparison of C99 Modulus of Rupture.***

*Commentary:* Modulus of rupture is a ratio of force-to-unit area where breakage occurs, and since stone is brittle and nonplastic, it fails not by yielding with any measurable "flow," but by breaking apart, or rupturing. Because of the thick "brick" shape and short proportion between supports with the C 99 sample and the concentrated load applied at the middle of the span, this modulus of rupture procedure fails the samples in primarily diagonal tension stress with a considerable shear component.

As compared with a C 880 test, where no shear exists in the middle of the span, the stress mechanics measured in this procedure are similar to those created locally at most anchorages' surfaces-of-influence, or "cone-of-failure" where considerable shear exists also. While there is some tension component involved, as there is in any "prying" phenomenon, which typically causes anchorage failure, there is a considerable shear component that combines with flexural tension to create a diagonal shear-tension failure plane. The short, deep beam shape of the C 99 stone sample best quantifies the ultimate unit stresses at rupture, which are then achievable at the engaged stone where the anchorage effectively contacts the stone. These values are helpful in preliminarily estimating anchorage sizes and capacities prior to the testing of actual anchors.

Because anchorages' influence surfaces are limited in size, their vulnerability to material strength inconsistencies is greater. Serious consideration should also be offered in recognizing that the actual area and the aspect ratio of the C 99 stone sample's surface-of-influence (failure plane) is quite large in comparison with the failure plane of most kerfs or pins. Thus there exists a greater potential for disguising material flaws or inconsistencies that can occur locally at an anchor if a designer depends upon the strength values resulting from the ASTM C 99 modulus of rupture test method. It is thus better to not underestimate, but instead overestimate the variability shown by the test samples when the consideration for variability and the engineering judgements are applied to predicting the individual stone anchorage capacities prior to their actual testing. Particular deliberation must be afforded toward the extent of variability that occurs in C 99 modulus of rupture tests because of the catastrophic nature of the shear-rupture type failure, especially when occuring at an anchorage. Also, because of the predominant shear component when reaching the stress that results in failure, C 99 strength values range from 20 to 35 % higher than C 880 flexural test values on the same material.

### *C 880 Standard Test Method For Flexural Strength of Dimension Stone*

This standard measures the stone's primarily tension strength by bending, which is most applicable in predicting the capacities of the stone panel itself between the anchorages, unless the stone is more "cubic" in configuration with closely located and secure supports, which might be modeled better by the C 99 procedure.

| |
|---|
| Considering flexural strength for the three material options (sawn finish): |
| Nevada Beige is strongest at 1961 lbs/in$^2$ (SD= 168) |
| Bismark Pearl is less strong at 1327 lbs/in$^2$ (SD= 117) |
| Texas Pink is least strong at 1282 lbs/in$^2$ (SD= 146) |

TABLE 7. *Comparison of C880 Flexural Strength.*

*Commentary:* Flexural strength is the unit stress capacity of the stone material while in bending at the point of breakage. Because of the thin "slab" shape of the C 880 sample and because the loads are applied at the third-points in the span, this flexural strength procedure creates a purely flexural tension-stress failure that is extreme at the tension face of the slab. The stress mechanics measured in this procedure are similar to those created regionally in the stone panel midspans between the anchorages. As the bending moment approaches maximum while the shear diminishes in a panel between its anchorages, the stone sample's middle third of its span is theoretically without shear and is purely in flexure. Given that stone is weaker in tension than in compression, the extreme fibers in tension fracture at their ultimate tensile strength. Removing the shear from this considerable "region" of the sample helps minimize overemphasizing the influence of rift, directional variability, or other stone imperfections.

Care must also be exercised to recognize that the surface area of the C 880 sample's failure surface plane may be quite small in comparison with the flexural-stress-failure plane that would exist for a full panel. Thus, opposite to the potential created by the C 99 sample and its relationship with applying values to compute anchorage failure capacities, in the C 880 flexural strength's method there exists a greater potential for exaggerating material flaws or inconsistencies that might occur in a full panel. It might then be appropriate to not overestimate, but instead underestimate the variability shown by the range of test samples when considering the influence of variability and the engineering judgements are applied to predicting the individual stone panel's flexural capacities prior to their actual testing.

### *Stress-State Comparisons*

Stress-state comparisons show that the mechanism that causes failure in the C 99 modulus of rupture method, because of its short-thick sample proportions and the concentrated center load over the short span, combines flexural and shear stresses. The mechanism that causes failure in the C 880 flexural strength method, because of its long, thin sample proportions and the quarter-point loads over the long span, isolates flexural stresses (Fig. 34).

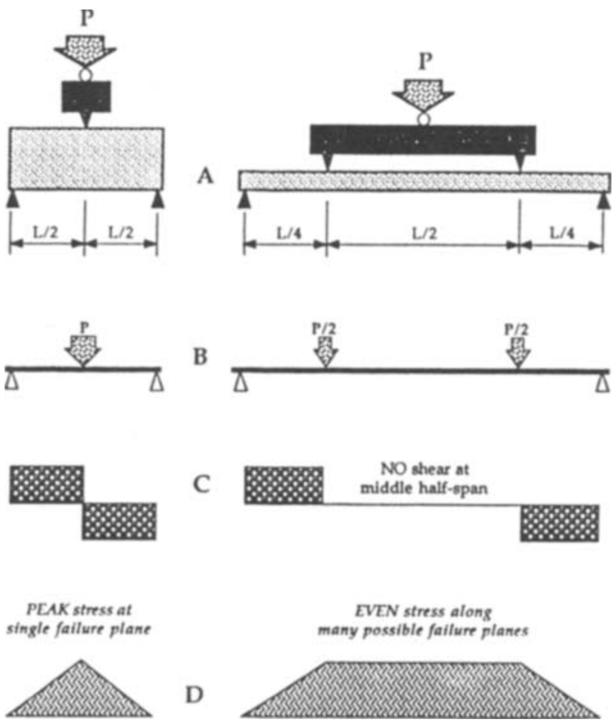



FIGURE 34: ***Stress-State Comparisons Between ASTM C99 and C880 Standard Test Methods.*** **ASTM C99 Modulus of Rupture and C880 Flexural Strength Standard Test Methods are frequently incorrectly used interchangeably to evaluate stone material strengths. Each evaluates similar stress states but in a completely different way rendering different results. The left column of diagrams represent the C99 method, the right column of diagrams represent the C880 method: *Diagram A* shows the test apparatus and the relative sample sizes. *Diagram B* shows the loading pattern on the samples. *Diagram C* plots the resulting shear diagram. *Diagram D* plots the resulting bending moment diagram. C99 tests a thick, brick-sized sample at a single theoretical failure plane which experiences combined shear and tension on the bottom face. C880 tests a thin slab strip-type sample through an entire region of an infinite number of theoretical failure planes which experience only tension on the bottom face.**

*Commentary:* In comparison to the C 99 modulus of rupture test results, the C 880 flexural strength results are typically 15 to 35% lower. Because a larger region of the material is at maximum-stress, more "opportunity," or potential exists for a natural weakness or feature to occur in that region when tested by the C 880 flexural strength method. Perhaps also because of the thinner sample, a greater opportunity exists for exerting the maximum stress at stone's surface imperfections in various locations, or be influenced by the aggregate mineral structures that would have to be located directly in the middle of the sample where the flexural stress is maximum to cause failure in a modulus of rupture C 99 method at a load equal to the C 880 capacity.

While the influence of a strategically located middle-of-the-sample flaw may untruly discount a C 99 test value, depending upon how that value is used within the engineering, its conspicuous absence from the failure plane in any samples might also render the modulus value to be unconservative. A greater variability might be expected for the C 99 modulus of rupture in a set of samples prepared at random orientations with the composition of the stone, because the region of highest stress in the sample is so small, thus only a small area of the material is actually being tested.

### *Modifications to Standardized Test Methods of Dimension Stone*

May be justified if the characteristics of the ASTM specified test samples may not offer confident correlation with the characteristics of the project's stone. Because the engineering analysis judgements are founded upon the conclusions of testing, the samples should represent the project material as closely as possible.

Any modifications to standard methods to facilitate project-specific conditions must be described within the test report. For instance, the specified C 880 sample is often modified in sample width, thickness, or finish to duplicate stone thickness and finish to better-represent project conditions. Sample thickness and width can influence test results depending upon the crystalline structure and crystalline size of the stone. Any sample characteristics that deviate from the specified standard must be noted in the test report documentation.

The conclusions from tests that include these deviations are not recommended to be directly correlated to other tests that either exactly follow the standard method, or deviate in some other aspect.

| |
|---|
| Considering flexural strength for the three material options (thermal finish): |
| Nevada Beige is strongest at 1772 lbs/in$^2$ (SD= 214) |
| Bismark Pearl is less strong at 1251 lbs/in$^2$ (SD= 102) |
| Texas Pink is least strong at 1223 lbs/in$^2$ (SD= 179) |

TABLE 8. ***Initial Conclusions from C880 Flexural Strength Results***

| |
|---|
| Considering decrease in flexural strength and increased variability from sawn to thermal surfaces for three options: |
| Nevada Beige decreased 189 lbs/in$^2$= 10%;(SD increased 27%) |
| Bismark Pearl decreased 76 lbs/in$^2$= 6%;(SD dev decreased 13%) |
| Texas Pink decreased 59 lbs/in$^2$= 5%;(SD increased 23%) |

TABLE 9. ***Initial Conclusions of C880 Flexural Strength Variability.***

*Commentary:* The stone surface finish for the project is requested to be thermal (or flamed), which is different from the sawn (or nominal) surface designated in the ASTM specification for stone sample specimens. The C 99 modulus of rupture procedure measures primarily shear, which is essentially resisted by the medium through the thickness of the "brick" shape, not the surface "fibers." Thus the rupture strength is insignificantly affected by the stone's surface finish.

The C 880 flexural strength procedure measures tensile strength induced by bending, which is highly affected by the stone's surface condition on the tension face. Thus, since the project's finish is thermal, which tends to increase surface microcracking—which separates crystals and partially eliminates molecular bond and then results in reduced capacity in tension—it is prudent to "modify" the prescribed test sample to represent the project's stone finish in order to as closely-as-possible correlate the test to the actual project conditions. Orient the thermal surface down, on the tension side (load is applied to the top). Rift direction (or bedding) can sometimes affect the directional strength of the material. While preparing and orienting test samples to be parallel or perpendicular to the rift is typically not a modification to the standard test method, groups of both are recommended to be included in the specified test group.

Besides representing actual project thicknesses in ASTM C 880 samples, their widths can also be important. Stones with large crystalline structures whose perimeters approach or exceed the slab thickness sometimes show tendencies to break at values premature of those attained in full panels unless the sample width is increased above four-to-six times the thickness. Thus for better sample-test strength-to-panel-strength correlation, wider samples are sometimes recommended (perhaps 6 or 8 inch widths for 1 inch (about 2.5 cm) or 1 1/4 inch (about 3 cm) project panel thickness modules respectively).

## Non Standard Unit-Evaluation Tests

Other tests, including modulus of elasticity, bowing tendency, resistance to chemical deterioration, weathering, or freeze-thaw cycling, dimensional stability and their effects on material strength properties can be determined by tests designed to evaluate these specific characteristics, which are in presently in development in Committee C-18. All are contemplating correlation methods with examples of work already existing in the field. Since the idea of tests is to evaluate performance in a simulated exposure, the results of the test should have realistic relationships with real exemplars. Until these tests correlate to both environmental conditions and existing stone's durability, the methods will likely remain under study in ASTM Committee C-18.

## Tests Designed to Evaluate Anchorages

Anchor tests (Figs. 35-40) are unique procedures specifically conceived and designed to isolate and quantify the capacity of each of the individual stone anchorages in the configuration to be installed specifically for that job. Special apparatus, pressure guages, and hydraulic jacks are employed to force the stone to fail at the engaging anchorage device, rather than the panel between the anchors. This is typically done by restraining the panel safely outside the anchor's expected plane of influence induced by the anchor's resistance on the stone (or where the stone is expected to fail around the anchorage device's engagement). This plane of influence will show the area of the stone effectively being resisted by the anchor, which

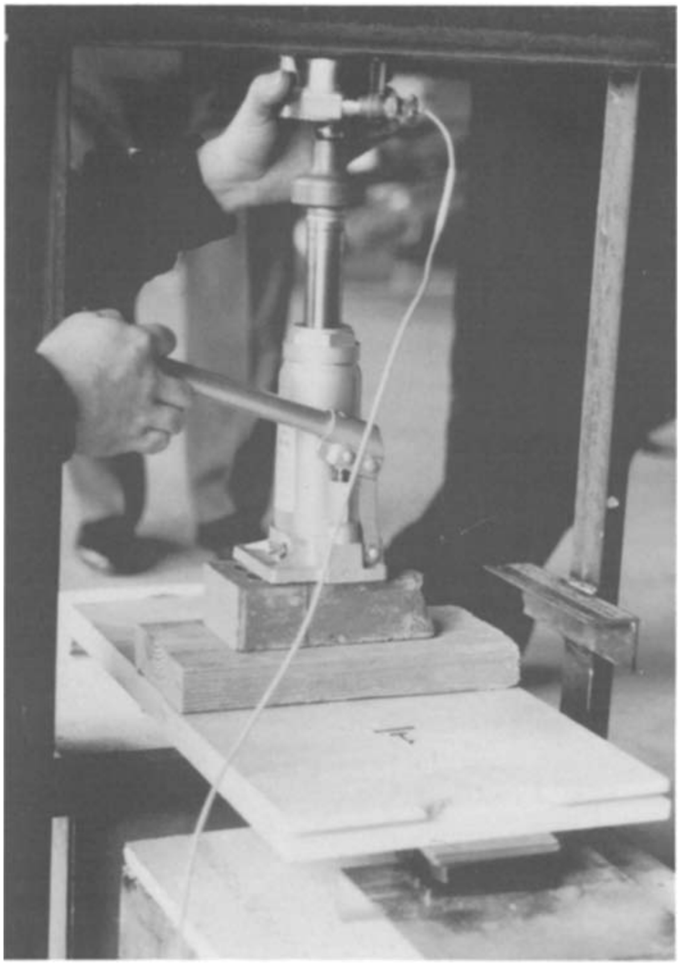


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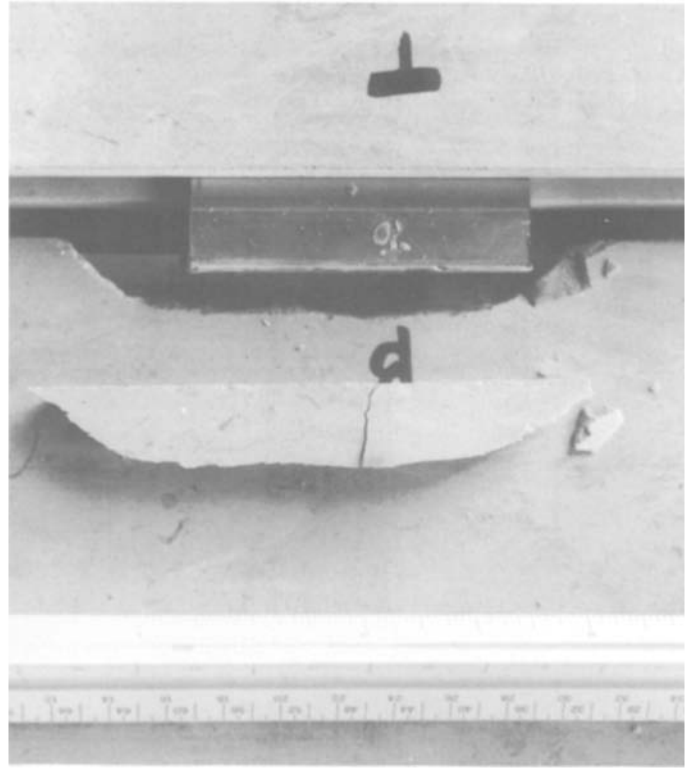
FIGURE 35: *Independent Kerf Anchorage Test.* The apparatus shows kerfed stone samples supported with an anchorage device engaged at their center. Similar to the wind pressure against the stone, a hydraulic jack applies load through the load cell and spreader beams onto the stone. The spreaders contact the stone directly outside the imminent failure surface emanating from the root of the kerf slot.



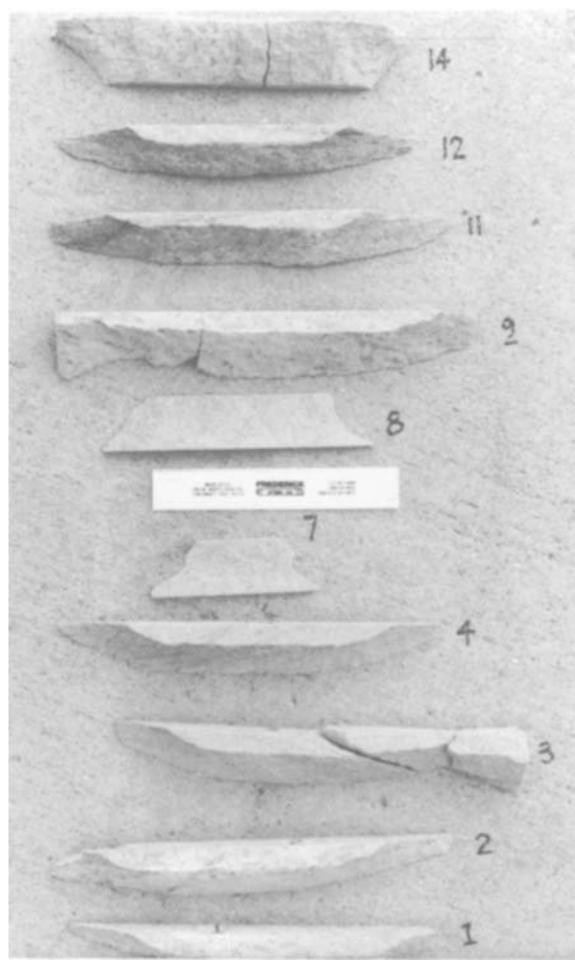
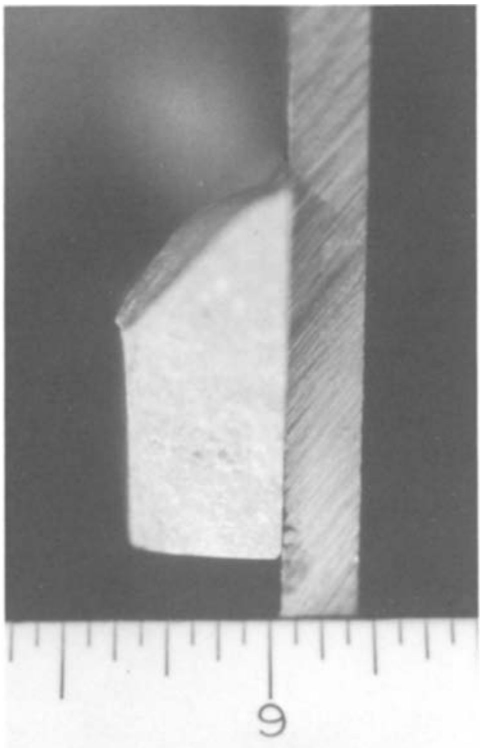


FIGURE 36: *(above, right) Breakout Pattern From Kerf Breakout Test.* The three-inch long extruded aluminum anchor is seen where the broken stone kerf fin was removed. This setup tested an anchor which was located at perhaps quarterwidths of the panel. The length of the breakout, which illustrates the effective length of engagement, is 5 1/2 inches.

FIGURE 37: *(right) Breakouts From A Series Of Anchor Tests.* Nine samples were tested to establish a statistically usable average of anchorage capacities and effective lengths for this project. The simplicity of the independent anchorage procedure not only allows proof of capacity early in the project, but allows inexpensive replication not possible in the full panel chamber test.

is understood to be the effective engagement. This effective engagement is not always the same as the actual engagement (Figs. 36 and 37).

The same load mechanics that the anchor will experience in use (during installation of the stone onto the exterior wall and in place in the exterior wall over the building's life) and identical project materials are duplicated by the testing apparatus and load-applying set-up. This allows the designer to associate the anchor's capacity to many panel sizes and anchorage locations as long as the mechanics of engagement into the stone and attachment to the supporting backup of the tested specimen are preserved.



◀ **Figure 38: *Cross-Section Of Broken Kerf Fin.* The segment of the stone fin broken from the kerf clip anchor. Notice the approximate 45-degree wedge shape which started at the root of the sawn groove in the stone.**

▼ **Figure 39: *Independent Pin Anchorage Test.* This apparatus tests the capacity of inclined pins engaged in the stone's back face and cast into the concrete panel. Similar to the wind's suction pulls on the panel the panel away from the four pins embedded in the concrete panel, by the hydraulic jack pulls on bars epoxied onto the stone surface. Flexible suspension assures equal load distribution to the pins.**

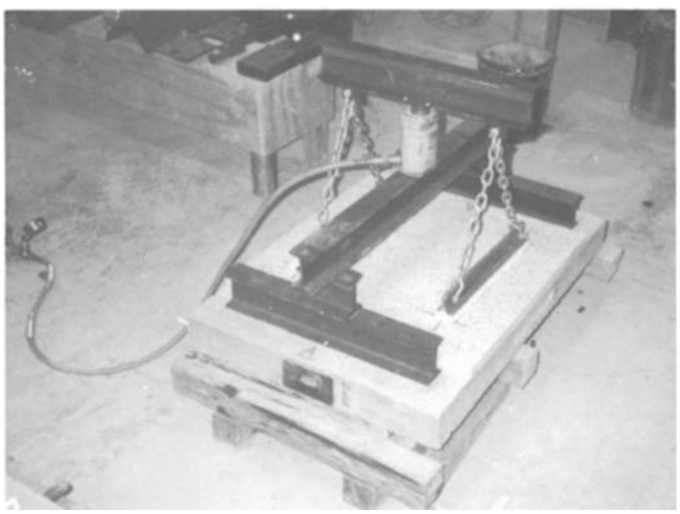


**Figure 40: *(below) Breakout Pattern From Pin Anchor Test.* Stone cones engaged by the pins still embedded in the concrete panel show the failure surface in the stone. Four pins engaged in the stone panel in opposing directions to mechanically lock the stone in place. A polyethylene slip sheet separated the stone from the cast concrete backup to prevent chemical bond and movement restraint.**

*Commentary:* Individual anchor tests evaluate the interactive capacity of the actual device with the stone it engages. It isolates the anchor and stone adjacent to it by limiting the panel size used in the test so that the failure surely occurs at the anchorage, not by stone panel breakage.

Because the example problem's exterior cladding system is a curtainwall glazing system with single stone infill panels between continuous vertical mullions, it is determined that the most economical installation process for this project is to retain the stone on a kerf bar that spans horizontally between the curtainwall mullions.

For the example of the special test method devised for evaluating this aspect, please reference the section on "*Anchor Capacity and Effective Engagement Length Test.*"

The capacity of the kerf must be attained by determining the actual length of the kerf that effectively engages and restrains, and thus supports the panel. With this span (width of the column cover between the vertical mullions) being up to six feet with only a narrow 1 inch of cavity behind the stone panel, and because the depth of the kerf bar is limited by the available cavity behind, the relative stiffness of the continuous horizontal kerf bar, calculated by its moment of inertia, will not exceed that of the stone panel it is supporting. With the bar material necessarily being aluminum because of corrosion resistance and compatibility criteria with adjacent related components as well as the actual final configuration of the kerf bar only being attainable by an extruding process, the aluminum kerf bar's modulus of elasticity is a relatively flexible 10.1 ksi, perhaps only twice what the stone's unit flexibility is. The narrow kerf bar's section modulus cannot approach that of the stone's slab profile for this example. The engineering conclusion is that, while the kerf bar engages the stone continuously at its top and bottom edges, with the kerf bar attached only at each of its ends to the vertical mullions, the stone is relatively stiffer than the kerf bar, and thus effective support occurs only at the ends (or corners) of the stone for some length in from where the kerf bar is supported by the mullion. That length had to be determined in order to calculate how long the surface of influence and thus the stone's resistance, would be.

To assess the anchor's capacity, two setups were built, one to test "end" conditions (kerf bar is attached to the mullion at the corner of the stone panel) and another to test "middle" conditions (kerf bar is attached to the mullion or other backup within the panel width, causing the panel to cantilever over the support). Reference the section on "Anchor Capacity and Effective Engagement Length Test" devised and executed to determine what that length would be. Two one-by-two foot stones with edge kerfs were engaged onto a kerf bar designed for the project. A load was induced onto the stone just outside the anchor's surface-of-influence where the fracture plane was expected to occur. A plane running diagonally (due to shear-tension stress) from the point of the stone's contact with the kerf bar to the surface where the load is applied was the predicted location of the plane.

Because the effective length of engagement is a function of the relative stiffness of the kerf bar compared with the stone panel, and because the stiffness of the stone's support (the kerf bar) is related to where it is attached to the mullion, the two conditions ("end" and "middle") were created by changing the kerf bars' support from the ends, or corners of the test stone panels for the end supports to the middle for the middle supports. Support was provided with blocking and fastening of the kerf bar solidly to the test apparatus.
Thus the actual engagement along the entire length of the stone is not the same as the effective engagement.

It was expected, and concluded from the test results, that the end supports yielded lower capacities due to less continuity of stone over the kerf bar's support. Further, for both conditions, and all kerf anchors in general, minimizing prying and minimizing the leverage distance between the "root" (bottom of sawcut) of the kerf and the point-of-contact with the device maximizes the anchorage's capacity. The "bulb" at the end of the fin on the engaging leg of the kerf bar (note "critical dimension 2" on the bottom of "Detail at Engagement of Clip in Kerf") helps assure that the contact point is both as close the root as possible, and also that contact between the anchor and the stone is prevented further down on the kerf fin, which would dramatically increase prying and cause premature failure.

This example's anchorage layout is for all four corners to be anchored, with equal resistances at each of these anchorage locations due to the kerf bars' attachment to the vertical window mullions that are of equal stiffnesses, equal support between the floors and thus equivalent spans, and each of the top and bottom stone anchorages attach to those mullions at equivalent locations within their spans.

Since attachment of stone anchorages is typically to another framework such as a strongback or curtainwall, it cannot be assumed that each of the stone anchorages has equal resistance simply because it is supported. In fact, each of these type supports is more accurately considered as a "spring" support since it is typically within the span of a separate vertical framework that has to transfer the individual anchorage reactions back to the base building superstructure.

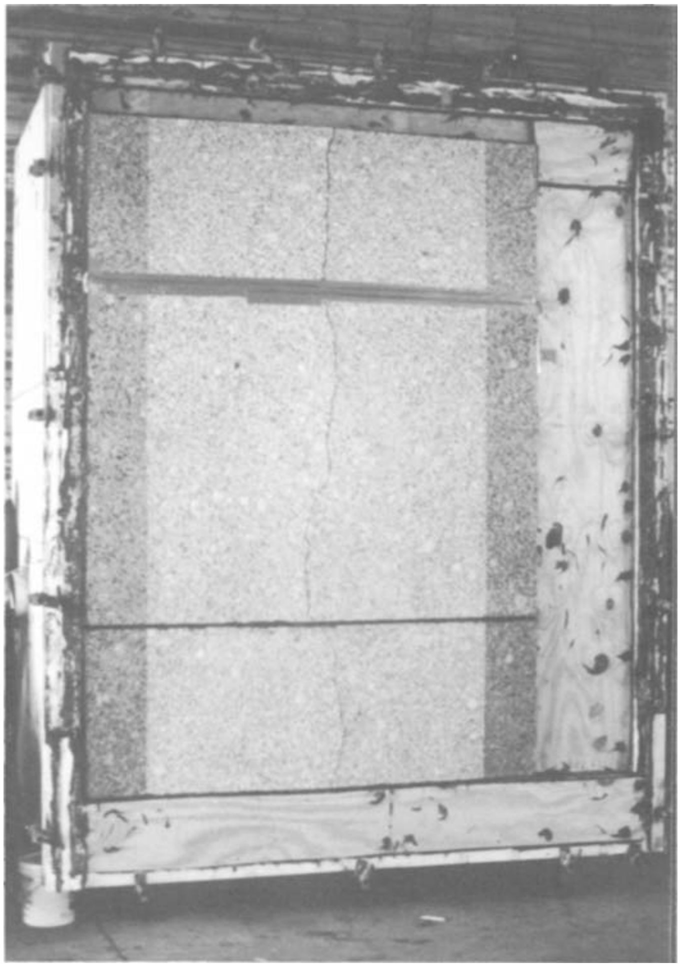
It is essential to analyze the relative stiffnesses of this backup framework that the stone anchorages attach to determine relative stiffness relationships between all anchorages in the same stone panel. Review member span distances, locations of the anchorage's attachment within that span, the member's moment of inertia, material, other reactions it supports, and its overall displaced shape and deflection magnitudes at the points where the stone anchorages attach. The displacement magnitudes themselves do not definitively identify the stone panel reaction distribution.

With the application of load onto the stone, there must be as nearly as possible a perfect plane created by the intersection of all the points of the location of the displaced attachments. Given that load travels toward stiffness, such a pattern of displaced supports that stay within a plane will allow equal distribution of the panel loads to its anchorages.

The consequence of a nonplanar displaced arrangement of panel supports will result in unequal load distribution between the stone panel's supports, thus load concentrations at probably atleast one of the other panel supports that was not anticipated in the anchorage design, and overstress situation, and an increased probability for failure. Doubling of loads at a specific anchor is realistically common in an inadequately conceived backup exterior wall structural framework. For nominal 1 1/4 inch thick sunset beige stone with a flamed surface, the kerf cut is 1/4 inch maximum width and 3/4 inch deep. These limits are attained by deducing the fabrication limits of the employed machinery that would produce the worst condition in the stone. With the maximum kerf width allowable into the edge of the panel and the thinnest acceptable slab thickness, the stone's kerf fin is the minimum 1/2 inch thick. Furthermore, the maximum allowable sawcut kerf depth of 3/4 inch results in the maximum distance between kerf root and point-of-contact with the kerf bar. This leverage distance was 3/16 inch plus 1/16 inch. This thinnest kerf fin with the greatest leverage of the force onto the fin creates the worst project condition and thus the minimum anchorage capacity values. Middle supported kerfs supported 1125 lbs and end supports are 850 lbs.

## Tests Designed to Prove the Capacity of an Assembly

These must be developed as unique tests, isolated from the common full-wall chamber test because it is conceived and designed to evaluate the complete actual-size panel with its individual stone anchorages (Fig. 41). Panels in the actual sizes and thicknesses are anchored in their proper layout configuration as they are to be installed specifically for that project. Because the loads within the overall curtainwall mock-up may only reach 1.5 times the design pressures before beginning permanent deformation, that procedure alone will not reach the load magnitudes necessary to prove that the



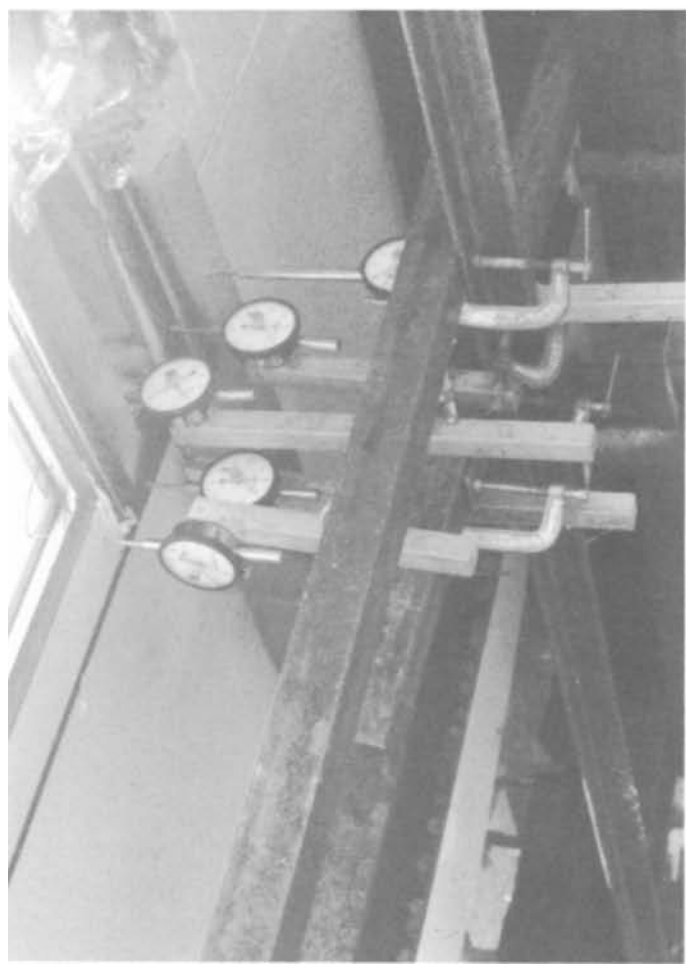


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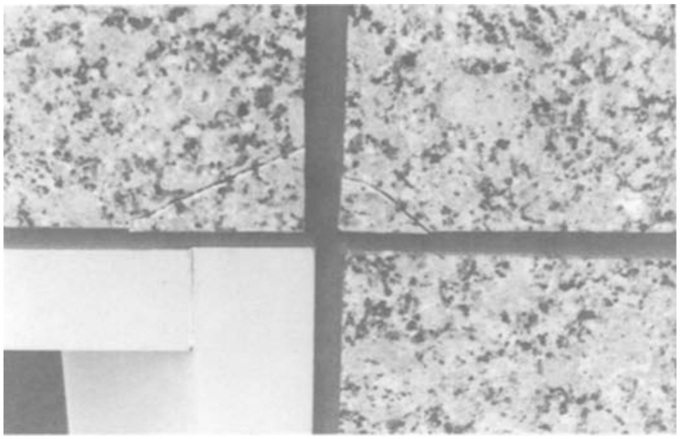
FIGURE 41: *Full Panel Chamber Test.* The complete assembly is tested after the anchorage and the stone panel are separately verified to conform to test requirements. Using a sealed chamber, pressures are sequentially increased and then reversed to simulate alternating wind forces on the facade until the panel breaks. Glass-and-metal curtainwall system mockups fail before reaching pressures required for stone. Thus, to evaluate the assembly's ultimate capacity, it is tested by a separate procedure.

FIGURE 42: *(above, right) Full Curtainwall Mock-Up Dynamic Water Test.* After the stone and anchorage assembly was proven, the stonework and support can be built into the remaining exterior building skin and tested for infiltration and water integrity. The propeller engine pulsates the wall to work water into potential gaps between components. Structural strength of the stone system is proven by the full panel test. This test intends to measure environmental integrity and structural compatibility between the systems composing the building skin.

FIGURE 43: *(right) Displacement Analysis Of Anchorages During Test.* Differential movements between the stone anchorage and the backup framing it attaches to must be studied closely. Described previously as a "boundary condition", incompatible behavior between the backup support and the stone anchorages can cause premature failure of the stone panel. Excessive twisting found at this location during the mockup was corrected by adding stiffeners at the connection.



◀ FIGURE 44: ***Broken Stone Kerf Fins Caused by End-Prying of the Anchor.*** **Differential movements between the stone anchorage, the backup framing it attaches to, and the displaced shape of the stone panel must match. Premature failures occurred here where the interactions of the three parts caused high stress concentrations at the panels' corners where the kerf rail engaged the stone. These conditions could not be diagnosed until after individual panel and anchor tests were completed. The full wall mock-up showed this deleterious behavior, which, if gone uncorrected, would zipper the kerf fin off the stone until the panel became loose.**

stone cladding assembly's strength has been attained at perhaps twice that required load (Figs. 42. 43, 44).

This test proves the panel-anchorage assembly alone will perform to the requirements that are elevated above those for the overall exterior wall. It is necessary to confirm the capacity of stone cladding above that of its adjacent systems because its higher relative variability requires higher overcapacity to maintain equal reliability to other curtainwall components.

To test the panel capacity initially, an expedient and inexpensive preliminary method can be conducted without a chamber or sophisticated instrumentation and apparatus. With rubber pads over wood blocks under the stone where the effective supports are expected to be located, orient a panel flat and uniformly load it with weight such as sand or bricks until the panel breaks. This will approximately measure the panel's single-mode ultimate capacity.

Within a sealed chamber encapsulating usually the backside of the stone, pressurization-and-depressurization in alternate and increasing loads to simulate positive and negative wind the procedure establishes the capacity of the overall stone-with-anchorage assembly. The test set-up duplicates the project's typical stone panel with all its production characteristics, its intended (and usually pre-tested) anchorage devices in the layout to be used within the project, all attached to a relatively rigid backup structure. The curvatures and displaced shape of the panel and its anchorage under load must resist the loads determined to safely eliminate the statistical opportunity for failure using the variability of the stone material's variability as the basis for establishing that load magnitude. Including dynamics of the backup support that closely model the movements at the full loads that only the stone and not the support structure are to be designed to withstand could compromise the true evaluation of the anchorage-panel interface performance. Designed structural resistance decreases with each successive system test, but must be executed in sequence to assure that each aspect of the system has proper overdesign to accommodate the potential of the other.

1. Because of the consequences of failure and its complicated behavior where it engages the stone and its dependency upon stone characteristics local to the anchor and the attachment to the backup support, anchorages require a higher designed load resistance than the stone it supports. These tests are run first, after theoretical analysis using ASTM unit-strength test values.
2. Because the anchorages are designed to be stable and with ample strength per above, stone panels require a lower designed load resistance than the anchorage supporting it. These tests are run second, after the anchorage tests have proven that they have adequate capacity. Full-panel tests should verify the flexural capacity of the panel between the anchorages. Failure at the anchorage indicates that either a local stone weakness not accounted for in the sizing of the engaging anchorage device exists or that the mechanics of the stone transferring load onto the anchorage device was not accurately modeled with the anchorage tests, which then rendered false conclusions.

*Commentary:* Once the capacities of the individual anchorages have been preliminarily proven by the individual anchorage tests, the true test interaction between the stone and its anchorage device is conducted using the sealed chamber to simulate positive and negative differentials that result from wind loads on the building's shape. Increased pressure is generated by increasing wind velocity that is tabulated by ANSI at $0.0296V^2$ (where V equals velocity in miles-per-hour) or by special wind tunnel testing, which is frequently used in congested urban sites. Affects of vortices caused by adjacent buildings can dramatically change the magnitudes of expected pressures and thus the forces required to be designed by.

Please reference the section on " Actual Panel Test For Preliminary Load Capacity" for the results of the example tests completed during the same time as the initial sample testing that were completed to establish material properties.

Usually dial guages that measure lateral displacements are placed at all anchorage supports to verify the support's expected zero-movement, as well as at midspans to measure stone panel curvatures under load. With the stone being full-size, it is essential to verify with this test that the displaced shape of the that the stone takes under load does not affect the effective support of its anchorages by somehow altering contact points at the engagement of the device into the stone or otherwise shifting contacts or load-transferring between the stone, its anchorage device, and the support framing the anchorage device is attached to.

This test is isolated from the full-wall mockup because the "overcapacity" required from the stone exceeds those on glass and other wall components, which because they are highly predictable engineering materials manufactured under controlled conditions, may only be designed to withstand 1.5 times design load before yield, permanent deformation, or failure. A consistent, "reliable" stone may require at least a 2.5 resistance factor over designed loads, thus in the wall mock-up, these loads could never be reached without breaking other wall components. Obtaining true capacity of the stone with its anchors establishes what the stone panel's capacity is with a given support anchorage pattern and type, and that the multiple anchorages in a panel are all working together in collecting the stone's reactions from the distributed load.

Please reference the section on "Complete Assembly Full-Panel Chamber Test" for the results of the example tests completed following the proof-of-capacities for the individual anchorages, and during the same time as the full-size wall mockup, but in a separate chamber. That procedure identifies that three setups, a 4'-7" by 6'-0" by 1 1/2 inch thick panel with only effective corner supports, a 4'-7" by 5'-0" by 1 1/4 inch thick also with only effective corner supports, and a similar size panel with corner supports on one side and also supports at third-points of its width towards the other side were tested.
All stones failed before the anchorages, which successfully established that the anchorage devices' mechanics performed well, as predicted, and allowed the panel itself to reach its full strength. Observing the break patterns verified that the position of the failure line (or highest stress plane) was where it was predicted to occur, at a load that closely correlated to the maximum flexural capacity of the material per the ASTM C 880 conclusions and the finite-element plate analyzing the panel at that load. This confirmation of authentic test results with mathematical analysis was then used to structurally verify other less-typical panel sizes and their anchorage configurations with finite element methods versus expensive and time-consuming full-size testing of multiple stone sizes and configurations.

SECTION 6
## ANCHORAGE DEVICE MECHANICS

The device designed to attach a stone panel to its support controls the assembly's structural capacity. How these anchorages are distributed about the panel determines its stresses. This layout locates where uniform surface loads are transferred to the exterior wall framing. Think about both when arranging anchors.

How the device engages the stone is more critical than where in the panel it is located. Attaining expected behavior means maintaining proper mechanical engagement despite varying installation positions, environments, and loads. The following anchorage fundamentals are reviewed.

- The Function of the Stone Anchor
- Proper Design and Installation Philosophy

- Correct Anchorage Device Configuration
- Handling Stone During Installation
- Basic Anchor Device Types
- Proper Application and Optimization of Kerfs
- Proper Application and Optimization of Dowels

*Commentaries* explain how these guidelines are applied to a design example.

## The Function of the Stone Anchor

An anchor must transfer the load from the stone to the supporting backup framing without causing the stone any distress. Where intended to carry the weight of the stone, the anchors' locations should divide the weight as evenly as possible. Their locations should distribute the flexural stresses as evenly as possible throughout the panel to optimize the panel's lateral load capacity.

While a single anchorage device may actually engage two separate stones, the device must isolate each stone structurally from another, that is, limit the actual loads to be superimposed on the panel body to the surface wind and weight of that stone itself.

The materials and constructibility of the anchorage must be adequately strong and simple to withstand design loads and also be reliably installed without compromising any of its functions over the full intended service life of the structure.

## Proper Design and Installation Philosophy

This *standard guide* suggests which basic engineering design considerations require evaluation to accomplish the design of stone anchorages, loading superimposition, climatic effects, superstructure behavior, and nature-of-material characteristics should be evaluated and provides a practical itemization to assure that the designer includes those design considerations at the prudent time during the design process.

A designer's general approach to anchorage design is predicated initially on the requirement to first understand the variable nature of stone material. This understanding must then orient an approach that exercises realistic scientific limitations in the engineering of this material, which is relatively inconsistent. This approach must then adapt appropriately to the engineering and compatibility limitations for the other materials that are adjacent to the stone and are part of the interfacing systems and components within the exterior wall.

Further, the overall construction in the overall skin's final assembly needs to be evaluated within the conditions and environment it is constructed in, and also the conditions and environment that it must endure within during the building's life expectancy. The climates, forces and exposures of these two periods of the building's existence are very different. Aesthetic and structural durability are the goals that a design that has integrity builds to through these exposures.

### *Stone Material Considerations*

Identify the general material physical properties unique to stone as a natural material used in building construction. Its structural performance characteristics must be reconfirmed for each considerably sized project and sometimes during that same project because its natural formation in unknown, uncontrolled conditions causes great variabilities, sometimes even within the same material color and quarry location.

Natural stone's physical properties are different than other engineered building materials, as are their consistencies and behaviors under stress and weathering. These properties need to be quantified individually for each stone type and generally for each new project. Then, evaluate these properties in whole within the context of the particular intended use conditions of that project in case some conditions imply different needs.

## Average Strength and Degree of Consistency

Because the properties of stone are relatively inconsistent due to its heterogeneous composition and varying conditions during its geological formation, variable reactions to loading and climate are likewise inconsistent. The stone's statistical, measurable consistency is as important a consideration to its performance as is its average strength. Low average strength alone does not determine a particular stone to be incapable of durable structural performance if it is consistent and appropriately designed so. Conversely, high average strength alone does not determine a particular stone to be a good structural material if it is widely inconsistent and appropriately designed so.

### *Nonlinear Elasticity and Failure of Stone*

Stone is nonlinearly elastic, that is, as stress increases, strain increases slightly disproportionately, resulting in a plot of the elastic modulus that is a curve. Stone does not yield or deform before it fails, and as a result it ruptures with almost no visible warning upon becoming overstressed. Because stone is nonisotropic and heterogeneous, it is difficult to predict where the "weakest" region of the material is.

### *Anchor Indeterminancy*

Factors influencing the stone's support behavior and the anchorage devices' arrangement within a stone panel's body should promote the simplest and the most predictable stone behavior possible. Extensive discussion includes, engineering stress analysis made simpler; avoiding approaches that stack stones; avoiding mid-panel anchorages that align multiple anchors in the same line or plane that become "moving and unstable foundations"; and anchorages' mechanics during installation. All are considered relative to safety.

### *Anchor Determinancy*

Unless engineering within the parameters of any particular system can prove otherwise, to attain determinant behavior, and thus achieve maximum predictability, the stone panel should be considered as a simple infill. Such an infill would have only the structural task of resisting loads applied to its surface while supporting its own selfweight.

Where movements occur, they must not occur where the anchorage device engages the stone, for shifts in contact points between the anchor and its stone or changed resistances could dramatically alter the intended stress distribution within the entire panel in addition to changing those locally at that anchorage. The panel must be allowed to independently move to avoid other facade elements prying against it and the stone prying against other elements such as mullions, strongback, and mounted components. Prying results in point-load concentrations and inevitable damage if not failure, usually to the stone. The building's normal expected skin movement itself should not induce any load into the stone.

An anchorage should establish its own stability by its attachment to its support structure that is not dependent on the stone.

An anchorage's designed form and installed engagement should isolate any added load or any unpredicted or unintended movements from adjacent components. They cannot influence the stone's performance. Anchorages also must accommodate the tolerances of adjacent construction while allowing for the dynamic movements within the building skin along with those movements that are introduced into the skin by the building frame.

### *Anchor Durability*

Given that climate, wind, precipitation, seismic effects, and material properties vary, anchor configuration is to be as stable, and its construction is to be as durable as necessary to endure the expected exposure to the environment while maintaining correct mechanical connection.

### *Anchor Inspectability*

Because anchorages are typically concealed, their capacity might be designed somewhat stronger than the expected performance of the stone panel itself. Should an anchor device fail or deform while being hidden within the exterior wall, it would likely be undetectable from the exterior or interior until the stone around that anchor began to crack.

Failure of the stone at the anchor might offer visual evidence of breakage on the surface if frequent facade maintenance and inspections are conducted. Once discovered, investigation could diagnose the cause of the deficiency, perhaps by studying the fracture pattern, correct the faulty anchor or substrate, and repair or replace the damaged stone material. Collapse of the stone panel is wisely averted.

## Correct Anchorage Device Configuration

To best preserve the described philosophy, apply the following rules during the design of the actual device

1. Simple connections are usually the best.

   *Summary:* Simple is meant in several key ways: The connection, or anchorage device, should be structurally "simple" versus fixed or rigid so as to not restrict any rotational freedom within the stone-to-anchorage engagement. Simple is also meant to keep the configuration of the components easy, that is easy to manufacture, and easy to install. The easier these tasks are, the more likely the anchorage devices will be made and built correctly and accurately. If it is built correctly, it will likely function and endure as designed.

2. Connections should be constructed with the fewest parts.

   *Commentary:* Not only is a connection that uses the fewest parts likely to be the least expensive to procure, but because there are fewer parts to handle, an anchorage with fewer parts is usually less expensive to install. Probably most important, though, is that with fewer parts, there is less opportunity for installation error or maladjustment between parts, which might result in faulty anchorage and stone performance.

3. Minimize the number of anchor types within the project to maximize quality workmanship.

   *Commentary:* Similar to the advantages of having the fewest number of parts in a single anchorage assembly, having consistent anchorage types within a single project means that economy is gained both with quantity manufacturing of parts and repetitious installation tasks by the craftsmen. Multiple part types, especially if they are "similar but different" can be more easily confused or misinterpreted in the field, and are more likely to be placed incorrectly.

4. Minimize the number of anchors within a single panel.

   Multiple anchorages along a single line or plane and within a single panel increases the potential for concentrated point loads because any anchor's position that deviates from the "perfect line" or "perfect plane" during either construction installation or in-place under load will tend to be ineffective or overeffective depending upon its position and backup's relative stiffness. Load distribution, and thus stress magnitudes, will not be as intended. Complex panel-body curvatures resulting from multiple-anchor lines result in indeterminate internal stresses within the panel unless the resistances (displacements under load) of all the anchorages and their backup structures are perfectly matched and balanced with the stone panel's original unstressed shape.

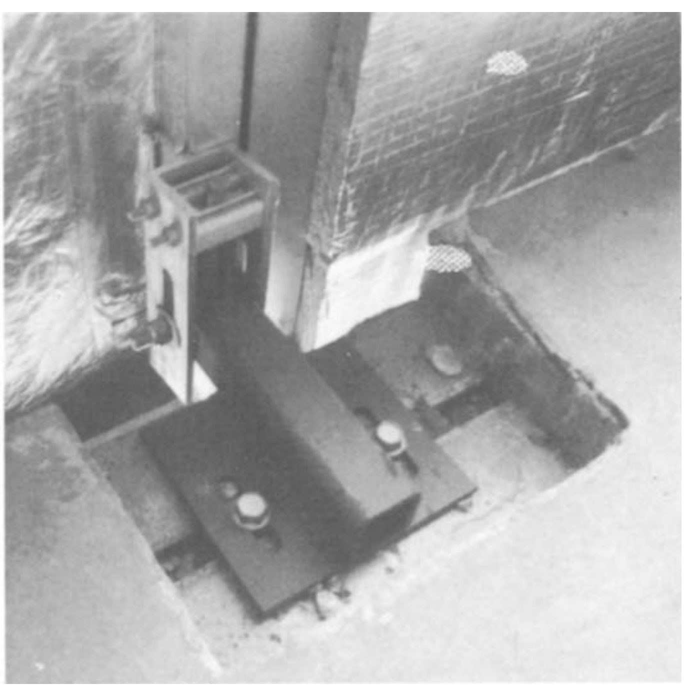
Even if practicing exacting workmanship and installation techniques, within the context of expected construction toler-

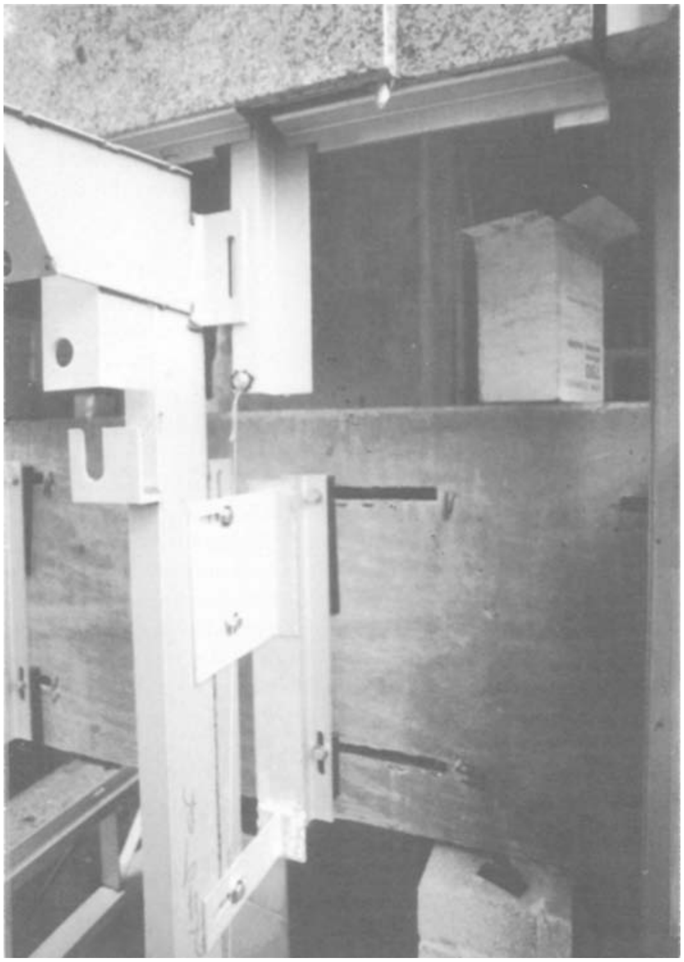
◀ FIGURE 45: ***Wall System Attachment At The Slab Edge.*** **An extruded tee attached to the slab edge anchors the vertical wall framing to the building structure. The serrated tongue with serrated washers over horizontal slots allow in-and-out adjustment without welding. Boxed-washers hold the flat faces of the hex nut inside to prevent back-off and the need for a wrench on that side. The skew-cut flat tube between the tee and slab attaches to an embedded channel-type insert cast into the structural slab. It may have only been structural shims, except the slab edge was too far out-of-tolerance.**

▼ FIGURE 46: ***Wall System Attachment At The Top-of Slab.*** **A fabricated tube-with-plate component attaches to a cast-in channel insert in the top-of-slab to provide attachment for a unitized stone-clad wall. The channel provides side-to-side adjustment, the plate slots allow in-and-out adjustment. Note the vertical slot at the panel connection, which allows vertical movement, or a roller connection. The fastener uses a nylon-insert locknut to stay secure with only slight torque.**

FIGURE 47: ***(below) Fabricated Metal Strongback At Return Corner.*** **With the swingstage rail system cantilevered at the top, workers align the vertical wall framing that will support stone cladding. The pre-alignment of the backup eliminated the need for time-consuming adjustments for each panel during setting. Note the cast-in horizontal channel inserts in the spandrel beam which provide side-to-side adjustment. Tees similar to that in figure 45 connect the vertical fabricated channel frames to the inserts in the beam. Plastic structural shims must separate the metal, especially aluminum, from the concrete to avoid corrosive disintegration.**

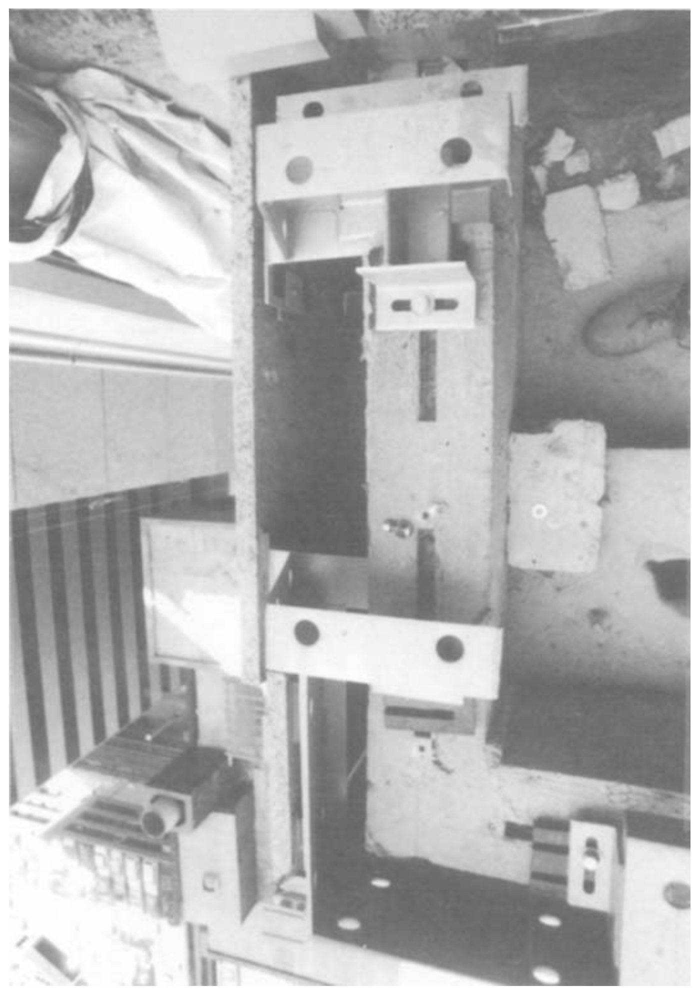
ances, the finished anchor locations will likely be out-of-the-plane relative to the unstressed flat stone they support. Thus, upon the panel's deformation under load, unless the panel's stiffness and displaced shape matches that of the anchors with their support backup, forces are distributed unequally and undesirable stress concentrations result at the anchors, usually making this approach unadvisable. Determining in-place stresses in the stone with such an indeterminate support resistance is thus impossible.





◀ FIGURE 48: *Fabricated Metal Strongback At Cornice.* Complicated facade planes were built from flat stone panels by constructing the profiles with prefabricated framing. Position variances (tolerances plus errors) between the poured concrete frame and the finished building planes require adjustability. Slots, serrated plates, and strut embedments allow quick alignment of the exterior skin to its correct location independent of the erratic frame behind. Structural plastic shims separate concrete and metal parts to protect against corrosion.

FIGURE 49: *(below) Fabricated Metal Coping Bracket.* Where the irregularly-shaped coping cap occurred at the top of the parapet wall, a specially fabricated bracket formed the profile, set the correct elevation, and allowed for encapsulation by the waterproofing system. Stainless-steel through-wall flashing fit between the stones and the bracket. Slots and inserts allowed for final adjustment between the concrete wall placement and the finished facade lines. Structural shims set the elevation. Type #31 anchors attach the stones to the bracket though oversized holes.



Because natural stone is such an unforgiving structural material that is extremely vulnerable to stress concentrations, the designer should prepare a support scheme that equalizes stress distributions within the panel. Lower peak stresses allow more economical use of the most commonly used flat-slab stone profile as a structural component. In order for a designer to predict the stresses that will result in a cladding component such as a stone panel, which is supported on a set of "spring" supports of potentially unequal resistances within the same panel, it is certainly unadvisable to introduce an inherently indeterminate support scheme like that produced with multiple anchors across a continuous panel edge or in a section through the panel body.

### Connection Components

Connection components must be adjustable to accommodate the cumulative tolerances of the stone, anchorage, and support framing to be able to create as-nearly-as-possible a "perfect" plane to support the stone (Figs. 45–49).

◀ Figure 50: ***Movement Accommodation Between Floors.*** **Horizontal movement joints accommodate differential long and short-term movements between floors. As the skin moves, the stone anchorage must accept the dynamics without transferring movement or force into the stone panel. Here, a flange extends down from the extruded window sill that engages a grooved clip. The serrated leg allows for adjustments and tolerances to provide the proper safe engagement.**

*Commentary:* Given that construction is the process of combining separate systems that perform separate functions into a final whole that fits together, it becomes one of the stone anchorage's function to accommodate the fit to its supporting substrate.

Most of the systems within the exterior wall and the base building get concealed between the exterior cladding and the interior finishes. Because they are eventually unseen, their locational accuracy is typically less critical, and thus are not installed to the accuracy required of the final skin.

Discrepancies then exist in location between the building structure, the exterior wall structure, and the final stone cladding. Where for instance the superstructure may vary in excess of 1 1/2 inches from its intended position, the stone's final position is not expected to deviate more than 1/4 or 1/8 inch from its theoretical position. The difference between these positions is resolved with adjustability in the anchorage components. Slots, serrated faces, and variably-interlocking components commonly combine to allow the deviations to be corrected.

Most important, though, are several aspects of anchorage component performance that cannot be compromised by the anchorage's adjustability. With adjustability, the position relationships of the different parts change. The anchorages' structural adequacy cannot change. The anchors' relative stiffnesses must remain constant through the full range of adjustability. And most important, through that range of adjustability, the behavioral mechanics of the anchorage device engaged in the stone must be preserved. Once the final position is attained, it should be "fixed," meaning that the adjustability should be locked-out to prevent slippage.

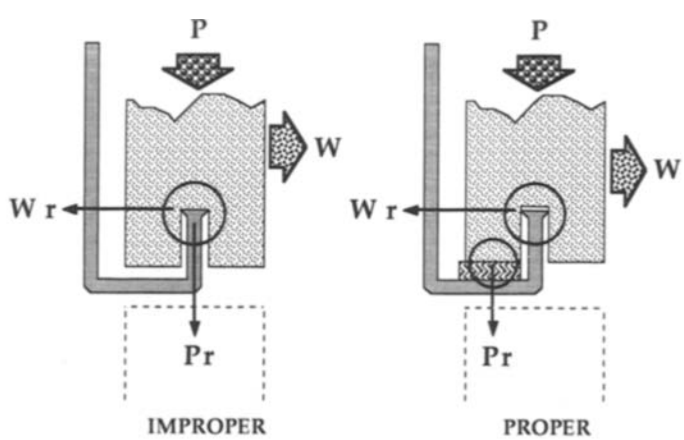
### *Weight Support*

Support the stone's weight on two connection points. Weight should be transferred by bearing physically on the anchorage device. It is advisable to not transfer both the gravity and lateral loads at the same contact point within the engagement.

*Commentary:* With the same reasoning offered for discouraging intermediate anchors between primary stone support locations within the section on "Anchor Indeterminancy," design the weight of the stone to be carried at two points. Alignment and equal stability of more than two locations will be unsure, resulting in probably only two effective supports.

Further, it is advisable to separate the contact points of the anchorage device and the stone that will carry either the lateral or gravity load. For instance, in the example kerf anchor, the vertical leg of the anchorage device, which engages the sawn slot in the edge of the stone, provides lateral resistance by its bearing on the side of the stone's kerf fin. This same leg should not carry the weight of the stone by bearing on the bottom of the sawn kerf slot. Bearing on the bottom, or "root" of the kerf and also on the side of the kerf fin creates complex stresses at this point-of-contact and multiplies the force magnitudes at the same plane-of-influence, which likely radiates diagonally away from the kerf root. A bearing condition of the stone at its kerf root not only tends to buckle or bend the device's engaging leg, but by maximizing eccentricity on the kerf angle, also induces the maximum torsional force. Both of these actions could deform the anchorage's shape and change the entire engagement behavior and thus the anchor's capacity. Instead, carry the stone's weight at the bottom of the stone fin in bearing on the horizontal leg of the kerf angle.

Adequate shims should be placed to assure that bearing does not occur at the kerf's root, but yet maintains the optimum engagement to minimize the prying forces of the anchorage's vertical leg onto the stone's kerf fin while resisting lateral loads.




◀ FIGURE 51: *Separate Lateral and Vertical Reaction Points of Contact in the Kerf.* **Complicated and unpredictable behavior results from improper support of the stones weight (P) and lateral wind load (W) at a single point of contact where gravity support (Pr) and lateral support (Wr) occur. Predictable, verifiable behavior results where gravity support (Pr) rests on the "ledge angle" independent of lateral support (Wr).**

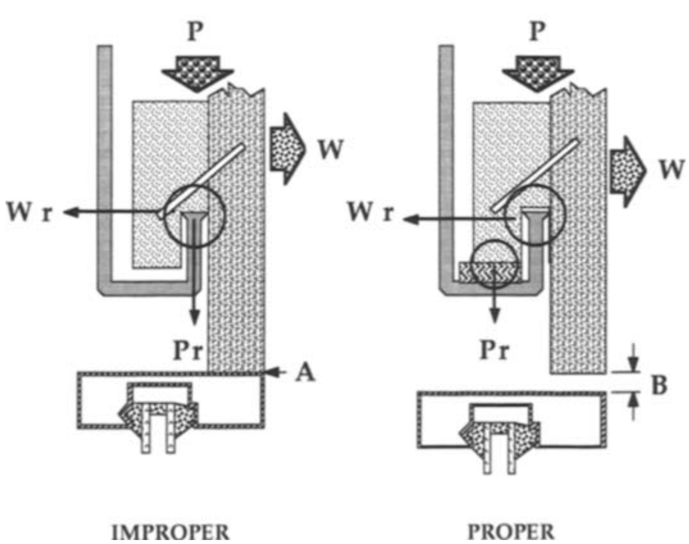



FIGURE 52: *(below, left) Separate Lateral and Vertical Reaction Points of Contact At Liner Blocks and Allow For Sufficient Movement.* **Avoid unpredictable capacities due to complex stress flows in the stone which result from an improper single point of contact. Insufficient movement allowance between stone panels and adjacent wall systems (at A) fail weatherseals, damage cladding, and can fail components. Proper anchorage design separates gravity and lateral reactions and allows sufficient space (at B) at perimeters for movement of stone panels, support systems, and all interfacing wall components.**

### *Points of Contact*

Separate the points-of-contact where the kerf clip anchorage device engages the stone within the sawcut kerf in the stone to independently transfer horizontal and vertical load components. At this incorrect single contact point, vertical and horizontal loads are transferred simultaneously, generating compound stresses, combined surfaces-of-influence, and prying that will likely reduce anchorage capacity and reliability. (Fig. 51)

Separate the points-of-contact where the kerf clip anchorage device engages the stone within the sawcut notch in the liner block to independently transfer horizontal and vertical load components. At this incorrect single contact point, vertical and horizontal loads are transferred simultaneously, generating compound stresses, combined surfaces-of-influence, and prying that will likely reduce anchorage capacity and reliability. (Fig. 52)

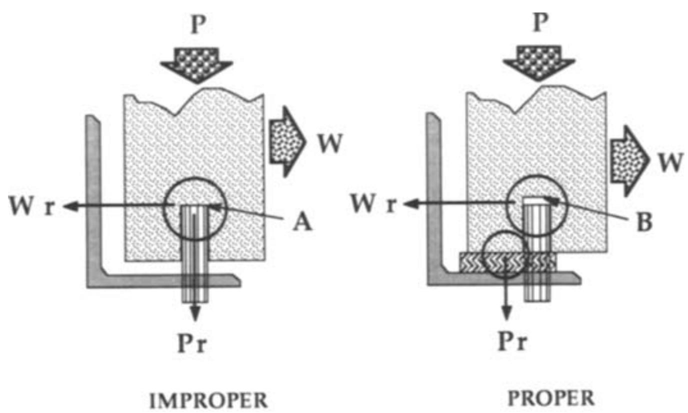
Separate the points-of-contact where the rod dowel pin anchorage device engages the stone within the drilled hole in the stone to independently transfer horizontal and vertical load components. At this incorrect single contact point, vertical and horizontal loads are transferred simultaneously, generating compound stresses, combined surfaces-of-influence, and prying, which will likely reduce anchorage capacity and reliability. (Fig. 53)

Separate the points-of-contact where the tooled rod headed pin anchorage device engages the stone within the milled slot in the stone to independently transfer horizontal and vertical load components. At this incorrect single contact point, vertical and horizontal loads are transferred simultaneously, generating compound stresses, combined surfaces-of-influence, and prying, which will likely reduce anchorage capacity and reliability (Fig. 54).

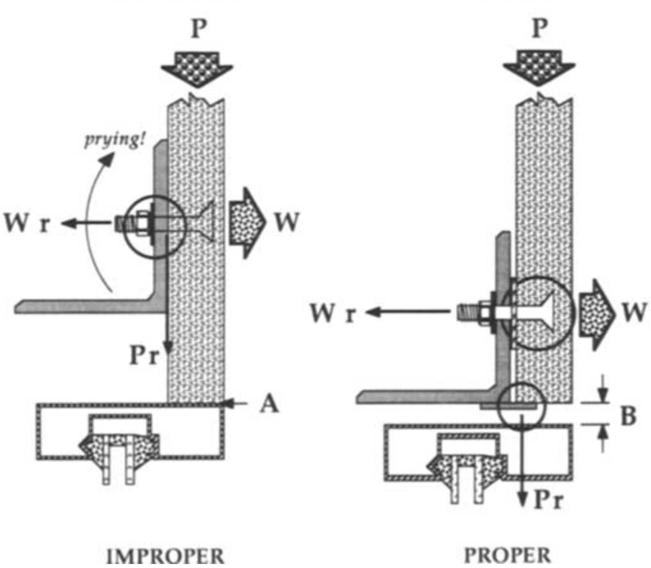
Anchors should be accessible—allow the craftsmen adequate access to the work. Provide clearance to the craftsman for his hands and his tools that are necessary to assure proper adjustment, engagement, installation, and then inspection of components and fasteners.

*Commentary:* With the same thinking that mandates that anchorages be simple, they must also be accessible to the craftsman installing them so that they can be manipulated relatively easily. Being able to see what is being done with fasteners and "fit" dramatically raises one's confidence that what is installed is what has been designed and tested to work.

Since third-party inspectors are frequently employed to oversee and verify the work of the stone cladding and exterior wall installers, visual and tactile access to the hardware is also helpful for them, and in the end beneficial to the installer, as fewer hidden aspects removes doubt. Anchorages that are difficult to access will take more time to install correctly and are likely to be more expensive to place as well.




◀ FIGURE 53: ***Separate Lateral and Vertical Reaction Points of Contact At The Pin.*** **Avoid unpredictable capacities due to complex stress flows in the stone which result from an improper single point of contact. Point contact (at A) on the pin head causes stress concentrations that cause premature breakage. A deeper hole (at B) provides clearance which prevents point contact.**




◀ FIGURE 54: ***Separate Lateral and Vertical Reaction Points of Contact At Headed Shanks and Allow For Sufficient Movement.*** **Avoid unpredictable capacities due to complex stress flows in the stone which result from an improper single point of contact. Insufficient movement allowance between stone panels and adjacent wall systems (at A) fail weatherseals, damage cladding, and can fail components. Proper anchorage design separates gravity and lateral reactions and allows sufficient space (at B) at perimeters for movement of stone panels, support systems, and all interfacing wall components. While these type anchors have been used to carry both lateral and gravity loads in the past, their true capacity in combined loading with movement while prying is unknown and untested in this condition. Multiple influences occurring simultaneously may eliminate overdesign which makes it susceptible to premature failure. Further, because no redundancy exists, a single anchor failure adds unanticipated loads to other anchors, that may cause a progressive, or domino failure of surrounding anchors due to overloads.**

### *Moisture*

Anchors should not collect or hold moisture. Anchorage components and their preparations within the stone must not collect or entrap moisture, which could stain, corrode, or freeze. Fill anchorage preparations in the stone that are not occupied by the anchorage, and all voids around the device that could collect moisture with a compressible filler material such as a sealant. The filler should be mutually suitable for the volume to be filled and be compatible with the materials it will contact and the environment to be endured.

*Commentary:* Voids that can collect moisture or retain moisture should be filled to avoid corrosion or freezing where either the ice or corrosion product could grow, expand, and spall the stone away from its engaging anchorage device, leaving the stone unsupported at that anchorage location.

A frequent procedure is to partially fill the kerf in the top of the stone with the same sealant to be used as a weatherseal, then embed the anchorage device into the sealant-filled kerf slot. Sufficient sealant should be in the kerf so that it is pushed completely to the top once the engaging leg is as deep as it is to be. Likewise, pin holes, plug holes, shank holes, and routed preparations are commonly filled with sealant.

Excess sealant must be cleared to assure that proper weatherseal proportions are allowed, and that three-sided adhesion is avoided. Compatibility and adhesion of that sealant to itself when it is cured (will the "uncured" sealant applied for the weatherseal adhere to its "cured" self at the kerf slot?) must also be verified prior to determining the kerf fill material.

After curing, the sealant also acts as a shock-absorbing medium that tends to distribute contact areas and dampens the effect of force reversals between the anchorage and the stone. Further, should any moisture invade the kerf between the stone and anchorage and does freeze, the compressibility of the sealant may absorb the expansion without consequence to the stone.

### *Slippage*

Anchors *should not* slip when designed to be "fixed." Prevent unintentional slippage at connections that are intended to accept tolerance, *not* movement. Interlocking serrated components, tack welds, lockwashers, lockflanges, or nylon insert locknuts prevent component-to-fastener slippage at connections after final adjustments to positions are made. Proper fastener size, the fastener's diameter's match with the hole or the slot, and accurate torque when tightening the fasteners are essential to accurate attachment durability and force resistance.

Anchors *should* slip when designed to "slide." Allow intentional slippage at moving joints with correctly oriented slots, separating slip shims, and limited-torque locked fasteners that allow guided slippage to accommodate anticipated movement at moving anchorage connections. This type of components must occur not only at the individual stone anchorage, but also where the backup framing attaches to the building structure. Movement, except in negligible magnitudes, must be accommodated between anchorage components outside the stone, not at the interface where the anchorage device itself engages the stone (Figs. 29 and 30 on page 54 ).

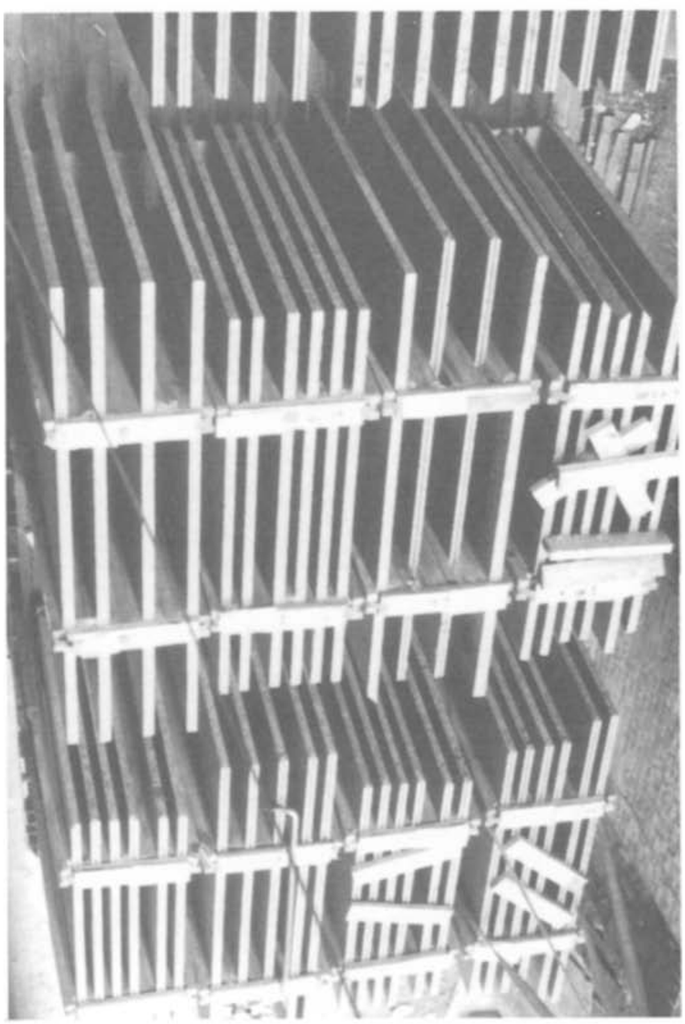
*Commentary.* Further to the section on "Factors That Influence Stone and Anchorage Performance," the designer must first acknowledge the difference between adjustability of an anchorage that accommodates tolerances between interfacing constructions, and movability within an anchorage that is intended to hold a position in some direction but allow freedom-of-movement in another. Adjustability is intended to be locked-out upon final positioning. Movability requires the same locking-out for the directions it is to hold, and is intended to remain "open" in the direction it is to allow movement to occur.

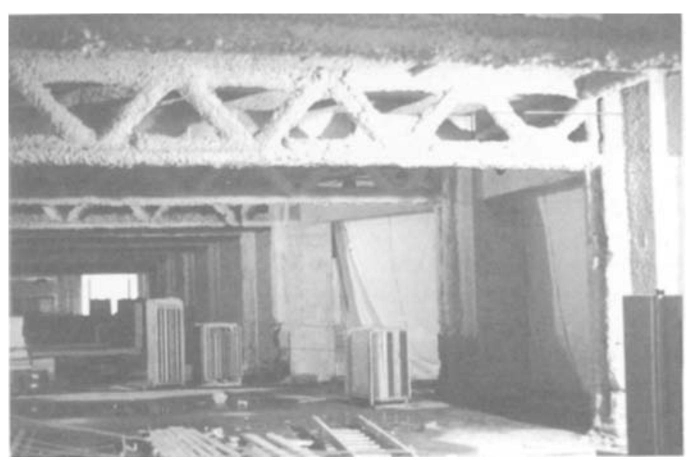
Adjustability may employ slots in all directions (in, out, and side-to-side) that is eventually to be a "pin" structurally, and thus requires all the slots to be locked-out to prevent slippage once final position is reached. An anchorage that allows movement will likely also employ slots in all directions within its components, however because it is theoretically a "roller" in some direction, that direction's slots cannot be locked. The adjustable directions are to be locked to form "pins" in order to restrain its intended directions.

◀ FIGURE 55: *A T & T Chicago's Facade Under Construction.* **In comparison to the mock-up shown in Figure 23, the typical areas of stone remain unchanged in stone type, finish, and joint configuration. Three swingstages near the top of the second set-back at the 47th floor continued the steady ascent to the roof. Above that, the aluminum curtainwall framing preceded before the stone, due to the stone being mullion-supported.**

FIGURE 56: *(below, left) Stone Panel Deliveries To The Jobsite.* **Stone production needed to be closely coordinated with installation. The sequence of fabrication, crating, and deliveries needed to be in the correct order to maintain progress, maximize productivity, and minimize rehandling of the panels. It required coordinating this sequence to the project site's stone installation activities with all the other building trades. Crates were hoisted with a cathead to each floor. Stabilizing cables, or skinner lines extended from the cathead to the outside of the flatbed trailer to guide the package up to over 900 feet to the roof.**

FIGURE 57: *(below) Crate Distribution On The Floor.* **After unloading from the truck, the crates are arranged on their respective elevations and facade bay. Fully coordinated marking and even the arrangement of the crates on the flatbed trailer or within an overseas container was required to maintain inventory and facilitate correct placement on the floors. The crates' weight with their handling equipment created considerable concentrated loads that had to be closely calculated to avoid damaging floor slabs. The crates were stored against the columns to keep them out of the way of ongoing construction activities, to locate their weight over the main floor trusses where the floor was strongest, and also to tie back the panels in an opened crate.**



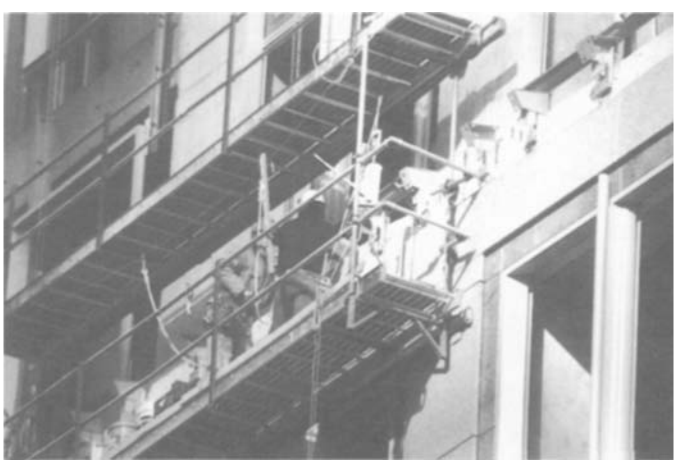


The free direction must encourage movement allowance with proper slip shims to assure that corrosion or moisture will not occur to lock the components unintentionally, nor will the tightness of the fastener hold the components intended to be able to slip together so that friction will prevent the intended movement. In this situation of unintended restraint, the weakest point of resistance will fail to allow the movement to occur. The designer must prescribe the system to not allow it to occur in the stone or its anchorage supports.

## Handling Stone During Installation

Consider the methods and mechanics of handling and placing the stone onto the anchorage support (Figs. 55 to 63). Where the stone is to be placed on a truss, frame, or wall unit off-site and then transported and erected in a unit, evaluate the dynamics of the handling of the pre-assembled unit through delivery, erection, and finalization on the building frame to be sure proper relative stiffnesses are maintained to not threaten the individual anchors. Consider the stage that the building construction will be in when stone is to be placed, the involvement of consultants and inspections overseeing and verifying the installation process, and the access to the connections.

Focus on support systems is on the relationship with the building frame's features. How that contemplated framework that the actual stone anchors attach to is structurally and environmentally compatible with the facade's architectural design, the building's superstructure construction and dynamic behavior's resulting movement, the framework's and stone's or device's installation approach, and their response to individual stone anchor requirements will determine the system's suitability.



▲
**FIGURE 58:** ***Exterior Swingstage For Cladding Installation.*** **Work platforms were suspended from monorails cantilevered through the openings above. These platforms held the men, tools, and miscellaneous parts needed to install the stone cladding. A solid platform above provided overhead protection from curtainwall and structural building frame construction that continued above.**

▲
**FIGURE 59:** ***Stone Panel Handling During Installation.*** **Workmen moved stone panels from their crate at a nearby column to the perimeter of the building with a walk-behind fork truck. The truck lifted each panel with a stone clamp, engaged a dimple in the back of the stone panel, in its correct sequence, the truck drove it to the unglazed opening in the curtainwall, and transferred the stone clamp and panel to a chainfall suspended from the same monorail that supported the swingstage. Workmen on the swingstage then moved the panel into position in from of the building column or spandrel, lowered it onto its anchorages, and fastened it into place. They also completed several quality control checks through this process.**

▲
FIGURE 60: *Ground Floor Entrance Stone Shake-Out.* At the entrances, where few of the stone panels were typical or repetitive, significant area was required to uncrate and shake-out materials. Since the entire width of the entrance was scaffolded, several crews worked simultaneously to erect the base. Multiple crews in different areas also required more crates to be open and available at the same time. Stones are usually crated by similar sizes and types in nontypical areas.

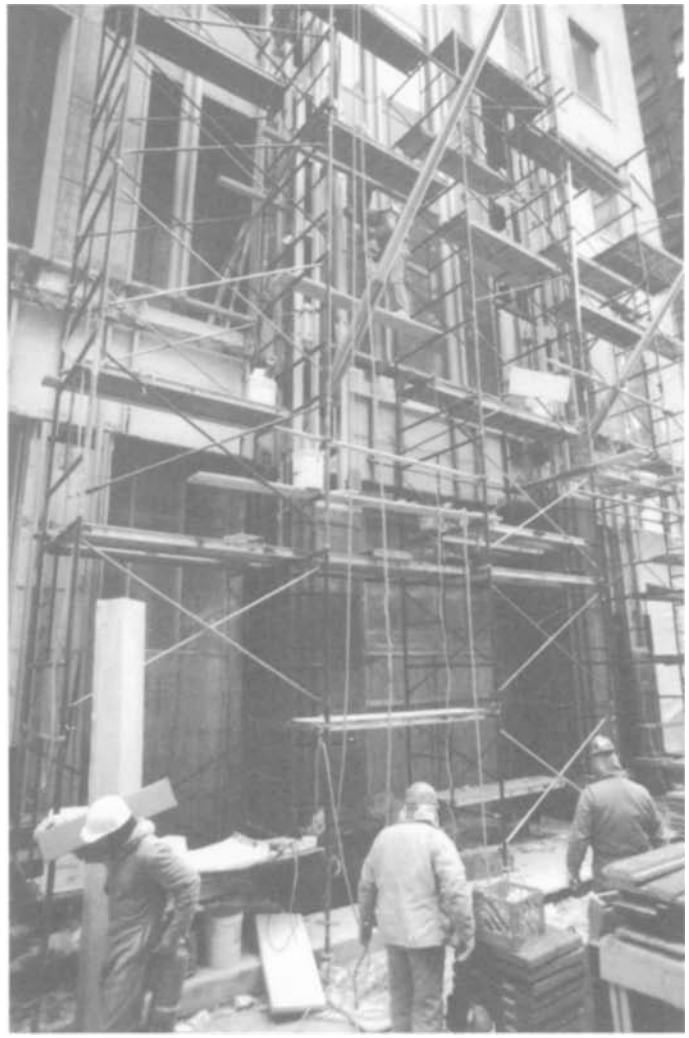


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FIGURE 61: *Ground Floor Entrance Stone Installation.* With the three-floor-height entrance saffolded, workmen on the top levels finalized the alignment of the prefabricated aluminum framing system while setters placed stone onto that framing near the top of the first floor. This is the entrance to the "bussel" connector between A T & T and what is now U S G.

### *Direct Attachment*

Anchorage device engaging and supporting the stone is also fastened directly to the building's structural frame.

### *Subframes*

An anchorage device engaging and supporting the stone fastens first to a subframe that is then attached to the main building frame. Intermediate structure such as curtainwall, strongback, trusses, or precast panels can support stone cladding, which then are themselves attached to the building structural frame. This approach in concept is referred to as panelization. A single subframe might have several stones attached to it along with other exterior wall components as well. Review considerations for subframe system performance to insure proper anchorage device performance. Preserve the same principles outlined for anchorage device performance. Include specific considerations for subframe behavior during assembly and handling to prevent stone damage.

*Commentary:* A facade unit configured to have several stones and perhaps other facade elements (such as windows) pre-attached at a site remote to the building may be engineered to resist design loads with two gravity-and-lateral load supports and then two additional lateral-only load supports that may be located toward the unit's corners. Accepting the support of the backup structure, these units may perform as planned and adequately resist deformities in stresses in their usually vertical orientation.

Depending upon the assembly and transportation means necessary to build and relocate the units to the site, orientations, forces, and supports are likely to be much different than those anticipated in the final in-place position. The relative stiffness of the backup is probably the most vital consideration in unitizing stone panels. A "stiff" panel on the building is not likely that stiff in the shop or over the road unless special provisions, stiffeners, or temporary backup is utilized to preserve and limit the forces and deforming activities that pre-assembly, loading, transporting, unloading, hoisting, hanging, and finalization operations can exert on a unit.

Careful study through each-and-every sequential operation a unit experiences between receiving stone and anchoring it in the shop through reaching its final location on the building must be devoted to the anchorage behavior to assure that structural integrity is preserved.

### *Installation Standards*

Rules or quality standards for installation outline objectives for placement tolerances accepted for the finished in-place stone as well as the anchorage devices and support system installation accuracy required to achieve these acceptable tolerances. Methods of documentation, supervision, and inspection of anchorages to assure that they conform to the design intent and that there is consistent quality is only possible if the designer is aware of the significant need for access, simplicity, and that the designer genuinely comprehends stone anchorage device behaviors through their range of allowable, or possibly placed, positions. Reference the section that addresses these installation standards and the other considerations that influence them.

### *Materials of Construction*

Material quality standards outline different performance characteristics for how to use prospective materials that are often included in the exterior wall system construction. Methods of documentation, evaluation, and assembly to assure that they conform to the design intent and that there is consistent quality is only possible if the designer is aware of the significant

◀ **FIGURE 62: *Cornice Installation At The Fourth Floor.* The cross-section of the cornice was built of five separate angled and offset stones. The multiple-piece cornice at the fourth floor aesthetically separated the lower register from the shaft of the tower. While all of the stone was fabricated from flats slabs, the complicated profile was created by the metal framing behind. Several different types of anchorages were used to attach the stone panels to the framing to make this profile constructible, including Cold Spring's type #31.**

▲
FIGURE 63: *Stacked Stones At The Entrance Columns.* Like the cornice, the column stones were fabricated from flat slabs, and the complicated assembled shape was created by the specially-fabricated metal framing supporting the stone panels. This backup framing hung from the floors above to keep movements compatible with interfacing systems, and additional weight off the utility tunnels beneath the sidewalks. The all-aluminum facade framing system for A T & T was designed by the stone engineer (the author with Industrial First, Inc.) detailed by a drafting specialist (S M Haw Associates), fabricated by Sigma Steel (Bedford In.), and the erected by the Stone Contractor (Industrial First). Cold Spring Granite quarried and fabricated the sunset red, cold spring black, mountain green, and sunset beige granite cladding installed by Industrial First.

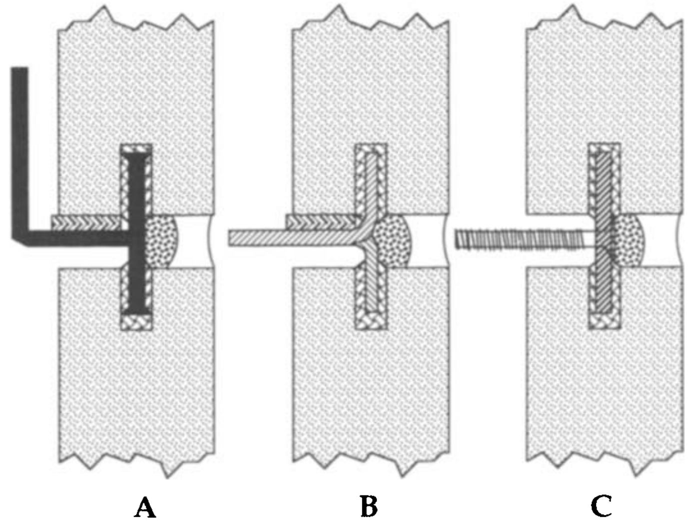


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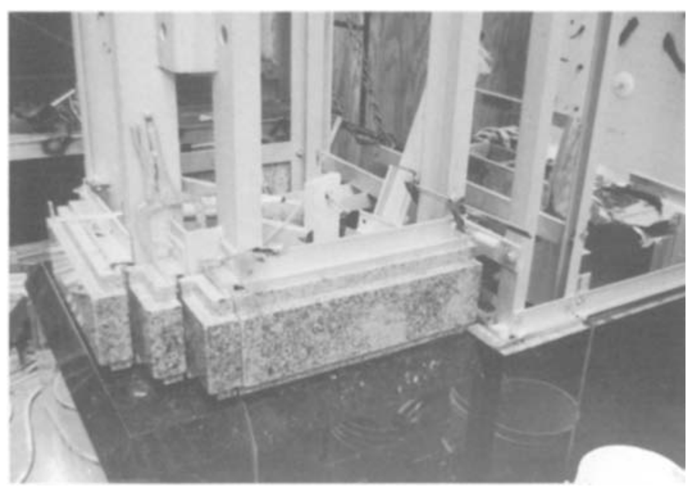
FIGURE 64: *Anchor Types Than Fit Into Kerfs.* Devices that engage a kerf sawn in the stone include an extruded profile (A), a brake-formed shape (B), and a plate, or disc with a fastener tie-back (C). They may be continuous (except C) or intermittent, and typically located in the top and bottom edges due to needs for access and alignment during installation.

FIGURE 65: *(below) Extruded Kerf Anchors At A Column.* Many stones arrange to form a stepped column shape. With each step having a different back-of-stone plane, the backup framing (vertical channels) pre-forms the column shape. The extruded kerf clips attach the stones directly to the framing. The stones are prealigned for better production and lower installation labor costs.



need for durability, compatibility, and that the designer genuinely comprehends stone, metal, and elastomeric chemical and metallurgical behavior through their range of potential environments exposed to through the inception period of construction through the building's life. Reference the section that discusses these material standards and other considerations that influence stone anchorages when used within the stone retention system.

## Basic Anchor Device Types

This *Guide Specification for Stone Cladding Systems* describes the categories of anchors and anchoring systems commonly used to support both the weight of the stone and any loads applied to the stone.

Basic anchor types are categorized according to the mechanical preparation made in the stone that accepts the device.

Behavior mechanics of the stone around the "preparation" are discussed according to how the device engages and contacts the stone.

These following considerations present the design principles involved in selecting those anchorage devices and identify those specific to the device and its attachment, separate from those previously presented regarding the stone material itself. Some fundamentally different anchor types include a kerf, rod, rod-and-plug, tooled rod, and wire tie. These are described in the following paragraphs.

### *Kerf Anchor*

Typically, a kerf is a sawcut groove or slot in the edge of the stone. A kerf clip or kerf bar is a flat bar or thin plate formed, extruded, or otherwise configured to engage a sawcut slot in the stone edge (Fig. 64). Its length is varied to match the magnitude of support reaction required to be resisted at that anchorage location. The interior end of the device is fastened to the support frame (Figs. 65, 66, 67).

### *Rod Anchor*

Typically a round-sectioned component that fits into a drilled hole in the stone's edge (Fig. 68). The component may be a short length dowel that fits through a hole in a strap to carry the load back to the support frame, or the component is long, and bent to fit into a stone hole, with the other end of the rod fastened to the support frame (Figs. 69, 70, 71, 72, 73).

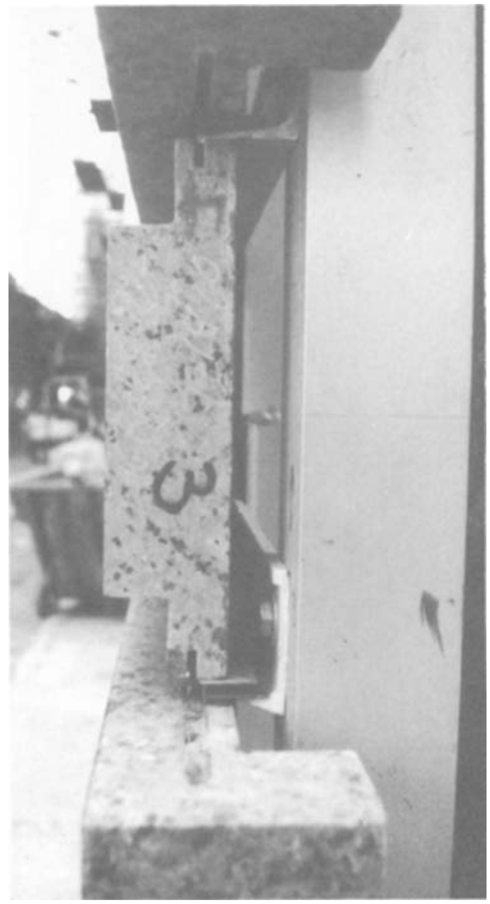


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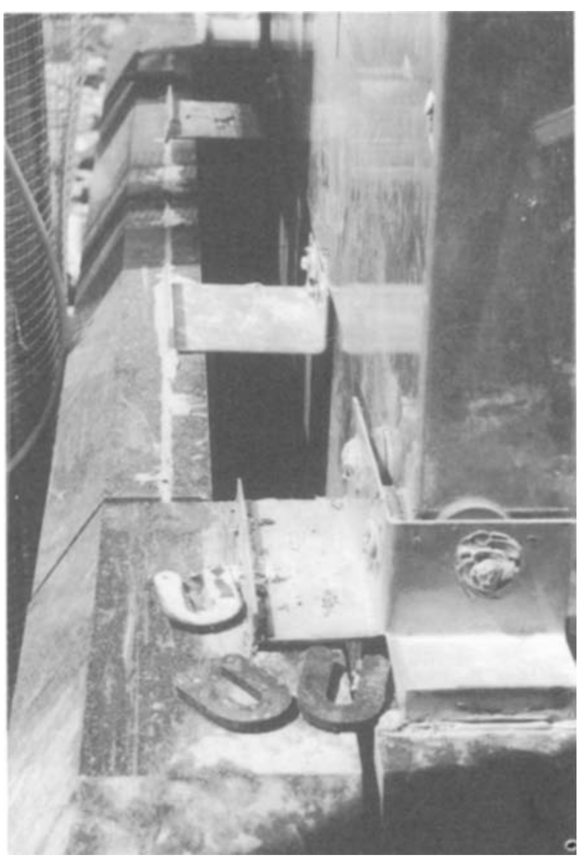
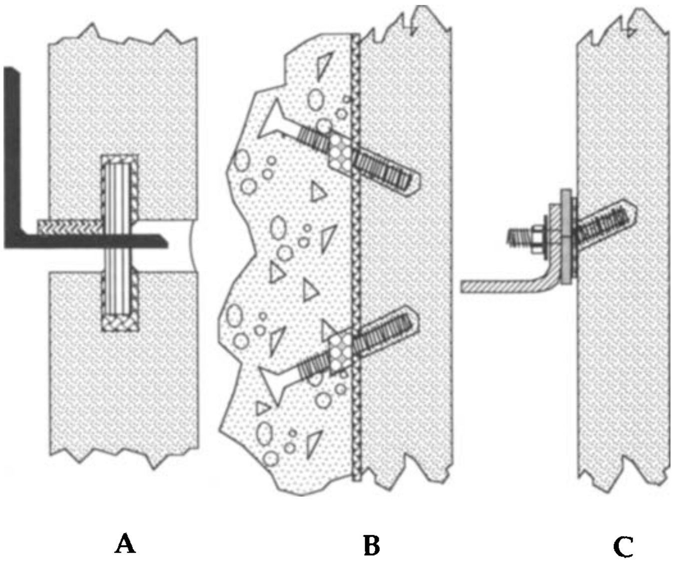
**FIGURE 66:** ***End View of Kerf Anchor In Stone Panel.*** **Using the same system shown in Figure 65, this view shows the stone engaged by the clip at the kerf, and the clip attached to the prealigning backup frame. Note the plastic setting shim used for leveling the stone and the bolted connection between the clip and the backup**



◀ **FIGURE 67:** ***Brake-formed Kerf Anchors At A Column.*** **Similar to the condition shown in Figure 65, this example loses many of that example's advantages because it wasted time and labor due to an archaic approach. Shims behind the anchors show that the framing was not pre-aligned, causing all stones to be individually aligned as set. Separate clips mean all stones were independently attached an aligned. Self-drilling screws perforated the second-line-of-defense which will allow infiltration. Caulk on the fastener heads don't block the path of air and water behind the shims. The finish on the galvanized sheet is destroyed by the fastener. Moisture will cause corrosion and galvanic activity there to eventually "erode" the anchor's attachment. The brake-formed shape fits loosely in the kerf and is not as strong as an extruded shape, though not critical in the thick-stone application shown.**




◀ Figure 68: *Anchor Types That Fit Into Single Drilled Holes.* Devices that engage a single drilled hole in the stone include a dowel typically in the top and bottom edges (A), inclined and opposing pins in the backface (B), and a bent threaded rod epoxied into the inclined hole in the back of the stone panel (C).

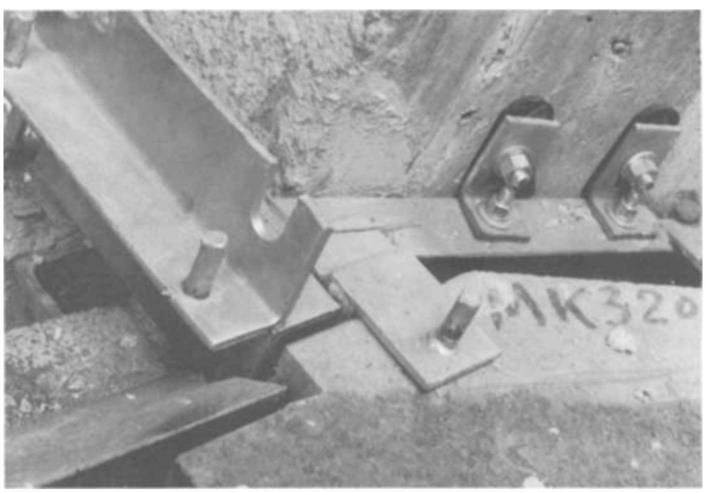



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Figure 69: *Pin Anchorage at Entrance Arch Stone Moulding.* A pinned bracket attaches to the contoured stone moulding to provide positive mechanical engagement. This bracket allows for attachment of equipment during erection as well as final adjustment of the stone relative to the concrete building frame backup.

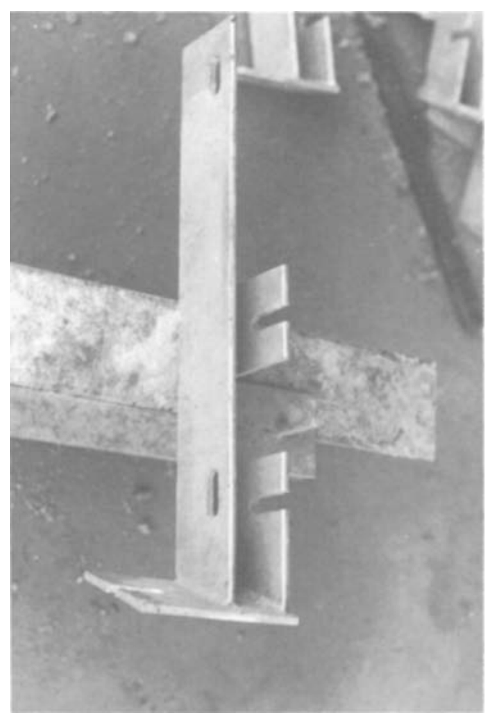


Figure 70: *(right) Pin Anchorage Bracket For Arch.* A custom-fabricated bracket for arch segments allowed pre-assembly of face stones, return stones, and inside arch pieces into proper alignments prior to placement on the wall truss. This method improved quality, efficiency, and adapted better to the structural backup configuration than individually handsetting the work.

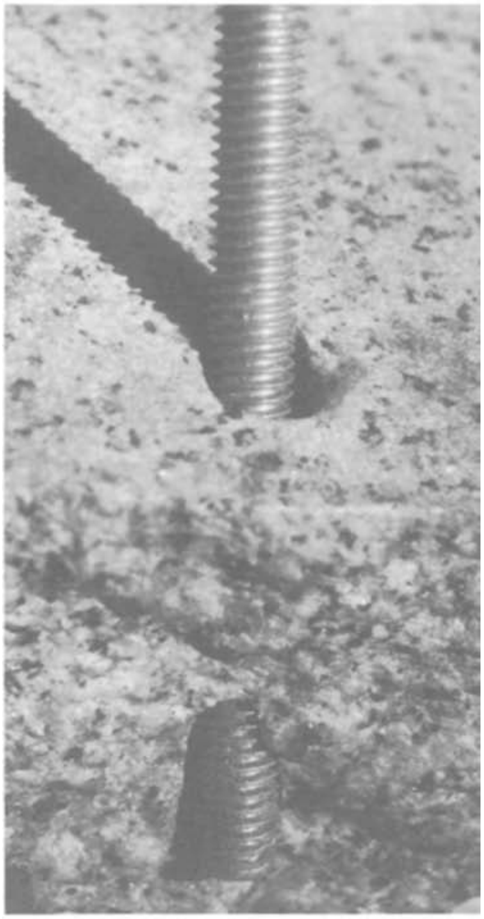
▲
FIGURE 71: *Parapet Coping Brackets To Receive Eisenshanks.* Slots receive eisenshank pins placed in the bottom of the stone coping caps in these parapet brackets. Access space between top-of-wall and bottom-of-stone limited tool and anchor clearance, requiring close pre-alignments. Notice the opposing slot directions that engage stones from the side while still providing adjustment.

▲
FIGURE 72: *Stone Facing For Precast Panels In The Bed.* Stone panels are placed finish-face down to form the bed of the precast panel form. After alignment and sizing, anchorage pins are placed into the inclined-drilled holes in the back face of the stone panel. Rubber grommets on the pins are required to isolate movements between stone, anchor and precast panel backup. A slip-sheet located beneath the grommets against the stone prevents chemical bond between stone an concrete.

## *Rod-and-Plug Anchor*

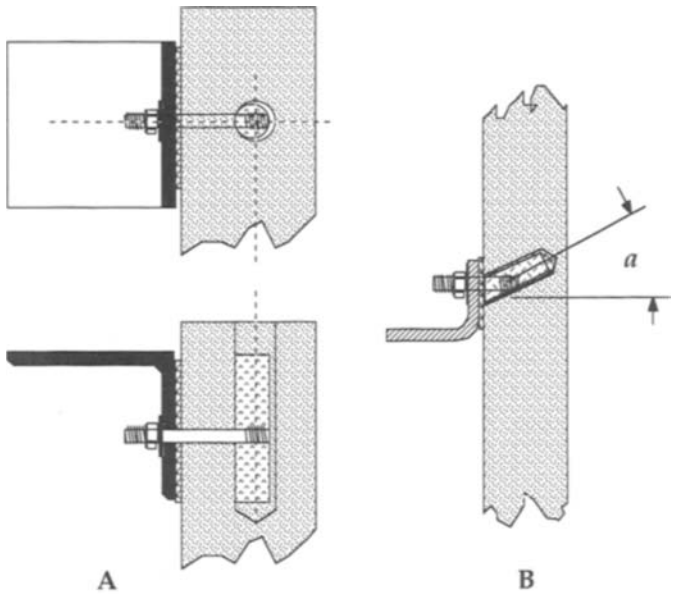
Sometimes also called an "eisenshank," the two engaging parts employ a single short threaded rod, (the "rod") which threads into a slightly larger diameter smooth rod (the "plug") either at a perpendicular or acute angle (Fig. 74). The preparation of the stone to accept rods-and-plugs requires closely coordinated and controlled drilled holes in the stone. If the rod and plug are oriented perpendicular, the hole in the stone for the plug is drilled into the stone's edge, and the hole in the stone for the rod is drilled into the stone's back so that it intersects the plug hole. The rod is inserted through the back hole to intersect and thread into the plug, which has been placed into the edge hole (Figs. 75, 76, 77).

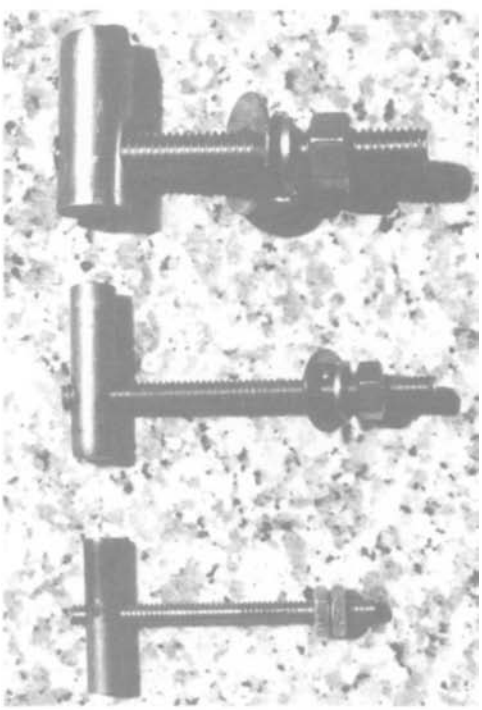
If the rod and plug are oriented in an inclined angle, a single hole is required to be drilled into the face of the stone where the threaded rod is to protrude from, with the pitch of the hole equalling the incline of the rod-and-plug. The depth of that hole must be sufficient to allow the threaded rod, once passing through its plug, to "lock" into the stone by wedging the plug against the hole wall. Because of the pre-stressing of the stone by the anchor by its installation before load is imposed, and because the concentrated prying could easily result from loads which would tend to twist the rod, use of this single-hole inclined rod-and-plug is appropriate for pullout-oriented lateral restraints only.



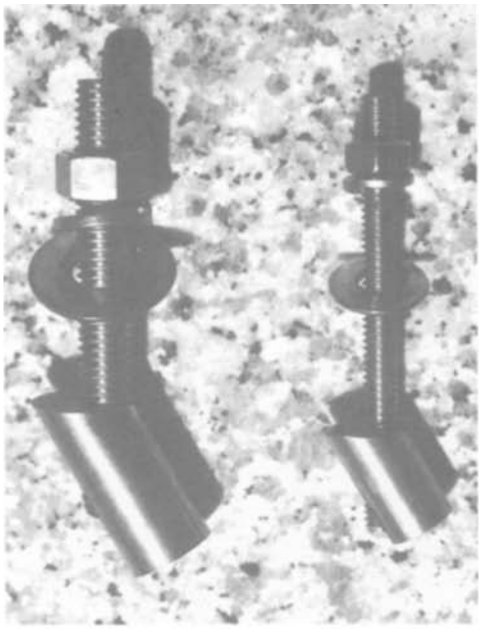
◀ FIGURE 73: ***Inclined Dowel Anchor In Stone.*** Where stone panels are supported on precast panels or will be secured to trusses with grout pockets, a threaded rod engages an inclined hole drilled into the back of the stone. Take care to attain sufficient engagement, yet maintain sufficient cover on the exposed face. Bed the rod in compressible fill, sheath the back of the stone with a bond-breaking slip-sheet, and use of rubber grommet on the threaded shank for proper anchor performance.

FIGURE 74: *(below)* ***Perpendicular Rod-And-Plug Type Anchor Diagrams.*** A perpendicular rod and plug, called an eisenshank, can be installed in opposing holes (two required) near the edge and back of a stone (A). An inclined rod-and-plug can engage the back of the stone away from its edge (B). Angle "a" is usually about 30 degrees and locked by the rod threaded snugly against the stone. Close fit and proper installation is required for positive performance.



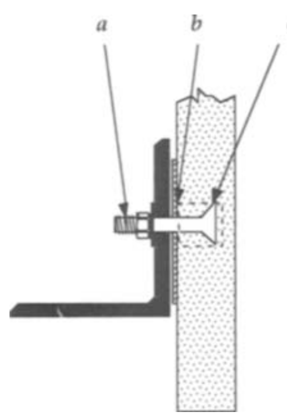
◀ FIGURE 75: ***Perpendicular Rod-And-Plug Type Anchor Devices.*** Three sizes of eisenshanks (3/8-9/16, 1/4-3/8, 1/8-1/4) rod-plug sizes. A threaded rod threads into a hole tapped perpendicular into the side of the smooth plug like a bolt fits into its nut. This anchor requires two separate holes, where the plug typically fits into the edge of the stone, and the rod fits into the plug through the hole in the back to lock into the plug. A deeper-set plug will engage more stone and potentially increase capacity. Close alignment and depth control between the holes drilled for the rod and plug is essential for the two components to meet. Some installation difficulty exists while attempting to align the hole in the plug with the threaded rod once inserted into the hole. The plug hole in the stone is sometimes slightly oversized to allow for some adjustment by an awl through the rod hole during alignment.



◀ FIGURE 76: *Inclined Rod-And-Plug Type Anchor Devices.* Two sizes of inclined plugs with shanks (3/8"-9/16", 1/4"-3/8") where the rod engages the plug at a 30-degree angle. The threaded rod threads into a tapped hole drilled at a 30-degree off-axis angle in the sloped end of the smooth plug like a bolt fits into its nut. This anchor requires one hole drilled into the back of the stone at an angle that matches the rod-and-plug. The threaded rod, or shank, protrudes perpendicular from the stone. The plug is locked into place by the threaded rod wedging against the stone as it tightens into the plug and wedges it against the other side of the hole. Close fit between plug and its hole even before it is tightened is essential. This type of anchor is more highly susceptible to dislodging due to its single-hole installation.

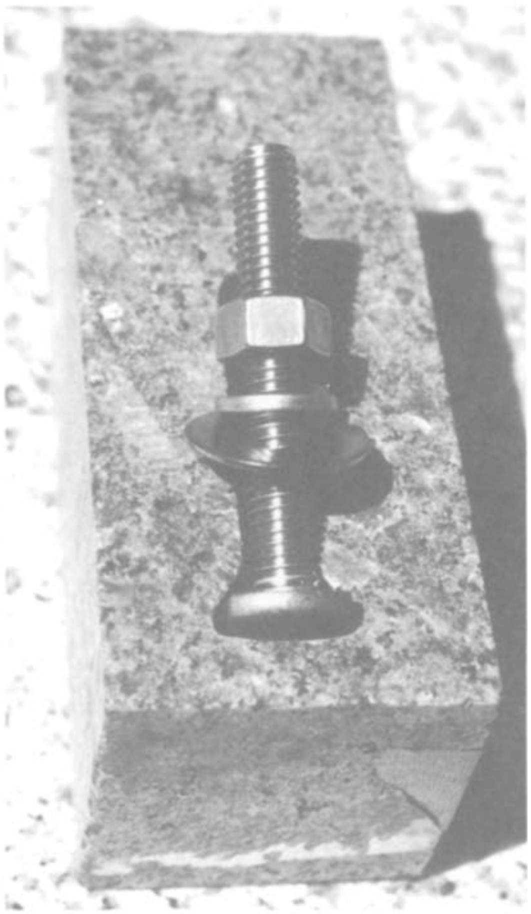
FIGURE 77: *(below, left) Installed Inclined Rod-And-Plug Anchor.* The plug fits snugly into the drilled hole. Slight tightening of the rod locks the plug in place against a relatively small failure cone surface within the stone.

FIGURE 78: *(below) Headed Shank Anchor Diagram.* This anchor type is characterized by a special head forged on the end of a threaded shank which fits into a matching slot in the stone. Mechanical engagement occurs when the special headed, or tooled rod (a) fits into a contoured groove (c) routed into the stone with a high-speed custom diamond tool. Any mounting fixture is best separated from the stone with a threaded washer, sometimes known as a stress-less disc (b), which maintains the shank's proper perpendicular alignment and seating in its groove. The routed groove should be oriented sideways or opposite the line of force to avoid disengagement by loads in the plane of the engaged face. The pictured anchor avoids the disadvantages of traditional types that expand in a hole with a wedging action against the stone. The wedging action maintains a high prestress in the surrounding fragile stone material which dramatically reduces long-term capacity and safety. "Kone" and "wedge"-type expansion bolts increase radial pressure as pullout reactions increase, multiplying internal shear stresses. Carrying in-plane loads such as the stone's weight further aggravates this undesirable condition.

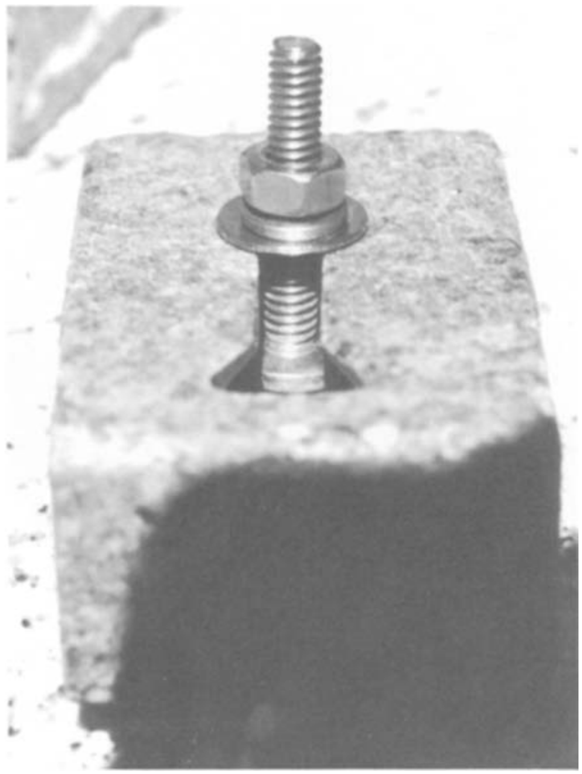



## *Tooled-Rod Anchor*

Typically a bolt-type device with the "head" of the bolt tooled to fit firmly into a patterned slot routed into the back or edge of the stone (Fig. 78). The threaded end is fastened onto a connecting device such as a clip angle then back to the supporting frame, or it can attach directly back to the supporting frame (Figs. 79, 80, 81, 82).



◀ FIGURE 79: ***Headed Shank Anchor.*** **Also popular as a "type #31" patented by Cold Spring Granite Company, this effective device uses the advantages of an eisenshank's large potential failure cone and an inclined rod-and-plug's placement anywhere in the stone. A special diamond bit routes a slot into the stone, and an oblong head of the shank slides into the slot. The type #31 provides positive mechanical anchorage without strict dependence upon double-drilled hole align ment, torque, and rigidity of the supporting structure maintaining the shank's stability.**

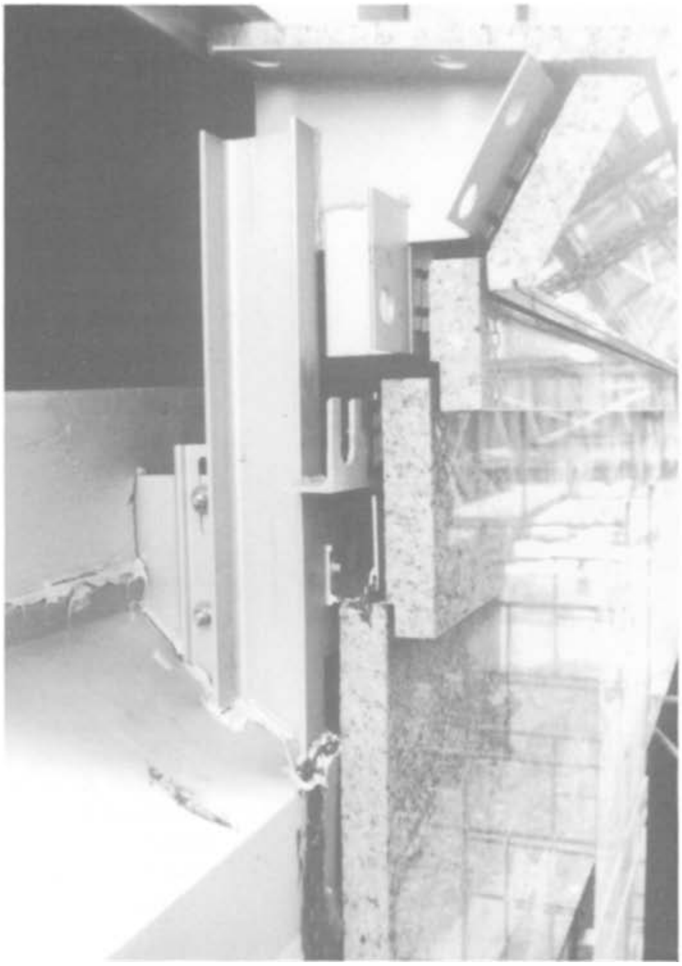


◀ FIGURE 80: ***Installed Headed Shank Anchor.*** **Fitted properly into the routed slot, the shank extends perpendicular from the back of the stone. Always orient the routed slot 90° of the direction of the reaction's force to prevent potential disengagement.**

## *Wire Tie Anchor*

Typically a heavy gage wire, perhaps #8, of a ductile metal such as copper or annealed stainless steel is formed into a "gooseneck" that has a hook (Fig. 83). The hook end passes through a hole angling into the stone to penetrate the back and edge, allowing the wire hook to be drawn back and wrapped below the gooseneck to form a loop. An epoxy (not polyester) or waterplug mortar, not plaster, is used to fill the hole, fixing the loop in the stone which also keeps moisture from collecting in the hole to potentiate freezing or corroding, which could expand and break the stone.

The gooseneck is layed into a pocket or reglet in the backup, usually concrete, which itself is packed with epoxy or waterplug mortar for the same reasons. Stone position must be fixed secure until the packing matrix cures to strength. Applications are generally limited to low-rise or storefront because of the typical variability associated with the anchor in its drilled stone preparations, wire loop-and-gooseneck configurations, and irregularities of the pocket or reglet and its packing.



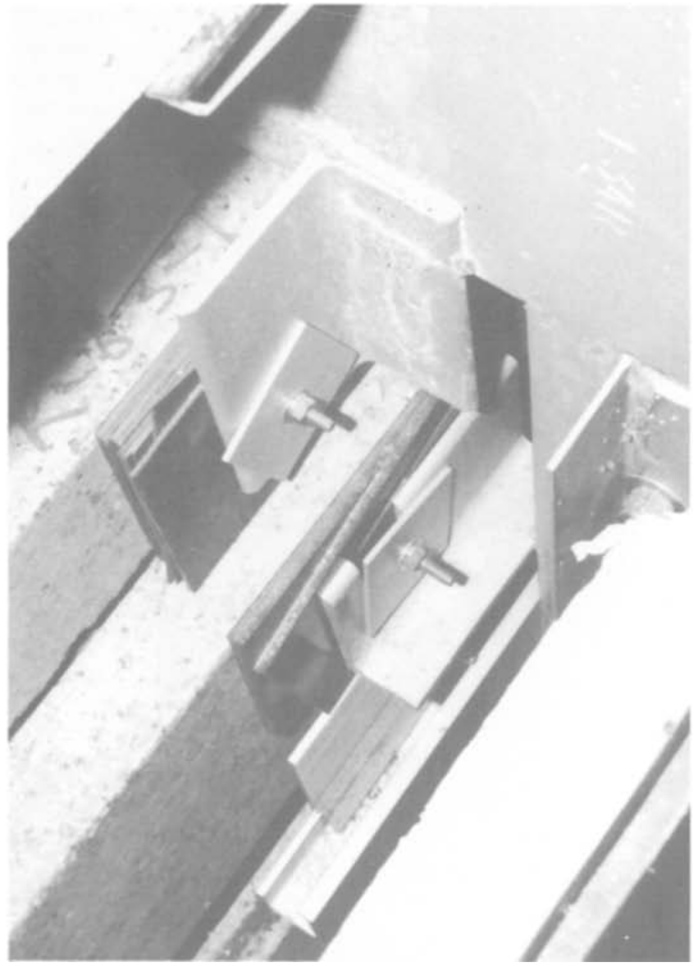


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FIGURE 81: *Cornice Assembly.* A single fabricated frame supports opposite sides of twelve individual stones with a variety of kerf, pin, and type #31 headed-shank anchors.
The oddly-shaped shop fabricated metal frame met the back-of-stone to form the complicated finished front face-of-stone cornice and fascia configuration built from flat slab stock. Different anchors were used to meet requirements of the stone's size, location, access during setting, and sequence of installation. Alignment of the frame pre-aligned the stones for out-of-plane position. Setters then only had to adjust for side-to-side locations. Properly matching anchor types to stone installation requirements allowed the installer to capitalize on the frame's pre-aligned accuracy by tripling typical stone installation production rates because individual stone adjustment was virtually eliminated. Jigs in the metal fab shop expedited the framing, saving costs on its manufacturing.

FIGURE 82: *(above, right) Backside View of Cornice Anchorages.* Washer plates fit over the oversize holes in the backup frame to accommodate stone anchor drilling tolerances combined with finished frame fabrication and erection tolerances. The flashing directs cavity moisture into the internal gutter to be weeped at the window heads.

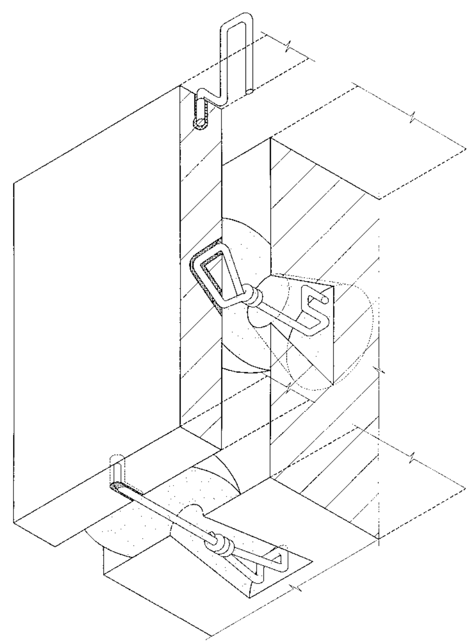
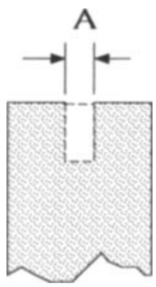
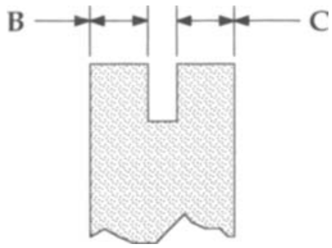


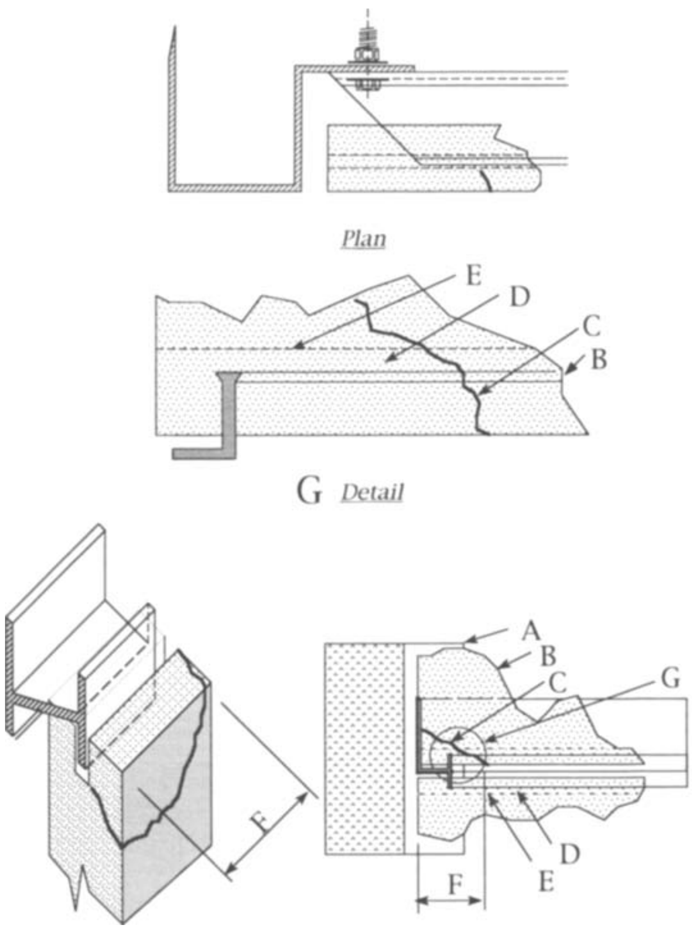
FIGURE 83: *(right) Wire Tie Anchor.* In low-rise lateral tie-back anchor conditions with cast-in-place concrete or masonry backup, wire ties may be appropriate to anchor the stone. Top and side views (bottom diagram) show the wire loops or hooks locked into the concrete. A dovetail or flared cone (a) drilled into the concrete captures a mortar or "waterplug" (fast-set portland cement mix) spot (b) which wedges into the cone. A heavy gage copper or ductile stainless steel wire (c) engages intersecting holes drilled into the edge and back of stone. The anchor installation is similar to traditional interior applications. The exterior exposure, however, requires durable spot material instead of plaster of paris. The drilled holes in stone are filled with sealant to avoid moisture damage, and the gooseneck's embedment in the backup must be deep and "wedged" enough to resist imposed loads in addition to those related to stone panel stability.




◀ FIGURE 84: *Kerf Slot Width.* Control the location and width of the kerf slot (A) sawn in the edge of the stone. Maximize the strength of the remaining fins by locating the slot in the middle third of the stone's thickness, adjusting front-to-back dimensions for unbalanced lateral loads or architectural considerations. Remember that fabricating tolerances will vary both overall thickness, placement, and width. Measure widths with inside calipers to verify conformance with realistically specified and attainable limits.




▼ FIGURE 85: *Kerf Fin Width.* The slenderness of the stone fin remaining on the panel edge after the kerf anchored assembly determones its potential capacity. Where side (B) is the exterior fin, the width should be held relatively constant to keep exposed finished faces aligned. The groove should be guaged from the finished face to maintain flush exposed faces. Side (C), as the interior fin, varies with the accumulated variance of the exterior fin width (B), the kerf groove, and the overall slab thickness. Slab thicknesses may range between +/-1/8 inch (+/-3mm) for nominal 1-1/4 inch (3cm) thick material and perhaps +/-1/4 inch (6mm) for 2 to 3 inch (5-8cm) thick material. The interior fin usually resists wind loads causing negative pressure, or suction towards the outside, plus the panel's weight by bearing upon a ledge support. In most cases, lateral suction typically exceeds inward positive pressure from wind. Take care that the narrowest fin remaining after the tolerances combine remains substantial enough to be structurally safe. Avoid problems with the interior fin, for any damage to it is concealed in the cavity, and fracture could result in the stone separating from the facade.




Isometric at End

Elevation

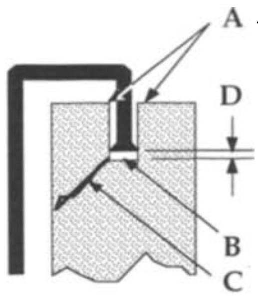
◀ FIGURE 86: *Effective Length of Engagement.* An anchorage will only provide support for the stone where it provides resistance. The anchor provides resistance where it is stiffer than the stone body it is supporting. The anchorage will not provide support where it more flexible than the panel even if it is physically connected or embedded in the stone. The anchor's effective resistance results from where the device is attached to its support, and its cross-section's moment of inertia (measuring stiffness) relative to the stone panel it engages. "Effective" length of engagement is the length of anchor actually providing resistance, and thus support, to the stone. The diagram's components include the facade framing (A) that supports the kerf anchor, which engages the stone panel (B) with a sawn kerf in its edge (D). Maximize engagement to minimize the prying distance (D) to the kerf root (E). The kerf fin ruptures along an interior plane (C) where the anchor is stiffer than the stone and ultimate stresses develop due to that resistance. Total length of this plane (F) is called the effective length, and results from the complicated interactive behavior of load, anchor, support, and stone. Increasing this length increases the surface area of the failure plane where the ultimate stresses occur, and thus increases the capacity of the anchor.

## Proper Application and Optimization of Kerfs

Kerf anchor design considerations outline the objectives that designers focus upon when conceptualizing kerf-type anchorages and also when determining when to use a kerf in a certain application.

Structural capacity of kerf anchors is limited by the combined shear and flexural strength of the stone fin remaining after sawcutting the kerf into the edge of the stone that the kerf clip anchorage device fits into. Parameters affecting the capacity follow.

1. Kerf slot's maximum width: (Fig. 84)
   The distance across the sawcut varies due to original sawblade thickness, the blade's trueness of plane and rotation, and degree of wear, which, over time results in a thinner blade which also tends to wander.




◀ Figure 87: *Maximize The Anchor's Depth of Engagement In The Kerf.* How the anchorage device engages the stone determines how the stone anchorage assembly will perform. Stone material strength and fin size directly influence the stone's potential capacity at the anchor. Anchor contact with the kerf and the effective length of resistance along that contact directly influence the transfer of the panel's surface pressure to concentrated reactions at the anchor locations. Prevent contact of the anchorage against the toes of the kerf fins (A) where the prying force on the fin is greatest due to the maximum leverage arm distance to the root. Instead, minimize the distance (D) between the anchor's contact point and root or bottom (B) of the kerf to minimize prying against the eminent failure plane (C). This plane is the "surface" within the material where maximum combined diagonal shear and tensile stresses develop and eventually rupture occurs. It is sometimes also called the potential plane of influence.

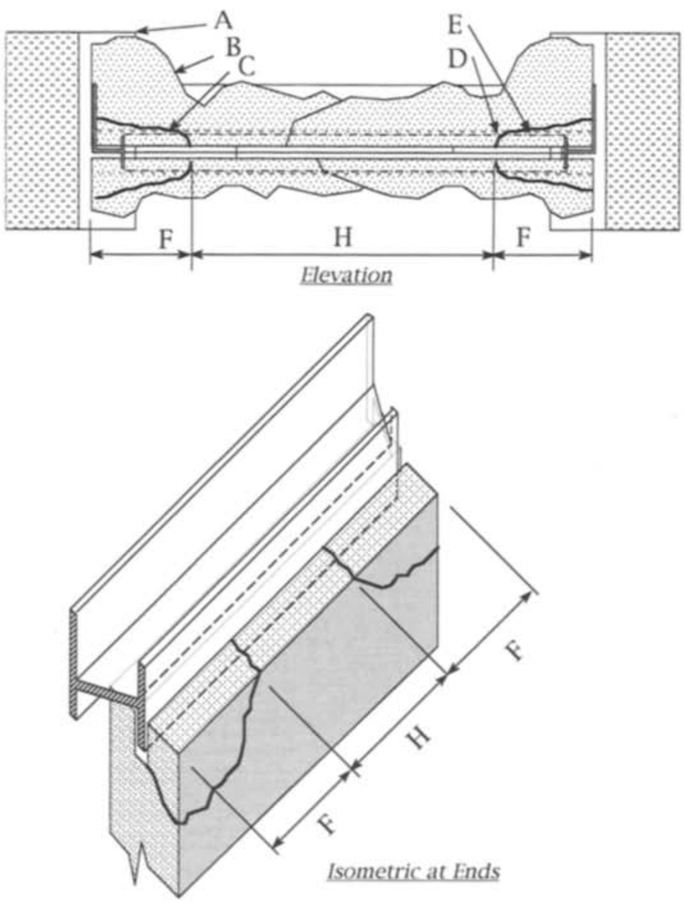



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Figure 88: *Effective Length Of Engagement At Continuous Kerf Rail Anchors.* Further to the mechanics described in Figure 860, an anchorage rail that engages the stone continuously will only provide support for the stone where it provides resistance. The anchor is stiffer than the stone where it attaches to the facade framing (A) at its ends, which corresponds to the stone panel corners (B). Maximize engagement to minimize the prying length (D) to the kerf root (E). The potential planes of failure (C) develop where the stone is transferring its load to the ancho. Effective lengths (F) occur at both ends of the rail, resulting in an "ineffective" distance at midwidth that engages the stone, but does not transfer load back to the facade framing because it is more flexible than the stone panel.

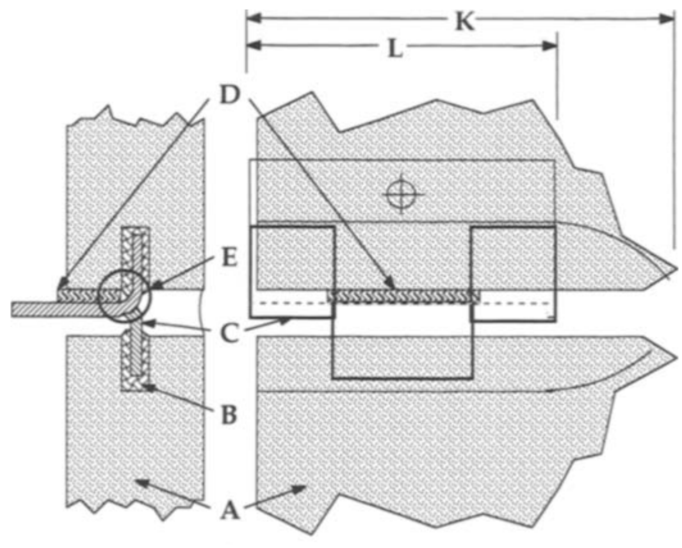


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Figure 89: *Formed Split-Ear Anchor.* Also called a split-tail anchor, it may be the most common and simplest anchor device used to engage a kerf slot. It is fabricated from sheet metal stock in a brake-press which folds tabs in opposite directions to fit into stones on either side of the anchor. The stone panel (A) is prepared with a sawn kerf slot (B) and filled with a compressible filler (usually sealant) to receive the tabs from the split-ear anchor (C). A setting shim (D) bears the weight of the stone panel above to level the stone and hold the toe of the kerf fin above the corner (E) of the folded metal. Actual anchor length (L) is different from the engaged length due to the different lengths of the folded tabs on either side if the anchor. Where kerf slots are not continuous, the kerf's sawcut length (K) must be sufficient to achieve full depth at the location of the anchor, and is dependent upon the blade's diameter.

2. Stone kerf fin's minimum thickness: (Fig. 85)
   The distance from inside face-of-kerf to face-of-finish varies due to guaging of stone panel from sawblade cutting the kerf, stone panel thickness, actual flatness of the panel itself, and all the same variables that influence Figure 84.

3. Length of kerf clip's leg actual engaged: (Fig. 86) The distance of anchorage device engagement into the stone kerf is dependent upon the fabricated length of the clip's engaging leg. Note that the actual engagement length is not necessarily the effective length of engagement.
4. Depth of contact: (Fig. 87) The depth where the kerf clip's leg contacts the stone within the stone kerf depth is the depth of contact. It is determined by the variances in the overall panel size that changes its location relative to its supporting anchorages, the depth of sawcut kerf, the tolerances involved with installing the kerf clip device, the trueness of positioning of the kerf clip into the sawcut slot in order to preclude prying or point contact at the toe of the kerf fin instead of the kerf fin's root. It is most desirable to minimize the leverage distance between the root of the kerf slot and the point where the anchorage device contacts the stone within the kerf slot. The objective is to maximize depth of contact depth, which maintains minimum leverage distance, and thus maximizes capacity;
5. Minimum stone material strength: Known as the actual material strength at the most critical contact point at the kerf's root and through potential failure plane, which predictably radiates at about a 45 degree angle from the point of highest bending moment at the root of the kerf (unless a material weakness occurs elsewhere. This plane of influence has been verified by tests.
6. Distribution of the kerf clip's leg's contact: (Fig. 88). Anchor device engagement into the stone alone does not constitute effective support of the stone along that entire length of engagement. The continuous kerf rail in the stone edge does not function as a continuous support for the stone unless the kerf rail is stiffer than the stone.

   This aspect measures the effective length of contact of the kerf clip along the engaged length of the stone panel. It is controlled by the stone panel's and kerf clip's relative stiffnesses away from the clip's support or attachment to its backup. When the realized stiffness of the support rail (a function of where the rail is supported, the cross-section's moment of inertia, and the material's modulus of elasticity) is less than the stone panel's stiffness (a function of the thickness and the stone's modulus of elasticity) across the span of the stone, the entire length of the continuous kerf rail will not act as a support. The ratio of the kerf rail's stiffness and the stone panel's stiffness as well as the locations of the support behind the rail will determine the effective engagement or effective support length;
7. Actual capacity: The true capacity of an anchor is difficult to accurately predict mathematically. It should be determined with tests specific to the actual kerf component engaged in the stone for the project. A continuously engaged kerf clip along a full edge of an entire stone does not mean that full length is effective, or actually supporting the stone. Relative stiffnesses of stone, anchor, and backup control how much of the anchorage device is actually resisting load, transferring load from the stone's kerf leg to the kerf clip's leg which the stone bears upon, and thus is effective.

   Reference *Anchor Capacity and Effective Engagement Length Test* in section 7 for specific considerations

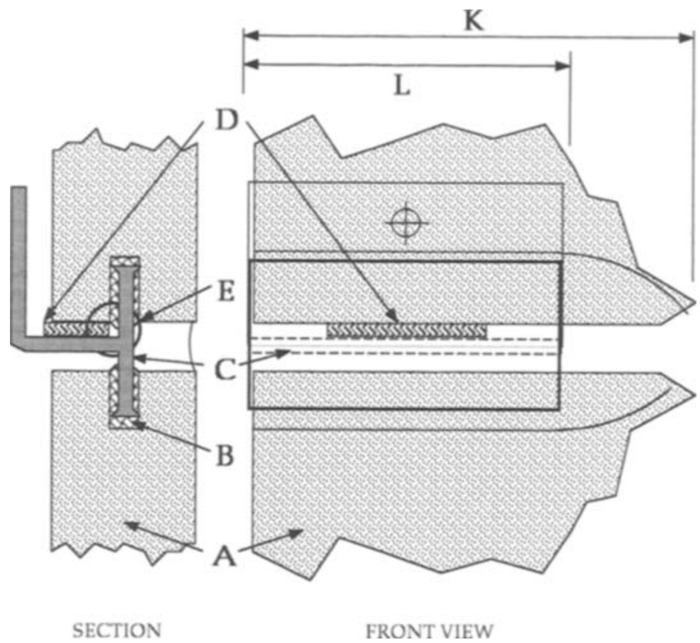


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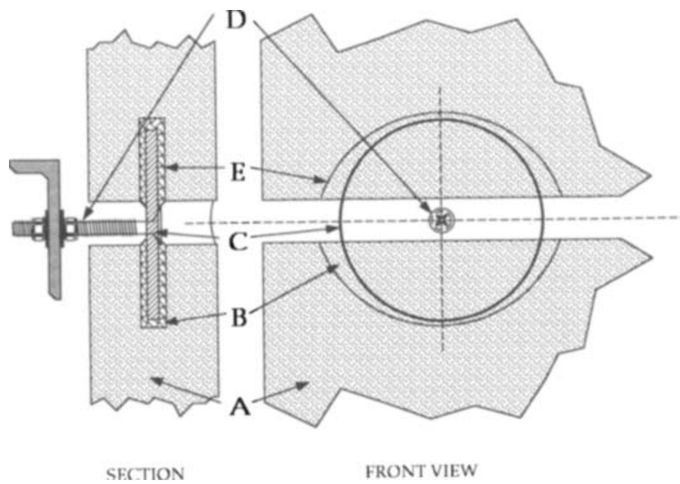
**Figure 90: *Extruded Anchor.* The cross-sectional profile of an extruded anchor can be articulated to optimize both the material used and more importantly, the interactive mechanics between anchor and stone. Typically aluminum, and sometimes stainless steel is used, though stainless steel extrusions are very difficult to obtain and costly. Features not possible by punching and brake-forming can be achieved in the extruding process. Extruded sharp inside corners (E) prevent point contact of the "toe" of the kerf with the anchor to avoid leveraged prying on the stone fin. Anchor leg thicknesses can be varied to match stresses and minimize material. Nibs can be added to the tips of the engaged anchor legs to assure that contact occurs between the anchor and the stone near the "root" of the kerf. This minimizes prying leverage and maximizes capacity even when slight rotations, misalignment, or lateral deflection due to loads or installation occur in the engaging tab. Because both top and bottom tabs are the full length of the anchor, actual clip length (L) equals the engaged length for both stones. (The profile features derived by these principles are patent pending.) Like the split-ear anchor, the stone panel (A) is prepared with the sawn kerf slot (B) and filled with a compressible filler (usually sealant) to receive the tabs from the extruded anchor (C). A setting shim (D) bears the weight of the stone panel above to level the stone. Where kerf slots are not continuous, the kerf's sawcut length (K) must be sufficient to achieve full slot depth at the location of the anchor, and is dependent upon the blade's diameter.**

FIGURE 91: *(right) Shank With Disc Anchor.* Disc anchors may be appropriate lateral supports for thicker stones (A) of moderate size. Anchor capacities are directly proportional to the surface are of the potential plane of failure, thus thicker stones require less "effective length" (line along E) to develop necessary surface area. Radial slots (E) cut within the length of the stone do not create edges and corners which are vulnerable to breakage from handling. However, slot locations must be closely coordinated with locations of the disc (C) where it fastens to the supporting backup with the shank (D). The fit of the disc in the slot must also accommodate the tolerances that accumulate there by a slight oversize (B). Accept required adjustment where the shank (D) attaches to the backup to maintain as close a fit as possible between disc and slot to minimize prying and lost capacity.



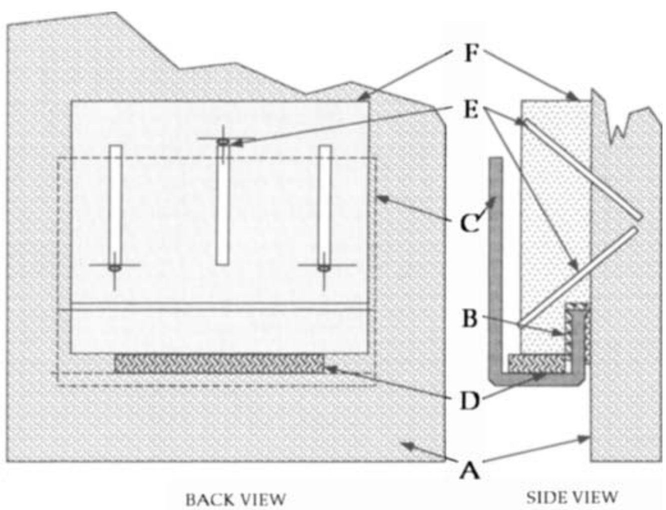




FIGURE 92: *Kerfed Liner Block Anchor Diagram.* In conditions where the architectural configuration or stone arrangement prevents engaging the bottom edge of the stone, or not enough of the bottom edge is covered by the joint to conceal the anchor, a separate block is laminated to the back of the face stone panel to provide engagement for the anchor. A liner block is the only method that observes the fundamental principle to bear the stone's weight at a point-of-contact separate from that of its lateral restraint in this condition. Pin, kone, headed shank, and rod-with-plug anchors by themselves do not separate gravity and lateral forces into determinant, predictable supports. The liner block (F) is fabricated into sufficient size to transfer its reaction into the anchor (C) through its pins into the face stone (A). Fill the kerf slot (B) in the liner with a compressible material to cushion contact and use bearing shims (D) to level the stone and/or avoid "toe" contact, depending upon whether split-ear or extruded anchors are used. Attachment of the block (F) to the face stone (A) is a critical connection requiring *both* durable adhesive and mechanical means. Opposing dowels (E) mechanically lock the block onto the back-of stone, and adhesives chemically bond the separate stones together and seal out moisture. Primary dowels (bottom) are pitched in the direction of the load to prevent disengagement, while secondary dowels (top) are pitched opposite to lock the block into place. Secondary dowels may not be needed if primary dowels are toenailed horizontal to prevent disengagement.

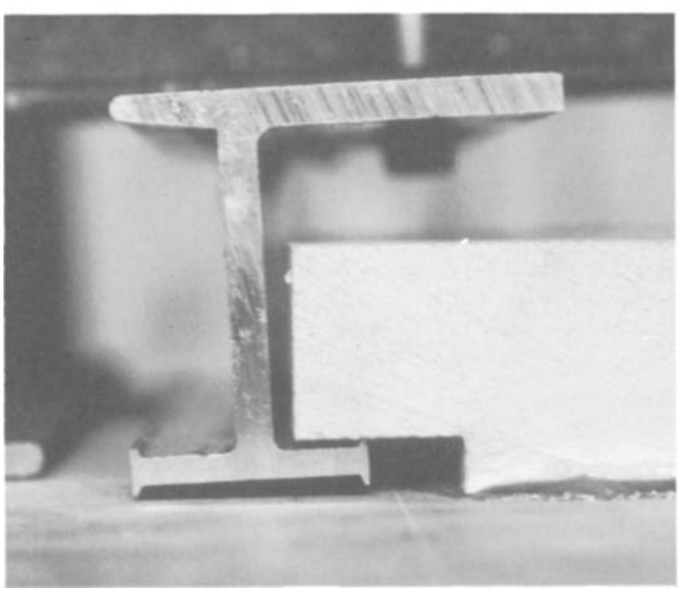
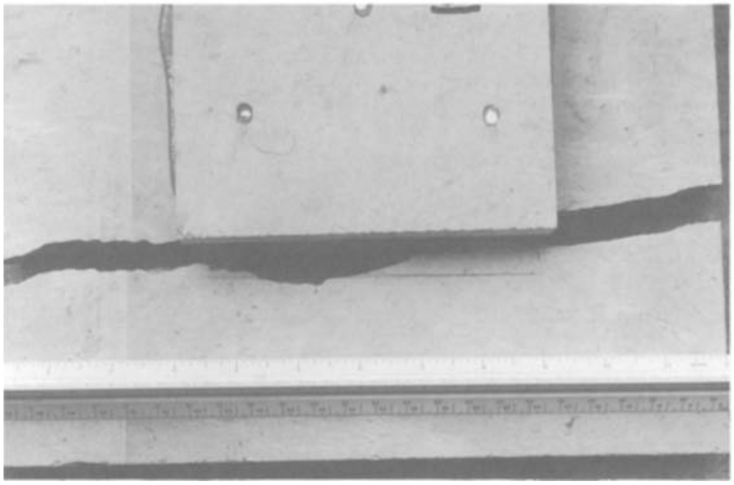


FIGURE 93: *Kerfed Liner Block Anchor.* Side view of the anchor engaging the block attached to the back of the face stone in a soffit condition.

required for an independent kerf anchorage test, which is intended to verify the capacity of the anchor itself as well as its effective length of engagement within the kerf slot.

Example Applications of the Kerf Clip and Kerf Slot

1. Brake-Formed Kerf Clip Anchor (Fig. 89)
   A brake-formed clip is commonly called a "split-ear." It uses a brake-formed sheet metal component to engage the stone, of appropriate non-ferrous and non-corrosive material, of sufficient guage to provide adequate strength, and formed with accuracy to control undesirable point contact at the "toes" of the stone's kerf fin.
2. *Extruded Kerf Clip Anchor* (Fig. 90)
   An extruded kerf clip or rail uses a custom-shaped



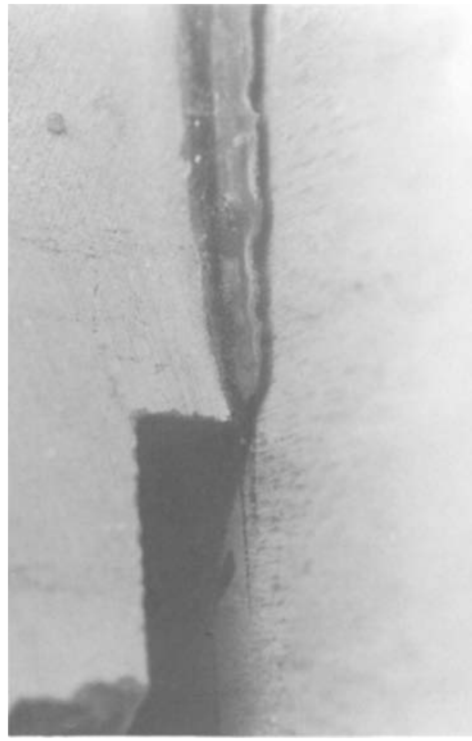


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FIGURE 94: ***Properly Designed Liner Block.*** **Back view of the liner block attached to the back of the fractured face stone. The liner's capacity should exceed the strength of the face stone to avoid concealed damage behind the facade and prevent separation of the panel from the building. Note the three dowel pins visible in the block. The bottom pins are primary dowels that carry the load between the stones, the single top pin is the secondary dowel that physically locks the block onto the face stone. Proper pin installation should show the pins' ends flush with the back face of the liner block without having been ground off after insertion.**

FIGURE 95: ***(above, right) Liner Block Detachment From Face Stone.*** **In this close view of the seam where the liner is laminated to the back of the face stone, the block separated from the face stone due to excessive load. While the lamination should remain intact for the full capacity of the face stone, the mechanical embedment of the pins prevented the block from becoming totally separated even after the epoxy adhesive failed. Pins must be precut to proper length and fully inserted into their holes through the liner block and into the face stone to assure proper embedment.**

extruded component to engage the stone. It is constructed of appropriate material alloy, wall thicknesses, and profile shape to eliminate undesired point contact and to promote desired contact as deep into the stone's kerf slot as possible.

3. *Shank-with-Disc Slot Anchor* (Fig. 91)
   A shank-with-disc slot is commonly used in thicker module stones such as limestone, and for lateral support only. The device uses a round metal disc of appropriate metal material, guage, and size to engage a deep sawcut "slot" rather than a continuous kerf in the stone, primarily because these types of stone panels are too large to be retained on light framing or curtainwall mullions perhaps like some granites. The disc is attached back to the framework by a threaded shank.
4. *Kerfed Liner Block Anchor* (Figs. 92, 93, 94, 95)
   A kerfed or slotted liner block is sometimes used when the bottom edge of the stone is not accessible to an anchorage device, either because that edge is exposed to view, or other component interferes. A stone block of the same material as the face stone is attached to the face stone with inclined-and opposing pins arranged to mechanically prevent the liner from dislodging. The block is also laminated with chemical adhesives such as epoxy for redundancy. Block and kerf sizing is accomplished identically to the previous brake-formed or extruded kerf clip examples.

### *How Kerf Anchors Secure the Stone to its Supporting Structural Backup*

A kerf clip or rail retains the stone panel onto the structural backup framing by engaging a kerf cut into the stone's top, bottom, or side edges, or a combination of these edges. The portion of the anchor inserted into the slot accepts load from the stone by bearing against the kerf walls. As has been discussed, the location of actual bearing of the kerf clip within the kerf slot is extremely influential as to the capacity of the anchorage.

### *Conceptual Analysis of the Kerf Anchor*

Follow the theoretical capacity analysis of the anchorage concept with actual testing of that device. This proves that the analysis was accurate and to confirm the actual capacity of the anchor. In a relatively consistent material, the cone (or surface)-of-influence (or tension/shear failure plane) emanates at an approximate 45 degrees from the root (bottom) of the stone kerf slot below the point-of-contact with the kerf clip. Depending upon the mineral composition of the specific stone being considered, an adjusted assumed plane may be required for preliminary conceptual mathematical analysis.

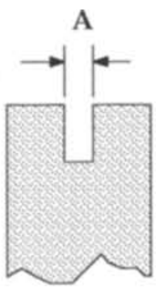



FIGURE 96: *Dowel Hole Diameter.* Control the location and diameter of the hole drilled in the edge of the stone to receive the dowel (A) . Maximize the strength of the remaining stone surrounding the dowel hole by locating the hole in the middle third of the stone's thickness, adjusting front to back for unbalanced lateral loads or architectural considerations. Remember that fabricating tolerances will vary both overall thickness, placement, and diameter. Measure diameters with inside calipers to verify conformance with realistically specified and attainable limits.

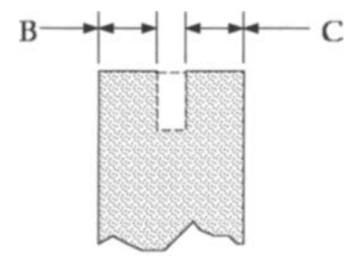



FIGURE 97: *(right) Edge Distance to Dowel Hole.* The volume of the stone remaining on the panel edge after the hole for the dowel is drilled determines the potential capacity of the stone portion of the anchorage assembly. Where side (B) is the exterior edge distance, its width should be held relatively constant to keep exposed finished faces of adjacent stones aligned. The hole should be gauged from the finished face to maintain gaged flushed finished faces. Side (C), as the interior edge distance, varies with the accumulated variance of the exterior edge distance (B), the hole diameter, and the overall slab thickness. Slab thicknesses may range between +/- 1/8 inch (+/-3mm) for nominal 1-1/4 inch (3cm) thick material and perhaps +/-1/4 inch (6mm) for 2 to 3 inch (5-8cm) thick material.
The interior edge distance usually resists wind loads causing negative pressure, or suction towards the outside, plus the panel's weight by bearing upon a ledge support. In most cases, lateral suction exceeds inward positive pressure from wind. Assure that the narrowest edge distance remaining after the tolerances combine remains substantial enough to be structurally safe. Avoid problems with the interior edge distance, for any damage to it is concealed in the cavity, and fracture could result in the stone separating from the facade.

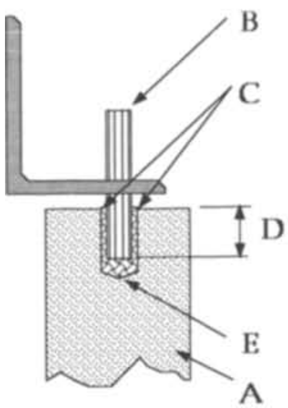



FIGURE 98: *Dowel Anchor Mechanics.* Dowel engagement is the length of anchor (B) embedded (D) in the stone (A). Maximize embedment of the anchor into the drilled hole (E) without bottoming the dowel. Fill the hole with compressible material (typically sealant) that cushions the anchor and distributes the pressure of the support reaction along the hole's full depth. Alignment of the dowel (B) and the hole (E) is critical to preventing prying points-of-contact at the rim of the hole (C).

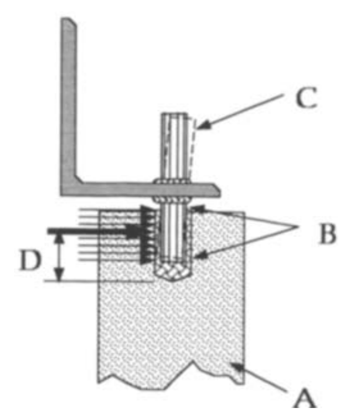



FIGURE 99: *(right) Depth of Dowel Contact in the Hole.* Maintaining proper alignment of the dowel (C) in the hole in the stone (A) (avoid the phantomed position) allows proper (even) distribution of forces along the entire height of the dowel with even bearing between points B. Relatively even distribution results in a midheight reaction for both positive and negative direction lateral forces and minimizes leveraged height (D) causes prying on the stone. While a deeper hole and greater embedment theoretically increases the potential surface area of the plane of influence, it also requires more perfect alignment to achieve proper bearing and avoid prying. As the reaction increases, length D unavoidably increases as the dowel bends elastically (it is a cantilever).

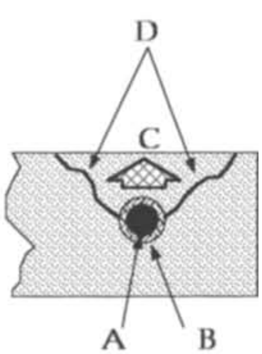



FIGURE 100: *Distribution of Dowel Contact Within The Drilled Hole.* The compressible fill in the drilled hole (B) helps distribute the forces (C) transferred from the stone to the dowel in the anchor (A) bearing upon the side of the hole. Shear-tension stresses develop along a diagonal plane emanating from the line of bearing contact (D). This surface area is relatively small, developing through the height of the hole and through the edge distance in the lone of the support reaction.

### *Actual Stress on the Stone*

Predicting actual stress on the stone is difficult without proving exactly where the kerf leg contacts the stone, which establishes the mathematical prying distance, bearing length along the kerf slot, and the surface area of the cone-of-influence (potential failure plane) emanating from the hole.

### *Kerf Slot Width*

Widths of kerfs should be kept to a minimum, only wide enough to accommodate fitting the anchor's kerf leg into the slot during stone placement without pinching the stone kerf fin, meanwhile anticipating the combined placement-with-fabrication tolerances.

### *Kerf Clip's Engaging Leg's Thickness*

The thickness of the leg of the kerf clip that engages the stone slot should be sufficient to be as stiff (not simply as "strong") as the stone through its height to effectively engage either the full slot depth for a brake-formed flat kerf clip, or to not deform enough to cause point contact at the kerf fin toe for an extruded kerf clip anchor. Too thick a kerf clip leg yields a stiff bar that will not bear continuously along the depth of the slot, which could reduce capacity.

Kerf slots should be filled. Fill the kerf slot with an elastomeric filler compatible with the stone, anchor, and joint sealant to prevent rattling and shock loading resulting from force reversals. The filler should be resilient enough to allow the bearing location of the kerf leg against the stone to remain as designed and not redistribute load transfer over the entire stone kerf fin height.

Minimum slot width with maximum stone kerf thickness maximizes the available surface area of the cone-of-influence (potential failure plane) and thus maximizes anchor capacity.

### *Kerf Slot Depth*

Depths of kerf slots should be kept to a minimum, only deep enough to accommodate the anchor's structural engagement and anticipated combined placement with fabrication tolerances. This approach also minimizes prying potential and allows practical workmanship practices and installation techniques in placing the stone. Shorter stone kerf fins minimizes the hazard of chipping during handling and placement.

Minimum kerf slot depth with maximum anchor engagement maximizes the surface area of the cone-of-influence (potential failure plane) and thus maximizes kerf capacity.

### *Kerf Slot Length*

Lengths of kerf slots should be determined after width and depth are optimized, and proportioned, with consideration of the resistance factors and capacity required, to match the stone's reaction due to superimposed design loads at that support location. While increasing kerf slot length also increases potential anchor capacity, relative stiffness of the strap along its engaged length must be matched to the stone to prevent point contact.

When the anchorage device is stiffer than the stone: for clips that cross joints, if the length of the kerf clip is too long and its shape is relatively stiffer than the stone, effective contact could occur only at only the kerf anchor's ends, which could reduce anchor capacity by restraining the panel's flexural curvature, depending on the flexural shape the panel takes under load.

When the stone is stiffer than the anchorage device: for clips that cross joints, if the length of the kerf clip is too long and the stone is relatively stiffer than the kerf clip's shape, effective contact could occur only within the kerf anchor's attachment to its backup supporting structure. Therefore, the realized anchor capacity is not proportional to the kerf anchor's full length.

A kerf anchor's length and its section's stiffness is to be approximately proportioned to match the stone panel's support reaction magnitude so that the necessary full potential of the surface area of the cone-of-influence within the stone kerf fin (potential failure plane) and is developed.

### *A Kerf Anchor Must Maintain Its Designed Shape Under Load*

Proportion the shape of the anchorage device to maintain its shape under full load to not create undesirable point loading more distant from the designed and tested engagement. This creates more leverage, and concentrates the load to increase local prying, which reduces the anchor capacity.

Avoid prying that could be caused by the anchor's kerf leg's engaged length twisting within the kerf slot.

Avoid prying that could be caused by point contact of the anchor's kerf leg's radiused bend contacting the outermost toe of the kerf fin where the engaged kerf leg exits the slot. Brake-formed anchors have radiused inside corners, the radius of which increases with metal gage, thus it is advantageous to minimize the anchor kerf leg thickness to minimize the folding radius and prying opportunity, yet maintain sufficient anchor kerf leg thickness to prevent twisting or bending within the engaged depth.

Avoid prying that could be caused by point contact with the kerf toe by chamfering the kerf fin's toe at the radiused inside corner where the anchor is folded or by shimming the stone up from the anchor to clear the radius.

### *Location of Anchors Within the Panel*

Kerf slots for kerf anchors should be located at points along the stone's length or width where their support minimizes the panel's span between the anchors. Typical quarter point locations nearly equalize positive and negative panel bending moments. Locations that use the fullest potential capacity of the uniformly thick panel are highly encouraged. Stress concentrations are also avoided with this approach.

## Proper Application and Optimization of Dowels

Dowel or "rod" anchor design considerations outline the objectives that designers focus upon when conceptualizing dowel-type and rod-type anchorages. They also determine when to use a dowels or rods in a certain application.

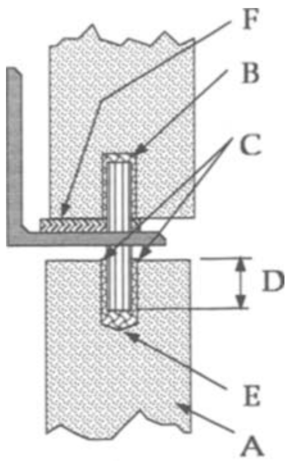



FIGURE 101: *Dowel Anchor Diagram.* The pin is probably the oldest type anchor used to restrain or stabilize stones. A stone (A) has a hole (E) drilled in an edge which is exposed during construction to receive a dowel (B) that is fixed to a strap or ledge support. The dowel should align with the hole axis to prevent point contact at the hole's rim (C) and embed deep enough in the hole to develop sufficient capacity. Use a bearing shim (F) between stones and stones-and-gravity-support anchors to prevent end-of-dowel-bearing and other single points of contact.

## *Structural Capacity*

Dowel anchor structural capacity is limited by the combined shear and flexural strength of the stone body at the cone-of-influence extending from the drilled hole in the stone edge that the dowel fits into. Parameters affecting the capacity are:

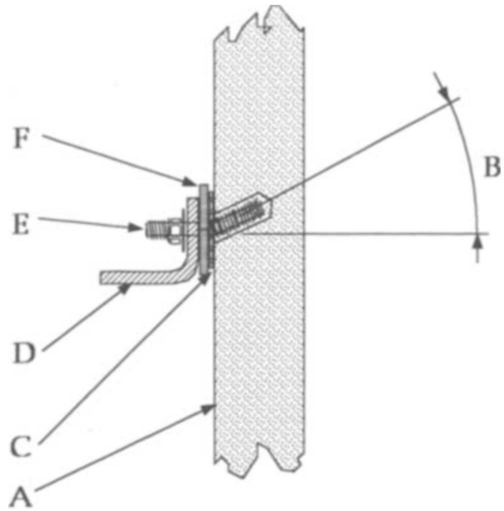
1. *Dowel hole's maximum diameter* (Fig. 96)
   The maximum diameter of a hole drilled into stone is affected by the trueness of the drilling bit as it traverses into the stone, perpendicularity of the hole with the stone's edge and thus the true straightness of the hole, accuracy of bit.
2. *Dowel hole's minimum distance from the inside edge-of-hole to the face-of-finish:* (Fig. 97)
   The distance from the edge of the stone to the edge of the hole, or guage accuracy of drill bit to its intended centerline, is affected by the "walk" of the drill bit from its intended centerline, and also the perpendicularity of the hole with the stone's edge and the true straightness of the hole, accuracy of bit;
3. *Length of dowel engaged into the hole:* (Fig. 98)
   The distance that the dowel pin is engaged into the drilled hole in the stone is dependent upon the fabricated length of the dowel extending from its strap or clip. Note that the actual point-of-contact greatly influences what the effective length of the dowel is.
4. *Depth of contact of the dowel:* (Fig. 99)
   The depth that the dowel that actually contacts the stone within the hole in the stone can be determined by the variances in the overall panel size, which changes its location relative to its supporting anchorages, the depth of drilled hole, the tolerances involved with installing the strap-and-dowel device, the trueness of positioning of the dowel into the drilled hole in order to preclude prying or point contact at the tip or fillet of the dowel pin instead of the dowel hole's sidewall.

   It is most desirable to minimize the leverage distance between the base of the drilled hole and the center point of the contact distribution area where the dowel pin anchorage device bears on the inside of the hole in the stone. The objective is to maximize depth of contact depth, which maintains minimum leverage distance, and thus maximizes capacity.
5. *Distribution of the contact of the dowel:* (Fig. 100)
   The surface around the hole depth that the engaged dowel pin contacts the stone is simply a function of the dowel pin's axial alignment with the centerline axis of the stone's drilled hole and the diameter differential between the hole and the dowel. To the extent that the dowel does not beat evenly onto the drilled-hole's wall, and will not remain somewhat "flexible" to self-align. There will be point-contact of the dowel at the rim of the drilled hole instead of its root.
6. *Minimum stone material strength.* Known as the actual material strength at the most critical contact point at the stone's cone-of-influence emanating from the anchor hole and through failure potential failure plane. At the kerf's root and through potential failure plane, which predictably radiates (see diagram above) at about a 45 degree angle from the point of highest bending moment at the root of the hole and at the hole's sides depending upon where the dowel bears on the stone (unless a material weakness occurs elsewhere. This plane of influence has been verified by tests.
7. *Actual capacity.* The true capacity of an anchor is difficult to accurately predict mathematically. It should be determined with tests specific to the actual dowel pin component in the stone for the project. A continuously embedded dowel is into an edge of the stone does not mean that full pin height is effective, or actually supporting the stone.

   Relative stiffnesses of stone, anchor, and backup control how much of the anchorage device is actually resisting load, transferring load from the stone surrounding the stone anchorage to the dowel pin's leg that the stone bears upon, and thus is effective. Reference the section on *Anchor Capacity and Effective Engagement Length Test* for specific considerations required for an independent kerf anchorage test, which is intended to verify the capacity of the anchor itself as well as its effective height of engagement within the drilled dowel hole.

## *Example Applications of the Dowel Pin and Drilled Hole*

1. *Dowel Pin Anchor*
   A dowel pin anchor (Fig. 101) is a round bar, pin, or dowel configured to engage a drilled hole in the stone edge, sometimes used when the required lateral resistance of the condition is relatively small (Figs. 114-117), or the stone is relatively thick (Figs. 104-113). It is usually a




◀ FIGURE 102: ***Inclined Threaded Dowel Anchor in Framing.*** **A variation of the edge dowel used for lateral support engages a single drilled hole in the back of the stone. The stone panel (A) has a hole drilled at an inclined angle (B) to receive a threaded rod (E) bent to match the drilled angle. The dowel is bedded into epoxy in the hole and attaches to the support backup framing (D). To maintain the dowel's proper alignment in the drilled hole and to avoid point prying, a compressible pad (C) cushions an oversize threaded plate (F) called a "stress-less" disc. This also prevents "prestressing" the dowel in the stone by being tensioned directly against the backup. The backup (D) tightens against the plate (F) instead of the stone (A) to maintain only enough tension between the bent dowel and the stone to keep it secure.**

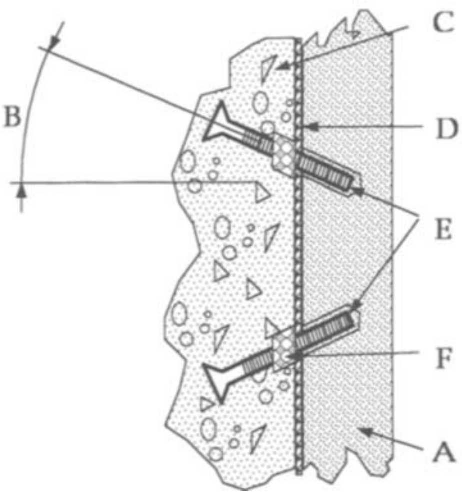


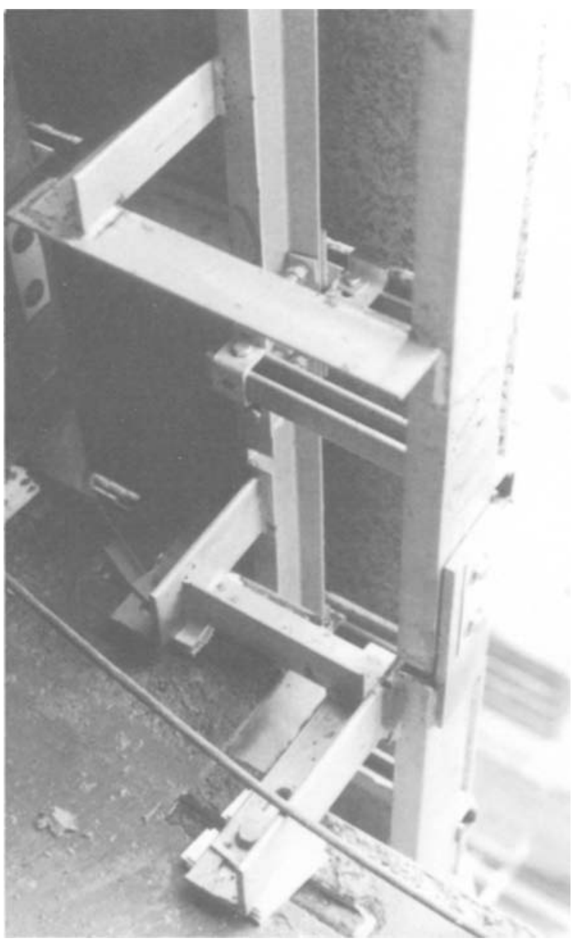

FIGURE 103: ***(left, bottom) Cast-In Inclined Threaded Dowel Anchor in Panels.*** **Used precast concrete panels or trusses with cast grout pockets support the stone. The stone panel (A) has holes drilled on an inclined angle (B) at a frequency determined by the loading and the dowel pin capacity. Headed threaded pins (E), sometimes typical hex-head bolts, are inserted into the holes at opposing angles to lock the stone onto its support (C). To accommodate differential thermal movement, separate the stone from the cast concrete with a "slip sheet" bond breaker (D) such as etha foam plastic sheeting and place rubber grommets (F), or collars, around the dowels where they enter the stone. The bond breaker prevents chemical bond between the stone and the concrete and allows expansion of trapped frozen moisture. The grommets prevent consolidation of the rigid concrete around the pin where it enters the stone and gives the dowel a short length of flexibility to bend to accommodate the movement between the different layers. Without these features, the stresses caused by thermal changes would either shear the pins or rupture and spall the stone surrounding the dowels.**

FIGURE 104: ***(right) 190 South LaSalle Street Typical Floors Under Construction.*** **To enclose the building as soon as possible, usually the typical floors' facade is installed first. Also, because the prototype mock-up models the typical areas, complete engineering and testing results are available for the typical system first.**
**The repetitive sections of the facade are quickest to fabricate and thus soonest manufactured and ready for installation. Here, stone cladding on floors six to thirty-seven are completed, except for the hoist bay. While these floors are being installed, non typical areas at the top floors and the base are engineered, submitted, approved, and then manufactured.**

smooth metal rod of appropriately compatible material bedded into a compressible material to improve bearing pressure distribution of the dowel within the hole. The limited area of the cone-of-influence due to the point support limits the potential of this anchor type. Alignment of the pin with the drilled hole is critical during the placement of the stone.

2. *Inclined Threaded Dowel Pin Anchor in Framing* (Fig. 102) An inclined threaded dowel pin anchored into framing is sometimes used in the back face of the stone for lateral

◀ FIGURE 105: *190 South LaSalle Street Transition Floors Framing.* **Just below the typical floors, an ornate colonnade rings the building at the fourth floor. Ladder frames are hung from the edge of the slab above to provide the exterior skin with structural support. Of the two floors of framing shown, notice that the upper floor's frames (the fifth floor) are repetitive and simple. The lower floor's frames (the fourth floor) are special, and more complicated to accommodate the irregular shapes profiles and anchorages of the stones in the fourth floor colonade band.**



▼ FIGURE 106: *190 South LaSalle Street Corner Ladder Frame at Curtainwall.* **The ladder frames span vertically between floors. Stub angles welded to the main channel vertical extend to and bear on the slab edge to be fastened onto the cast-in strut-type insert. The slot in the angle provides in-and-out tolerance adjustment between the building and facade. The channel in the strut insert provides side-to-side tolerance adjustment, and bearing shims provide up-and-down elevation adjustment between the building and facade. Horizontal unistrut spans between the main vertical to provide attachment for the individual stone anchors located in the horizontal stone joints.**

support when the top edge of the stone is inaccessible. The dowel pin is bent at about 30 degrees to provide positive mechanical engagement, and the threads (to im prove mechanical bond with the adhesive matrix) bedded in an epoxy adhesive to keep the pin fitting tightly within its matching inclined hole. A "stressless" disc, which is simply a tapped washer plate with sufficient thickness to resist the rod's twisting, is added to maintain the rod's alignment in the stone without inducing prying.

3. *Inclined Threaded Pin Anchor in Panels* (Fig. 103)
An inclined threaded dowel pin is typically used to attach stone to precast panels or trusses with grout pockets. The threaded rod is embedded into holes drilled into the back of the stone at about 30 degrees to provide positive mechanical engagement, and the threads (to improve mechanical bond with the adhesive matrix) bedded in an epoxy adhesive to keep the pin fitting tightly within its matching inclined hole. Alternating holes are placed in opposite-and-opposing angles to "lock" the stone onto the panel, once the panel is cast over the pins. Reference the section on Anchor Capacity and Effective Engagement Length Test Preparations, Setup, and Execution for explanation of components' roles in avoiding restraint.

A rubber "grommet," which is simply a compressible sleeve wrapping the dowel's shank for perhaps the first half-inch protruding from the hole in the back of the stone over the bond breaker membrane, is added to maintain the stone's separation from the panel backup to allow micro-movement (infinitesimal differential movements) between the panel and the stone without inducing prying.

### *How Dowel Anchors Secure the Stone to Its Supporting Structural Backup*

A dowel pin retains the stone onto the structural backup framing by a rod or dowel engaging a hole drilled into the stone's top, bottom, or side edges, or a combination of these edges. The rod (or dowel) of the anchor inserted into the hole accepts load from the stone by bearing against the hole wall.

### *Conceptual Analysis of the Dowel Anchor*

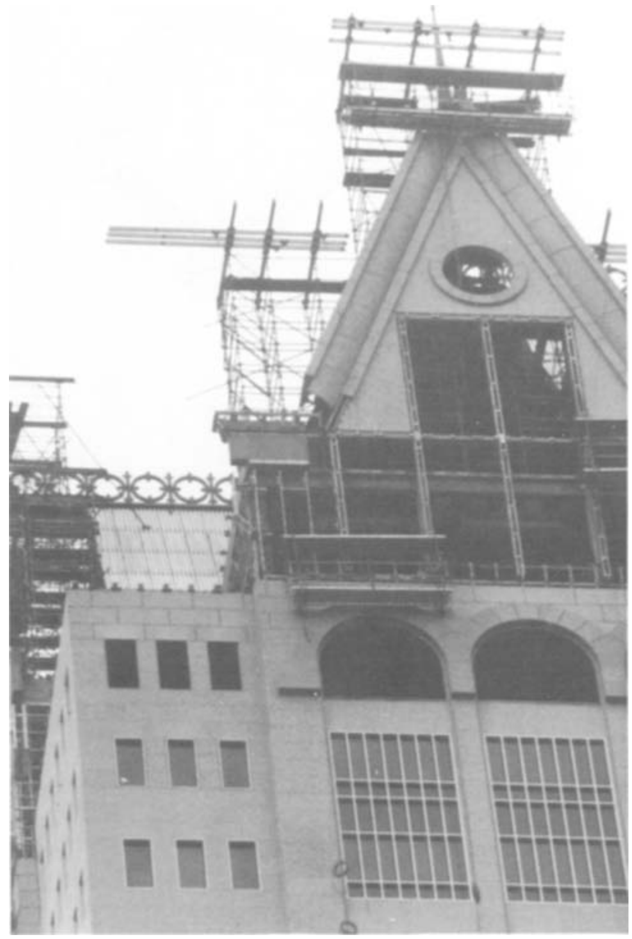
Follow the theoretical capacity analysis with actual testing of the device is required to determine the capacity of the anchor. In a relatively homogeneous material, the cone (or surface) - of-influence (or tension/shear failure plane) emanates at an approximately 45 degrees from the anchor's bearing point in the stone hole. Depending upon the mineral composition of the specific stone being considered, an adjusted assumed plane is advised for conceptual mathematical analysis.

### *Actual Stress on the Stone*

Predicting actual stress on the stone is difficult without proving exactly where the rod (or dowel) contacts the stone, which establishes the mathematical prying distance, bearing depth within the hole, and the surface area of the cone-of-influence (potential failure plane) emanating from the hole.

### *Hole Diameter*

Diameter of the hole should be kept minimum, only large enough to accommodate fitting the rod into the hole during stone placement with anticipated combined placement with fabrication tolerances without pinching the surrounding stone.



▲
FIGURE 107: *190 South LaSalle Street Top Gable.* Six steep gables compose the top three floors of the 42-story building to give the tower cap a signature silhouette. The completed composition shows may of the architectural features which required different structural systems for support: large floor-to-floor height arches at the 38th floor, three belt courses and a cornice, stacked story-high rectangular windows, diagonal tile fields, the circular oculus, a parabolically-contoured fascia and a pair of curved and cantilevered rollouts. The special systems had to interface and move properly without interfering with the function, or installation, of surrounding systems.

◀ FIGURE 108: *190 South LaSalle Street Gable Under Construction.* The partially-assembled gable reveals the exterior wall framing and the specialized staging erected to provide access to the work. Because only ridge beams, spandrel beams, and alternate-bay columns composed the building frame, all the facade features needed to be developed as an intermediate structural system to conform to the back-of-the-cladding's configuration. Between the large rectangular windows, the stone ribs are supported by a column built-up of rectangular windows, the stone ribs are supported by a column built-up of plates and channels, which attach to the floors. Main horizontals between these columns are structural tubes. "Table" frames cap the rollout and side brackets attached to the top of the concrete parapet wall are mounted in place, located relative to the other facade parts, ready to receive stones. This framing is still primarily supported at the slab edge.

▲
FIGURE 109: *190 South LaSalle Street 38th Floor Arched Truss.* The vertical curtainwall of the shaft rises to large arches at the 38th floor, which is the bottom of the 190's "top". The trusses span between columns and are hung from their horizontal wide-flange top chord. The bottom chord is a rolled steel channel. Angle diagonals are welded between the chords to triangulate the arrangement of the members. Each stone wedge is independently attached to the truss with separate brackets in the radial pattern. The truss also supports the window unit. There is a moving joint at the bottom of the truss where it connects to the typical ladder frames below.

▲
FIGURE 110: *190 South LaSalle Street Truss With Stone Arch Attached.* To give definition to the arch, the stone segments ranged between five and nine inches thick. Brackets utilized dowels and eisenshanks to mechanically engage the stone. In the foreground, individual stone segments are uncrated and shaken-out to have their brackets attached.

FIGURE 111: *(right) 190 South LaSalle Street Arch Return and Inset Stone Attachment.* In addition to supporting thick "cubic" stone segments for the arch itself, the windows were also set further into the interior to read greater depth at their over 500 ft. height above the LaSalle and Adams Streets' sidewalks. Inset Jamb stones and return stones were radiused on their finished edges, but straight on their back edges. Adjustable formed-plate brackets receive the eisenshanks from the stone and connect the assembly to the arched truss.

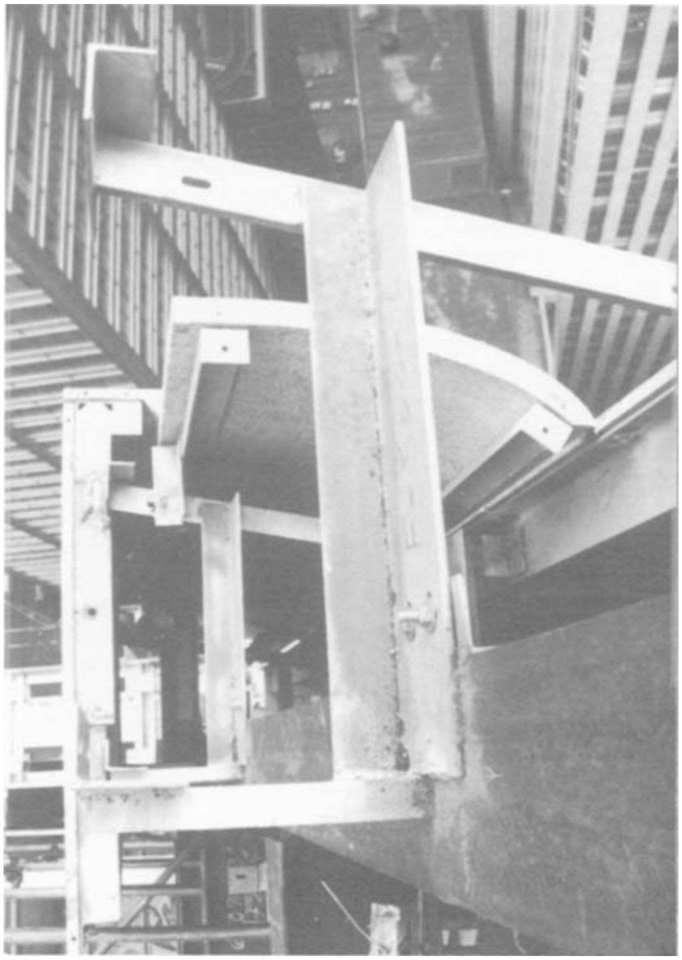


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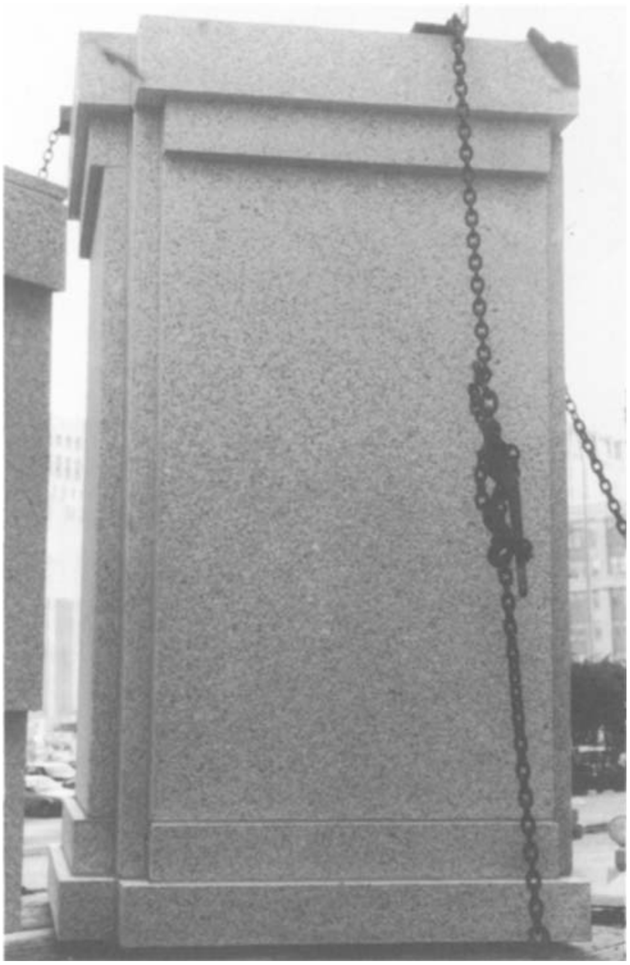
FIGURE 112: *190 South LaSalle Street Gable Fascia and Coping Cap.* The complicated stone profile at the perimeters of the gables combined with the ashlar jointing pattern mandated three separate intermediate framing systems to attach the stone. This condition was further aggravated by structural steel rafter beams being up to four inches out-of-tolerance. The triangular infill panels were solid metal with angles at the sloped edge to receive the squared-edge "molding" edge of the fascia. The middle, curved panel oriented on the slope, attaches to angle frames bolted to the rafter beams for alignment, then welded to fix the bracket to the beam. Table frames on the left assemble the three-piece coping cap, which also bolts to the rafters' angle frames.

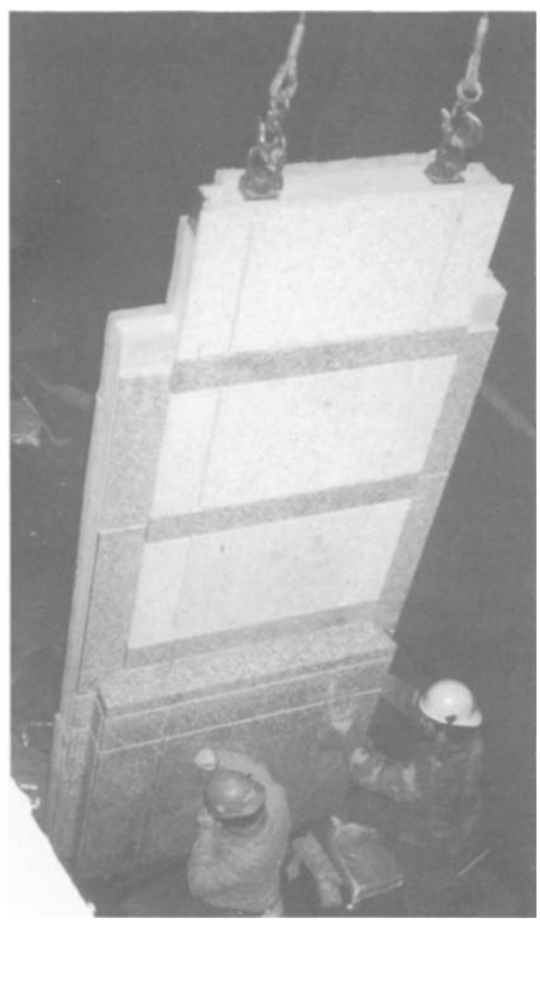
FIGURE 113: *(above, right) 190 South LaSalle Street Oculus Assembly.* A round oculus window is surrounded by the diagonal tile field at the peak of each gable. The steel members at the bottom and right were later clad with the belt and fascia. Visible at the center is the exposed front face of the rectangular tube that supports the window surround (or frame) at the head of the rectangular windows. The swing stage and chain fall are suspended from the monorail of scaffolding shown in Figure 108.

FIGURE 114: *(right) 255 Fifth Street Base Granite Laid Out in Precast Panel Bed.* Where architectural configurations result in small pieces of stone, several changes in plane, mixed flat and cubic shapes, or combines all these aspects into a single panel, precast backup has been used as the most economical intermediate support system. In Petropan's precast panel plant, stone panels were arranged on the form work to be the front mold for the concrete panel. Stiffening ribs were epoxied onto the return and side panels to secure their position during concrete placement and vibration. The penciled grid drawn on the back of the stone locate the inclined drilled holes for the anchor pins. Before concrete is poured, a bond-breaker slip-sheet lined the form, pins and grommets were placed, panel anchorage hardware mounted, and panel reinforcing placed.



◀ FIGURE 115: *77 West Wacker Chicago Corner Pilaster Precast Panel.* After the stone-clad precast panel wass fabricated at the plant, it was trucked to the building site. The 52-story neoclassical skyscraper on Wacker completed in 1992 was another example of complicated stone configuration mandating a precast backup. Architect DeStefano Goettsch also designed tight, clean corners instead of the typical quirk miters within a composition of multiple thickness roval white thermal finish granite stones. This criteria required an "l-shaped" panel, which Petropan ingeniously cast in the vertical position. This seven-foot tall panel was clad with thirteen different stone pieces which range from 2 1/2 inches to 12 inches thick.

FIGURE 116: *(below) 255 Fifth Street Completed Panel Being Hoisted Into Position.* Upon unloading from the truck, the panel in the form of Figure 114 has been lifted from its truck bed by crane to be placed in its position on the building facade. To expedite construction and utilize the tower cranes to their fullest extent, the building's structural frame was constructed during the day shift while the precast was erected during the night. Petropan finalized and detailed the precast during the day. The completed panel includes 31individual stones cast as one panel. Panels were also preinsulated because the concrete building frame prevented access to the cavity between the back of the panel and the exterior face of the concrete column that it covered. Ferrule loops for hoisting, prependicular stub plates for gravity anchors, and inserts for lateral anchors are also cast into the panel.



### *Dowel Diameter*

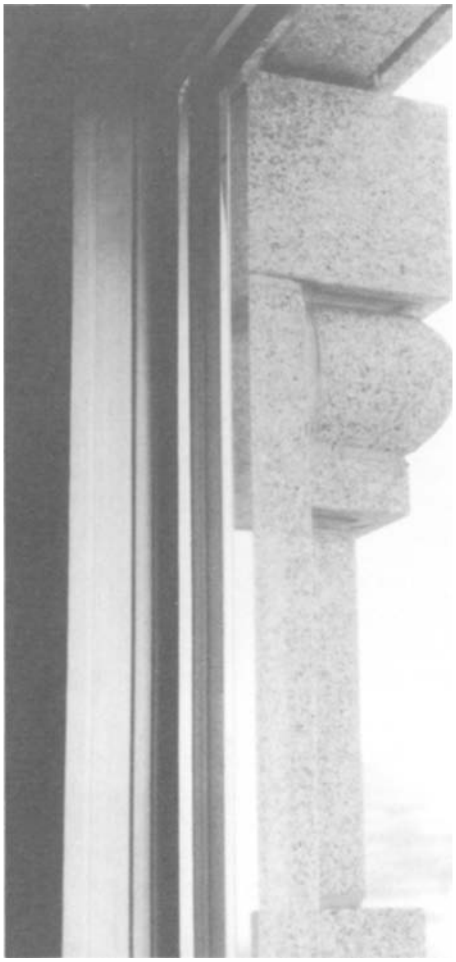
The diameter of the dowel in the anchor that engages the hole in the stone should be sufficient to be as stiff as the stone through its length to effectively engage the full hole depth. Too large a diameter yields a stiff pin that will bear at a point instead of along the depth of the hole and could reduce capacity.

Fill the drilled hole with a filler compatible with the stone, anchor, and joint sealant to prevent rattling and shock loading resulting from force reversals. The filler should be resilient enough to allow the bearing location of the rod against the stone to remain as designed.

Minimum hole diameter with maximum stone thickness maximizes the available surface area of the cone-of-influence (potential failure plane) and thus maximizes anchor capacity.

### *Hole Depth*

Depths of drilled holes should be maximized, but must be compromised with its minimized diameter to allow practical workmanship practices and installation techniques in placing the stone. Maximum hole depth without threatening stone damage local where the pin inserts with maximum pin engagement maximizes the surface area of the cone-of-influence (potential



◀ FIGURE 117: *77 West Wacker Precast Column Cover Stone Capital.* **In its final position on the building, as it appears through the glazed curtainwall, the granite-clad precast panel shows another complicated stone configuration. This panel is two floors tall, gravity supported at its midheight, with seven different stone cross-sections cast together. To expedite weatherproofing and improve quality control, all joints between stones with the precast panel were caulked in the precast factory. Further, a second line of sealant behind the perimeter weatherseal acted as a second-line-of-defense against infiltration. The void between them is vented.**

failure plane) and thus maximizes anchor capacity. However this approach also maximizes prying potential and thus the interaction of pin-and-stone must be closely examined.

### *A Dowel Anchor Must Maintain Its Designed Shape Under Load*

Proportion the length and diameter of the dowel pin and the anchor it is part of to maintain its shape under full load to not create point loading further from the designed and tested engagement, as the load concentrations increase local prying and reduces the anchor capacity.

Avoid prying caused by the rod's engaged length twisting within the hole; and avoid prying caused by point contact of the rod's fillet weld attachment to its strap contacting the outermost rim of the hole where the engaged dowel exits the hole;

Avoid prying caused by point contact with the outermost rim of the hole by chamfering the hole's rim at the fillet or by shimming the stone up from the strap to clear the fillet.

Avoid prying caused by overtightening nuts over supporting hardware devices that attach to the threaded rod portions of the anchor. Correct torque on the threaded rod must be achieved to avoid over-pretensioning the fastener which then pre-loads the stone, and as a result leaves less reserve capacity in the stone's strength to resist superimposed loads it is designed and intended to carry.

### *Location of Anchors Within the Panel*

Holes for rods should be located at points along the stone's length or width where their support minimizes the panel's span between the anchors. Typical quarter point locations nearly equalize positive and negative panel bending moments.

## SECTION 7
## CASE STUDY TESTING APPLIED TO THE DESIGN PROCESS

Testing determines engineering values needed to determine the structural adequacy of construction. Several different types of tests are involved in the stone system design process.

Initial tests are ASTM standard methods that establish unit material properties. Full-size load tests to failure can establish the panel's ultimate capacity. These can be correlated to theoretical analyses used to model other configurations. Individual anchor devices are tested separately to prove engagement behavior. These procedures are difficult to standardize because they are unique to the project and represent many variables specific to only that condition.

Overall assemblies should be tested to prove that all parts function together properly. Special tests that evaluate components separately allow for fine-tuning before developing the next phase. To refine the stone system and minimize testing costs, keep procedures simple, quick, and objective so that values are relevant and economy is achieved. Compare all work with exemplars to apply known performance to the project under consideration.

There are large granite panels in this presented example. They are supported only at their corners because the exterior wall framing members are vertical curtainwall mullions. While not very efficient for the stone panel structurally, this approach combined windowwall and opaque surface support in one system, significantly reducing labor and material costs for the overall wall.

The stone panels typically are flat plates. Cubic pieces can be analyzed similarly. The plate capacity was tested empirically first. Unit strength values from historical and initial tests were used to preliminarily set granite panel thickness. Finite element analysis modelling this plate computed stresses to be correlated with the full load test and unit-strength values.

Individual anchors were designed and tested to meet the reactions recorded in the panel analyses. Once proven, the anchors were built with the full-size system and tested in a chamber that simulated reversing pressures.

The sequence of tests are accomplished as follows:

Examining Exemplars for Proper Selection
- ASTM Standard Tests for Material Strengths
- Theoretical Panel Test by Finite-Element Structural Analysis
- Actual Panel Test for Preliminary Load Capacity
- Anchor Capacity and Effective Engagement Length Test
- Complete Assembly Full-Panel Chamber Test

## ASTM Standard Tests for Material Strengths

All engineering procedures require accurate information regarding the structural characteristics of the material. Because stone is a natural and variable material, its properties must be re-established and confirmed for each new significant application. Probably more important than "what are the values of the compressive or flexural strengths?," is "what are those strength's variabilities?"

In order to assure replicatability of these tests and also maintain consistent data, the fundamental strength and durability characteristics of the stone are to be evaluated with standardized, industry-recognized test methods developed and endorsed by consensus of the experts in the stone industry, the American Society for Testing and Materials. Legitimate statistically based conclusions from these tests will be used to make initial panel and anchorage design decisions within the upcoming mathematical finite-element analyses.

### *Objective and Purpose of the Standard Tests for Material Strengths*

Unit strength and durability material properties are measured using ASTM standard methods on identically sized samples of the actual material intended to be used for the project. Several types of stone materials might be considered for the same application initially in the project.

Because variability is a critical parameter for engineering design decisions because it strongly influences the usable strength, multiple small samples are tested by identical procedures and equipment prescribed by ASTM standard test methods' guidelines to enable the designer to consider and compare these material properties.

| | |
|---|---|
| C 97 | Absorption and Bulk Specific Gravity |
| C 170 | Compressive Strength |
| C 99 | Modulus of Rupture |
| C 880 | Flexural Strength |

Several other applicable test methods exist for dimension stone, however, they are seldom considered definitively to actually select the material for a project unless special concerns or exposures exist.

Major differences in C 97 absorption values between prospective stones or simply an absorptivity value above one-half percent could indicate relative difficulties in those stones' ability to endure water effects or freeze-thaw. Averages of C 880 flexural strength tests (thin, wider-slab samples with quarter-point load application to create an area of uniform peak flexural bending stress over the middle-half of the span) could be used in panel calculations. Averages of C 99 modulus of rupture tests (brick-sized samples with a center-midspan concentrated load application to create a single plane of peak flexural stress with some contribution of shear resistance due to the short span and thick specimen) could be used in preliminary anchorage capacity calculations.

Standard material strength tests are typically used for three purposes:

1. Historical tests are values from past test reports of the same stone material once used on a different project, but now being considered for "this" project. These are compared with each other, among the other major concerns such as plentifulness, availability, quarry, block size, and of course, aesthetic appeal.
2. Initial tests are values from groups of samples, usually of sufficient quantity to gain a statistical distribution and satisfy confidently the structural predictability appropriate for the significance of the project.
3. Quality control tests are values from smaller groups of samples duplicating those test methods that are executed

| | | | | DIRECTION | FINISH | TEST AVERAGES | | | STANDARD DEVIATIONS | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| *Test Set* | *Qty of Samples* | *C880 or C99* | *Wet or Dry* | *parallel or perpendicular* | *Sawn or Thermal* | *A* | *B* | *C* | *A* | *B* | *C* |
| 1 | 5 | C99 | wet | parallel | sawn | 1,732 | 2,236 | 1,702 | 177 | 140 | 106 |
| 2 | 5 | C99 | dry | parallel | sawn | 1,778 | 2,266 | 1,798 | 72 | 139 | 83 |
| 3 | 5 | C99 | wet | perpendicular | sawn | 1,278 | 2,458 | 1,538 | 154 | 85 | 140 |
| 4 | 5 | C99 | dry | perpendicular | sawn | 1,492 | 2,238 | 1,616 | 163 | 106 | 123 |
| | | OVERALL AVERAGES FOR EACH TYPE = | | | | 1,570 | 2,300 | 1,664 | 142 | 118 | 113 |
| | | COEFFICIENT OF VARIANCES = | | | | 0.075 | 0.062 | 0.068 | | | |
| 5 | 5 | C880 | wet | parallel | sawn | 1,114 | 1,928 | 1,256 | 142 | 130 | 135 |
| 6 | 5 | C880 | dry | parallel | sawn | 1,278 | 1,990 | 1,374 | 82 | 74 | 157 |
| 7 | 5 | C880 | wet | perpendicular | sawn | 1,326 | 2,010 | 1,288 | 232 | 71 | 53 |
| 8 | 5 | C880 | dry | perpendicular | sawn | 1,408 | 1,914 | 1,388 | 213 | 112 | 64 |
| | | OVERALL AVERAGES FOR EACH TYPE = | | | | 1,282 | 1,961 | 1,327 | 167 | 97 | 102 |
| | | COEFFICIENT OF VARIANCES = | | | | 0.075 | 0.085 | 0.077 | | | |
| 9 | 5 | C880 | wet | parallel | thermal | 1,028 | 2,003 | 1,272 | 96 | 120 | 68 |
| 10 | 5 | C880 | dry | parallel | thermal | 1,152 | 1,892 | 1,294 | 230 | 76 | 102 |
| 11 | 5 | C880 | wet | perpendicular | thermal | 1,326 | 1,670 | 1,144 | 74 | 137 | 61 |
| 12 | 5 | C880 | dry | perpendicular | thermal | 1,386 | 1,568 | 1,292 | 65 | 110 | 107 |
| | | OVERALL AVERAGES FOR EACH TYPE = | | | | 1,223 | 1,783 | 1,251 | 116 | 111 | 85 |
| | | COEFFICIENT OF VARIANCES = | | | | 0.091 | 0.065 | 0.068 | | | |

▲
FIGURE 118: ***Comparing Historical Strength Tests for Preliminary Selection.***
**In the beginning of the stone selection process, the material's strength characteristics should be considered at the same time aesthetic traits are evaluated. Examine the most recent available results of standard tests such as ASTM C99 and C880 of stone from that area of the quarry to get a preliminary indication of material strength, and more importantly, variability. Assure that the structural characteristics of previous material from that area of the quarry are compatible with the intended application. For small projects or applications that do not challenge the strength of the stone, recent historical tests may be appropriate as a basis for engineering. Stones are Nevada Beige (A), Bismark Pearl (B), and Texas Pink (C).**

through the actual production of the stone for the project, which evaluate those most critical properties to the particular design. This step in the sequence will affirm how the actual properties of the material being quarried and fabricated at that time compare with the initial values adopted by the structural design, and thus assure that all required safety margins are being maintained. Quality control tests' values that cannot satisfy the previously adopted minimum parameters suggests that modifications to the original design be adopted to accommodate the actual strength of the quarried stone, whether it be to thicken the slab, segregate the stone to less-loaded facade areas, or refine the stones' support.

## Preparations, Setup and Execution for Standard Tests for Material Strengths

These standardized tests have methods prescribed by ASTM and are to be accomplished and executed by certified testing laboratories who possess proper load equipment and recording devices that they are experienced in operating. Sizes of stone samples for each of the methods vary, and are stated within each method's text. These samples are prepared by the prospective stone fabricator from stone stock extracted from the same area and "table" of the quarry as the stone for that project. Samples must be collected from scattered blocks or slabs though, to prevent confining results to just a limited representative sample.

Wet or dry, parallel or perpendicular to the rift, and variable finishes are options within each procedure, particularly flexural strength, to attempt to quantify the material's natural variability. Larger projects will require that "quality control" samplings be tested at predetermined intervals to confirm that the structural characteristics of the stone material have not changed appreciably from that stone material originally tested, which may influence the required safety of the design.

## Standard Tests for Material Strengths Data Collection Requirements

Format, computations, and record of the results are outlined in the ASTM test method procedure. The load at which each sample tested failed is recorded along with its condition (wet or dry, parallel or perpendicular to the rift, and finishes). A specified standard formula to convert that load into stress is computed. Typically, arithmetic averages are reported. ASTM procedures do not, however, direct the designer on how to apply these values or how to interpret their relative strength or variability.

## Standard Tests for Material Strengths Data Evaluation for Historical Tests

Historical strength data is summarized in the table on the following page "Comparing Historical Strength Tests For Preliminary Selection," for the properties most pertinent to the stone's structural capabilities (C 99 modulus of rupture, and C 880 flexural strength, with their relative variabilities). Initial consideration of the three stones included in Figure 118 were based upon aesthetic architectural judgement.

## Standard Tests for Material Strengths Interpretations and Conclusions for Historical Tests

If one were to assume that no other selection criteria such as the material's availability, its appearance, or its cost supercede this strength advantage, Rockville beige would be preliminarily selected for the project, as its strength will offer the possibility of perhaps thinner panels, larger panels of a standard thickness module, or perhaps more uniform thickness even in varying higher wind zone areas. Because stone strength at the anchorages is particularly important, and since it is at this interface with the anchorage device and the support framing that loads and movements must be transferred, this strength advantage can offer increased opportunity for economies within the support framing configuration and anchorage device layout.

Professional Stone Testing
Dimension Stone Road
Quarrytown, Minnesota 00002

Laboratory No. 1234 56-789

Test of Stone Samples for Unit Strength

**Introduction:**
Project:
Owner:
Architect:
Stone Supplier/Fabricator:

This report presents the results of strength testing work performed on the nevada beige granite accordin to the standard test method **ASTM C99**-(most recent approved version): *Modulus of Rupture of Dimension Ston*

**Material:**
Trade Name of Stone: nevada beige
Quarry Identification: nevada beige
Quarry Location: Granite City, Nevada, USA
Date Sampled:

**Test Results:**

| pc. no. | width in. | thkns in. | length in. | span in. | load lbs. | **MOR wet psi.** | pc. no. | width in. | thkns in. | length in. | span in. | load lbs. | **MOR dry psi.** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 121 | 4.05 | 2.25 | 8.01 | 7.0 | 3130 | 1600 | 91 | 4.04 | 2.15 | 7.97 | 7.0 | 2640 | 1480 |
| 122 | 4.05 | 2.25 | 8.02 | 7.0 | 2890 | 1480 | 92 | 4.03 | 2.24 | 7.94 | 7.0 | 2570 | 1330 |
| 123 | 4.05 | 2.18 | 8.00 | 7.0 | 2810 | 1530 | 93 | 4.02 | 2.25 | 8.11 | 7.0 | 2910 | 1500 |
| 124 | 4.06 | 2.22 | 7.96 | 7.0 | 3060 | 1610 | 94 | 4.05 | 2.21 | 7.93 | 7.0 | 2770 | 1470 |
| 125 | 4.06 | 2.25 | 8.00 | 7.0 | 3220 | 1640 | 95 | 4.01 | 2.22 | 7.95 | 7.0 | 3010 | 1600 |
| 126 | 4.05 | 2.31 | 7.94 | 7.0 | 2900 | 1410 | 96 | 4.04 | 2.24 | 8.10 | 7.0 | 2590 | 1340 |
| 127 | 4.04 | 2.26 | 8.00 | 7.0 | 3050 | 1550 | 97 | 4.05 | 2.25 | 7.93 | 7.0 | 3270 | 1680 |
| 128 | 4.03 | 2.24 | 7.98 | 7.0 | 3040 | 1580 | 98 | 4.06 | 2.30 | 8.07 | 7.0 | 3420 | 1670 |
| 129 | 4.05 | 2.25 | 8.00 | 7.0 | 3280 | 1680 | 99 | 4.05 | 2.24 | 7.96 | 7.0 | 2610 | 1350 |
| 130 | 4.05 | 2.33 | 7.96 | 7.0 | 3090 | 1470 | 100 | 4.07 | 2.24 | 7.97 | 7.0 | 3120 | 1600 |
| 131 | 4.06 | 2.18 | 7.90 | 7.0 | 2770 | 1510 | 101 | 4.06 | 2.25 | 7.98 | 7.0 | 3040 | 1550 |
| 132 | 4.07 | 2.26 | 8.00 | 7.0 | 3090 | 1560 | 102 | 4.05 | 2.24 | 7.95 | 7.0 | 3310 | 1710 |
| 133 | 4.06 | 2.26 | 7.96 | 7.0 | 2760 | 1400 | 103 | 4.05 | 2.36 | 7.95 | 7.0 | 3330 | 1550 |
| 134 | 4.05 | 2.22 | 8.01 | 7.0 | 2990 | 1570 | 104 | 4.06 | 2.23 | 8.02 | 7.0 | 3250 | 1690 |
| 135 | 4.05 | 2.25 | 7.99 | 7.0 | 3090 | 1580 | 105 | 4.05 | 2.25 | 8.03 | 7.0 | 3100 | 1590 |
| 136 | 4.03 | 2.24 | 7.96 | 7.0 | 3070 | 1590 | 106 | 4.04 | 2.24 | 8.03 | 7.0 | 3170 | 1640 |
| 150 | 4.05 | 2.25 | 7.95 | 7.0 | 2880 | 1470 | 120 | 4.06 | 2.23 | 7.99 | 7.0 | 3320 | 1730 |
| | | **average of 30 samples wet** | | | | **1540** | | | **average of 30 samples dry** | | | | **1550** |

**Remarks:**
The test samples will be retained by the laboratory for a period of two weeks following issue of this report for observation by the client. Samples are the discarded unless the laboratory receives further instructions from the client.

Respectfully submitted,
Professional Stone Testing

◀ FIGURE 119: *C99 Modulus of Rupture Unit-Strength Test Report.* After the stone is selected for the project and quarrying begins, samples must be prepared for initial tests to establish the actual unit strength and variability of the stone material. For small projects or applications that do not challenge the strength of the stone, the stone cladding engineer may determine that separate initial tests may not be required. Prior to actually testing the anchors, C99 tests may best simulate the stresses occurring at anchorages because of the failure mechanism cause by the test. For a large cladding project, the example tests 30 wet and 30 dry samples according to the ASTM C99 procedure. Samples must be from random blocks and slabs representative of the variety of material to be supplied for the project. The example sampling established a good statistical measurement of variability due to the different conditions and quantities represented.

FIGURE 120: *(right) C880 Flexural Strength Unit-Strength Test Report.* Initial C880 tests should begin with initial tests when applications warrant new testing. Prior to preliminary full panel tests, C880 results best simulate the stresses occurring at midpanels because the wider, flatter sample and quarter-point loadings put the bottom face of half the sample's span in pure flexural tension without shear. Finishes, rift direction, and moisture variations should be included in the sampling to determine the lowest strength values and the weakest-performing condition. Do not mix standard deviations or coefficients of variability of different conditions to establish overall material consistency. Assure that sufficient number of samples of each condition exist before making conclusions about cladding material variability.

*Commentary.* For C 99 strength, Rockville Beige is nearly half-again stronger than the other stones considered

For C 880 strength, Rockville Beige is also nearly half-again stronger than the other stones considered

Coefficient of variation is nearly equal for all stones, for the standard deviation averages about 7% of the mean.

The previous is a summary of the conclusions of the data from the table *"Comparing Historical Strength Tests for Preliminary Selection"* as figure 118. Superior structural performance of Rockville beige would favor it as the preferred material. However, engineers rarely select cladding materials. The architect's opinion of the stone's appearance for this example found that the contrasts of the black and dark gray occlusions and their concentrations were objectionable, and Nevada beige was selected. Nevada beige also was less costly. Thus, more extensive testing was accomplished for Sunset beige, which is the material to be used for the project, as begun in the section *"Standard Tests For Material Strengths Report Example."*

### *Example Report the Initial Standard Tests for Material Strengths*

Following are sample test reports for ASTM C 99 Modulus of Rupture Test Method (Fig. 119) and C 880 Flexural Strength Standard Test Method (Fig. 120) used as initial tests for a project. The actual procedures are published by the American Society for Testing and Materials (ASTM). These are the first of a sequence executed for a specific project, which are followed by the finite-element, preliminary panel capacity, inde pendent anchorage, and final assembly tests once means strengths, variabilities, and other characteristics are concluded.

**Professional Stone Testing**
Dimension Stone Road
Quarrytown, Minnesota 00002

Laboratory No. 1234 56-789

Test of Stone Samples for Unit Strength

**Introduction:**
Project:
Owner:
Architect:
Stone Supplier/Fabricator:

This report presents the results of strength testing work performed on the nevada beige granite according to the standard test method **ASTM C880**-(most recent approved version): *Flexural Strength of Dimension Stone.* Polished and diamond-10 (flamed with water-jet) finishes as front exposed faces and sawn finish as backside face were tested to represent project conditions.

**Material:**
Trade Name of Stone: nevada beige
Quarry Identification: nevada beige
Quarry Location: Granite City, Nevada, USA
Date Sampled:

**Test Results:**

| pc. no. | width in. | thkns in. | length in. | span in. | load lbs. | FlexStr wet psi. | pc. no. | width in. | thkns in. | length in. | span in. | load lbs. | FlexStr dry psi. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ***polished face down*** | | | | | | | | | | | | | |
| 1 | 3.05 | 1.40 | 14.02 | 13.0 | 810 | 1310 | 6 | 3.08 | 1.45 | 13.99 | 13.0 | 760 | 1140 |
| 2 | 3.09 | 1.41 | 14.01 | 13.0 | 770 | 1220 | 7 | 3.06 | 1.43 | 13.99 | 13.0 | 700 | 1090 |
| 3 | 3.06 | 1.46 | 13.99 | 13.0 | 785 | 1170 | 8 | 3.06 | 1.42 | 14.01 | 13.0 | 610 | 970 |
| 4 | 3.05 | 1.41 | 13.99 | 13.0 | 770 | 1240 | 9 | 3.05 | 1.34 | 13.97 | 13.0 | 580 | 1030 |
| 5 | 3.10 | 1.40 | 14.03 | 13.0 | 720 | 1160 | 10 | 3.00 | 1.30 | 14.01 | 13.0 | 580 | 1120 |
| | | **average of 5 samples wet** | | | | **1220** | | | **average of 5 samples dry** | | | | **1070** |
| ***diamond 10 face down*** | | | | | | | | | | | | | |
| 11 | 3.08 | 1.31 | 14.01 | 13.0 | 680 | 1250 | 16 | 3.07 | 1.31 | 14.01 | 13.0 | 625 | 1160 |
| 12 | 3.08 | 1.32 | 14.03 | 13.0 | 635 | 1150 | 17 | 3.07 | 1.32 | 14.01 | 13.0 | 580 | 1060 |
| 13 | 3.10 | 1.28 | 14.00 | 13.0 | 615 | 1180 | 18 | 3.09 | 1.30 | 14.06 | 13.0 | 560 | 1050 |
| 14 | 3.06 | 1.29 | 13.98 | 13.0 | 600 | 1150 | 19 | 3.09 | 1.30 | 14.00 | 13.0 | 580 | 1080 |
| 15 | 3.09 | 1.31 | 14.01 | 13.0 | 610 | 1120 | 20 | 3.07 | 1.29 | 14.01 | 13.0 | 570 | 1090 |
| | | **average of 5 samples wet** | | | | **1170** | | | **average of 5 samples dry** | | | | **1090** |
| ***sawn face down*** | | | | | | | | | | | | | |
| 21 | 3.03 | 1.30 | 14.00 | 13.0 | 625 | 1190 | 26 | 3.07 | 1.29 | 14.00 | 13.0 | 390 | 740 |
| 22 | 3.06 | 1.32 | 14.02 | 13.0 | 630 | 1160 | 27 | 3.08 | 1.29 | 13.98 | 13.0 | 570 | 1080 |
| 23 | 3.00 | 1.29 | 14.01 | 13.0 | 600 | 1170 | 28 | 3.02 | 1.33 | 13.99 | 13.0 | 550 | 1000 |
| 24 | 3.01 | 1.32 | 14.00 | 13.0 | 610 | 1120 | 29 | 3.05 | 1.30 | 14.02 | 13.0 | 590 | 1120 |
| 25 | 3.09 | 1.33 | 14.00 | 13.0 | 590 | 1040 | 30 | 3.07 | 1.31 | 13.99 | 13.0 | 600 | 1110 |
| | | **average of 5 samples wet** | | | | **1140** | | | **average of 5 samples dry** | | | | **1010** |

**Remarks:**
The test samples will be retained by the laboratory for a period of two weeks following issue of this report for observation by the client. Samples are the discarded unless the laboratory receives further instructions from the client.

Respectfully submitted,
Professional Stone Testing

## Data Evaluation Standard Tests for Material Strengths

Initial tests are conducted to ascertain the actual structural properties of the selected stone, the same primary standard unit-strength tests are executed on a larger set of stone samples, these being prepared from material stock fabricated from the same quarried blocks that the project work will be fabricated from. These values, included in the table in Fig. 121 "Comparing Initial C 99 and C 880 Tests to Evaluate Consistency" summarize the test reports.

## Interpretations and Conclusions of Initial Standard Tests for Material Strengths

Testing of 60 samples of both C 99 Modulus of Rupture and C 880 Flexural Strength with a variety of wet, dry, parallel and perpendicular to the rift, and thermal and sawn finishes for Nevada beige are summarized as follows: (from Figure 121)

Average C 99 strength is 1564 lbs/in$^2$ and is most appropriately applied at the anchorage engagements;

Average C 880 strength is 1172 lbs/in$^2$ and is most appropriately applied within the panel spans;

Coefficient of variation is nearly 7.7 %, for the standard deviation averages just under 8% of the mean and is most appropriately considered for safety factor formulation.

| | | | RIFT | FINISH | TEST AVERAGES | | STANDARD DEVIATIONS | |
|---|---|---|---|---|---|---|---|---|
| *Test Set* | *Qty of Smpls* | *Wet or Dry* | *Parallel or Perpendicular* | *Sawn or Thermal* | *C99 Modulus of Rupture* | *C880 Flexural Strength* | *C99 Modulus of Rupture* | *C880 Flexural Strength* |
| 1 | 5 | wet | parallel | sawn | 1,628 | 1,220 | 185 | 60 |
| 2 | 5 | dry | parallel | sawn | 1,676 | 1,070 | 122 | 70 |
| 3 | 5 | wet | perpendicular | sawn | 1,496 | 1,158 | 99 | 77 |
| 4 | 5 | dry | perpendicular | sawn | 1,532 | 1,030 | 121 | 166 |
| | | OVERALL AVERAGES FOR EACH TYPE = | | | 1,583 | 1,120 | 132 | 93 |
| | | COEFFICIENT OF VARIANCES = | | | 0.083 | 0.083 | | |
| 5 | 5 | wet | parallel | sawn | 1,572 | 1,192 | 65 | 73 |
| 6 | 5 | dry | parallel | sawn | 1,476 | 1,053 | 97 | 77 |
| 7 | 5 | wet | perpendicular | sawn | 1,538 | 1,206 | 104 | 79 |
| 8 | 5 | dry | perpendicular | sawn | 1,528 | 1,222 | 170 | 64 |
| | | OVERALL AVERAGES FOR EACH TYPE = | | | 1,529 | 1,168 | 109 | 73 |
| | | COEFFICIENT OF VARIANCES = | | | 0.071 | 0.063 | | |
| 9 | 5 | wet | parallel | sawn/thermal | 1,548 | 1,228 | 103 | 120 |
| 10 | 5 | dry | parallel | sawn/thermal | 1,514 | 1,205 | 113 | 76 |
| 11 | 5 | wet | perpendicular | sawn/thermal | 1,584 | 1,185 | 133 | 127 |
| 12 | 5 | dry | perpendicular | sawn/thermal | 1,676 | 1,297 | 145 | 99 |
| | | OVERALL AVERAGES FOR EACH TYPE = | | | 1,581 | 1,229 | 124 | 106 |
| | | COEFFICIENT OF VARIANCES = | | | 0.078 | 0.086 | | |

▲

**FIGURE 121: *Comparing Initial C99 and C880 Tests to Evaluate Consistency.* After the stone is selected for the project and quarrying begins, samples must be prepared for initial tests to establish the actual unit strength and variability of the stone material. For small projects or applications that do not challenge the strength of the stone, the stone cladding engineer may determine that separate initial tests may not be required. Prior to actually testing the anchors, C99 tests may best simulate the stresses occurring at anchorages because of the failure mechanism cause by the test. For a large cladding project, the example tests 30 wet and 30 dry samples according to the ASTM C99 procedure. Samples must be from random blocks and slabs representative of the variety of material to be supplied for the project. The example sampling established a good statistical measurement of variability due to the different conditions and quantities represented.**

For allowable strength design, variances in material understrength is presumed to be "covered" by the safety factor. A rational approach to ascertaining risk could determine a minimum strength using probabilistic methods with comparisons to existing cladding.

Variance is critical in calculating the risk of material understrength. While examining the variability of the results from the tests of the selected Nevada beige material, and in assuming a normal distribution, the probability of the material's strength occurring less than the exclusion value must be determined in order to calculate risk.

Depending upon how strictly the design needs to limit understrength to control risk, this probability is selected to translate into an exclusion value. Using the normal distribution tables, the random variable "$z$" corresponding to that required probability ( $f(z)$ ) is multiplied by the standard deviation derived from the tests and then subtracted from the average strength attained also from the tests to attain the exclusion value. This exclusion value is the minimum strength that correlates to the probability of an occurrence, in this case, material understrength, that is below "$z$" number of standard deviations below the average strength.

Given that the C 880 mean is 1172 lbs/in$^2$ and the standard deviation is 90 lbs/in$^2$ (a 7.7% variance), the probability of Nevada beige flexural strength being more than 1.5 standard deviations {1172 lbs/in$^2$ - (1.5) x 90 lbs/in$^2$ }, or 1037 lbs/in$^2$ or less is:

Probability of flexural strength occurring less than 1.5 standard deviations below the mean strength < 1037 lbs/in$^2$ = less than 7%

Therefore, more than 93% of the stone will have a C 880 flexural strength greater than 1037 lbs/in$^2$.

Similarly, the probability of Nevada beige flexural strength occurring less than two standard deviations below the mean strength {1172 lbs/in$^2$ - (2) x 90 lbs/in$^2$ }, or 992 lbs/in$^2$ is less than 2%. And the probability of Nevada beige flexural strength occurring less than three standard deviations below the mean strength {1172 lbs/in$^2$ - (3) x 90 lbs/in$^2$ }, or 902 lbs/in$^2$ is less than 1/10%, or one-tenth of one percent.

*Commentary:* Remember that material understrength is only one of the many considerations that are to be considered in determining the overall safety and, inversely, the probability of failure. A 2.5 safety factor used in the ASD approach translates into 7.8 standard deviations for this design example, or less than 1/10,000th of one percent probability of understrength, or nearly 100% reliability.

Under ASD, this single parameter "covers" all other variables, and actually provides for 100% probability of those other variables like movements and maximum load (which, at design load of 45 lbs/ft$^2$ means a wind velocity of 134 miles-per-hour). Obviously, likelihood of this occurrence in this project's American midwest site is "somewhat" less than 100%. Other risk-rendering situations should be considered similarly when establishing a safety factor .

## Finite-Element Structural Analysis

The advent of matrix methods and automated computer calculations has allowed the testing of the stone panels expediently and effectively "on paper" before having to be proven with the actual stone. Multiple versions of panel sizes, anchorage configurations, panel thicknesses, and load magnitudes can be mathematically tested without the trial-and-error expense of setups, equipment, stone stock, and staff hours required to run the physical tests of the panels.

The results of these tests, which are calculated principal flexural and shear stresses only, are compared with the actual strengths concluded from the standard method tests to interpret if the safe margin is maintained. The mathematical matrix model is varied and refined until all the systems, including the anchorages, support framing, and the stone panels themselves are optimized.

### *Objective and Purpose of the Finite-Element Structural Analysis*

First, the finite-element theoretical "test" will suggest what limitations the support schemes and panel sizes can be considered. Actual panel stresses and anchorage capacities are determined by applying the specified loads to a theoretical panel model, which has an assumed thickness, size, and support layout given the panel sizes established by the architectural design, the support configuration estimated by preliminary schematic engineering, and using the strengths derived from the historical testing. The plate element boundaries are established by gradually refining the mesh near the supports, and again near the midspans, both horizontally and vertically. Design loads are applied, and the resulting stresses are computed.

If using allowable stress design-type analysis, these stresses must be less than those of the initial tests after they are reduced by the specified safety factor. If stresses indicated within the analysis conclusions are too large (and the model's mesh is adequately refined to avoid false "spikes"), than the panel's section modulus must be increased by thickening the stone, its overall face size perhaps is reduced, or the anchorage arrangement is revised.

Mathematical analyses allow multiple alternative versions of a panel to be examined upon successful derivation of the primary panel model with its backup. Mathematical tests of the typical and nontypical panels on the project within their varying wind pressures, after correlation for accuracy against actual testing (these analyses will follow testing in the section on *Actual Panel Test for Preliminary Load Capacity,* the section on *Anchor Capacity and Effective Engagement Length Test,* and the section on *Complete Assembly Full-Panel Chamber Test*), inexpensively determine the range of panel thicknesses required for the project. Uniformity in production, while not perhaps the most economical structurally, offers considerable dividends to the stone supplier and installer.

Finite Element Panel Analysis

***6'-0" Window Jamb Stone Panel:*** (Fabricator's Identification Mark #)

*STONE PANEL MODEL*

***material*:** nevada beige; diamond 10 finish in panel body
polished band in anchor areas at windows in corners

***minimum thickness*** = 1 1/2 inches (-0",+ 3/16") for 45 lbs/ft$^2$ typical wind pressure

***minimum section modulus*** = $S_{x,y} = \frac{bd^2}{6} = \frac{1" \times 1.5"^2}{6} = 0.375$ in$^3$/inch width

***structural analysis model*:** 6'-0" wide by 4'-3" high panel with a graduated rectangular finite element plate mesh with 130 nodes, 108 members.

***panel supports*:** the stone panel behaves as a corner-supported plate with pinned (effectively) point supports at its corners only. At the panel bottom's and top's corners, support locations are determined by the kerf anchorages' effective engagement lengths where the kerf clip is secured to the window mullion jambs. Conservative assumptions of the model include:

1. The kerf clip bulb engagement is 1/2" into the stone from top and bottom edge, reducing the vertical span 1 inch.
2. The actual stone height is 4'-2 3/8", further reducing the vertical span 5/8 inch.
3. The effective engagement length results in a support location not at the extreme corner, but somewhere in from each end, further reducing the horizontal span. This will be determined during the Effective Engagement Length Test.

***design loading*:** maximum of 45 lbs/ft$^2$ in the typical wind pressure zones established by wind-tunnel testing.

***material strength*:** as derived from initial sample and panel testing:
maximum of 1172 lbs/in$^2$ flexural strength per ASTM C880.
maximum of 1564 lbs/in$^2$ modulus of rupture per ASTM C99.
maximum of 128 lbs/ft$^2$ per preliminary panel capacity test.

***maximum bending stress*:** fb = 572 lbs/in$^2$ at 45 lbs/ft$^2$ design wind pressure, per the finite element structural analysis.

*compared to ASTM C99:* $\frac{1564 \text{ lbs/in}^2}{572 \text{ lbs/in}^2} = 2.7 > 2.0$ min. required safety factor

*compared to ASTM C880:* $\frac{1172 \text{ lbs/in}^2}{572 \text{ lbs/in}^2} = 2.0 \geq 2.0$ min. required safety factor

*compared to full panel:* $\frac{128 \text{ lbs/ft}^2}{45 \text{ lbs/ft}^2} = 2.8 > 2.0$ min. required safety factor

***maximum deflection*:** $d_{max}$ = 0.0649 inch at nodes 65, 66
maximum span = $l_{max}$ = 64 in. between edge supports

$\frac{d_{max}}{l_{max}} = \frac{0.0649 \text{ inch}}{64.0 \text{ inches}} = \frac{1}{986} < \frac{1}{600}$ max. allowed panel deflection

*ANALYSIS SUMMARY AND COMPARISON TO DESIGN CRITERIA*

***support reactions*:** kerf anchorages at window jambs = 287.4 lbs. (from analysis output)
average minimum capacity of an edge kerf = 850 lbs. from kerf anchor tests.

*compared to anchor test:* $\frac{850 \text{ lbs. capacity}}{287 \text{ lbs. reaction}} = 3.0 \geq 3.0$ min. required safety factor

***element forces and stresses*:** (see stress map for distribution pattern)
For bending moments $M_x$ or $M_y$ in the plane of the panel in the ranges below, the following flexural stresses result:

> 120 in.-lbs./in. but < 140 in.-lbs./in. : $f_b = \frac{M_{avg}}{S_{x,y}} = \frac{130 \text{ in.-lbs./in.}}{0.375 \text{ in.}^3\text{/in.}} = 350$ lbs./in.$^2$

> 140 in.-lbs./in. but < 160 in.-lbs./in. : $f_b = \frac{150 \text{ in.-lbs./in.}}{0.375 \text{ in.}^3\text{/in.}} = 400$ lbs./in.$^2$

> 160 in.-lbs./in. but < 180 in.-lbs./in. : $f_b = \frac{170 \text{ in.-lbs./in.}}{0.375 \text{ in.}^3\text{/in.}} = 455$ lbs./in.$^2$

214.7 in.-lbs./in. maximum at 54, 55 : $f_b = \frac{215 \text{ in.-lbs./in.}}{0.375 \text{ in.}^3\text{/in.}} = 572$ lbs./in.$^2$

◀ FIGURE 122: *Finite Element Panel Analysis Model.* Test the panel mathematically to prove it is structurally adequate prior to actually loading an actual panel. Mathematical models are extremely valuable during schematic design of the exterior wall system. Once the facade support framing scheme is conceptually designed, which determines the anchorage locations for the stone panel, build finite-element model of the typical panels. Use material strengths from historical tests during initial system conceptualization to predict the panel's, and the proposed system's feasibility. These assumed values can be verified or modified later in the design process. Calculate required thickness by comparing loads with resulting stresses. The resulting support reactions can be superimposed onto framing structural design models and can preliminarily size anchorages. Finite element analyses are only approximations, and are sensitive to the matrix of elements and stress gradients. Stone is a heterogeneous material, and does not behave exactly as the mathematical analysis models it. However, fundamental overall facade design options, from architectural arrangements to structural system configurations can be effectively compared with expeditious, inexpensive, nondestructive mathematical models built by an experienced stone designer. The conclusions must be verified by and correlated with actual panel tests later in the engineering sequence.

## *Finite-Element Structural Analysis Preparations, Setup, and Execution*

Finite-element structural analysis is, on the most fundamental level, founded upon the premise that a stone panel can be subdivided into many regularly sized smaller plates called finite elements to determine the forces within the different regions of the panel body. A computer then uses matrix methods to solve the multiple simultaneous equations resulting from equilibrium boundary conditions for each individual element. As load is applied perpendicular to each element, flexural reactions within the body of the panel resist that load, displacing and deforming to restore equilibrium until internal forces equal the applied load and also the element's boundary forces equal those of the contiguous adjacent elements.

Gradation of the "mesh" of elements is optimum when it is tightest where the stress levels are changing the fastest. This *does not* necessarily mean where the stresses are at their greatest magnitude, but instead where the stress gradient is maximum, for example at supports and midspans. Individual elements should not exceed 4-to-1 aspect ratio and should not change proportions more than one-half from element-to-element to avoid inaccurate computed "spikes." Each rectangular element is numbered, as are its corner incidences, which are shared with adjacent elements. The model is input into a program such as STAAD-III. Loads must be input in the same units as the model's dimensional notation, for instance, 45 lbs/ft$^2$ must be converted to 0.313 lbs/in$^2$ since the panel is input in inches.

```
STAAD FLOOR 6 FT WINDOW STONE IN 45 PSF ZONE
OUTPUT WIDTH 72
UNIT INCH POUND
JOINT COORDINATES
1   0.  0.  0.;   2   0.  0.  6.;   3   0.  0. 12.
4   0.  0. 18.;   5   0.  0. 24.;   6   0.  0. 27.
7   0.  0. 33.;   8   0.  0. 39.;   9   0.  0. 45.
10  0.  0. 51.;   11  6.  0.  0.;   12  6.  0.  6.
13  6.  0. 12.;   14  6.  0. 18.;   15  6.  0. 24.
16  6.  0. 27.;   17  6.  0. 33.;   18  6.  0. 39.
19  6.  0. 45.;   20  6.  0. 51.;   21 12.  0.  0.
22 12.  0.  6.;   23 12.  0. 12.;   24 12.  0. 18.
25 12.  0. 24.;   26 12.  0. 27.;   27 12.  0. 33.
28 12.  0. 39.;   29 12.  0. 45.;   30 12.  0. 51.
31 18.  0.  0.;   32 18.  0.  6.;   33 18.  0. 12.
34 18.  0. 18.;   35 18.  0. 24.;   36 18.  0. 27.
37 18.  0. 33.;   38 18.  0. 39.;   39 18.  0. 45.
40 18.  0. 51.;   41 24.  0.  0.;   42 24.  0.  6.
43 24.  0. 12.;   44 24.  0. 18.;   45 24.  0. 24.
46 24.  0. 27.;   47 24.  0. 33.;   48 24.  0. 39.
49 24.  0. 45.;   50 24.  0. 51.;   51 30.  0.  0.
52 30.  0.  6.;   53 30.  0. 12.;   54 30.  0. 18.
55 30.  0. 24.;   56 30.  0. 27.;   57 30.  0. 33.
58 30.  0. 39.;   59 30.  0. 45.;   60 30.  0. 51.
61 36.  0.  0.;   62 36.  0.  6.;   63 36.  0. 12.
64 36.  0. 18.;   65 36.  0. 24.;   66 36.  0. 27.
67 36.  0. 33.;   68 36.  0. 39.;   69 36.  0. 45.
70 36.  0. 51.;   71 42.  0.  0.;   72 42.  0.  6.
73 42.  0. 12.;   74 42.  0. 18.;   75 42.  0. 24.
76 42.  0. 27.;   77 42.  0. 33.;   78 42.  0. 39.
79 42.  0. 45.;   80 42.  0. 51.;   81 48.  0.  0.
82 48.  0.  6.;   83 48.  0. 12.;   84 48.  0. 18.
85 48.  0. 24.;   86 48.  0. 27.;  .87 48.  0. 33.
88 48.  0. 39.;   89 48.  0. 45.;   90 48.  0. 51.
91 54.  0.  0.;   92 54.  0.  6.;   93 54.  0. 12.
94 54.  0. 18.;   95 54.  0. 24.;   96 54.  0. 27.
97 54.  0. 33.;   98 54.  0. 39.;   99 54.  0. 45.
100 54. 0. 51.;  101 60.  0.  0.;  102 60.  0.  6.
103 60. 0. 12.;  104 60.  0. 18.;  105 60.  0. 24.
106 60. 0. 27.;  107 60.  0. 33.;  108 60.  0. 39.
109 60. 0. 45.;  110 60.  0. 51.;  111 66.  0.  0.
112 66. 0.  6.;  113 66.  0. 12.;  114 66.  0. 18.
115 66. 0. 24.;  116 66.  0. 27.;  117 66.  0. 33.
118 66. 0. 39.;  119 66.  0. 45.;  120 66.  0. 51.
121 72. 0.  0.;  122 72.  0.  6.;  123 72.  0. 12.
124 72. 0. 18.;  125 72.  0. 24.;  126 72.  0. 27.
127 72. 0. 33.;  128 72.  0. 39.;  129 72.  0. 45.
130 72. 0. 51.
ELEMENT INCIDENCES
1   1  11 12  2;  2  2 12 13  3;  3  3 13 14  4
4   4  14 15  5;  5  5 15 16  6;  6  6 16 17  7
7   7  17 18  8;  8  8 18 19  9;  9  9 19 20 10
10  11 21 22 12; 11 12 22 23 13; 12 13 23 24 14
13  14 24 25 15; 14 15 25 26 16; 15 16 26 27 17
16  17 27 28 18; 17 18 28 29 19; 18 19 29 30 20
19  21 31 32 22; 20 22 32 33 23; 21 23 33 34 24
22 24 34 35 25;  23 25 35 36 26; 24 26 36 37 27
25  27 37 38 28; 26 28 38 39 29; 27 29 39 40 30
28  31 41 42 32; 29 32 42 43 33; 30 33 43 44 34
31  34 44 45 35; 32 35 45 46 36; 33 36 46 47 37
34  37 47 48 38; 35 38 48 49 39; 36 39 49 50 40
37  41 51 52 42; 38 42 52 53 43; 39 43 53 54 44
40  44 54 55 45; 41 45 55 56 46; 42 46 56 57 47
43  47 57 58 48; 44 48 58 59 49; 45 49 59 60 50
46  51 61 62 52; 47 52 62 63 53; 48 53 63 64 54
49  54 64 65 55; 50 55 65 66 56; 51 56 66 67 57
52  57 67 68 58; 53 58 68 69 59; 54 59 69 70 60
55  61 71 72 62; 56 62 72 73 63; 57 63 73 74 64
58  64 74 75 65; 59 65 75 76 66; 60 66 76 77 67
61  67 77 78 68; 62 68 78 79 69; 63 69 79 80 70
64  71 81 82 72; 65 72 82 83 73; 66 73 83 84 74
67  74 84 85 75; 68 75 85 86 76; 69 76 86 87 77
70  77 87 88 78; 71 78 88 89 79; 72 79 89 90 80
73  81 91 92 82; 74 82 92 93 83; 75 83 93 94 84
76  84 94 95 85; 77 85 95 96 86; 78 86 96 97 87
79  87 97 98 88; 80 88 98 99 89; 81 89 99 100 90
82  91 101 102 92; 83 92 102 103 93; 84 93 103 104 94
85  94 104 105 95; 86 95 105 106 96; 87 96 106 107 97
88  97 107 108 98; 89 98 108 109 99; 90 99 109 110 100
91 101 111 112 102; 92 102 112 113 103; 93 103 113 114 104
94 104 114 115 105; 95 105 115 116 106; 96 106 116 117 107
97 107 117 118 108; 98 108 118 119 109; 99 109 119 120 110
100 111 121 122 112; 101 112 122 123 113; 102 113 123 124 114
103 114 124 125 115; 104 115 125 126 116; 105 116 126 127 117
106 117 127 128 118; 107 118 128 129 119; 108 119 129 130 120
SUPPORT
1 10 121 130 FIXED BUT MX MZ
ELEMENT PROPERTIES
1 TO 108 TH 1.50
CONSTANT
E  7000000. ALL
LOADING 1 UNIFORM WIND LOAD OF 45 PSF
UNIT INCH POUND
ELEMENT LOAD
1 TO 108 PRESSURE -.313
PERFORM ANALYSIS
```

▲

**FIGURE 123: *Finite Element Analysis Input Model.* Sample model for a flat, corner-supported 6'-0"x4'-3" x 1 1/2" thick panel in a 45 psf wind pressure zone.**

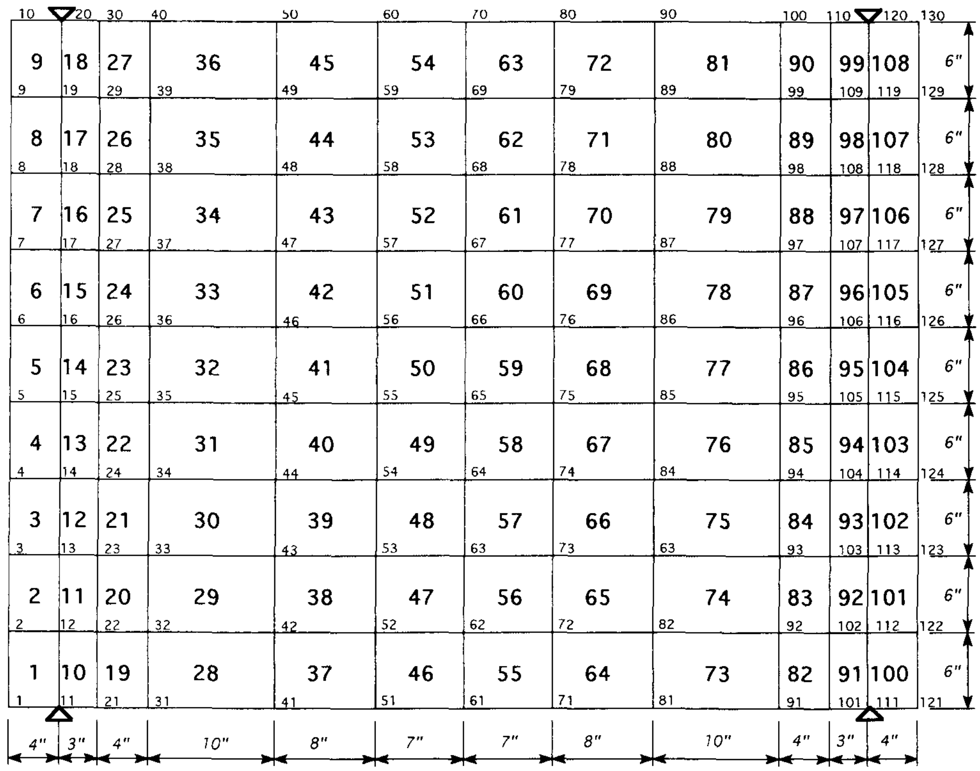
The example model in Figures 122–125 entitled "6'-0" Window Jamb Finite Element Model" illustrates the 51 inch-by-72 inch corner-supported panel. Because the anchorage test was not yet executed and the effective engagement length of the support remained unknown, the model conservatively assumed supports at the extreme corners, thus making the span somewhat longer than reality. Refinement of the mesh from the constant six-by-six will be executed once the preliminary panel capacity test and independent anchorage tests offer their conclusions for correlation.

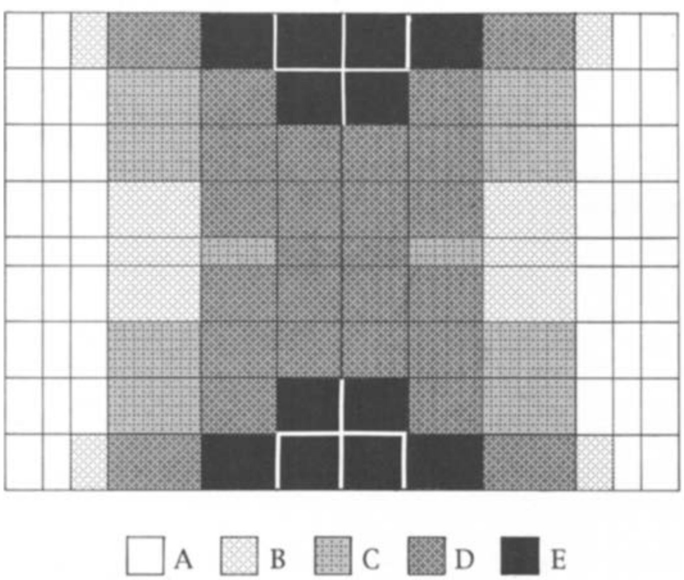
## *Finite-Element Structural Analysis Data Collection Requirements*

Finite-element structural analysis output states the shear and bending moment magnitudes in the three planes the plate exists, and by the units (lbs and in) consistent with the input data. Once the model is adequately refined, which can be evaluated through matrix method principles, the stress distributions can be computed from the force-reaction analysis output. These stresses can be compared to the magnitudes of the unit-strength tests to calibrate the required panel thickness or support layout.

## *Finite-Element Structural Analysis*

Figures 122, 123, and 124 show the sample Stone Panel Model analysis input, drawing model and initial design criteria for the 6'-0" window stone, 51" x 72" and 1 1/2 inches thick in a 45 lbs/ft$^2$ wind pressure zone. Figure 122 includes the Analysis Summary and Comparison to Design Criteria that checks the support reactions to the anchorage capacity (from the independent anchorage test) and compares the maximum bending stresses against the C 99 and C 880 limits. Different stress level thresholds are graphically illustrated in Fig. 125 on the panel model in order to quickly represent the stressed condition of the stone panel.




▲ FIGURE 124: *Finite Element Model Matrix.* To improve the accuracy of the theoretical mathematical model, the grid of plate bending elements was arranged to create the smallest elements where the largest stress gradients and maximums occured, typically near supports and midspans.

◀ FIGURE 125: *Finite Element Matrix Results.* The finite element analysis not only predicted approximate maximum flat-plate behavior stresses and reactions, it also indicated stress distributions. "Safety factors" were factored against a C880 minimum average value of 1172 psi derived from initial testing. Any comparisons must be in conformance with specific criteria that established the appropriate Safety Factors for that project.

A: <350 psi (SF>3.4)
B: 350 psi to 400 psi (SF= 3.4 to 2.9)
C: 400 psi to 455 psi (SF=2.9 to 2.6)
D: 455 psi to 505 psi (SF=2.6 to 2.3)
E: maximum average 572 psi (SF=2.0 minimum)

## *Finite-Element Structural Analysis Data Evaluation*

The initial model included consistent six-inch square elements except for three-inch tall elements at the panel midheight. In assuming a 1 1/2 inch thickness for a 45 lbs/ft$^2$ wind pressure zone (0.313 lbs/in$^2$ ) and extreme corner supports, maximum stresses occurred at the horizontal midspan top and bottom edges. The stress distributions are computed by these ratios of the ultimate C 99 Modulus of Rupture Test Method's stress from the initial testing shown following.

Comparing finite-element stresses with C 99 unit-strength test results (note that because the maximum stresses being evaluated occur near midspans, they are true flexural stresses and should be compared to the C 880 results):

572 lbs/in$^2$ (analysis maximum stress)
1564 lbs/in$^2$ (maximum unit-strength) =
37% of C 99 strength, Safety factor =2.7
Four elements (two nodes) aquire this stress level
(4% of panel is affected)

505 lbs/in$^2$ (analysis stress)
1564 lbs/in$^2$ (maximum unit-strength) =
32% of C 99 strength, Safety factor =3.1
31 elements aquire this stress level
(30% of panel is affected)

455 lbs/in$^2$ (analysis stress)
1564 lbs/in$^2$ (maximum unit-strength) =
29% of C 99 strength, Safety factor =3.4
11 elements aquire this stress level
(11% of panel is affected)

The stress distributions are computed following, and also illustrated by these ratios of the ultimate C 880 Flexural Strength Test Method's stress from the initial testing shown following:

Comparing finite-element stresses with C 880 unit-strength test results:

572 lbs/in$^2$ (analysis maximum stress)
1172 lbs/in$^2$ (maximum unit-strength) =
49% of C 880 strength, Safety factor =2.0

505 lbs/in$^2$ (analysis stress)
1172 lbs/in$^2$ (maximum unit-strength) =
43% of C 880 strength, Safety factor = 2.3

455 lbs/in$^2$ (analysis stress)
1172 lbs/in$^2$ (maximum unit-strength) =
39% of C 880 strength, Safety factor = 2.6

400 lbs/in$^2$ (analysis stress)
1172 lbs/in$^2$ (maximum unit-strength) =
34% of C 99 strength, Safety factor = 2.9

It is erroneous to deduce that, because the C 880 comparison results in a lower numerical safety factor, that a lower margin of safety exists. The comparison to C 880 is more appropriate due to the stress state considered. It means, that with a 134 mph wind (ignoring corner, edge, altitude, or other special effects), the stone of only average strength will be stressed to half its capacity if all other influences remain constant.

### *Finite-Element Structural Analysis Interpretations and Conclusions*

Because this example design is based upon an allowable stress-type design (ASD) specified minimum factor-of-safety of 2.5 in comparison to ASTM C 880. The example is in conformance with the project requirements and is an acceptable design (middle of Fig. 122). In traditional ASD, there is no stipulation as to what other considerations that overcapacity is to "cover." While other influences can potentially affect this realized stress level in the actual in-place panel, these influences should be evaluated independently; it is the arbitrary approach that should be avoided.

### *Comparing the Finite-Element Analysis to the Initial Standard Strength Method Tests*

Correlation between the finite-element model analysis results and the unit-strength tests is difficult without some degree of confirmation from the preliminary full panel test, because the finite-element model assumes an isotropic body and a homogenous material, which the stone is not. Further, true refinement of the mesh is not possible without the results from testing the real stone panel. Initial, unrefined finite-element analyses will be prominently valuable in getting atleast within one-third of the likely stress conditions, which, when considering panel size (bending moments are a function of the square of the span) and stone thickness (section modulus is a function of the square of the thickness) likely attains a design within 10% of its ultimate configuration. Obviously, this analysis, as a mathematical test is possible to execute within minutes, and without destruction, and is invaluable in conceiving an overall facade retention system.

After correlation with the full panel capacity tests, the finite-element analysis becomes extremely valuable in preliminary testing, and then ultimately in proving the structural adequacy of the non-typical configurations that cannot all be tested because of the limited time and costs of a construction project.

### *Preliminary Panel Test*

Before expensive and time-consuming specialized setups for individual anchorage tests and chamber testing of panels is pursued, it is appropriate to preliminarily prove general correlation between the mathematical finite-element model and the values and conclusions of the standardized sample tests. This preliminary full panel test is, while unsophisticated and relatively unexact, quite a valuable and efficient (comparing the cost and time involved in conducting the test with the information it provides) indicator of overall stone panel behavior.

### *Objective and Purpose of the Panel Test Preliminary*

Standardized testing that determines unit-strength properties of the natural material help deduce the stone material's actual variabilities and actual strengths for the small samples. Exemplar claddings may suggest durability, or which properties are vital. These standardized methods *do not* determine, however, stress distributions and their effects on the geological composition of the material itself.

Finite element computerized structural analysis models help deduce stress distributions within the structural member's body given assumed superimposed loads, support locations and support functions. Yet these theoretical models are *not* accurate alone without correlation to true behavior. The actual panel test for preliminary load capacity establishes that panel body's capacity, which is a nonisotropic and heterogeneous matrix. It provides the necessary correlation for tested load resistance to the finite element mathematical model. Together they provide the "truest possible" representation of actual panel body behavior short of testing every panel.

Once the analysis has true tested basis of the actual panel test for preliminary load capacity, mathematical analyses can "test" other panel configurations to predict their structural capacity.

The "Actual Panel Test for Preliminary Load Capacity" will establish how accurate the conclusions of the ASTM standardized test methods were, and how refined the mathematical finite-element model should be in predicting actual stone panel performance. If close correlation between these test methods is not proven, then adjustments must be made in either the strength values translated from the unit-strength tests to the finite-element model, or in the finite-element model itself.

## *Preparations, Setup, and Execution for Preliminary Panel Test*

Select the most typical stone configuration, which means the most common panel size with its anchorage layout, thickness, and finish for the project. Arrange "blocks" to serve as effective panel supports beneath the edges or locations of the stone where anchorages are expected to be located. This must closely correspond with the finite-element model, which in turn must agree with the projected design of the overall facade support system.

Place the stone panel face down (what will be the exterior finished face of the panel, downward) onto those blocks. Bricks or sand can be onto the flat laid stone, or a Goodyear inflatable airbag with a table fixed above it, to apply the weight, or pressure onto

FIGURE 126: *(right) Preliminary Panel Capacity Test Report.* To "calibrate" the mathematical finite element test results, a panel can be loaded evenly until it fails to determine its breakage pattern and rupture load. For projects where the stone's application develops high stresses, completing this test sequence immediately following the finite element analysis and final stone type selection can avoid later problems if the panel breaks at lower loads than expected. On the other hand, if the configuration breaks at higher than expected loads, the test may allow improved economy or dependability. This is a preliminary test intended to predict stone capacity as a panel loaded perpendicular to its face. Early and economical verification of the actual capacity of the slab itself is necessary before the full assembly is tested. Placed in the horizontal position and supported by edge blocks where anchors are to occur, the method evaluates the true plate behavior of the stone slab excluding anchorage interactions. Quantifying the limitations caused by any strong directional rift or finish influence during the actual panel test will help verify the accuracy of the finite element analysis. This correlation between actual test and theoretical model will allow relatively accurate prediction of the performance of non-typical panel configurations that will not be tested.

### Professional Stone Testing

Dimension Stone Road
Quarrytown, Minnesota 00002

Laboratory No. 1234 56-789

Preliminary Panel Capacity Test

**Introduction:**

Project:
Owner:
Architect:
Stone Installer:
Stone Supplier/Fabricator:

This report presents the results of load testing work performed on the nevada beige granite panels in a typical 45 psf wind zone. The purpose of this test is to establish preliminary capacity of the panel in plate-bending action supported as intended in the exterior wall design. The stone panels rest horizontally across blocks that are located where support anchorages would be. Load is applied evenly to the face of the panel to simulate wind loads until the panel breaks. This procedure tests the panel capacity separate of the influence of the anchorages with a simple and inexpensive method which can establish the relative viability of a panel size, thickness, and anchorage layout for a certain loading early in the project. Complete assembly behavior should be proven after panel and anchorage capacities are independently verified.

**Scope:**

This test establishes the ultimate capacity of the stone panel at the point of fracture. It is limited to load testing the granite panel while resting on four corner supports, monitoring the stone panel's deflection during loading, and presenting the factual results in this report. Test a minimum of three samples to establish a range of expected behavior.

This test will not prove the overall stone panel capacity, meaning the load resistance of the panel in-place within the exterior wall system. Influence of stability, strength, or overall integrity of the backup support and the anchorages is to be verified in a final test of the complete assembly.

**Test Apparatus Setup:**

*(Reference following figure showing setup)*

1. Stone panel (actual size for project). For this test, use three panels 6'-0" x 4'-3".
2. Wood blocks (length of anticipated *effective* anchorage).
3. Goodyear Air Bag.
4. Dial guages.

**Test Procedure:**

1. *Place stone panel onto supports.* Set stone in horizontal position, arranging blocks at corners of panel. Use 4-inch blocks as conservative effective length. Place the finished side down if the greater design load is suction on the facade.
2. *Arrange the support blocks into proper position.*
3. *Place dial indicators at midspan s of edges.*
4. *Apply a uniform load onto the stone panel's full face.* Inflate the Goodyear air bag system to apply the pressure to the top face of the stone, which places the bottom side in tension. Increase the air pressure steadily while taking deflection readings until the panel breaks. Record the total load at break, divide it by the stone's face area to compute the unit pressure resisted by the panel. Compare the ultimate pressure resisted to the design load multiplied by the appropriate safety factor(s).

**Test Results:**

***panel 1 (thkns. = 1.513") thermal finish side in tension***

| *load* | | *deflection* | |
|---|---|---|---|
| lbs. | psf | width | length |
| 532* | 20.9 | 0.000 | 0.000 |
| 890 | 34.9 | 0.005 | 0.001 |
| 1070 | 42.0 | 0.016 | 0.005 |
| 1250 | 49.0 | 0.025 | 0.010 |
| 1430 | 56.1 | 0.034 | 0.020 |
| 1610 | 63.1 | 0.036 | 0.043 |
| 1790 | 70.2 | 0.036 | 0.060 |
| 1970 | 77.3 | 0.037 | 0.078 |
| 2150 | 84.3 | 0.037 | 0.097 |
| 2330 | 91.4 | 0.038 | 0.118 |
| 2510 | 98.4 | 0.038 | 0.142 |
| 2690 | 105.5 | 0.039 | 0.190 |
| 2870 | 112.5 | 0.039 | 0.211 |
| 3050 | 119.6 | 0.041 | 0.249 |
| 3230 | **126.7** | 0.041 | **0.291** |
| | ***=2.8 x design*** | | ***L/247*** |

* = panel selfweight

***panel 2 (thkns. = 1.488") sawn finish side in tension***

| *load* | | *deflection* | |
|---|---|---|---|
| lbs. | psf | width | length |
| 530* | 20.8 | 0.000 | 0.000 |
| 890 | 34.9 | 0.001 | 0.001 |
| 1070 | 42.0 | 0.001 | 0.002 |
| 1250 | 49.0 | 0.002 | 0.005 |
| 1430 | 56.1 | 0.005 | 0.009 |
| 1610 | 63.1 | 0.010 | 0.019 |
| 1790 | 70.2 | 0.017 | 0.037 |
| 1970 | 77.3 | 0.023 | 0.054 |
| 2150 | 84.3 | 0.026 | 0.067 |
| 2330 | 91.4 | 0.032 | 0.078 |
| 2510 | 98.4 | 0.035 | 0.095 |
| 2690 | 105.5 | 0.037 | 0.119 |
| 2870 | 112.5 | 0.039 | 0.144 |
| 3050 | 119.6 | 0.040 | 0.183 |
| 3230 | 126.7 | 0.043 | 0.225 |
| 3410 | **133.7** | 0.046 | **0.282** |
| | ***=3.0 x design*** | | ***L/255*** |

***panel 3 (thkns. = 1.500") thermal finish side in tension***

| *load* | | *deflection* | |
|---|---|---|---|
| lbs. | psf | width | length |
| 534* | 20.9 | 0.000 | 0.000 |
| 890 | 34.9 | 0.000 | 0.000 |
| 1070 | 42.0 | 0.000 | 0.002 |
| 1250 | 49.0 | 0.003 | 0.010 |
| 1430 | 56.1 | 0.007 | 0.025 |
| 1610 | 63.1 | 0.013 | 0.035 |
| 1790 | 70.2 | 0.019 | 0.069 |
| 1970 | 77.3 | 0.026 | 0.087 |
| 2150 | 84.3 | 0.031 | 0.108 |
| 2330 | 91.4 | 0.037 | 0.160 |
| 2510 | 98.4 | 0.042 | 0.178 |
| 2690 | 105.5 | 0.046 | 0.208 |
| 2870 | 112.5 | 0.052 | 0.234 |
| 3050 | **119.6** | -- | **0.263** |
| | ***=2.7 x design*** | | ***L/274*** |

*All samples failed by fracturing into "halves" at midwidth across their length.*

**Conclusions:**

**Comparing the results show that the average capacity of the three panels is 126.7 psf, or 2.8 times the project design load. The three breaks were within 5.5% of the average. The lengthwise displacements average 0.279", or L/258 at breakage., and were within 5.7% of the average. The interactive behavior between the stone and continuous anchorage kerf rail over the full 6' span will be proven with the full-size chamber test.**

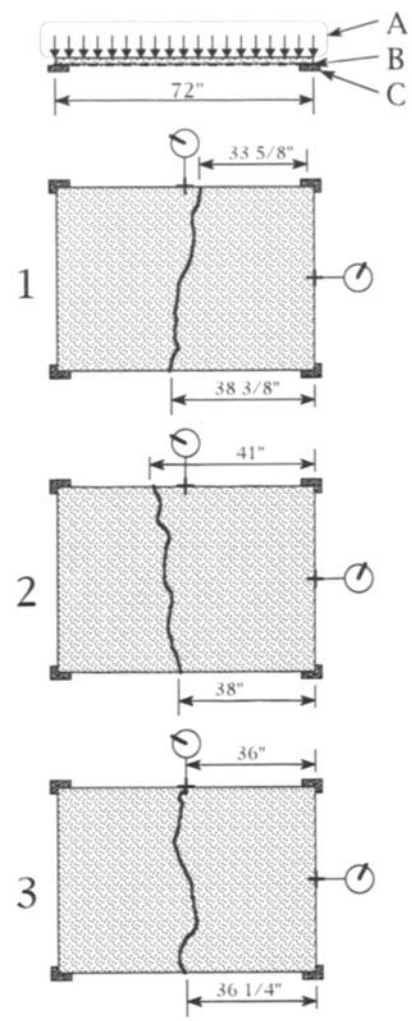
**Maximum flexural stresses at failure are:**

| panel | load | bending moment | section modulus | approximate flexural stress |
|---|---|---|---|---|
| 1 | 126.7 psf | 29,078 in.-lbs. | 19.46 $in.^3$ | 1494 psi |
| 2 | 133.7 psf | 30,684 in.-lbs. | 18.82 $in.^3$ | 1630 psi |
| 3 | 119.6 psf | 27,448 in.-lbs. | 19.13 $in.^3$ | 1435 psi |

**Remarks:**

The test samples will be retained by the laboratory for a period of two weeks following issue of this report for observation by the client. Samples are the discarded unless the laboratory recieves further instructions from the client.

Respectfully submitted,
Professional Stone Testing




▲
FIGURE 127: ***Preliminary Panel Capacity Test Setup and Results.*** **Place the flat slab finish side down (B) on support blocks (C) located where anchors will be on the supporting backup and sized according to the anchors' expected length or bearing. Inflate to pressurize the airbag (A) to apply an evenly distributed load to the face of the panel, which simulates wind on a facade. Three typical size panels were tested to compare failure. Results were compared to finite elements and unit strength tests early in the conceptual design of the facade system to correlate the conclusions and assure consistent test interpretation.**

the stone. If weight such as bricks or sand is used, know its density beforehand and continuously monitor quantities added to the stone panel. When using the airbag, the pressure guages must be calibrated and pressure increased very gradually. Be reminded that 30 lbs/ft$^2$ facade pressure equates to only 0.2083 lbs/in$^2$ for the airbag pressure. If the sand is relatively dry, every inch of loose sand added onto the stone will weigh between 7.5 and 9 lbs/ft$^2$. Modular (2 3/8 in. thick) brick will weigh about 24 lbs/ft$^2$. Do not forget that the stone's selfweight ranges between 14 lbs/ft$^2$ for granite to 9 lbs/ft$^2$ for dry sandstone for every inch of the material's thickness. If a pan was built at the stone's perimeter and a "pool" built for water, every inch of water would create 5.2 lbs/ft$^2$. Keep careful record of the load present on the stone at all times so that at the point of sudden breakage, accurate knowledge of that ultimate total load is known.

### *Data Collection*

The pattern of the stone's break is dimensioned, drawn, and photographed. The position of the break should agree with the highest-stressed elements in the finite-element mathematical model unless the stone's geological irregularities, such as veining or rift, influence the fracture pattern. With a micrometer, measure the stone thicknesses at several locations along breaks.

The more the support pattern tends to be point-or-corner oriented, the greater the benefit of two-way "plate" bending and stress distribution there will be, which may increase capacity from that predicted by the small samples, depending upon the aspect ratio of the face's height-to-its-base. The more that the support arrangement suggests a beam, that is, one-way bending, the closer the capacity may be predicted by the small sample C 880 test results.

Using the actual thickness to compute the stone's section modulus ($S$) with the actual total stone's load at failure ($w$) to calculate the bending moment ($M_f$) will render $M_f / S = f_b$. This section modulus divided by the load will calculate flexural stresses ($f_b$). These stresses should agree somewhat with those results of the finite-element analysis to establish a correlation if that model is properly refined. The load at which the stone breaks might likely occur about the load that corresponds to that which generates the same stress in the critical elements of the finite-element model at the point-of-failure for the C 880 conclusions. If the thicknesses and stress distributions correspond, the degree of correlation can be established. Once this relationship is confidently resolved, this correlation could validate the mathematical analysis method for proving less typical panel sizes without actual full panel testing of those configurations. It is vital to verify this correlation early in the project, so that panels can be arranged within the exterior wall support scheme and so that panels of that certain size can be released for slabbing production in the modules proven by these analyses and tests.

## Example Report of Preliminary Panel Test

Figures 126 and 127 show is the sample *Load Test of Natural Building Stone* actual panel test for preliminary load capacity report for the 6'-0" window stone, 51" x 72" and 1 1/2 inches thick. This procedure blocks the corners to simulate the anchorages' support, then imposes an increasing load with a pressurized airbag that duplicates lateral wind pressure until the stone fails. The total load at failure is recorded with the pattern of breakage.

## Data Evaluation of Preliminary Panel Test

Three separate panels were tested in order to gain a relative level of confidence for the consistency and reliability of the panel's actual capacity. The panels were corner supported as presumed to be during their final placement within the facade, and loaded, finished face (diamond 10, which is a water jet over thermal, or flamed treatment) on the tension face, downward, as summarized in the following table:

| SAMPLE | STONE | TOTAL LOAD | SHORT-SIDE | LONG-SIDE |
|---|---|---|---|---|
| *Number* | *Thickness* | *At Failure* | *Deflection* | *Deflection* |
| 1 | 1.513 in. | 3,230 lbs. | 0.041 in. | 0.291 in. |
| 2 | 1.488 in. | 3,410 lbs. | 0.046 in. | 0.282 in. |
| 3 | 1.500 in. | 3,050 lbs. | 0.052 in. | 0.263 in. |

TABLE 10. ***Preliminary Panel Test Data.***

Using the 50 3/8 in. by 72 in. panel size, there are 25.19 square feet of surface area. The design load is 45 lbs/ft$^2$ for this example with an adequate factor-of-safety (this example is within the ASD format). The short-side deflection ratio is based upon 50.38 inches less the 4 inches bearing at each corner resulting in a 46 3/8 span, and long-side deflection ratio is based upon 72 inches less the 4 inches to the center-of-bearing at each corner, resulting in a 64 inch span:

| SAMPLE | FAILURE LOAD | SAFETY FACTOR | SHORT-SIDE DEFLECTION | LONG-SIDE DEFLECTION |
|---|---|---|---|---|
| *Number* | *Per Square Foot* | *Factor-of-Safety* | *Deflection* | *Deflection* |
| 1 | 128.2 | 2.8* | L/1,131 | L/220 |
| 2 | 135.4 | 3.0* | L/1,008 | L/227 |
| 3 | 121.1 | 2.7* | L/892 | L/243 |

*> 2.5 minimum required

TABLE 11. ***Preliminary Panel Test Summary.***

If one-way bending was assumed to be the limiting structural mode, then the failure moment $M_f$ would be computed as $wl^2 / 8$, where:

$w$ = unit failure load (lbs/in$^2$) = lbs/ft$^2$ / 144 in$^2$/ft.$^2$ x 50.38 in wide)
$l$ = long-side span; (64 inches)

and the failure stress $f_b$ would be computed as $M_f / S$ where:

$S$ = the stone's section modulus; (bd$^2$ / 6) where:
$b$ = short-side width in inches; (50.38 inches)
$d$ = stone's thickness in inches measured from test sample

## Interpretations and Conclusions of Preliminary Panel Test

The following table summarizes the application of these formulas with the calculation of the apparent ultimate flexural stresses reached at panel failure:

| SAMPLE | FAILURE LOAD | BENDING MOMENT | SECTION MODULUS | FLEXURAL STRESS |
|---|---|---|---|---|
| *Number* | *Per Inch Width* | *(Mf)* | *(S)* | *(fb)* |
| 1 | 44.85 lbs/in. | 22,964 in.-lbs. | 19.22 in.$^3$ | 1,195* lbs./in.$^2$ |
| 2 | 47.37 lbs./in. | 24,254 1n.-lbs. | 18.59 in.$^3$ | 1,305* lbs./in.$^2$ |
| 3 | 42.37 lbs./in. | 21,693 in.-lbs. | 18.89 in.$^3$ | 1,148* lbs./in.$^2$ |
| | | | Average = | 1,216* lbs./in.$^2$ |

*compared to C880 average of 1,172 lbs./in$^2$

TABLE 12. ***Preliminary Panel Test Conclusions.***

### *Comparing the Preliminary Panel Test to the Initial Standard Strength Method Tests*

Given that the initial testing sequence rendered a 1172 lbs/in$^2$ ASTM C 880 average flexural strength, and the derived actual flexural stress average of the full panels was 1216 lbs/in$^2$, less than 4% difference, there exists a strong, direct correlation between these two test examples.

If possible, it might also be suggested that C 880 samples be fabricated from the failed panel and C 880 tests conducted on these samples to gain correlation of the material properties of the full panel to the standard strength method values. Fallacies exist with this approach however, because that stone material is possibly already partially pre-stressed. Strength of those samples from that "weakened" portion near the failure plane therefore cannot be representative of the whole stone. And because the material is heterogeneous, it cannot be conclusive that samples fabricated from the intact perimeter segments genuinely duplicate the strength properties of the remaining stone either. Perhaps a combination of the two ought to be investigated. Nonetheless, C 880 tests of several samples from either the broken stone, or from the "drops" off the same rough slab that the panel was fabricated from, should be encouraged to assist in the inter-correlation process.

### *Comparing the Preliminary Panel Test to the Finite-Element Analysis Structural*

Given that the finite-element analysis concluded that the maximum stress at the design load of 45 lbs/ft$^2$ was 572 lbs/in$^2$, and that the average failure load of the preliminary full panels was 128.8 lbs/ft$^2$, one could proportion the two tests as follows to show correlation:

1216 lbs/in$^2$ (calculated failure stress)
128.8 lbs/ft$^2$ (panel failure load)

572 lbs/in$^2$ (finite-element stress) =
45 lbs/ft$^2$ (finite-element design load)
But 572/45 (12.7) does not equal 1216/128.2 (9.5)

Because the panel allows 25% more load than the finite element the finite-element is conservative by predicting stresses that are higher than those realized. Further refinement of the mesh could improve this distant mathematical relationship. Closer inspection of resulting stresses in those critical elements immediately surrounding the nodes where the peak occurs are about 10% lower, and would correlate at about 26%, but still conservative.

Further refinement later showed a closer correlation of the analysis to the full panel by adapting the support locations from the independent anchor tests and tightening the mesh at the supports and panel midspans. Because the effective engagement length of the support was centered four inches in from each corner along the top and bottom edges, the span was reduced to 64 from 72 inches. Further, better gradation of the finite-element mesh near the higher stress gradients combined for a nearly 33% improvement in comparison to yield a 1210 lbs/in$^2$ maximum flexural stress, only 4% different from the 1172 lbs/in$^2$ unit-strength method value. Also, observing the averages of the tables, the failure load is 0.89 lbs/in$^2$, the section modulus of the stone is 18.9 in.$^3$ for the entire stone width, and the average moment is 22,971 in-lbs/in, to result in the 1215 lbs/in$^2$ flexural stress. These further refinements in the finite-element model were not fully possible without the conclusions of the independent anchor tests that defined the effective engagement length.

## Anchor Capacity and Effective Engagement Length Test

Determining the capacity of the anchorages by tests that are independent of the other stone features is essential in proving that the capacity of the stone panel can be transferred to the supporting backup structure. While preliminary capacities might be estimated by deriving the failure surface area in the stone where it is engaged by the anchorage device, then applying the appropriate material strength, the interaction of the device and the stone, their relative stiffnesses, and the points where that device actually contacts the stone are difficult to accurately determine.

This particular example problem uses a continuous kerf rail to engage the stone panel continuously across the stone's top and bottom edges. This anchorage system is employed because of its compatibility with the backup framework, which also supports the remaining facade fenestration along with retaining the stone onto the building skin. Further, the continuous rail requires only simple operations for proper alignment of the stone during its installation, which results in economical and consistent placement of the anchorage to assure dependable performance. Should a different type of anchorage have been determined to be the best opportunity to satisfy the project's design and installation criteria, the approach to testing those anchorages independent of the stone panel would be identical to that objective and procedure proposed for this kerf anchor. Upon making conclusions from the anchorage test, they must be incorporated into the previous finite-element and preliminary full panel tests to ensure that all critical conditions such as anchor location, anchor shape, behavior, and engagements are consistent among the mathematical models, panel setup, and actual anchorage tests, respectively.

### *Objective and Purpose of the Anchor Capacity and Effective Engagement Length Test*

Both the true capacity of the anchor and its effective, active length of engagement in the stone must be determined independent of the actual panel capacity.

First, the actual realized capacity of the anchorage device-and-local-stone-material at that engagement must be proven to be sufficient to resist the anticipated panel reactions as derived from the design loads superimposed onto the panel during panel analysis.

Second, actually most important, and probably less obvious, the region of actual support, not just actual engagement or contact between the anchor and the stone, must be determined to establish where the anchorage is effective, and therefore, where the support is active.

Because of differential stiffnesses of anchorages due to their shapes, materials, and attachments back to the supporting framework, and the deformed shape of the panel plane relative to the anchorages' locations within the body of the panel, an anchorage device can be engaging the stone without providing resistance to load, and thus, does not provide support for the panel. This can especially be the case with linear-type anchorages such as kerfs, and could also be the case with point-type anchors such as pins, if all points of contact between multiple anchors and the stone they engage do not form a perfect plane throughout the experienced load imposition. That case would exhibit redundant, but inactive supports, which are nearly impossible to discover before a failure, and is therefore the primary reason multiple anchors are not recommended

It simply *cannot* be assumed that placing an anchor in the stone will always provide support wherever it is engaged. Independent anchorage tests will determine both the anchorage's capacity in the stone for the project and where it is effective.

## Anchor Capacity and Effective Engagement Length Test Preparations, Setup, and Execution

The test apparatus should isolate the anchorage behavior from the panel behavior and be able to measure only the stone's behavior mechanics at the anchorage engagement. The stresses at the surface-of-influence within the stone material, emanating from the physical point-of-contact between the anchorage device and the stone, must be greater than the flexural stresses within the panel between the anchorages to be tested in order to preclude the stone failing prematurely for reasons other than the intended anchorage's ultimate failure. To accomplish this, small panels are used, and the load is applied directly outside that plane where the surface-of-influence is expected to occur.

The actual stone selected for the project must be used, and the test specimens be fabricated in the specified thickness. The designed anchorage hole, slot, or kerf preparation must be

## Professional Stone Testing

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Laboratory No. 1234 56-789

Independent Stone Kerf Anchorage Capacity Test

**Introduction:**

Project:
Owner:
Architect:
Structural Engineer:
General Contractor:
Exterior Wall Backup Contractor:
Stone Installer:
Stone Supplier/Fabricator:

This report presents the results of load testing work performed on the nevada beige granite and kerf type anchoring system.The purpose of this test is to prove the capacity and the effective support length of the stone portion of the anchorage system. The kerf rails continuously engage the full length of the stone. The rails attach to the backup at their ends, which correspond to the panel corners. The procedure establishes the stone material strength at the anchorage device and its failure mechanism around the device engaged in the stone's kerf. Structural calculations engineer and verify the attachment of the device to the supporting backup. The anchorage device is an extruded aluminum kerf clip.

**Kerf Anchorage Capacity:**

Allowable kerf clip capacities used in design are proportioned according to the capacities attained by this test and appropriate safety factors. Stone anchor sizes for this project are calculated by proportioning load reactions to tested anchor capacities. Theoretical loads applied perpendicular to the clip (components include wind, snow, or panel weight if the panel is eccentric, or nonvertical) establish support reactions based upon tributary areas for each component. Allowable ultimate capacity of an anchor is the average of the test values reduced by the safety factor appropriate for the application, material, cladding design, and support.

**Effective Length of Engagement:**

Effective length of engagement is the actual length of the clip that *effectively* supports the stone panel. Depending upon relative stiffnesses of rail and stone and backup, the entire length of the clip cannot always be assumed to support the panel along its full length. Because the extruded aluminum rail in this example is less stiff than the stone panel it engages, the rail will only support the panel along part of its length. Some distance away from the rail's attachment to its backup, the engaged clip no longer provides resistance, or support, for the panel. The breakout pattern shows what this distance from the member attachment is. The overall length of breakout is the

The anchorage is stiffest where it attaches to its backup. Because the backup for this example corresponds with the corners of the panel, the panel bends biaxially (across height and width) like a plate between its corner supports.

To conservatively model the anchorage members as more flexible, and thus less resistant to bending between the short span supports, back legs of rails TST1, 2, 3, and 4 were coped various lengths. The effect of these copes over the short two-foot span would be compared and correlated to the various stone widths in the project to determine the actual effective length of engagement.

**Scope:**

This test confirms the ultimate capacity of the engaged anchorage device in the stone at the point of fracture. Due to the complicated material mechanical behavior where the resistance force of the anchorage device is transferred from the anchorage device at the points of contact onto the stone, and then distributed through the stone body, the capacity can only be predicted by testing.

This test will not prove stone panel capacity. , meaning the ability of the stone panel itself to span between the anchorages. This test alone also cannot prove the stability, strength, or overall integrity of the backup support for the anchorages.

**Test Apparatus Setup:**
*(Reference following figure showing setup)*

1. Steel frame test apparatus to mount stones, anchor, and load cell with ram.
2. Structural plastic shims for kerf clip (same size as job condition bearing area).
3. Cee clamps.
4. Kerf anchorage clips.
5. Stone panels with same preparation as job; 1' x 2' x job thkns. Number each edge and then top face "A" and bottom "B".
6. two 1/2" thick x 1"x 2' long neoprene pads for full bearing and contact on stone.
7. 2" x 8" x 2' loading distribution beam(wider if the stone is thicker).
8. 4" x 4" x 2' loading distribution beam.

**Test Procedure:**

1. *Measure critical thickness of kerf fin.* Measure the stone fin at the kerf root with dial calipers to the nearest 100th (0.01 in.) at each end of each panels for both sides of the stone.
2. Engage the kerf clip into the slot cut into the edge of the stone. Assemble remainder of test apparatus as shown in figure.
3. *Measure critical depth of kerf clip engagement.* Measure the distance between the contact point of the kerf clip and the kerf slot's root with dial calipers to the nearest 100th (0.01 in.) at each end of the clip. If the clip does not meet the ends of the stone, this may have to be done by subtraction between the bottom of the kerf fin and the "shelf" angle on the clip. Set engagement at typical project conditions +/- designed tolerances.
4. Repeat above for all kerf leg anchorage corners.
5. Apply a unifrm load across the joint clip, with the contact area outside the anticipated failure zone (a diagonal crack upward and away from the kerf root). Distribute the load from the ram through a 3x3, 2x6, and pads (one inch wide-by-1/2 inch thick neoprene pads) the full length of the joint to model the superimposed loads. Load until the first kerf cracks, recording the following:
   1. Total load at failure to the nearest 5 lbs.
   2. Sketch top and side views of the broken kerf, with dimensions showing crack pattern, lengths, and angles of cracks.
   3. Show the critical dimensions measured for the corners before the test.
6. Turn failed kerf's stone upside down and retest the same panel with the opposite faces' kerfs.

**Test Results:**

| test number | panel number | granite finish | anchor number | load at failure | break length |
|---|---|---|---|---|---|
| 1 | 5B / 2B | sawn / sawn | TST2 | 3665 | 8.5" |
| 2 | 5B / 3B | sawn / sawn | TST2 | 3875 | 6.5" |
| 3 | 5B / 4A | sawn / flamed | TST2 | 2380 | 6.0" |
| 4 | 5B / 4B | sawn / sawn | TST3 | 3370 | 7.0" |
| 5 | 5B / 15A | sawn / flamed | TST2 | 3500 | 7.5" |
| 6 | 6A / 7A | flamed / flamed | TST4 | 3540 | 7.5" |
| 7 | 6B / 7B | sawn / sawn | TST4 | 3095 | 6.5" |
| 8 | 8A / 9A | flamed / flamed | TST1 | 3125 | 6.5" |
| 9 | 8B / 9B | sawn / sawn | TST1 | 3395 | 7.0" |
| 10 | 14A / 16A | flamed / flamed | WIF1 | 6065 | 10.0" |
| 11 | 15B / 14B | sawn / sawn | WIF1 | 3425 | 7.0" |
| 12 | 12A / 13A | flamed / flamed | WIF1 | 4460 | 9.0" |
| 13 | 13B / 12B | sawn / sawn | WIF1 | 5425 | 10.0" |
| 14 | 10A / 11A | flamed / flamed | WIF1 | 3980 | 8.5" |
| 15 | 10B / 11B | sawn / sawn | WIF1 | 3625 | 7.5" |

...(arrange data in 2 columns to condense this)

**Conclusions:**

1. *Summary:*
   2 samples wet with clip TST1, average = 3260 lbs./assembly, breakout = 6.5"
   4 samples wet with clip TST2, average = 3355 lbs./assembly, breakout = 6.5"
   1 samples wet with clip TST3, average = 3370 lbs./assembly, breakout = 6.5"
   2 samples wet with clip TST4, average = 3320 lbs./assembly, breakout = 6.5"
   6 samples wet with clip WIF1, average = 4830 lbs./assembly, breakout = 6.5"
   **15 samples wet total average = 3400 lbs./assembly, breakout = 8.0"**

   Use 3400 lbs./assembly as a conservative average for this type extruded kerf anchorage in nevada beige granite. Only three tests did not meet this capacity:

   3260 lbs./assembly average
   4 kerf anchorages per assembly = **850 lbs. per individual kerf clip anchor**

2. **Comparing the results using the various stiffness clips TST1, 2, 3, and 4 shows that the capacities and the breakout lengths are nearly unchanged. This suggests that the relative stiffness of the anchorage clip itself where it attaches to the supportdoes not effect the capacity. The uncoped WIF1 section showed 65% increased capacity. The interactive behavior between the stone and continuous anchorage kerf rail over the full 6' span will be proven with the full-size chamber test.**

**Remarks:**

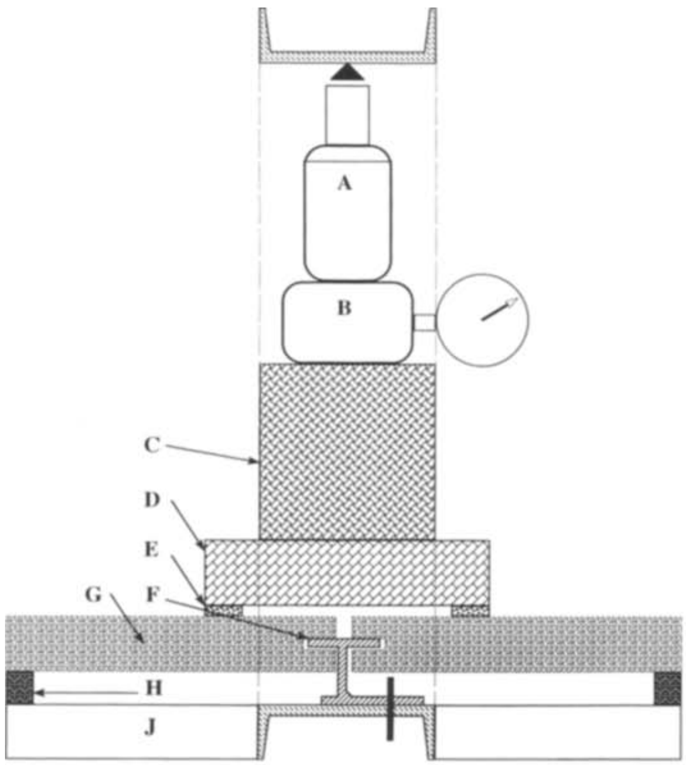
The test samples will be retained by the laboratory for a period of two weeks following issue of this report for observation by the client. Samples are the discarded unless the laboratory recieves further instructions from the client.

Respectfully submitted,
Professional Stone Testing

▲
◀ FIGURE 128: *Stone Kerf Anchorage Capacity Test Report.* The strength of the panel and the capacity of its anchorages should be established separately before testing the entire assembly. This test isolated the behavior of the anchorage device where it engaged the stone. Without knowing the capacity of a single anchor and its failure mechanism in the stone independent of panel bending influence, it is improper to proportion different size anchorages for different size stones or make final conclusions on the anchor's ultimate strength. While this example report represents a procedure and rationale for obtaining capacities for kerf-type anchors, similar sequences should be followed for other type anchors. "Kerf Anchorage Capacity" identifies the significance and use of the test. "Effective Length of Engagement" explains the influence of relative stiffnesses between stone and anchor. "Setup" and "Procedure" outline a process which could be customized if necessary to fit specific project conditions.

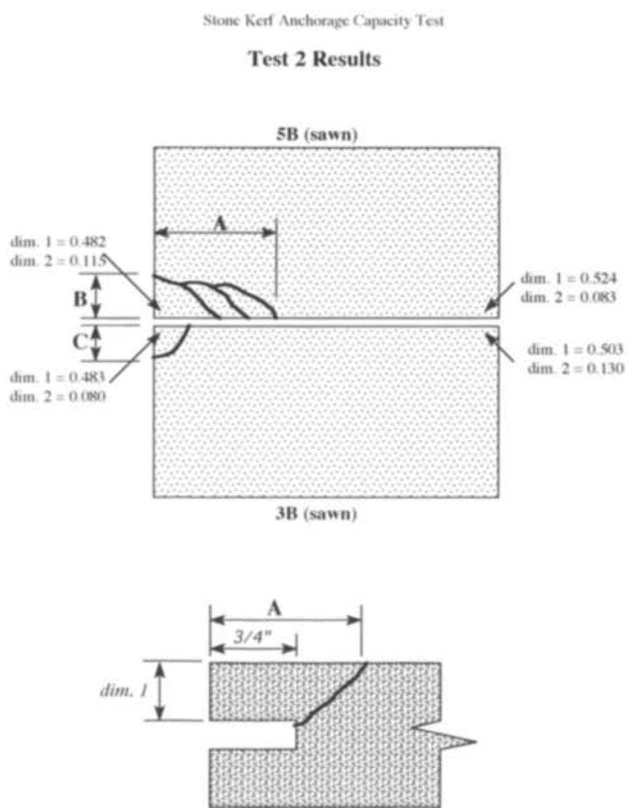
fabricated within the limited ranges of fabrication tolerances for that operation. It is recommended that the test specimens should represent the full range of the tolerances expected and identified for the actual anchorage preparation in the stone. Substantial enough samples should be conducted to offer some statistical reliability, and should be commensurate with the risk involved and the degree of maximum capacity that will be required for the project. Twelve tests were attempted for each of the "end anchors" of the example, with four anchors per each setup, giving the minimum capacities of nearly fifty anchorages.

Because each of these independent anchorage tests usually involves a unique test apparatus setup in order to accommo-




◄ FIGURE 129: ***Independent Anchor Test Setup for Kerfs.*** **The apparatus shown applies load to the stone directly outside the imminent plane of influence of two stones on either side of the joint between panels. In the example conditions, the anchors attached to their backup at the jambs of the stone so the effective length begins at the edge of the panel. The kerf rail engaging the kerfs and supported the stones fastened at its ends, thus received the load from the stones at four different reaction points (both stones (2) at both ends (2)). The setup includes:**

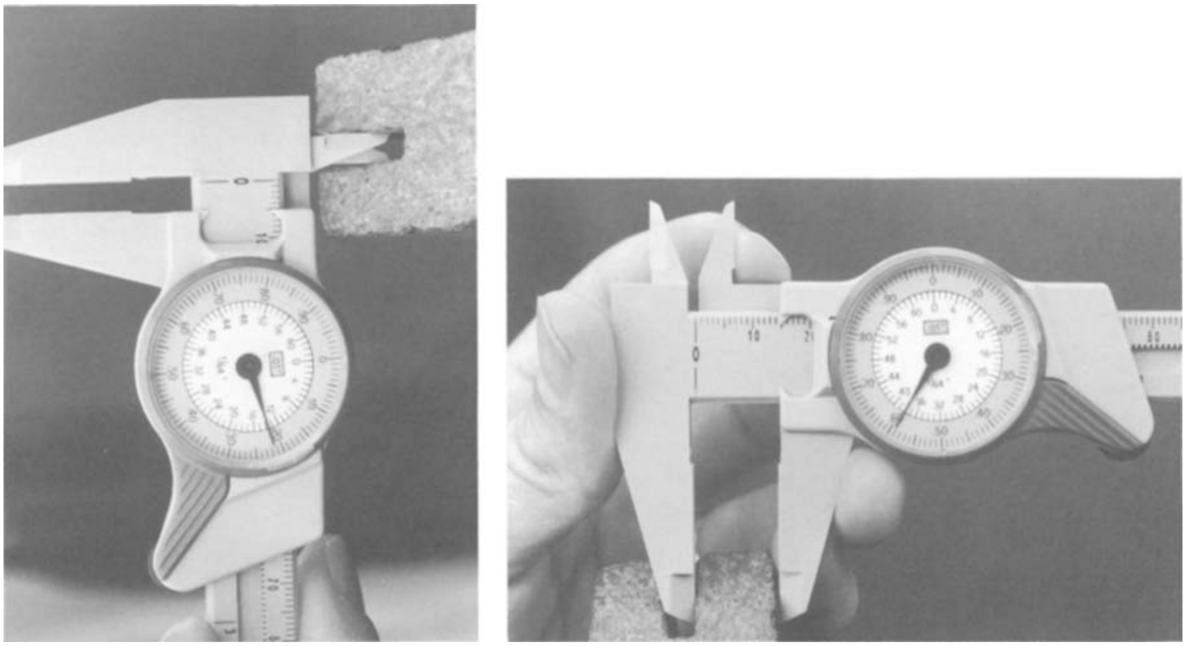
**A: Hand-pumped hydraulic ram to apply load.**
**B: Calibrated load cell with digital readout in pound.**
**C: 4x4 unwarped oak block to spread load across width of stone.**
**D: 2x6 spreader to distribute load across joint outside failure plane.**
**E: Continuous compressible bearing pads to prevent load contact onto stone fins.**
**F: Anchor device engaged in kerf. Assure that the relationship between the device and stone agrees with project conditions.**
**G: Stone Panel. Measure critical dimensions. Assure conformance with project limits.**
**H: Corner blocking to stabilize the outside corners, with clamps are required.**
**J: Apparatus frame (Including channel above ram).**




◄ FIGURE 130: ***Stone Kerf Anchorage Capacity Test Results.*** **The conclusions of this test should be reported graphically to represent the results of the test. Each "setup" actually evaluates four sample anchorages in this setup. The fifteen test runs tested sixty kerf anchorages. The results summarized the average capacity of the fifteen weakest, since most setups failed one kerf at rupture, while the other three remained intact. In Test #2 shown, the kerf that failed is logically the one with the thinnest stone fin (0.482 in.) with the longest leverage arm (0.115 in.). Using the total breakout length as the effective length of engagement (A), with its diagonal surface in section, the surface area of the failure plane could be calculated and the failure stress compared to C99 test values. The report's conclusions should document both capacity and effective length at a minimum.**

date and isolate the specific devices, stone, and support configurations unique to that particular project, this test is difficult to standardize. However, four important procedural concepts must be included:

1. The stone must be representative of that material and quality-of-fabrication that is to be used for the project, including, and especially regarding the features of the stone at the sawcut, milled, or drilled anchorage preparation.
2. The anchor must be representative of that device that is to be used for the project, including, and especially regarding the material, alloy, and shape of the device where it engages and also contacts the stone.
3. The point where the anchor contacts the stone must be represented in the test the same way it is designed to be installed in the field. If possible, it might be suggested to vary that point-of-contact equally through the range of designed tolerances that can be experienced in the field. These variances must be carefully measured and recorded.
4. The means of anchorage attachment to its support and its location must be represented in the test the same way it is designed to be installed in the field. Because this attachment of the anchor to its support is critical to the



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FIGURES 131 *(left)* and 132 *(right)*: *Field Measurement of Stone Panel and Kerf Thickness*. **The stone fabricator needs to conduct a thorough quality assurance program to check tolerances, panel thicknesses, kerf thickness, and all other types of anchorage preparations. These aspects are critical to panel strength and structural integrity, and are usually measured several times across the edges with calipers prior to crating for shipment. Figure 131 measure the width of the kerf slot at its "root". Figure 132 measures the thickness of the kerf "fin".**

anchorage device's torsional and flexural stability and stiffness, which in turn directly influences the engagement performance both in capacity and effective engagement. Both the location of that support and its means of attachment must be duplicated from the designed and intended system condition. The following example proves why this is important, as the use of the rubber pad allowed instability, which resulted in premature failures.

### *Anchor Capacity and Effective Engagement Length Test Data Collection Requirements*

Load is applied directly outside the predicted plane of influence until either rupture occurs or the anchorage device itself fails, and the magnitude of the load at failure is recorded. If the breakage occurred beneath where the load was applied, and was not due to the function of the anchor, the test is considered invalid.

The maximum load magnitude at failure is divided by the number of anchorages in the setup that are resisting the load, as some setups may employ multiple anchors. This quotient then becomes the capacity of the anchor that failed, which is the minimum capacity in comparison to the others anchors in the setup that had not yet failed, and therefore had not reached their capacity. Closely document by sketch, scale, and photograph (which is helpful when a legible measuring device is included within the frame) the failure plane pattern and the device. Note any deformations, abrasions, or other feature of the device that might indicate it failing before the stone, which would discount the potential capacity of the stone. Measure specifically the length and patterns of the cracks at the failed stone. This length of the failure is where the anchorage device was effectively supporting the stone up to the failure load, and is known as the effective length of engagement. Record of the load, cracks, and anchorage features are the contents of this test report.

### *Anchor Capacity and Effective Engagement Length Test Report Example*

Figures 128 and 129 include the Effective Length of Engagement Test Procedure for a kerf anchor in granite panels. Figures 130 through 133 include the reported laboratory test results.

### *Anchor Capacity and Effective Engagement Length Test Data Evaluation*

The capacity of the tested anchor must exceed the required support reaction of the panels by the prescribed margin-of-safety. For ASD, the tested capacity is divided by the specified factor-of-safety, whose quotient cannot be exceeded by the reaction results from the panel analysis.

The effective engagement length is compared with the support provisions assumed in the panel analysis to be sure that all boundary condition assumptions correlate to conclude a safe design through this stage of testing. While it is not expected that all tests will have assumed equivalent anchorage locations from the beginning, one could interpolate the results of the full panel tests and the finite-element analyses or revise and rerun the finite element analysis to improve correlation and then compare conclusions.

### *Anchor Capacity and Effective Engagement Length Test Interpretations and Conclusions*

Comparing the independent anchor capacity and effective engagement length test to the finite-element analysis model—the tested anchor capacity must exceed the reaction of the stone panel, which is intended to support by the prescribed margin-of-safety. Less than adequate capacity requires strengthening the anchor.

If the stone failed, then capacity can be increased by configuring the anchorage device to increase the surface-of-influence's area by increasing effective engagement length or simply by increasing stone kerf fin thickness. If neither of these is promising, then the panel size should be reduced to lower the magnitude of the support reaction. As discussed previously, it is not recommended to increase the quantity of the anchorage locations. Additional or secondary anchorages, because more often than not, they do not fall into the same plane with the other primary anchorages, are not always supplementary or even complimentary.

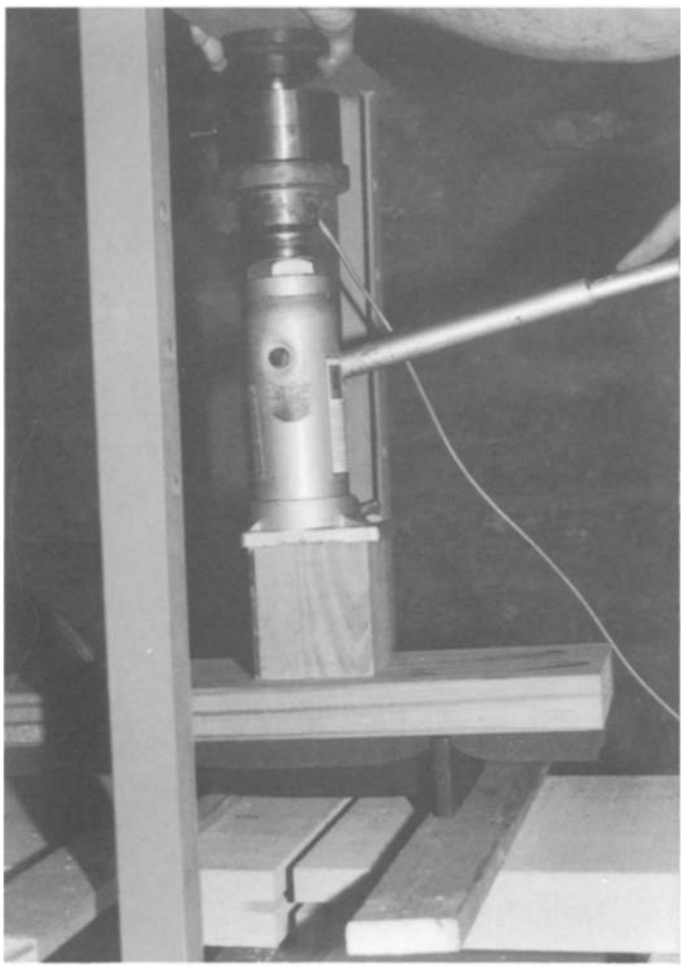

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FIGURE 133: *Independent Kerf Anchorage Test.* The apparatus shows kerfed stone samples supported with an anchorage device engaged at their center like the diagram in Fig. 129. Similar to the wind pressure against the stone, a hydraulic jack apples load through the load cell and spreader beams onto the stone. The spreaders contact the stone directly outside the imminent failure surface emanating from the root of the kerf slot.

Conversely, if the tested capacity far exceeds the required resistance, redesign could offer greater economy with smaller or lighter anchor devices or perhaps support structure. Indepth study of these value engineering considerations must be thoroughly integrated through the previous testing steps to assure proper correlation, which probably requires the tests to be executed again.

Locations of the anchorages should also be verified with the previous tests. Centers of effective engagement lengths are recommended to be used as the support locations in the finite-element models. If these locations result in shorter spans than the finite-element analysis had modeled, then the resulting bending moments and stresses resulting from that analysis will also reduce. It is recommended that the finite-element model be refined to include the conclusions from the anchorage test and the preliminary full panel test to optimize their correlation before proving the entire design with the chamber test. This may require that several different mesh configurations be attempted and compared. For the example problem, the mesh was refined to these horizontal node incidences, since the predominant flexural stresses were generated across the longer horizontal span: 0, 4, 7, 11, 21, 29, 36, 43, 51, 61, 65, 68, and 72 inches as shown on the finite-element model diagram. Given the four-inch center for the effective engagement length, the support was relocated to nodes 11, 20, 111, and 120, which reduced the net span to 64 inches and improved correlation with the other test methods.

Comparing the independent anchor capacity and effective engagement length test to the preliminary panel total load capacity test. Like correlation with the finite-element model, the anchorage locations used in the panel capacity test should be relatively close to the centers of the effective engagement lengths concluded from the kerf anchor test. If the panel broke at the midspan during the panel capacity test, then the anchorages should support at least the load that failed the panel. Design anchors to be stronger than the panel.

### Complete Assembly Full-Panel Chamber Test

Being the last test in the stone-testing series, the Complete Assembly Full-Panel Chamber Test is intended to verify that all the parts work together. It applies wind loads with pressure difference in a sealed chamber similar to the reaction of a cladding on a building wall. The materials, the panel, and the anchorages were all tested and analyzed individually to assure that each of their performances and capabilities were adequate for the overall stone anchorage system. Their performances

## Professional Stone Testing

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Laboratory No. 1234 56-789

Complete Assembly Full Panel Chamber Test

**Introduction:**

Project:
Owner:
Architect:
Structural Engineer:
General Contractor:
Exterior Wall Backup Contractor:
Stone Installer:
Stone Supplier/Fabricator:

This report presents the results of load testing performed to establish the structural capacity of full-size stone panels assembled complete with their anchors and backup intended to be used in the project. The test included actual size nevada beige granite panels and aluminum anchorages engaged in their kerfs representative of job and building conditions. These components should represent the conclusions of previous engineering and testing of individual parts of the exterior wall stone system. The stone anchorages were attached to a metal frame similar to the mullion system backup of the project, which was then installed into a sealed test box (chamber. The test box would be pressurized and depressurized alternately to simulate wind pressure gradients and pressure reversals that occur on a building facade.

**Test Setup:**

The test assembly inc luded three stones, a 6'-0" wide by 4'-3" tall center stone with a half panel at the top and bottom to simulate the adjacent stone panels (see diagram). Continuous extruded aluminum kerf rails engaged the stones along the horizontal joint. These anchorages attached to the mullions at the jambs, which were anchored to the box. Displacements were measured at five locations (midwidth and midheights of each edge) and center of panel. Average span ratio represents the approximate displacement at midpanel across the diagonals for reference purposes only. Because stone panel anchorages were fastened to mullions that moved with the pressure, these coefficients represent overall movement of the *system*, not the panel itself.

**Test Procedure:**

All tests were performed in accordance with test procedures outlined in ASTM **1201**- (most recent approved version) *Test Method for Structural Performance of Exterior Dimension Stone Cladding Systems By Uniform Static Air Pressure Difference* as included in the project requirements.

All loads shall be reached and released promptly with no period of sustaining at maximum pressure. Project safety factor of 1.5 for the complete assembly in a design wind pressure area of 45 psf allows no stone breakage or permanent anchorage deformations below 67 psf.

**Test Results:**

| test | direction, pressure | percent of design | failure | deformations 1 | 2 | 3 | 4 | 5 | avg. span |
|---|---|---|---|---|---|---|---|---|---|
| 1 | +22.5 psf | +50% | none | none taken | | | | | |
| 2 | - 22.5 psf | - 50% | none | none taken | | | | | |
| 3 | +45.0 psf | +100% | none | 0.23 | 0.18 | 0.21 | 0.15 | 0.17 | 1/419 |
| 4 | - 45.0 psf | - 100% | none | 0.29 | 0.27 | 0.25 | 0.17 | 0.20 | 1/352 |
| 5 | +56.3 psf | +125% | none | none taken | | | | | |
| 6 | - 56.3 psf | - 125% | none | none taken | | | | | |
| 7 | +67.5 psf | +150% | none | none taken | | | | | |
| 8 | - 67.5 psf | - 150% | none | none taken | | | | | |
| 9 | +78.8 psf | +175% | none | none taken | | | | | |
| 10 | - 78.8 psf | - 175% | none | none taken | | | | | |
| 11 | +90.0 psf | +200% | none | 0.58 | 0.53 | 0.45 | 0.38 | 0.42 | 1/196 |
| 12 | - 90.0 psf | - 200% | none | 0.64 | 0.60 | 0.58 | 0.41 | 0.49 | 1/215 |
| 13 | +101.3 psf | +225% | none | none taken | | | | | |
| 14 | - 101.3 psf | - 225% | none | none taken | | | | | |
| 15 | +112.5 psf | +250% | none | none taken | | | | | |
| 16 | - 112.5 psf | - 250% | none | none taken | | | | | |
| 17 | +123.8 psf | +275% | none | none taken | | | | | |
| 18 | - 123.8 psf | - 275% | none | none taken | | | | | |
| 19 | +135.0 psf | +300% | none | 0.50 | 0.68 | 0.70 | 0.51 | 0.62 | 1/126 |
| 20 | - 135.0 psf | - 300% | none | 0.82 | 0.87 | 0.80 | 0.57 | 0.78 | 1/110 |
| 21 | - 146.3 psf | - 325% | none | none taken | | | | | |
| 22 | - 157.5 psf | - 350% | vertical crack through middle of panel at - 143.0 psf | | | | | | |

**Conclusions:**

**The panel with its anchorages as an assembly reached 346% (156 psf) of design load when it was required to exceed it 150% (127 psf). Failure occurred upon re-pressurization at 326% (143 psf), which exceeds the 150% requirement established by the project documents.**

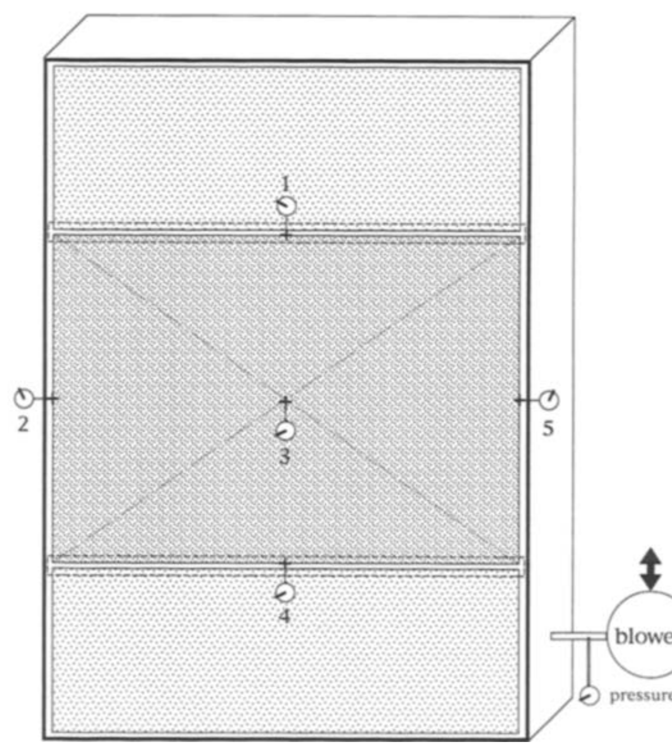
**Remarks:**

The test samples will be retained by the laboratory for a period of two weeks following issue of this report for observation by the client. Samples are the discarded unless the laboratory recieves further instructions from the client.

Respectfully submitted,
Professional Stone Testing

◀ FIGURE 134: *Complete Assembly Full Panel Chamber Test.* Also known as ASTM C1201, this procedure *proves* how well the interactive behavior of backup, anchor device, panel flexure, and reversing loads work together to achieve the capacity required for the facade. Without completing panel tests and anchor tests separate from this more expensive and time-consuming procedure, it may be difficult to both isolate system deficiencies and also to correlate the results to other size panel and anchorage configurations. Previous panel and anchor tests also allow discrete engineering economy measures to reduce costs where possible. The test involves constructing an airtight chamber onto one face of the prototype assembly of a portion of the wall. Alternating positive (windward) and negative (leeward) pressures are included at incrementally increasing proportions of the design load multiplied by the safety factor derived for that project. Loads should be increased until failure if possible. Since stone overdesign SF (perhaps 2.5 to 10) exceeds curtainwall ultimate limits SF (1.5), the chamber must be robustly built to prevent it from influencing the behavior of the wall assembly.

FIGURE 135: *(below) Chamber Test Setup.* The test setup included constructing the stone panel on its anchorages and its backup using exactly the same components to be used in the project. Sizes should duplicate the conclusions of previous tests and all structural backup systems interfacing the prototype exterior wall. Half-panels are included above and below the full panel so model balanced loads on the anchorages. Deflections were measured at midpanel, midwidths, midheights, and also anchor points to verify relative movements. Pressure gauges and a mercury level should monitor blower pressure rate change and maximums.



during their testing were precisely evaluated, and each of those components was adjusted and refined to best "fit" their role within the system. However, not until all are tested together can they be proven to work symbiotically to the same level as they had independently. Further, force reversals created in the chamber will expose these parts to effects not modeled in previous tests, which could cause behaviors not predictable from those tests.

This Complete Assembly Full-Panel Chamber Test is a time-consuming and expensive test to setup and conduct because of chamber construction, full-size components, and monitoring apparatus and instrumentation. Causes of failures during this test are not always obvious without the previous knowledge of actually tested performances of each of the components in the system, which is why the other tests are recommended to successfully precede this one in order to establish an objective confidence for each of those components' capabilities. This test completes the proof for the stone panel and anchorage system.

Several of the most typical configurations are usually selected for this model, which proves as much of the facade as possible, yet is somewhat economical. Test pressure levels are established by the project's performance criteria, which is based upon the ASD-type philosophy for this case study. Using load-and-resistance factors, once developed, may alter the criteria and parameters the components and system are tested to, but will not alter the techniques and process of testing that establishes proof of structural adequacy.

### *Objective and Purpose of the Complete Assembly Full-Panel Chamber Test*

Using the conclusions from the Actual Panel Test for Preliminary Load Capacity, which verifies the stone panel's capacity separate from its anchors, and the Anchor Capacity and Effective Engagement Length Test, which verifies the anchor's capacity and effective support location separate from the stone panel, the Complete Assembly Full-Panel Chamber Test will prove the interactive performance of the panel with its anchorages with alternating-sense loads that simulate actual loadings that will be experienced by the cladding in-place.

The nature of stone and its anchorages will require higher ultimate resistances than any glass-and-metal portion of the exterior wall because the stone has a greater range of uncertainties. Maintaining a consistent confidence for adequate performance and a consistent margin-of-safety will mean that higher proof loads are required for stone than are required to be resisted by the remaining exterior wall system during a test. This load requirement mandates the totally separate testing procedure.

Typical mock-up chamber tests for curtainwalls will pressurize their systems to one-and-one-half times design loads to prove that the combination of all the parts act together with enough reserve strength to resist permanent deformation. Under the allowable-strength design premise, stone testing may need to surpass two-and-one-half times design loads to prove their reliable capacity. This load level would destroy other curtainwall components if they were combined in the same actual test.

### *Complete Assembly Full-Panel Chamber Test Preparations, Setup, and Execution*

Procure the same stone size, shape, thickness, and finish used for the Actual Panel Test for Preliminary Load Capacity, which also should be those same parameters used in the project for the condition to be tested. Use the same anchorage devices that were proven in the Anchor Capacity and Effective Engagement Length Test, including all fasteners, shims, sealants, and installation methods to be executed for the project. These anchorages should be distributed onto the panel in the identical pattern proven to be successful in the Theoretical Panel Test by Finite-Element Structural Analysis as well as the Actual Panel Test for Preliminary Load Capacity, or the conclusions derived from the correlations of these tests with the final development of the supporting exterior wall backup framing that will be retaining the assembly onto the building. For the final assemble test, these anchorages must be engaged within the stone in the identical fashion proven to be successful in the Anchor Capacity and Effective Engagement Length Test, which will be executed during the installation of the work into the project.

All is assembled onto a sealed chamber that will be pressurized to simulate differentials caused by changing wind velocities and directions in the same orientation as the building's, meaning vertically for a vertical wall. These naturally occuring force reversals are created by incrementally increasing and alternating positive and negative pressures within that sealed chamber by using a controlled air-blower. Sequence of loads should proceed in multiples of the design load of the panel and anchorage (+/- 50%, +/- 100%, +/- 125%, +/- 150%, +/- 175%, and so on to be continued until failure). These loads are precalculated according to the design load magnitude, and are published with the test procedure specifically developed for this project condition before the test is run.

To record the relative movements of the panel and its anchorages, erect a rigid grid in front of the chamber-mounted stone, with dial guages placed at each anchorage (theoretically at the point where the anchor engages and contacts the stone ) and along the edges and midwidths between anchorages where the panel is expected to reach maximum stresses and eventually fracture. It is also suggested, if possible, to place dial guages at locations where the anchorages attach to the chamber to both monitor chamber dynamics and subtract them from anchorage movements, and also to confirm that the anchorage is stable between its attachment point and the point where it engages and contacts the stone. Displacements should be recorded at even increments of design loadings (+/- 100%, +/- 200%, +/- 300%, and so on to be continued until failure).

### Complete Assembly Full-Panel Chamber Test Data Collection Requirements

Upon reaching the ultimate capacity of the assembly, and either fracturing the panel or breaking its anchorage, the pattern of the failure is recorded by sketch, measurement, and photograph and compared with the results of previous tests. If anchorages are designed for greater resistances than the panel itself, then it should be expected that the panel will fail prior to the anchorage, or fails before the stone around the anchorage, as long as all interfacing behaviors have been correctly modeled during the previous tests.

Because the primary purpose of this procedure is to prove proper interaction between the anchorage and the panel under loading conditions similar to those the assembly will experience over its life upon the building, although the magnitudes are multiples of the design loads, closely examine the stone around the anchorages during each dial guage reading. Dial guages at each of the anchorages should indicate negligible movement if the chamber is sufficiently stiff and strong, with perhaps only some flexure of the engaged kerf rail leg. This displacement *must not* allow premature contact of another part of the kerf rail with the toe of the stone kerf fin, which would change the point-of-contact within the stone's kerf slot from that intended and result in premature failure.

With displaced panel curvature, or twisted kerf rail that causes a changed point-of-contact, prying results that will likely cause immediate and premature failure. This full-panel assembly test will enable the full ranges of movement between the panel and anchorage to be realized, and any potential for interactive behavior that would threaten performance of the system will be simulated in the test.

The assembly should survive test pressures equal or in excess of the required designed resistance, which accounts for the variabilities and uncertainties of the stone material, anchorage device, and exterior wall system interactive dynamics as deduced from the project's specified performance criteria and modeled by the sequence of tests.

### Complete Assembly Full-Panel Chamber Test Report Example

The following test procedure conducted and reported by Figures 134 and 135 parallels the ASTM E 331 Procedure for Structural Performance of Curtainwalls and ASTM C1201. It uses a pressurized chamber, around a mock-up prototype of a wall to prove its strength against differential wind pressures. The adaptation of this standard procedure for stone maintains the same proof criteria, which states that "no stone breakage at anchorage shall be allowed or distortion below –90 psf = (2.0 times design load for stone). Loads shall be reached and released promptly with no period of sustaining at pressure." Using a chamber separate from the glazed wall for the stone allows loads to be increased above proof loads to determine capacity at breakage without being limited to glass, gasketing, or window framing limitations. Loads were recorded in the alternating sequence, with displacements at predetermined intervals also recorded. The –90 lbs/ft$^2$ represents the overall exterior wall criteria's limit that requires that no component sustain any permanent deformation at 150% of design load (1.5 x 45 lbs/ft$^2$ = 67.5 lbs/ft$^2$) and no stone breakage at 200% (2.0 x 45 lbs/ft$^2$). Since this example involved retention of the stone by the curtainwall which is engineered to 67 lbs/ft$^2$, it is likely that this system may be the limiting component of the facade.

### Complete Assembly Full-Panel Chamber Test Data Evaluation

Three different typical configurations were tested. One duplicated the preliminary panel test to establish a correlation. Two other configurations, also expected to be critical, are tested to prove their adequacy. Results of the previously reviewed 6'-0" Window Stone configuration follows:

| | | LOAD MAGNITUDES | | | | | | DEFLECTION |
|---|---|---|---|---|---|---|---|---|
| | *Location* | *+100%* | *-100%* | *+200%* | *-200%* | *+300%* | *-300%* | *max / span* |
| 1 | Mid-Span at Head | 0.080 | 0.150 | 0.175 | 0.385 | 0.300 | 0.410 | Horiz= |
| 2 | Mid-Span at Sill | 0.060 | 0.105 | 0.145 | 0.275 | 0.230 | 0.395 | L/200 |
| 3 | Mid-Span at Left | 0.065 | 0.100 | 0.130 | 0.250 | 0.220 | 0.370 | Vert= |
| 4 | Mid-Span at Right | 0.070 | 0.075 | 0.145 | 0.230 | 0.220 | 0.295 | L/183 |
| 5 | Center of Panel | 0.095 | 0.025 | 0.195 | 0.330 | 0.385 | 0.490 | L/199 |

TABLE 13. ***Full-panel Chamber Test results***

### *Complete Assembly Full-Panel Chamber Test Interpretations and Conclusions*

Given that the performance criteria for this example study stipulated that, under the ASD-type regime, the minimum safety factor is to be SF = 2.0, for the panel and the minimum safety factor is to be SF = 3.5, for the anchorages, panel breakage at -127 $lbs/ft^2$ is 282% (realized SF = 2.82) of the designed 45 $lbs/ft^2$ load, which exceeds the specified 200% of 90 $lbs/ft^2$ (SF = 2.5). Therefore, the assembly's performance is acceptable and structurally in conformance with the design criteria.

Successful completion of this series of tests has confirmed that the design of the stone panels, their materials and sizes, the anchorage devices and installation techniques do meet this performance criteria specified under the Allowable Stress design approach. The arbitrary nature of this criteria does not assure safety; therefore, this design's conformance to this criteria also does not guarantee safety. It is the best approach, however, until LRFD develops.

Sequentially evaluating all the prescribed tests and analyses in a logical manner outlines how to assess the system's expected performance. Rational derivation of performance criteria that these tests could compare to, whether by Allowable Strength Design or Load-Resistance Factor Design philosophies, is the topic of further study. The testing process is valid for both.

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# INDEX

**Michael D. Lewis,** AIA

## ABOUT THE AUTHOR

AS LEAD CONSULTANT for the Facade Group at THP Limited in Cincinnati, Ohio, Lewis works on both existing and new building facades. He investigates existing facade distress and its causes and develops rehabilitations to restore their integrity. Using knowledge from these exemplars, he develops new stone facade systems that simplify production and installation techniques while optimizing quality and durability. His combined environmental, structural, architectural, and installation expertise allows a comprehensive approach to facades. He is also an Adjunct Professor of Architecture at the University of Cincinnati, teaching construction theory, structural economy, and curtain wall science.

Mr. Lewis is a registered architect and holds a Master of Science in Structural Engineering. His facade expertise includes historic preservation, construction management, and building technology. Notable recent projects include repair of the terra cotta facade of Cincinnati's Central Trust Tower (34 stories designed by Architect Cass Gilbert), rehabilitations for the College Conservatory of Music at the University of Cincinnati. (Architects Pei Cobb Freed and NBBJ-Roth), the stone skin for the Dubai National Bank Headquarters on the Persian Gulf shore, Harrah's Jazz Casino in New Orleans (Perez Ernst Farnet, Architects) and the Federal Reserve Bank of Cleveland (Architects Hellmuth, Obata, Kassabaum with VanDijk Pace).

Lewis began his professional career researching and developing special lightweight dome, cable, and envelope structures with the engineering firm of Geiger-Berger Associates PC of New York. He then was project engineer for specialty facade subcontractor Industrial First and directed engineering of the stone facades of two Chicago high-rises; 190 South LaSalle (42-stories by Architect Philip Johnson and John Burgee) and AT&T Corporate Center (70-stories by Architect Skidmore, Ownings, and Merrill). Later, Lewis joined Harmon Contract W.S.A.'s Commercial Construction Division as a Project Manager in the Major Projects Group. With Harmon he directed the engineering, manufacture, assembly, installation, and contract administration of total envelope systems for Cincinnati's Chemed Center (32-stories by Architect Skidmore Owings and Merrill) and the University of Cincinnati's Engineering Research Center (8-stories by Architect Michael Graves and KZF).

As chairman of ASTM C18.06 on Dimension Stone Anchors and Anchoring Systems, Lewis directs standard development for cladding engineering such as ASTM C1242 *Standard Guide for the Design, Selection, and Installation of Exterior Dimension Stone Anchors and Anchoring Systems,* under the advisement of many experts on the committee. He is also an active member of other C18 committees responsible for specifications, testing, and durability standards for dimension stone. As a member of Committee E6 on Building Performance, he is also involved with promulgating standards for exterior building wall performance, historic building technology, and rehabilitation.

Mr. Lewis is a member of the American Institute of Architects, the Marble Institute of America, the National Trust for Historic Preservation, the Association for Preservation Technology, and Terra Cotta Conservation Group. He has authored and contributed to articles published in *Stone World, Dimensional Stone, Architecture,* and *Standardization News* and has presented lectures on exterior cladding to many audiences.

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