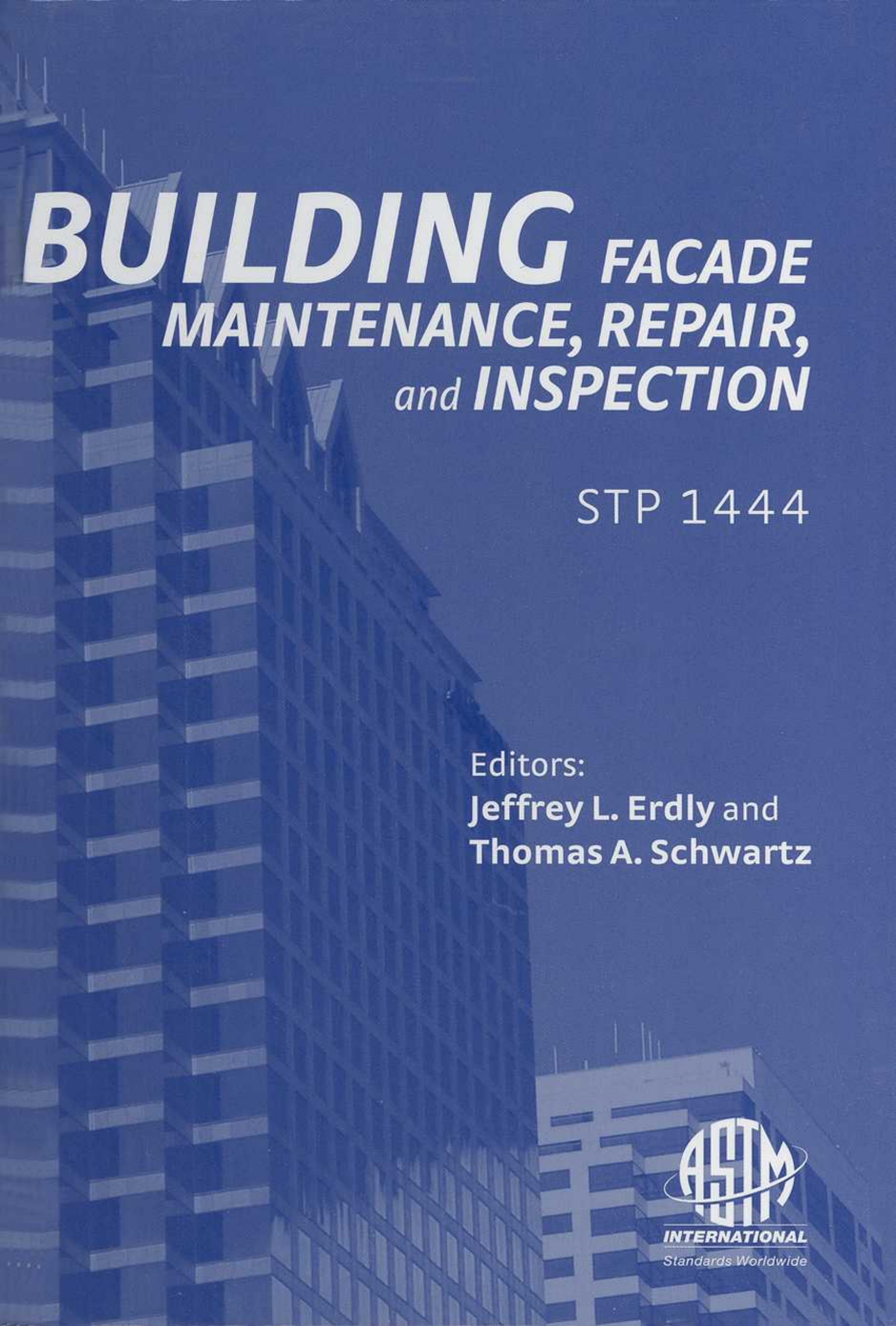

BUILDING FACADE MAINTENANCE, REPAIR, and INSPECTION

STP 1444

Editors:
Jeffrey L. Erdly and
Thomas A. Schwartz

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Building Façade Maintenance, Repair, and Inspection

Jeffrey L. Erdly and Thomas A. Schwartz, editors

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May 2004

Foreword

The Symposium on *Building Façade Maintenance, Repair, and Inspection* was held in Norfolk, VA on October 12–13, 2002. ASTM International Committee E06 on Performance of Buildings served as its sponsor. Symposium chairmen and co-editors of this publication were Jeffrey L. Erdly and Thomas A. Schwartz.

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Overview

Building facades are not static. They move in response to wind effects and temperature changes. They interact with the structural frames that support them. They degrade with age and, occassionally, lose attachment to the building. Loss of façade materials is a growing problem. Only eight U.S. cities have adopted some form of local ordinance requiring inspection of building facades to detect unsafe conditions, and these ordinances vary considerably in thoroughness, effectiveness, and enforcement. In some cases, façade ordinances have done little to reduce the threat and, in fact, have resulted in a false sense of security concerning the safety of building facades. Facades that have been inspected have lost significant façade materials within a year or two of the inspection.

The papers published in this special technical publication (STP) were presented at a symposium entitled *Building Façade Maintenance, Repair and Inspection,* held in Norfolk, Virginia on October 12–13, 2002. ASTM International Committee E06 on performance of buildings sponsored the symposium as a parallel effort with the final development of ASTM's Standard E 2270, *"Standard Practice for Periodic Inspection of Building Facades for Unsafe Conditions,"* which received final approval in the spring of 2003.

The first known building code, Hammurabi's Code of Laws (1700 B.C.), included the following: "if a builder build a house for someone and does not construct it properly and the house which he build fall in and kill it's owner, then that builder shall be put to death." While the sentence of death seems harsh, the underlying implication of a responsibility to protect those using our buildings during their everyday life is clear. It is the intent of the papers in this book, combined with ASTM Standard E 2270, to provide a rational guide for building owners and governing authorities to help ensure the safety of our aging building infrastructure.

The papers contained in this publication provide insight with regard to four major headings. They include: 1) Purpose and Background to Façade Ordinances; 2) Addressing Historic Buildings; 3) Investigation and Data Collectino Techniques; and 4) Material and Repair Techniques. The authors who generated these papers, architects, Engineers, public and private institutional facility owners, and contractors, bring to their work first hand knowledge and experience that covers the wide diversity of architecture within North America.

These papers, combined with ASTM Standard E 2270, represent a starting point for this important work. ASTM committee E06.55, through its ongoing task group, will be expanding its work to include additional annex information. The proposed topics include, but are not limited to, public sidewalk protection, safety of inspections, hazardous materials, safety considerations for inspection openings, mechanisms of distress, structural movement, and material-specific guidelines.

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Section I: Purpose and Background to Façade Ordinances

Jeffrey L. Erdly[1] *and Gregg M. Bekelja*

Reporting Unsafe Conditions at Public Schools and Private Structures

Reference: Erdly, J. L. and Bekelja, G. M., "**Reporting Unsafe Conditions at Public Schools and Private Structures,**" *Building Façade Maintenance, Repair and Inspection ASTM STP 1444*, J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: As a building restoration contractor specializing in historic masonry repair and restoration, building owners and architects request that we review building facades with regard to unusual conditions. Over the past 20 years, we have observed structures where life safety is of immediate concern. While we acknowledge our responsibility to notify those responsible for these conditions, we are sometimes frustrated by owners and professionals who are ambivalent to the risks identified.

This paper will review public school buildings and private institutions where, in our opinion, public safety was compromised. A national standard requiring the periodic inspection of building facades is needed to protect the public, especially children who attend our public schools.

Keywords: façade, public schools, life safety, masonry, unsafe conditions

Introduction:

The exterior walls (façade) of a building require periodic maintenance like all other major systems within a structure. The roof (horizontal closure) is widely recognized as needing preventative maintenance Arepair@ to extend its useful service life, along with a structured replacement program intended to protect the structure from the affects of water leakage. This in turn is also intended to maximize the useful life of the structure as a whole. Few owners understand that the vertical closure (façade) also requires a similar commitment to preventative maintenance.

Mr. Samuel T. Harris, PE, AIA, Esquire, in his book entitled *ABuilding Pathology Deterioration, Diagnostics and Intervention@* puts forth the following concept of the deterioration mechanism of buildings. Mr. Harris identifies six (6) major subsystems of a building and their respective effective life spans:

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[1] President and Vice President, respectively, Masonry Preservation Services, Inc. (MPS), P. O. Box 324, Berwick, PA 18603.

Structure	100 year effective life span
Vertical closure	40 year effective life span
Horizontal closure	20 year effective life span
Climate stabilization	15 year effective life span
Hydraulic	20 year effective life span
Energy	30 year effective life span

During the operation of a building, four of these six subsystems receive preventative maintenance and/or replacement to allow for the continued use of the structure, and are generally considered as normal maintenance. These include: 1) horizontal closure (roof), which when failed allows liquid water into interior spaces, making building operation difficult or impossible; 2) climate stabilization (HVAC), which directly impacts the comfort of the buildings users; 3) hydraulic (plumbing), which must be maintained to ensure hygiene and sufficient supply of water; and 4) energy (electrical and/or communication networking), required to provide lighting, communications and life safety subsystems. The remaining two subsystems, structure and vertical closure, are rarely, if ever, considered as requiring "normal maintenance."

Public structures including government facilities, primary and secondary schools, institutions of higher learning (colleges and universities) and religious facilities all share common problems associated with ever tightening budgets and failure of those entrusted with their care to understand the need for an all-inclusive maintenance program. For example, government facilities can always be patched up to provide for that quintessential "no raise in taxes" promised by politicians, but public school boards faced with upward spiraling needs, coupled with declining tax bases can and do neglect their buildings' facades. Colleges and universities focus on generating revenue for expanded programs and new facilities while growing a deferred maintenance budget on existing facilities and religious structures, often relying on divine intervention to protect their aging architectural inventory.

Private structures also suffer from insufficient maintenance planning and expenditures. Common to both public and private structures, building facade maintenance repair and inspection should be required on a national level to ensure public safety. By the implementation of a national facade inspection standard, specific benefits could be realized: 1) Those responsible for the maintenance and repair of buildings would be required to address a structured facade maintenance, repair and inspection protocol that would motivate owners to be proactive with respect to preventative maintenance; 2) public safety ensured; and 3) uniform standards set enabling qualified professionals to generate universally understandable documentation.

Public School Facilities

Over the past 20 years, we have reviewed numerous public school buildings with regard to their masonry envelopes. At the time these structures were reviewed, they were in use and in our opinion, presented life safety concerns for students and pedestrians.

Northeastern Pennsylvania

This brick, stone and terra cotta structure was reviewed for the school superintendent, who was alerted by a roofing consultant (contractor) that problems with the structure's masonry parapets might cause problems (Figure 1). After a cursory review by our firm, we recommended that the school district engage a licensed professional architect and civil engineer and immediately construct overhead protection for pedestrians at building entrances and cordon off the remainder of the facility. Within one month of the submission of our report, the structure was abandoned and later condemned.

Figure1 – *Northeastern Pennsylvania Middle School*

Southeastern Pennsylvania

We were requested by the school district's attorney to inspect cracks in this structure's stone parapets. Our cursory review identified incipient terra cotta spalls which posed an immediate life safety concern (Figure 2). A licensed civil engineering firm was quickly engaged to inspect the façade and protective netting was installed prior to the next school year. Currently, the facility is being refurbished, including an extensive repair/preservation program to address masonry envelope deficiencies.

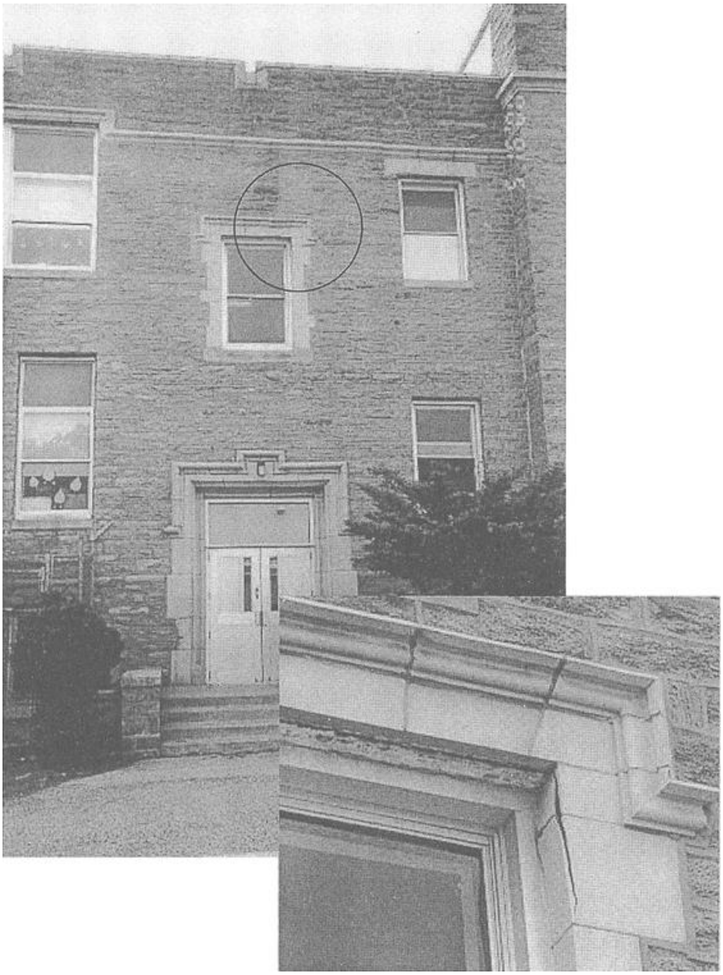

Figure 2 – *Southeastern Pennsylvania Grade School*

East Central Pennsylvania

Two school structures were informally reviewed for the district's architect. The brick and limestone facility (Figure 3) exhibited severe wall displacement caused by oxide jacking and a brick and terra cotta structure (Figure 4) exhibited widespread incipient terra cotta spalling. After expressing our concerns regarding these structures, the architect was hesitant to forcefully react to the identified deficiencies. To fulfill our ethical obligation, the State Department of Education was independently notified with regard to the structure's condition.

All of the projects listed above were mass masonry structures constructed during the early twentieth century. The primary cause of deterioration was the corrosion of embedded steel anchors and supports caused by a general lack of good preventative maintenance. These facades were allowed to deteriorate to a point where their occupants, school children and the public, were needlessly exposed to unsafe conditions.

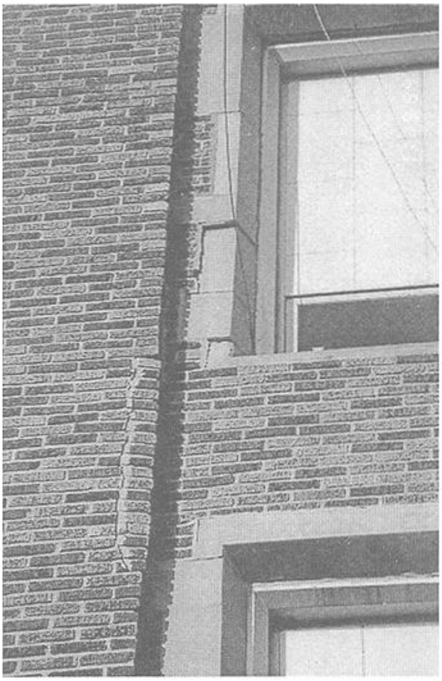

Figure 3 – *East Central Pennsylvania*

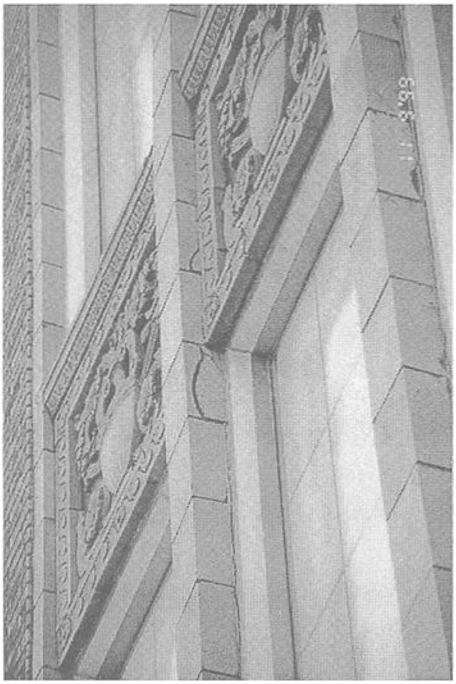

Figure 4 – *East Central Pennsylvania*

Discussions and Conclusion

While this paper primarily deals with public school buildings, there are many buildings, including churches, corporate owned high and low rise structures, condominiums, and mixed use retail/apartment buildings (Figure 5) that pose serious life safety threats to pedestrians every day.

While the examples listed above deal with masonry facades constructed during the early twentieth century, all building envelopes do require repair and intervention to maximize their useful service life and ensure the safety of pedestrians and those who use the facilities. The general consensus among professionals who specialize in building façade repair and remediation is that more current façade assemblies, including EIFS, single wythe veneers, etc., due to their construction and lack of redundancy will require more aggressive intervention at an earlier point in their expected service life.

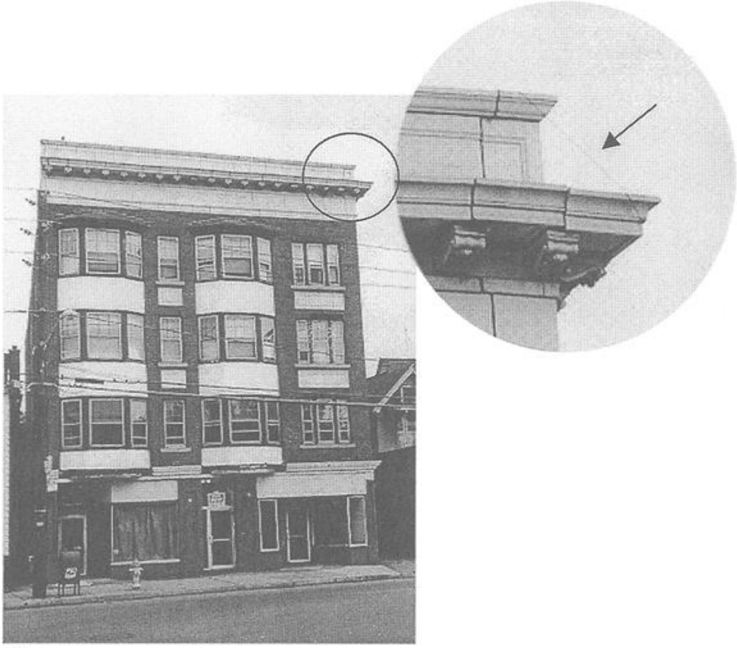

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Figure 5 – *Four-story mixed-use (retail/apartment)*

We, as a society, must provide sufficient resources to maintain our public buildings and ensure that those entrusted with the responsibility of their maintenance have the tools to accomplish this task. For privately held structures, it remains the owner's responsibility to provide the resources to maintain their buildings. As responsible professionals, the preparation of conscientious minimum required standards like the "*Standard Practice for Periodic Inspection of Building Facades for Unsafe Conditions*" will provide a catalyst to start and address this important task on a national level.

Ian R. Chin[1] *and Holly Gerberding*[2]

Evolution of the Development of the Chicago Facade Inspection Ordinance

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Reference: Chin, I. R., and Gerberding, H. "***Evolution of the Development of the Chicago Facade Inspection Ordinance,***" *Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444,* Erdly J. and Schwartz T., Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: In Chicago, IL, there are hundreds of high-rise buildings that were constructed starting in the 1890s. The exterior facade on these buildings includes terra cotta panels, thick stone panels, thin stone panels, brick veneer, precast concrete, poured-in-place concrete, steel panels, aluminum panels, and glass/aluminum curtain wall. After pieces of a terra cotta facade fell from a building in 1974 and killed a pedestrian, Chicago prepared its 1978 facade inspection. This ordinance was the first facade inspection ordinance in the United States. This ordinance was subsequently repealed. Due to subsequent facade failures, Chicago prepared its 1996 facade ordinance and amended this ordinance in 2000, 2001, and 2002. The amended ordinance is currently the most comprehensive facade ordinance in the United States. Approximately 70% of eligible buildings in Chicago have complied with the ordinance. This paper presents information on the evolution and development of the ordinance.

Keywords: Facade, failures, inspection, ordinance

Introduction

Chicago, IL, was incorporated as a city on 4 March 1837. At that time, the population of Chicago was about 4 000 people. By the fall of 1871, Chicago was a "boom town" with a population of about 334 000 people and about 59 500 structures [1].

On 8 October 1871 at approximately 9 p.m., the Great Chicago Fire began in Patrick O'Leary's barn, located southwest of the central business district. Fanned by a strong, steady, dry southwest wind, the fire was driven towards and through the center of the city, and across the Chicago River. The fire was finally put out by a steady rain on the morning of 10 October 1871, approximately 35 hours after it began. During the fire, at

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times, 20 blocks and 500 buildings were on fire at the same time. The fire destroyed all of the buildings in the central building district. A total of about 1 800 buildings in the city were destroyed by the fire [2].

Rebuilding of the city began within days after the fire, and by 1875, approximately four years after the fire, Chicago was once again a dominant city in the United States. The post-fire rebuilding of Chicago provided the world with innovations in architecture and structural engineering that included skeleton frame construction, caisson foundations, and the use of terra cotta to fireproof steel or cast iron columns and beams [3]. The advent of skeleton frame construction and the development of the elevator resulted in the construction of taller buildings in Chicago and in the use of thin claddings of terra cotta, brick, and stone veneers, and glass and aluminum curtain walls on the facades of buildings.

The Department of Buildings of the City of Chicago (City) reported in 2000 that there are approximately 2 000 buildings in Chicago that are more than six stories or more than 80 feet tall. The exterior facade on these buildings generally includes: terra cotta, stone (limestone, granite, marble), brick, and concrete veneers; aluminum and steel panels; glass in aluminum and steel framed curtain walls; glass in aluminum and steel framed windows; and exposed cast-in-place reinforced concrete members. The age of these buildings varies from approximately one year to approximately 110 years.

Genesis

The genesis of the Chicago Facade Inspection Ordinance (ordinance) occurred on 22 October 1974. On this day, "two pieces of glazed tile measuring 12 by 20 by 1-1/4 inches" fell from the cornice of a 17-story building on West Madison Street in Chicago, IL, killing a pedestrian walking on the sidewalk in front of the building [4].

Prompted by this accident, personnel from the City in 1975 made a cursory inspection of the exterior facade on 2 458 buildings with binoculars from grade level. This inspection "detected loose and potentially unsafe building materials" in the exterior walls of approximately 1 105 or approximately 45% of the buildings inspected. This inspection revealed that not only were terra cotta buildings experiencing problems, but also that virtually all types of buildings were involved in failures. This inspection also revealed that "visual inspections from ground level or from adjacent buildings were inadequate to spot all loose facing materials," because "subsequent checks from scaffolding were revealing many loose sections of various materials that were not visually detectable" [5].

The 1978 Ordinance

As a result of the above findings from the façade inspections performed by the City in 1975, the City turned to the Structural Subcommittee of the Mayor's Advisory Commission (Commission) to assist the City in developing a solution to the problem. Subsequently, the commission formed a committee consisting of architects, attorneys, city inspectors, and structural engineers (committee) to evaluate the problems. The Committee's evaluation included the following:

1. The results of the city's visual inspection of the facades on 2 458 buildings from grade level and their follow-up close-up inspection as described above.
2. Studies performed by the General Services Administration (GSA) in evaluating the history and performance of their building facades.
3. Studies of the design of various types of building facades.
4. Studies of non-destructive methods (sonic, infrared, metal detectors, remote detection) to detect problems in building facades.
5. The findings of investigations performed by architects and structural engineers to determine the cause(s) of failures in building facades.

Regarding the potential for a piece of a facade to fall from a building, the committee stated that "as buildings age, experience shows methods of fastening facades to the back-up walls or structures weaken and deteriorate. In time the bond or anchorage fails and parts of the facade fall, endangering people and property below". Consequently, "to warn a building owner of potential danger of life and property so that necessary repairs can be made before any accidents occur," the Committee in their recommendations emphasized a complete systematic examination with repeat periodic surveys of building facades.

The committee prepared a facade inspection ordinance, which was presented and discussed at a public hearing in the City Council chambers on 10 May 1978. No person appeared in opposition. The ordinance was adopted by the City Council on 13 September 1978 [6]. This ordinance was reported to be the first facade inspection ordinance in the United States. This 1978 ordinance applied to all buildings that were five (5) stories or more in height because "the building department has the staff, equipment and expertise to inspect exterior building walls only up to five (5) stories in height" [7].

The 1978 ordinance required that the entire facade of buildings five (5) stories or more in height be critically inspected hands-on and close-up from a suspended scaffold by, or under the supervision of, a registered architect or structural engineer. A detailed, comprehensive report on the inspection was to be submitted, describing in detail all the conditions observed and any repair work recommended.

Building owners were required to comply with the ordinance within two years of its adoption by the City, and to perform similar critical examinations every ten (10) years thereafter. Buildings that were more than 35 years old were required to be critically examined every five (5) years.

Repeal of the 1978 Ordinance

Subsequent to the adoption of the 1978 ordinance, a few building owners had the exterior facade on their building critically examined and filed the reports with the City.

However, in 1979, the recently-enacted requirements for inspection of all buildings over five stories was rescinded by action of the City Council, with little publicity. It was replaced with a requirement that "If there is any doubt as to the structural stability of any building or structure or parts thereof, the Commissioner of Buildings may request such building or structure, or parts thereof, to be critically examined by a licensed architect or registered structural engineer employed by such owner, agent or person in charge, possession or control of any such building, structure or parts thereof" [8].

The Proposed 1981 Ordinance

As a result of the repeal of the 1978 ordinance, the City formed another committee to prepare another facade inspection ordinance. This committee consisted of architects, building owners, City inspectors, and structural engineers.

In 1981, this committee prepared an ordinance which was similar to the former 1978 ordinance, except that the need to inspect the facade hands-on and close-up from a suspended scaffold and the extent of such inspection were left up to the discretion of the architect or structural engineer supervising the inspection. The proposed 1981 ordinance was submitted by the committee to City Committee on Buildings and City departments for review, where it remained tabled and was never submitted to the City Council for consideration for adoption.

The 1996 Ordinance

In August 1994, on separate occasions, a "7-foot chunk" of terra cotta fell from a 16-story building onto the sidewalk adjacent to West Van Buren Street; "a small section of brickwork fell from the 23rd floor of a building onto the sidewalk adjacent to North Wells Street; and pieces of glass fell from the 33rd floor of a building onto the sidewalk adjacent to West Wacker Drive [9]. Fortunately, no one was injured by these pieces of falling building facade.

However, due to these three incidents, the City contacted the Chicago Committee on High Rise Buildings (CCHRB) and requested that CCHRB form a committee to evaluate the situation and to prepare recommendations for a facade inspection ordinance that could minimize the potential of pieces of building facades falling to the ground.

CCHRB is an interdisciplinary group of architects, builders, developers, engineers, and owners experienced in all aspects of high-rise building design, construction, and maintenance. CCHRB was formed in the fall of 1969 to initiate, support research, and disseminate information on design, construction, and maintenance of high-rise structures.

In response to the request from the City, the Exterior Wall Task Force of CCHRB formed a committee that consisted of representatives of Apartment Building Owners and Managers of America (ABOMA), Building Owners and Managers of America (BOMA), Chicago Chapter of the American Institute of Architects (CAIA), the Structural Engineers Association of Illinois (SEAOI), and CCHRB members with experience in design, construction, maintenance, investigation, and repair of exterior facades on buildings.

During the period of September 1994 through June 1995, the committee met several times to discuss the intent and requirements of the proposed ordinance. The content of the existing New York City Facade Inspection ordinance as well as the content of Chicago's original 1978 ordinance and Chicago's proposed 1981 ordinance were reviewed and discussed by the committee in these meetings. Discussions focused on a range of issues.

1. The high cost of performing the hands-on, close-up inspection of the facade from swing stages.
2. Effective alternatives to the hands-on, close-up inspection. Infrared thermograph and close-up photography were discussed and found not to be acceptable alternatives.
3. The qualifications required for the inspecting architect or structural engineer. The City stated that from their point of view, all architects and structural engineers that are licensed in

Illinois are eligible to perform the inspections, recognizing that licensing laws restrict these professionals from performing services outside the area of their own competence.

4. The extent and frequency of the close-up, hands-on inspections of older buildings and building facades of certain materials. BOMA wanted the inspections to be the same for all buildings regardless of age and material so that one building ~~will~~ would not have an economic advantage over another building.

Based upon the discussions that were held during the meetings, the committee prepared recommendations for a facade inspection ordinance, which was submitted to the City for consideration on 13 June 1995.

Subsequently, the City prepared its "Maintenance of Exterior Walls and Enclosures" ordinance based upon their knowledge and upon the CCHRB's recommendations. On 4 January 1996, the Building Committee of the City Council held a public hearing to discuss the City's proposed ordinance and to listen to any party wishing to voice an opinion on the ordinance. At the conclusion of this meeting, the Building Committee recommended the ordinance for adoption by the City Council.

The City Council adopted the ordinance on 10 January 1996. The requirements of the 1996 ordinance include the following~~:~~.

1. The ordinance applied to all buildings in Chicago, Illinois that are six (6) stories or more, or 80 feet or more in height above grade. There are approximately 2 000 buildings within the city limits of Chicago that meet that criteria and are required to comply with the ordinance. All exterior walls, and parts thereof (including balconies, cornices, etc.), regardless of height on these buildings are subject to examination under the ordinance.
2. The ordinance required building owners to maintain the exterior facade on their buildings "in a safe condition," and either to establish and report on a yearly "Ongoing Inspection and Repair program" <u>or</u> perform and report on "Periodic Critical Examinations" of the facade every four years.
3. The first "Ongoing Inspection and Repair Program" report was due within one year after the adoption of the ordinance.
4. The first "Critical Examination" was due within two years after the adoption of the ordinance.
5. The "Ongoing Inspection and Repair Program" does not require close-up inspection of the facade.
6. The "Critical Examinations" require close-up inspection of the facade. The location and extent of the required close-up inspection were left up to the discretion of the professional (licensed architect or structural engineer) performing the Critical Examination.
7. The ordinance requires that the Commissioner of Buildings be notified promptly by the professional upon determining that an exterior wall or enclosure or part thereof is in an "unsafe and imminently hazardous" condition.
8. The ordinance requires building owners to remove, reinforce, and/or make permanent repairs to "unsafe and imminently hazardous conditions" found in the facade in a timely manner.

Limitations of the "Ongoing Inspection and Repair Program" and of the "Critical Examination Program"

These programs are intended to give the owner the information he/she needs in order to keep his/her building from becoming a hazard to the public. They are a part of the owner's plan "to maintain the building exterior walls and enclosures in a safe condition." The "Ongoing Inspection and Repair Program" and the "Critical Examination Program" are intended to identify visible "unsafe and imminently hazardous" conditions that need to be immediately addressed, and to determine the general visible condition of the facade. These programs are not intended to be investigations that fully diagnose the cause of and extent of the "unsafe and imminently hazardous" conditions and of other observed external distress conditions in the facade; to identify concealed distress conditions; or to gather information to prepare repair designs of distress conditions observed. Special investigations beyond the scope of these programs are necessary to obtain this information.

Amendment to the 1996 Ordinance in 1998 and 1999

Minor amendments were initiated by the Commissioner of Buildings and passed by the City Council in November 1998, and February 1999, to strengthen the compliance requirements of the ordinance.

Amendment to the Ordinance in 2000

On 30 August 2000 the City amended the ordinance to address significant facade failures that had occurred subsequent to the adoption of the 1996 ordinance. These conditions included: the following.

Collapse of Facades

On Friday 8 October 1999, a piece of glass fell from the west facade at the 29th floor of a 30 year old, 45 story building on South Wabash Street in Chicago, IL, and fatally injured a pedestrian walking on the sidewalk opposite the building [10]. The cause of the glass failure was determined to be high temperature differentials that led to high tensile stresses in the glass edges, which exceeded the ultimate edge strength of the glass and caused the glass to crack. On Sunday evening, 2 July 2000, a 2 foot by 10 foot by 4-inch thick limestone panel fell from the west facade at the 36th floor of a 70 year old, 45 story office building on South LaSalle Street in Chicago, IL. This panel fell onto the adjacent street and damaged several parked cars [11]. The cause of the limestone panel collapse was determined to be corrosion of the mild steel shelf angles and of the ties that support the panel from the structure of the building.

Non-Compliance with 1996 Ordinance

According to the City, by the end of 1999, approximately 70 percent (1400) of the approximately 2 000 eligible buildings in Chicago, IL, had not complied with the 1996 ordinance. Approximately 80 percent (480) of the 600 buildings that had complied had filed

"Ongoing Inspection and Repair Program" reports, and the other 20% (120) of the buildings had filed "Critical Examination Reports."

These significant facade collapses and the large number of eligible buildings that had not yet complied with the ordinance initiated an amendment to the 1996 ordinance by the City on 26 September 2000. The amended ordinance (changes are underlined) required the following:

1. Building owners shall arrange for periodic "Critical Examinations" at four year intervals ~~or~~ and establish an "ongoing inspection and repair program" for each of the intervening years.
2. The initial critical examination shall be performed on buildings constructed prior to 1 January 1950 within two years of the amendment, and on all buildings constructed on or after 1 January 1950 within four years of the amendment. (The date of 1 January 1950 is a date, which the City believed marks the approximate time when the use of non-corrodible or corrosion resistant metals to support building facades became prevalent).
3. The periodic "Critical Examinations" require close-up inspection of "the entire area of all elevations of the exterior walls".
4. The building owner and the professional shall promptly notify the City upon determining that exterior wall or parts thereof is in an unsafe or imminently hazardous condition or if any failure of the exterior wall has occurred.
5. The Owner of any building which constitutes an imminent danger and hazardous to the public shall take immediate action to have a critical examination performed and provide a report to the City.
6. Any costs incurred by the City in taking emergency actions due to unsafe and imminently hazardous conditions in an exterior wall shall be recoverable from the Owner.
7. The City may issue Rules and Regulations for administration and enforcement of the ordinance. Rules and Regulations for the amended ordinance prepared by the City became effective on 10 October 2000.
8. The Critical Examination shall determine whether a wall is "unsafe and imminently hazardous," "safe with a repair and maintenance program," or "safe".
9. Critical examinations shall begin with the elevation(s) parallel to any public way.
10. Examination of the substrate in walls with no externally visible distress shall be performed at no less than three inspection openings per elevation in buildings which meet all of the following conditions:
    a. Building was constructed prior to 1 January 1950;
    b. Components and cladding of the building is comprised of masonry, concrete, stone, or terra cotta;
    c. Material is attached to building with concealed metal fasteners
11. Limitations: Rules and Regulations recognized that, due to the limitations of detecting concealed internal wall distress, submittal of the critical examination report is not a representation that all "unsafe and imminently hazardous conditions in a wall have been identified".

Amendment to the Ordinance in 2001

On 1 November 2001, the Rules and Regulations for the 1996 ordinance were amended to include the following requirements.

1. The owner/agent of the building shall promptly begin repairs or stabilization of an "unsafe and imminently hazardous condition," and shall submit a schedule for the repair or stabilization work within 72 hours of notification to the Owner by the professional.
2. Light courts enclosed by walls on all sides need not be included in the Critical Examination unless there are openings or skylights at the bottom of the light court.
3. After the initial Critical Examination, subsequent Critical Examinations may be conducted over a period of up to four years (one elevation per year).

Compliance with Ordinance

In the fall of 2001, the City reported that approximately 70% (1 400) of the approximately 2 000 eligible buildings in Chicago have complied with the ordinance.

Amendment to the Ordinance in 2002

On 4 September 2002 and 21 November 2002, the Facade Ordinance and the Rules and Regulations for the Ordinance, respectively were amended to address concerns of building owners on the cost of the facade inspection program. This amendment included the following:

1. Buildings which are six stories or more but less than 80 feet in height are no longer required to comply with the ordinance. This amendment eliminated the requirement for hundreds of small, six story residential buildings to comply with the ordinance.
2. "Critical Examination" for non-terra cotta facades is redefined to be a close-up inspection of the facade at alternate scaffold drops on each elevation and including corners. This amendment reduced the cost of Critical Examinations of non-terra cotta facades by approximately 40 percent.
3. The interval between Critical Examinations was changed from every four (4) years to vary from four (4) years to twelve (12) years based upon the corrosion potential of metal elements that are in direct contact with the facade as designated in the Rules and Regulations as follows:

    - Category I Buildings: Twelve (12) years
      Category I buildings are buildings with facades that are primarily reinforced with or in direct contact with non-corrodible metals (stainless steel, aluminum)

    - Category II Buildings: Eight (8) years
      Category II buildings are buildings with facades that are primarily reinforced with or in direct contact with corrosion resistant metal (galvanized, epoxy coated, coated steel)

    - Category III Buildings: Four (4) years
      Category III buildings are buildings with facades that are primarily reinforced with or in direct contact with corrodible metal (carbon steel, uncoated reinforcing bars, shop-primed steel)
    - Category IV Buildings: Eight (8) years

Category IV buildings are buildings with facades that are primarily secured to the substrate by adhesive bond or with masonry headers

4. The interval between the ongoing inspection and repair program inspections was extended from one to two years.

Current Ordinance

The complete amended Chicago Facade Inspection Ordinance and its Rules and Regulations that are in effect as of 21 November 2002 are presented in the Appendix.

Conclusions

Over the past 24 years, between 1978 and 2002, the Chicago Facade Inspection Ordinance has evolved into the most comprehensive facade ordinance in the United States. Currently, approximately 70 percent of eligible buildings in Chicago have complied with the ordinance. Generally, the ordinance requires the following:

1. Buildings that are 80 feet or more in height are required to comply.
2. Complying buildings are required to have a Critical Examination performed and a report filed with the City every four (4) to twelve (12) years, depending on the construction type of the building which is based upon the corrodibility of the metal that reinforces or is in direct contact with the facade on the building.
3. Complying buildings are required to have an Ongoing Inspection and Repair Program every other year between Critical Examinations.
4. The Critical Examination requires a close-up, hands on inspection of 100 percent of terra cotta facades; and of alternate scaffold drops, including corners of non-terra cotta facades.
5. The Ongoing Inspection and Repair Program does not require a close-up, hands-on inspection of the facade.
6. Unsafe and imminently hazardous conditions found in the facade are required to be promptly reported to the City and be promptly repaired or stabilized.
7. A repair program for other types of distress conditions found in the facade is required.

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[3] Randal, Frank A., "History of the Development of Building Construction in Chicago," Second Edition, University of Illinois Press, 1999, p. 12.

[4] *"Loop Building Tile Falls, Kills Woman,"* Chicago Tribune, October 23, 1974.

[5] "Guidelines for the Critical Examination of Building Exterior Walls and Enclosures as Required in Subsections 78-3(e), 78-3(f), and 78-3(g), Building Exterior Walls and Enclosures," published in *Journal - City Council - Chicago, Illinois*, September 13, 1978, pages 8357 and 8358, G-1.

[6] *Journal - City Council - Chicago, Illinois*, September 13, 1978, p. 8357.
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Appendix

1. Chicago Facade Ordinance, Amended, effective 1 October 2002

2. Rules and Regulations for the Chicago Facade Ordinance, Amended on 21 November 2002

MAINTENANCE OF EXTERIOR WALLS

13-196-031 Maintenance of exterior walls and enclosures—Definitions. These terms shall have the following meanings when used in Sections 13-196-031 through 13-196-037, which sections shall be known as the minimum requirements for maintenance of exterior walls and enclosures, as further clarified by such rules and regulations promulgated by the commissioner of buildings pursuant to Section 13-196-038:

"Critical examination" shall mean a close-up visual examination of the condition of all elevations of the exterior walls and enclosure. For buildings constructed of material other than terra cotta, the examination may be satisfied by scaffolding utilizing alternate drops to cover at least 50% of the area of each elevation, and including all corners of the building. For buildings constructed of terra cotta material, the examination shall cover the entire area of all elevations. All examinations shall be performed by or under the direct supervision of a professional employed by the owner/agent for the purpose of determining if remedial work is required.

"Ongoing inspection and repair" shall mean a program of bi-annual inspections by a professional retained by the owner/agent, with accompanying report by the professional to the commissioner of buildings and repair work by the owner/agent as necessary.

"Owner/agent" shall mean the owner, agent or person in charge, possession or control of the building.

"Professional" shall mean an Illinois licensed architect or Illinois licensed structural engineer.

13-196-032 Maintenance of exterior walls and enclosures—Application. Exterior walls and enclosures and parts thereof of buildings that are 80 feet or more in height above grade, shall comply with Sections 13-196-033 to 13-196-037.

13-196-033 Maintenance and reporting required.

It shall be the owner/agent's duty to maintain the building's exterior walls and enclosures in a safe condition and to provide periodic reports to the commissioner of buildings. In furtherance of that requirement, the owner/agent shall: 1) arrange for periodic critical examinations at intervals designated in rules and regulations promulgated by the commissioner of buildings pursuant to 13-196-038 and 2) establish an ongoing inspection and repair program at two-year intervals for each of the intervening years. When the report indicates that repair or remedial work is necessary, the report shall include a proposed schedule for completion of such work.

13-196-034 Maintenance of exterior walls and enclosures—Critical examination program.

(a) The initial critical examination shall be submitted for all buildings constructed prior to 1/1/50, by December 1, 2003, and on all buildings constructed on or after 1/1/50, by December 1, 2004. The initial critical examination for newly constructed buildings shall be submitted no later than December 1 of the fourth year following completion of the construction.

(b) Following the initial critical examination, the exterior walls and enclosures and parts thereof on all buildings shall be subsequently critically examined and a report submitted at intervals designated in rules and regulations promulgated by the commissioner of buildings pursuant to Section 13-196-038 of this code. Any building which cannot be categorized according to the information contained in a previously submitted critical examination report shall be required to supplement such report with a certification by a professional as to which category the building belongs.

(c) The critical examination shall include a review of all previous reports.

(d) The professional shall prepare a report in writing on the critical examination, describing the condition of the exterior walls and enclosures on the building and including a record of the components and cladding including, without limitation, any broken glass and loose or missing glazing components; loose masonry, concrete, EIFS, metal, stone or terra cotta; and all significant deterioration and displacement observed. Additionally, the report shall indicate any imminently dangerous conditions. If any remedial work is recommended the report shall indicate the nature and urgency of such work.

13-196-035 Maintenance of exterior walls and enclosures—Ongoing inspection and repair program.

(a) The ongoing inspection and repair program shall provide inspection, reporting and preventive maintenance of the exterior walls and enclosures and parts thereof.

(b) No later than November 1 of every second calendar year, the professional employed by the owner/agent shall prepare a report in writing on the ongoing inspection and repair program, describing the condition of the exterior walls and enclosures and parts thereof on the building and on any inspections, surveys or repair work performed or to be performed on the exterior walls and enclosures.

(c) Failure to submit an ongoing maintenance and repair report that is acceptable to the commissioner of buildings shall trigger the requirement of a critical examination report in accordance with Section 13-196-035. Such report shall become due within six months after the due date of the missing report.

13-196-036 Maintenance of exterior walls and enclosures—Reports to the commissioner. The owner/agent shall submit to the commissioner of buildings two copies of the report required under Section 13-196-034 or 13-196-035. The report shall bear the professional's seal and signature. If acceptable, one copy of the report shall be returned to the owner/agent, bearing a stamp indicating acceptance by the commissioner of buildings. The owner/agent shall maintain such reports in a permanent building file for future reference. The fee for examination of reports shall be $10.00 per report. However, reports requiring extensive review of technical information by licensed professionals within the department of buildings shall be examined for a fee of $200.00. All reports shall identify any persons or entities involved in the preparation or completion of the examination and report under both the critical examination program and the ongoing inspection and repair program. All reports shall also include as exhibits or attachments any and all documents, notes, summaries, memoranda, letters or ancillary reports submitted by the professional to the owner of buildings subject to these requirements.

13-196-037 Unsafe exterior walls and enclosures.

(a) Every exterior wall and enclosure and parts thereof found to be in an unsafe condition shall be subject to notice by the commissioner of buildings to the owner/agent to take appropriate precautionary measures and effect such repairs or reinforcements in a timely manner as will bring the building exterior walls and enclosures and parts thereof into a safe condition. The owner/agent of any building which constitutes an imminent danger and hazard to the public shall take immediate action to have a critical examination performed upon such building and provide the ensuing report to the Department of Buildings. Additionally, the owner/agent shall promptly begin and complete the removal, reinforcement and/or permanent repairs necessary to make the premises conform to the building provisions of this code, and provide structurally safe conditions. Any costs incurred by any department of the city in taking emergency actions due to the dangerous and hazardous condition of an unsafe exterior wall, including, but not limited to: closure of vehicular traffic in a public street; rerouting of pedestrian traffic on a public sidewalk; erection or installation of partitions, canopies, sidewalk sheds, barricades, scaffolding or netting, shall be a debt due and owing to the City and recoverable from the owner/agent of such building.

(b) It shall be the joint and several duties of the owner/agent and the professional to notify the commissioner of buildings promptly by phone and later in writing upon determining that an exterior wall or enclosure or part thereof is in an unsafe and imminently hazardous condition or if any failure of the exterior enclosure is noted.

13-196-038 Rules and regulations. The commissioner of buildings may issue rules and regulations for the administration and enforcement of the minimum requirements for maintenance of exterior walls and enclosures. Any person violating such rules and regulations shall be subject to the fines prescribed in Section 13-196-039.

13-196-039 Fines and penalties. Any violation of, or interference with the enforcement of, any of the provisions of Section 13-196-031 through and including Section 13-196-038 shall be punishable by a fine of not less than $500.00 and not more than $1,000.00. Each day that such violation shall continue shall constitute a separate and distinct offense for which a fine as herein provided shall be imposed.

RULES AND REGULATIONS FOR EXTERIOR WALL MAINTENANCE

SECTION I. DEFINITIONS AND REQUIREMENTS

Rule 1 "Category I buildings" shall mean those buildings constructed with exterior walls and parts thereof that are primarily reinforced with or are in direct contact with non-corrodible metal.

Rule 2 "Category II buildings" shall mean those buildings constructed with exterior walls and parts thereof that are primarily reinforced with or are in direct contact with corrosion resistant metal.

Rule 3 "Category III buildings" shall mean those buildings constructed with exterior walls and parts thereof that are primarily reinforced with or are in direct contact with corrodible metal.

Rule 4 "Category IV buildings" shall mean those buildings constructed with exterior walls and parts thereof that are primarily secured to the substrate by adhesive bond or with masonry headers.

Rule 5 "Corrodible metal" shall mean unprotected carbon steel, shop-primed steel, uncoated reinforcing bars and other metals that can corrode.

Rule 6 "Corrosion-resistant metal" shall mean corrodible metal that is galvanized, epoxy coated, or painted specifically to resist corrosion with that finish intact.

Rule 7 "Non-corrodible metal" shall mean stainless steel, aluminum and other metals that do not corrode under atmospheric conditions.

Rule 8 For exterior walls constructed of material other than terra cotta, a "close-up visual examination" means that: a) the professional; or b) the architect-in-training, engineer-in-training, technician, contractor or skilled trades people, under the professional's direct supervision, must actually touch those portions of the exterior wall reachable by hand or tool while utilizing scaffolding of alternate drops spanning at least 50% of the area of each elevation, and including all corners of the building.

For exterior walls or parts thereof constructed of terra cotta material, a “close-up visual examination” means that: a) the professional; or b) the architect-in-training, engineer-in-training, technician, contractor or skilled trades people, under the professional's direct supervision, must actually touch 100 % of the terra cotta material by hand or tool.

Other methods, such as, but not limited to, photographic magnification techniques, remote observation equipment, or infra-red or thermography cameras, which can demonstrate reasonable reliability may be approved to supplement the close-up visual examination by the Commissioner of Buildings on a case-by-case basis. Such approval must be granted prior to the examination.

Rule 9 An "unsafe and imminently hazardous condition" in an exterior wall and enclosure shall mean a condition that has no reliable means of structural support, and that is dangerous to people or property.

a) The owner/agent and professional shall promptly notify the Building Department upon determining that a wall is in an "unsafe and imminently hazardous condition", by phoning (312) 746-8501 during business hours or, if no answer, by phoning (312) 744-6460. It shall also be the responsibility of the professional to personally examine the condition and determine the appropriate repair and/or stabilization procedures. The owner/agent of the building shall promptly begin repairs or stabilization of an "unsafe and imminently hazardous condition."
b) A schedule of the repair or stabilization work shall be submitted to the Department of Buildings within 72 hours of notification to the owner by the professional.
c) An application for a building permit for the repair work shall be submitted no later than 30 days after the professional notifies the owner/agent and Building Commissioner of such conditions.

Provided, however, that if the severity of conditions warrant more immediate action, the Department of Buildings may prescribe an earlier date by which an application must be submitted.

Rule 10 A "safe with a repair and maintenance program condition" in an exterior wall and enclosure shall mean a condition that is considered by the professional not to be in an "unsafe and imminently hazardous condition" at the time the critical examination is performed, but requires repair and maintenance within a time period designated by the professional in order to prevent its deterioration into an "unsafe and imminently hazardous condition".

Rule 11 A "safe condition" in an exterior wall and enclosure shall mean a condition observed in a wall that exhibits neither an "unsafe and imminently hazardous condition" nor a "safe with a repair and maintenance condition" at the time of the critical examination.

Rule 12 "Failure of the exterior enclosure" shall mean that any portion of the cladding or component of the facade has broken away from the exterior wall and is dangerous to people or property.

Rule 13 "Repair" or "Repair work" shall mean such work performed on a building which is permanent in nature and intended to bring any condition into a state of reliability.

Rule 14 "Stabilization" shall mean such work performed on a building which is temporary in nature and intended to contain an unsafe and imminently hazardous condition until permanent repairs can be effected.

The location and description of any stabilization work shall be reported to the Department of Buildings. The report shall include a description of a recommended repair program and schedule and a discussion of any temporary 'make-safe' work performed or required.

SECTION II. REPORTS

A) CRITICAL

Rule 15 Based upon any previous critical examinations, the professional shall categorize the building according to the categories as defined by these rules and include such information in subsequent reports. Buildings which are primarily category I, II or IV, but which have some terra cotta elements, shall have the terra cotta inspected on a 4 year cycle, and the remaining facade shall be inspected as required for the primary category.

Rule 16 Category I buildings shall be required to submit their critical examinations by December 1 of the 12th year following the last submitted critical report.

Rule 17 Category II and IV buildings shall be required to submit their critical examinations by December 1 of the 8th year following the last submitted critical report.

Rule 18 Category III buildings shall be required to submit their critical examinations by December 1 of the 4th year following the last submitted critical report.

Rule 19 All critical examination reports shall include the following documents or information:

a) name and address of building;

b) site plan of building showing adjacent streets and/or alleys and relationship of building to property lines and to adjacent buildings;
c) principal building occupancy and type of mixed use, if any;
d) complete name, mailing address and phone number for the Owner/Agent, including primary contact person on site and at the management company, if applicable;
e) name, business address and phone number of Professional preparing the Critical Examination Report;
f) description of building, including: number of stories; height; plan dimensions; age and type of exterior wall construction, describing (as applicable) cornices, soffits or similar overhangs or features;
g) overall photographs or drawings of the four elevations of the building;
h) detailed description of the Critical Examination in narrative form, that must include characterization of the building as: "unsafe and imminently hazardous"; "safe with a repair and maintenance program"; or "safe"; and start and completion dates of the exam;
i) drawings and/or photographs to describe the locations and extent of all significant distress or deteriorated conditions observed in the exterior walls;
j) description and location of observed unsafe and imminently hazardous conditions in the exterior wall; and description of recommended repair program and schedule to address these conditions; and a discussion of any temporary 'make-safe' work performed or required;
k) description of recommended repair work, if any, and the urgency of such repairs;
l) where appropriate, a comparison of conditions of exterior walls on building with conditions observed during previous examinations;
m) recommendation for future examination, if earlier than otherwise required by Code;
n) signature and seal of Professional who performed or supervised the Critical Examination;
o) date of the report;
p) other documents, notes, summaries, memoranda, letters or ancillary reports pertinent to the critical examination report prepared by the professional and submitted to the owner.
q) categorization of the building as determined by the professional pursuant to the definitions contained in Rules 1, 2, 3 and 4.

B) ONGOING MAINTENANCE

Rule 20 All ongoing maintenance reports shall include such information as requested on a form approved by the Commissioner of Buildings. It shall also include other documents, notes, summaries, memoranda, letters or ancillary reports pertinent to the ongoing maintenance report prepared by the professional and submitted to the owner.

Rule 21 Any such form which indicates the need for ongoing maintenance must include a thorough description of recommended repairs, maintenance or corrective actions and a timetable for completion. The performance of the recommended repairs and maintenance work in accordance with the timetable shall be confirmed in subsequent reports.

Rule 22 Any pre-1950 building for which a critical report was due on November 1, 2002, shall be required to submit an Ongoing Maintenance and Repair Report in the year 2002.

Any post-1950 building for which a critical report was due on November 1, 2003, shall be required to submit an Ongoing Maintenance and Repair Report in the years 2002 and 2003.

SECTION III. CRITICAL EXAMINATION

Rule 23 The close-up visual examination shall determine whether a wall or portion thereof should be characterized as "unsafe and imminently hazardous"; "safe with a repair and maintenance program"; or "safe".

Rule 24 Critical examinations shall begin with the elevation(s) parallel to any public way.

Rule 25 Light courts enclosed by walls on all sides need not be included in the scope of a critical examination, unless there are openings or skylights at the bottom of the courtyard. In any case, such light court shall be included in any ongoing maintenance and repair report.

Rule 26 After the initial critical examination as required by section 13-196-034, subsequent critical examinations may be conducted over the periods designated in Rules 15, 16, 17 and 18. Provided, however, that, a) Any partial critical examination of single or multiple elevations must encompass the entire elevation within that year; and b) A report shall be submitted by the filing deadline of each year for any elevation(s) completed in that year. Buildings completing the Critical Examination beyond one calendar year shall provide an Ongoing Maintenance and Repair Report for those elevations not included in the Critical Examination for that year.

Rule 27 Examination of the substrate of typical wall areas with no externally visible distress at no less than two inspection openings per elevation shall be required in buildings which meet all of the following conditions:

r) Building was constructed prior to 1/1/50;

s) Component and cladding of the building is comprised of masonry, concrete, stone or terra cotta;
t) Material is affixed to the building with concealed metal fasteners.

Rule 28 Examination of the substrate shall require the physical removal of small portions of the components or cladding at the inspection openings as recommended by the professional.

Rule 29 The owner shall notify the Department of Buildings of any deviation from a schedule of repairs recommended by the professional either in the critical examination report pursuant to Rule 13 or in any emergency schedule recommended pursuant to Rule 9.

Rule 30 Repairs may be performed concurrent with the performance of a critical examination. Such repair work may be done under the authority of a "general repair permit" demonstrating the anticipated standard repair details to be utilized. The permit application must be signed and sealed by the professional.

Rule 31 Any repair work conducted under the auspices of a "general repair permit" which is structural in nature shall cause the owner to submit proper plans and documentation and obtain a revised building permit at the completion of the work. Structural repair work conducted under a general repair permit shall be allowed solely at the risk of the owner and professional. It shall be the burden of the owner or professional to remove any work found by the Department of Buildings not to be in compliance with the Chicago Building Code.

Rule 32 The repair or replacement-in-kind of any materials or types of structural support systems, such as, but not limited to: lintels or shelf angles, shall require a permit but no structural plans or structural review. Repair work which may be characterized as nominal or cosmetic, such as, but not limited to: sealing or patching, shall not require a permit.

Rule 33 Repair work shall not be performed from or upon any fixed scaffolding which does not meet a minimum live load of 30 lbs. per square foot.

SECTION IV. STANDARD OF CARE

Rule 34 Services rendered by professionals pursuant to the provisions of Sections 13-196-031 through 13-196-039, inclusive, and these Rules and Regulations shall be exercised with reasonable care and competence.

Rule 35 The standard of care of the professional performing the critical examination of an exterior wall shall include the following, with the understanding that, because of the physical properties of the many materials commonly used for constructing exterior walls, and the limitations on detecting concealed internal wall distress, the critical examination may not find "unsafe and imminently hazardous conditions" in the wall that are not visible from the exterior. Therefore, submittal of the critical examination report is not a representation that all "unsafe and imminently hazardous conditions" in a wall have been identified.

a) The professional need not be physically present on the platform or device when the close-up, visual inspection is made. Under the professional's direct supervision, architects, engineers, architects-in-training, engineers-in-training, technicians, and skilled trades people trained to perform this work may be delegated select tasks.

b) The professional responsible for the critical examination shall be qualified by education and experience in design, inspection, or repair design of the type of exterior wall system(s) on the building being examined and shall perform services only in the areas of his/her competence, as required by the Illinois Architecture Practice Act and/or the Illinois Structural Engineering Licensing Act.

c) The professional shall review and be familiar with the "Maintenance of Exterior Walls and Enclosures" ordinance in sections 13-196-031 through 13-196-039 of the Chicago Building Code.

d) The professional shall review pertinent available drawings and specifications of the building to determine the specified design of the exterior wall system(s) on the building.

e) The professional shall review available as-built drawings and specifications of the building and as-built conditions exposed by inspection openings cut in the wall under Rules 27 and 28, to determine the as-built construction of the exterior wall system on the building.

f) The professional shall review available drawings and specifications and maintenance reports on previous repair work performed on the exterior facade to obtain information on the maintenance history of the facade wall.

g) The critical examination of the exterior wall shall be performed from a platform or device, which allows for an arms-length inspection of the wall.

h) The professional shall document the condition of the exterior wall by photographs and drawings.

i) The professional shall notify the owner/agent of wall areas that are bowed, bulged, displaced, or leaning inward or outward, and that examination of the condition of a sufficient number of metal ties, anchors, and shelf angles that support the wall at these locations shall be performed.

j) The known history of the building, the nature of the materials used, and the observed condition of the wall will dictate the extent of the critical examination. The professional shall seek to detect "unsafe and imminently hazardous conditions" in building facades and shall ascertain the cause(s) of these conditions.

k) The professional shall notify the owner/agent of observed "safe with a repair and maintenance program" conditions and shall recommend to the owner/agent the inspections and/or tests that may be required to seek the cause(s) of these conditions and the time period within which these observed wall conditions should be repaired.

l) The professional shall notify the owner/agent of his/her obligation to maintain an ongoing permanent file on the condition of the building facade.

David May, RA[1]

New York City's Local Law 10 at Twenty: Critical Issues for Critical Examinations

Reference: May, D., **"Critical Issues for Critical Examinations: New York City's Local Law 10 at Twenty,"** *Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444*, J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: Facade inspection laws exist at the complex intersection of politics, economics and engineering practice. Nowhere can this be better demonstrated than in a critical examination of New York City's facade inspection laws: their development and how they are practiced. Our experience in New York City can inform the development of a Standard Practice for Periodic Inspection of Building Facades by ASTM International, as well as contribute to the development of facade inspection laws in other municipalities. In New York City, which has the oldest continually enforced facade inspection law, Local Law 10 of 1980, there are over 11,800 buildings subject to inspection. This paper will address issues related to the code mandated "critical examination" of building facades, based on 20 years of front-line experience with New York City's facade inspection laws. During the most recent inspection cycle, the author's firm performed 225 critical examinations of building facades in New York City.

Keywords: Local Law 10, Local Law 11, facade, inspections, critical examination, ordinances, New York City

Introduction

Facade inspection laws exist at the complex intersection of politics, economics and engineering practice. Nowhere can this be better demonstrated than in a critical examination of New York City's facade inspection laws: their development and how they are practiced.

Our experience in New York City can inform the development of a Standard Practice for Periodic Inspection of Building Facades by ASTM International, as well as contribute to the development of facade inspection laws in other municipalities. For example: In June of 2001, as reported in the *Philadelphia Inquirer*, a piece of a stone cornice crashed to the sidewalk in downtown Philadelphia. Fortunately, the incident

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occurred at 4:30 AM, in what during the day, would have been a crowded area. No one was injured [1]. However, the incident triggered questions that have been asked and answered in other cities with tall buildings: New York, Chicago, Boston, Detroit and Columbus Ohio.

New York City's Local Law 10 of 1980, as amended and amplified by Local Law 11 of 1998, is the oldest continually enforced facade inspection law in the nation [2]. In Chicago, there are approximately 2,500 buildings subject to inspection, in Boston 600 buildings, in Columbus, Ohio, 480 buildings, and in Detroit 450 buildings [3, 4]. These numbers pale in comparison with New York City, where there are over 11,800 buildings subject to inspection [5]. New York City has an environment conducive to the accelerated deterioration of building walls and structures: salt-air, freeze-thaw cycles, and air pollution, all create an inhospitable environment for building envelopes. Furthermore, almost all buildings suffer from neglect, charitably termed "deferred" maintenance.

During the most recent inspection cycle, the author's firm performed 225 critical examinations of building facades in New York City. This paper will address issues related to the code mandated "critical examination" of building facades based on 20 years of front-line experience with New York City's facade inspection laws.

Background

Local Law 10 of 1980

On the evening of May 16, 1979, an 18 year-old Barnard freshman, Grace Gold, was struck on the head and killed by a portion of an ornamental terra-cotta lintel that fell from the seventh floor of a building located at 601 West 115th Street, in Manhattan.

A week after the tragedy, on May 23, a Board of Inquiry was convened by New York City's Commissioner of Buildings, Irwin Fruchtman, P.E. The Board of Inquiry examined in detail, all available physical evidence and administrative documentation (violations, repair records, testimony of witnesses), contributing to the situation that resulted in the tragedy. In his report to the Mayor, Commissioner Fruchtman summed up the need for a facade inspection law, its benefits and limitations:

> "My conclusion, based on the above and on the fact that the deteriorated conditions uncovered after the accident occurred are so extensive, is that even in a building that is reasonably well maintained by an owner who attempts to respond to complaints, a more thorough repetitive maintenance procedure is necessary to prevent such extensive deterioration. That is why I recommend that the preventive maintenance legislation bill be adopted so a professional is responsible for evaluating the exterior condition at five-year intervals. This will not insure against all potential accidents, but will I am certain, help spot serious deficiencies such as cracked or loose facades or parapets in many instances [6]."

Commissioner Fruchtman further noted, "In addition to improved safety, several other benefits are derived by the city and the owners.

> "Early identification of potentially unsafe conditions can considerably reduce the total cost of future remedial measures.
>
> Monitoring the condition of the building's exterior provides the owner with the tools to develop a maintenance program which will extend the life of his building and preserve our building stock.
>
> The program will provide information to professionals, contractors and owners on conditions which are likely to give maintenance problems which will result in careful exterior design.
>
> The feedback of information on the actual performance of various wall designs will assist the Building Department in developing effective and reasonable regulations for the design, construction and maintenance of exterior walls [7]."

The Board of Inquiry's report was submitted to the Mayor on October 29, 1979, and a bill was drafted by the Department of Buildings (DOB). The law as proposed was reasonable, but not without opposition. New York City's Local Law 10 was passed unanimously by the City Council on February 5, 1980, nine months after the tragedy. At the February 21 public hearing, John Belt, the Director of the Management Division of the Real Estate Board of New York, representing building owners, requested that the proposed legislation be reconsidered. Some of his reasons were self-serving, others proved to be prescient.

> "Firstly, we feel the building code already requires that owners maintain the outside of their buildings. We feel that there is 'duplication.'
>
> Secondly, we feel that the liability on the architects and engineers in signing documents that might be developed by the Building Department will create a situation where there would be great hesitancy by the architects and engineers in signing a document."
>
> Thirdly, that most building owners in the City of New York are already maintaining the outside of their buildings."
>
> Fourthly, we feel strongly we do not know what the requirements or what the standards that the Building Department is going to draw up and in this we are most hesitant and scared about, because we feel that many legislative situations the legislature passes can be a broad stroke bill and then when it comes to implementing the bill, the city administration, Department of Buildings, for example, starts putting their own interpretation of it.

> We would like to have...our own input in helping the city draw these rules and regulations, but we do not want to have things that are just totally onerous to our own industry [8]."

The Mayor reassured Mr. Belt that he and any responsible party could keep in touch with the Commissioner and provide input into the regulations to be promulgated. He then signed the bill into law, which became effective immediately [9].

In summary, Local Law 10 required that, "in order to maintain a building's exterior walls and appurtenances thereof in a safe condition," that for buildings greater than six stories in height, a critical examination of the building's exterior walls and appurtenances be made by a licensed architect or professional engineer at periodic intervals, as set forth by the DOB, but at least once every five years. Exterior walls set back more than 25 feet (7.62 m) from streets and/or paved pedestrian walkways were exempt from the inspection requirements.

The law required the submission of written reports under the seal of the inspecting professional, documenting the condition of the walls and appurtenances, recording all significant deterioration, unsafe conditions and movement observed, as well as a statement concerning the water-tightness of the exterior surfaces. The law further required that the owner immediately commence such repairs, reinforcements or precautionary measures required to ameliorate the observed defects. The law provided an exemption from critical examination for buildings with an "on-going maintenance program," which was defined as a program of preventive maintenance conducted under the supervision of a licensed architect or professional engineer [10].

DOB Rules are Promulgated

On July 7, 1980, the DOB promulgated *Rules and Procedures Relating to the Periodic Inspection of Exterior Walls and Exterior Appurtenances of Buildings.* These rules expanded the general requirements of the new law and provided more specific procedural requirements necessary for compliance. The first "cycle" inspection reports were due prior to February 21, 1982.

The rules left the method of inspection to the inspecting professional, but noted that "the use of a scaffold or other observation platform is preferred." The rules made an example of terra cotta with respect to how the known history of building materials used and conditions observed should dictate the extent of the critical examination. "For example: a special effort shall be made to detect splitting or fracturing of terra cotta on buildings, or cracking of masonry and brickwork in brick faced buildings, etc. [11]"

The rules were very broad. "Unsafe condition" was not defined. There were many ambiguities concerning what constituted a building of greater than six stories. (In the NYC building code, a "basement" is considered a story, while a cellar is not - but this was not widely known.) The extent of "paved pedestrian walkways" was not clear. For example, an interpretation was made that for "setback" facades otherwise exempted because they were more than 25 feet (7.62 m) from a pedestrian walkway, inspection would be required if access to the setback roof was provided by a door. If access to the setback roof were provided by window, the facade would remain exempt. During the initial cycle, professionals muddled through, with many, at the recommendation of their attorneys, opting for the "on-going maintenance plan," rather than declaring a building "safe."

The rules were amended and expanded in February 1987, just prior to the fifth anniversary of the initial due date (the second inspection "cycle"). Definitions of terms, including "critical examination," and "unsafe conditions" were included. Inspection "cycles" were clarified. The term "precautionary condition" was introduced, to classify conditions that while not unsafe, "may lead, if not treated, to an unsafe condition." Owners were directed to repair precautionary conditions as required, and not leave them to deteriorate into unsafe conditions before the end of the next critical examination. Report filing procedures, and procedures for ongoing-maintenance programs, and buildings with "unsafe" conditions were expanded [12].

Local Law 11 of 1998

Over the course of the third and fourth inspection cycles, the DOB continued to issue clarifications, most of which pertained to administrative items, such as changes in filing procedures, forms, and the tracking of violations issued for "unsafe buildings" and "No Reports Filed." For professionals in the marketplace, the "visual" inspection remained, in almost all cases, the method employed.

While the rules were amended, Local Law 10 itself remained unchanged for 18 years, until another dramatic facade failure triggered changes in the law. On December 7, 1997, a large section of a side wall of 540 Madison Avenue rained down on Madison Avenue from above the 33rd floor. Tons of debris were left hanging in a safety net that had been installed as an initial precautionary measure after the bulge in the wall was observed. Remarkably, injuries were relatively minor, but because Madison Avenue was bridged over and closed to traffic for weeks during the peak of holiday shopping season, the disruption was economically disastrous for local businesses. The portion of the facade that had failed had not been subject to the inspection requirements of Local Law 10 of 1980, because it was located on the side "lot-line," beyond 25 feet (7.62 m) from the "street line."

Less than three months after the failure, on January 30, 1998, proposed changes in the law were introduced at the City Council by Gaston Silva, RA, the Commissioner of the DOB. Commissioner Silva cited "a number of recent accidents reported within the City of New York involving falling debris from building facades, partial building collapses and construction site mishaps." He described the accident that occurred at 540 Madison Avenue. He also referred to a second incident in which a large chunk of concrete fell from an unnamed nine-story hotel on Manhattan's Upper West Side. In this case, no one was injured [13].

Two weeks later, on February 26, the City Council unanimously passed Local Law 11 of 1998. The main thrust of the legislation was to remove the prior exceptions to the examination requirements for exterior walls falling outside the scope of Local Law 10, and place even more of the burden on the inspecting professionals.

- Under the new law, virtually all exterior walls of applicable buildings were subject to examination. (The only exemption was for walls less than 12 in. (30.48 cm) from the wall of an adjacent building.) Although, this change was a direct political result of the facade failure at 540 Madison Avenue, common sense and engineering practice would have suggested that the original exemption from inspection for building walls more than 25 feet (7.62 m) from streets, for exterior walls anywhere between 20 and 60 or more stories in height, was arbitrary to begin with.
- Professionals, who, during a critical examination of the exterior walls of a building, became aware of an "unsafe" condition, were required to immediately notify the DOB.
- The law eliminated the previous "precautionary" category. The initial intent of the new law was to have only two categories: "safe" and "unsafe." The argument was made that the precautionary classification was a "hedge" for the inspecting professional, and that the conditions so noted were ignored by owners. Concerns were raised by the City Council that having to declare a building either "safe" or "unsafe" would limit the inspecting professionals' discretion too severely. (Also, it would probably have greatly reduced the number of professionals willing to tackle the critical examinations.) A third category, "safe with a repair and maintenance program" (SWRMP) was restored to the bill. For such conditions, the professional was to recommend a time frame in which the repair was to be performed. However, the law prevented the professional from carrying the condition over to the next filing period, as had been the case with the prior "precautionary" category. As envisioned in 1979, the inspections were to be a preventive maintenance tool, as well as safety inspection. In practice, the new SWRMP classification has provided a disincentive to address lesser, preventive maintenance items in the report. (Conditions reported as SWRMP that are not repaired by the end of the following cycle, automatically default to "unsafe.") Many of the items that were carried as "precautionary" are no longer reported. While these conditions might require remediation, they would not become physically unsafe by the end of the next cycle. Consequently, the report has become less of a preventive maintenance tool for building owners.
- Other provisions included the requirement that unsafe conditions be remedied within 30 days of the "immediate" DOB notification, with 90-day extensions available. The exemption from cyclical critical examination for buildings with an approved "ongoing maintenance program" was deleted. The requirements for this optional program were vague, and proved to be too difficult to enforce.

The proposed law received the "wholehearted" support of the Real Estate Board of New York. Testimony noted that "making additional exposed facades subject to Local Law 10 is a sound 'fail-safe' measure," and that it would have "captured the recent, most dramatic instance of non-compliance [14]."

On March 13, 1998, Mayor Rudolph Giuliani signed the bill into law, which became effective immediately [15]. The Department of Buildings set to work drafting the Amendment to its Rules, based on the new law.

Critical Issues: Technical, Political and Economic

As engineers and architects, we would like to see facade inspection laws written in a way that would allow for comprehensive inspections carried out in accordance with the highest engineering principles and protocols. However, in reality, the real estate community, which pays for inspections and repairs, is affected by market conditions, and is not always willing or able to pay for them. This mechanism is no different from what occurs every day during the design, value engineering, and even construction phases of new projects. Compromises are made.

The following are observations on the dynamics of how New York City's facade inspection laws play out on and above the streets of the city, based on the author's experience as a practitioner.

Public Reaction to Proposed Amendments to DOB Rule 32-03

Based on engineering principles and field experience, one would think that buildings between 80 and 120 years in age, decorated with terra cotta, cast iron, or limestone, would require comprehensive physical inspections.

The DOB did. After the enactment of Local Law 11, the rules were amended by the DOB to redefine the "critical examination" to eliminate specific references to "visual" examination [16]. Under inspection procedures, terra cotta and cast stone were singled out for "hands-on examination." *(italics* added):

> "The methods used to examine the building shall permit a complete inspection of same. The use of a scaffold or other observation platform is preferred, but the professional may use other methods of inspection as he/she deems appropriate *except in the case of all ornamental cast stone and/or terra cotta decorations projecting more than six (6) inches (15.24 cm) from the face of the exterior wall. Hands on examination from a scaffold or other observation platform is required for all ornamental cast stone and/or terra cotta cornices, and for other ornamental cast stone and/or terra cotta decorations projecting more than six (6) inches (15.24 cm) from the face of the exterior wall* [17]. "

Notice of a Public Hearing to Comment on the Proposed Amendment to Rule 32-03 Governing Periodic Inspection of Exterior Walls and Appurtenances of Buildings was published in the *City Record.* A public hearing was held on November 17, 1998, during which a firestorm of criticism was unleashed at the proposed rule changes.

Numerous organizations, including The New York Landmarks Conservancy, the Municipal Art Society of New York, Friends of Terra Cotta, the Federation of New York Housing Cooperatives, as well as several practitioners objected, strongly and in unison, to the singling out of terra cotta and cast stone to be inspected on a hands-on basis. Position papers were read into the record.

There was concern that the new requirements would cause a new wave of "facade stripping." After the passage of Local Law 10, a tragically large amount of terra cotta and other ornament was removed from buildings, even prior to inspection. In one of the most notorious cases cited, the facade of the Mayflower Hotel, located on Central Park West was denuded of almost all detail, including balconies as well as cornices and water tables. Photographic evidence of facade stripping was provided at the hearing by the Municipal Art Society.

The Chair of the City Council's Subcommittee on Landmarks, Public Siting and Maritime Uses termed the requirement "capricious," and stated that there was no legislative intent in the law itself to single out certain building materials for "hands on" inspection. He noted that the highly publicized failure that was the "driving force" behind the legislation involved a brick facade (540 Madison Avenue), and not cast stone or terra cotta [18].

The DOB promulgated the amended rules, substantially unchanged from those initially issued for comment, with the exception of the requirements for physical examination. Terra cotta and cast stone were no longer singled out for physical inspection. Now all subject buildings, both old and new, of all wall-types, required physical inspection. The text singling out terra cotta was replaced with the following: (*italics* added)

> "*....A physical examination from a scaffold or other platform is required for a representative sample of the exterior wall. The professional shall determine what constitutes a representative sample. The representative sample must include at least one physical examination along a path from grade to the top of an exterior wall on a street front using at least one scaffold drop or other observation platform configuration [20].*"

A new paragraph was added to protect landmarked buildings:

> "The removal of portions of the facade in order to facilitate the performance of tests may require a permit from the Landmarks Preservation Commission [21]."

"Deferred" Maintenance

Building owners, managers, residents are often not fully aware of the issues involved in the preventive maintenance required to forestall deterioration of facades. Thus maintenance of facades is deferred, in some cases indefinitely. Commercial owners want to spend money on new air conditioning and internet communications systems. For residential building managers, elevator upgrades and lobby beautification programs often have priority. Mandated facade repairs can cost hundreds of thousand dollars, and at the end of the day, if they are performed well, there is no visible evidence that work was done. Ironically, the one item that would be most marketable - the cleaning of the facade - is frequently beyond what the budget will allow.

Too Many Regulations

New York City is a very highly regulated place. Facade inspection requirements are often viewed by building owners as just another nuisance to deal with. For example, Local Law 10 of 1980 is the facade inspection law. Local Law 10 of 1981 mandates periodic inspections for elevators, which also injure and kill people from time to time. There are huge fines assessed for non-compliance with the elevator inspection law. If you were to ask a building owner has he or she complied with Local Law 10, the answer could very well be: "Which Local Law 10?"

The DOB is also caught in a bind. Enforcement of New York City's facade inspection laws is very complicated. The DOB must aggressively and even forcefully encourage compliance, while respecting the rights and reasonable financial concerns of building owners. It should also be understood that while the identification of facade defects, by mandated inspections, is an important step in providing safer streets - it is the first step. Ensuring that the required repair work is done expeditiously is really "the ball game" for the DOB, and the point of the facade inspection laws.

The Marketplace

Often, the amount of repair work required by a building is inversely proportional to the amount of income generated by the building. Older buildings are often more highly decorated, (terra cotta, limestone), and more poorly maintained than their newer and more streamlined counterparts. The older buildings, based on their location, their limited floor plates and lack of up-to-the minute mechanical systems, generally have lower rent rolls and their owners are less able to afford the cost of inspection and repairs.

The real estate industry, representing building owners, is seated on DOB committees that draft laws, and has a large say in determining the technical requirements of proposed legislation. Real estate market conditions determine the amount of money that might be available for inspection and repair by building owners. In better markets, the real estate industry is more willing to spend money on facade inspections and repair. The physical reality necessitating the inspections does not vary with market conditions. The deterioration of a building, while a function of the environment, is neither cyclical, nor linear. As a building ages, the rate of deterioration of systems of components tends to accelerate over time. Historically, it takes a tragic incident to trigger legislation.

Some segments of the real estate community view the inspections as a "commodity," and will generally accept any report that a licensed professional is willing to sign. From their perspective, not necessarily unreasonably, anyone with a license is qualified to perform the critical examination. For some owners, the fee will often determine who makes the critical examination - regardless of the technical competence, or experience of the professional. For architects and engineers, facade inspection laws provide a marketing opportunity - a chance to "get a foot in the door." The inspecting professional has to be able to strike a reasonable balance -- and be convincing to all parties: If you propose to do too much, (i.e. suggest more than the code-mandated single scaffold drop) you will not get the job.

Financial Constraints

In his 1979 introduction of the proposed Local Law 10, Commissioner Fruchtman estimated that "the cost of such a program to an owner would be in the range of 10 to 15 cents per square foot (0.0929 m^2) of building exterior, and would average $3,000.00 every five years [22]." (An average of 12.5 cents per square foot of exterior would yield a building with facade area of 24,000 square feet (2,229.6 m^2), or approximately 20 stories with 120 feet (36.6 m) of frontage.) The Commissioner's estimate, although more than 20 years old, is still in the ballpark - despite the fact that the scope of the examination has increased. Typical fees for a building of the noted magnitude, may range anywhere from $2,000.00 to $6,000.00, exclusive of rigging costs.

Lack of Background Information

The economics of the inspections do not permit a truly comprehensive investigation. For many buildings, the cost of the swing staging for "hands-on" inspections (for even a single scaffold drop), exceeds the fee charged by the inspecting professional. Drawings of existing conditions of building facades are rarely available for examination during the course of the critical examination. Owners simply do not have them. Drawing searches in DOB records for buildings over 30 years old are costly, and almost always unsuccessful. Exploratory probes are generally outside the scope of the initial critical examination. Probes are expensive, and while the scaffold drops for the physical examinations are generally made without the need of a sidewalk shed to protect the sidewalk below (the sidewalks are roped off), sheds are necessary for probes to be made. (Probes are included in the more comprehensive investigations made in connection with the preparation of construction documents to repair observed facade defects.)

Lack of Standards

New York City laws do not establish minimum standards of technical competence and experience for the inspecting professionals. There is no code requirement that the inspecting professional have any specific experience with exterior walls. The economics of the inspections as well as the lack of drawings of existing conditions, however, require that inspecting professionals are knowledgeable of the fabric of the buildings they are examining. They must make critical determinations, based on relatively little evidence.

Inspection is based on observed symptoms. It is assumed that the inspector is familiar with what is happening below the building surface, and can extrapolate the causes.

New York City laws do not provide standards, or protocols for facade inspections, beyond the requirement of at least a single scaffold drop on a street facade. The law relies on the inspecting professional to determine methods employed in the examination. These are left to the discretion of the professional, who shall "determine what constitutes a representative sample of the exterior wall, and use an undefined "professional standard of care" to detect facade defects. Additionally, there is a large variation in the level of knowledge, experience and competence of the licensed professionals making the inspections. As a result, the quality of mandated facade inspections varies greatly -- from "drive-by" inspections with minimal documentation to thoroughly documented inspection reports.

Interestingly, the rules promulgated by the DOB in connection with Local Law 11 introduced language stating, "The registered architect or licensed professional engineer shall utilize a professional standard of care to detect splitting or fracturing of terra cotta, or cracking of masonry and brick work..." This replaced text simply stating that, "...an effort shall be made." No standard was referenced.

The "lowest common denominators" of practitioners are known to the DOB, but are not prevented from making inspections and having constituencies among building owners. At the same time, some highly qualified engineering firms have shied away from providing Local Law 10 and 11 facade inspections.

Lack of Clarity

The law itself is not clear, and subject to interpretation at critical junctures. For instance: What constitutes an "unsafe" condition?" The current DOB Rules provide no working distinction between a condition termed "unsafe," which might be understood to be imminently hazardous, and conditions termed "safe with a repair and maintenance program" (SWRMP) that would have to be repaired before the onset of next winter.

A notice of an "unsafe" condition triggers a relatively onerous situation for the building owner with respect to repair timetables and financial penalties. It puts the inspecting professional in a difficult bind as well. An "unsafe" condition requires *immediate* notification "by letter or fax." The professional does not have the luxury of a considered decision. He or she must immediately make a decision regarding a potentially unsafe condition.

Frozen Toes

In New York City, the report-filing deadline is a function of the date the law was enacted. Reports are due in mid-February, generally resulting in a crush during the time of year when it is most difficult to inspect buildings.

Improvements

Improvements to New York City's Local Laws 10 of 1980 and 11 of 1998 and the related DOB Rules that would make them more comprehensive and effective are no mystery. Excellent suggestions were made on the record during the November, 1998 public hearing on the proposed amendments to the DOB rules. They include:

- Visual inspection of all buildings,
- Limited physical inspection of buildings less than seven stories in height,
- Changing the February filing date to a more friendly time of year,
- Making real-estate tax incentives available to building owners for making repairs in compliance with the laws [23].

Laws and rules are currently in place, that with relatively minor amendment, could result in a reasonably high quality of inspections and reports that would better protect the pedestrians and building owners of New York City. However, the rules exist in a system governed by economics, where changes are hard-fought, and for many of the reasons described, the quality of the results suffer.

Based on our firm's experience, perhaps the single most important improvement in the law, aside from clarifications of the definition and reporting requirements for unsafe conditions, would involve an increase in the minimum required amount of facade area covered by physical examination. There is no substitute for close-up inspection.

Conclusions

Have New York City's facade inspection laws served their intended purpose, as envisioned by Commissioner Fruchtman in 1979? The answer is that for the most part, they have. While the system may be imperfect, it is the author's belief that the overall safety of New York City has been improved as a result of its facade inspection laws. (Figures were unavailable from the DOB.)

The mandated facade inspections, as practiced, are more focused on the safety of pedestrians than preventive maintenance. However, Local Laws 10 and 11 have led to an increased level of awareness of the problems with building facades, and that has, in turn, resulted in increased preventive maintenance. Facade restoration has become a specialized area of practice.

Our firm's experience indicates that enlightened owners will, in the spirit of good building maintenance, pro-actively undertake comprehensive facade investigation and repair programs. Most of the inspections we have made have been for existing clients, in connection with pending restoration projects. Astute owners have come to understand that if a preventive maintenance program is in place - the required inspection can be perfunctory.

In the 22 years that have passed since the enactment of Local Law 10, a large percentage of buildings have had some facade repair work done on them. However, the building stock is now older and continues to deteriorate at an ever-increasing rate. Facade inspections will become an even more critical tool in maintaining the safety of pedestrians and protecting the building stock of New York and other cities.

Regarding Philadelphia, a recent follow up with the author of the newspaper article about the 2001 facade failure drew this remark: "After people saw the World Trade Towers collapse nobody was very interested in a few falling bricks here and there. Absent a disaster, I do not think the city administration here is likely to push a facade inspection ordinance[2]."

As for New York City, the "ongoing" Local Law 10 Review Committee that was in the process of reviewing proposed changes to the facade laws has not met since September, 2001, and as of the fall of 2002, no meetings have been scheduled. There has been a change in mayoral administration, and a new DOB commissioner. New initiatives are being examined by the DOB for high-rise safety (egress, structural) as a direct result of the September 11 disaster. The next Local Law 10 and 11 facade inspection cycle commences in 2005. The Local Law 10 Review Committee will probably get back to it - especially if there is another facade failure of note.

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[3] For background information on facade ordinances in these cities, see: Petermann, Michael A., and Normandin, Kyle C., "The Role of Facade Ordinances in Preserving the Recent Past," *The Proceedings of The National Park Service Conference: Preserving the Recent Past 2*, Philadelphia, PA, October, 2000, pp. 2-131 - 2-139.

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[6] Fruchtman, Irwin, PE, Commissioner New York City Department of Buildings, Board of Inquiry Report: 601 West 115th Street, October 29, 1979, p.1 (included in "Bill Jacket" for Local Law 10 of 1980, of the NYC City Counsel Archives, obtained for the author by New York Legislative Service, Inc., New York, NY)

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[8] Transcript of the Stenographic Record of the Hearing on Local Laws, etc., dated February 21, 1980, prepared by the Acme Reporting Service, New York, NY, pp. 4-5. (included in Local Law 10 Bill Jacket).

[9] Transcript of February 21, 1980 hearing, p. 5.

[10] Local Law 10 of 1980 (the 10th law enacted during the year 1980) was an amendment to the "Building Code" portion of the Administrative Code of the City of New York. Local Law 10 added section C26-105.3. The code was subsequently re-numbered, so that the provisions of Local Law 10 are currently contained in administrative code section 27-129.

[11] "Rules and Procedures Relating to the Periodic Inspection of Exterior Walls and Exterior Appurtenances of Buildings," promulgated by the Commissioner of the New York City Department of Buildings, dated July 7, 1980.

[12] "Rules and Procedures Relating to the Periodic Inspection of Exterior Walls and Exterior Appurtenances of Buildings," as amended by the Commissioner, dated August 10, 1987.

[13] From the Testimony of Gaston Silva, RA, Commissioner, New York City Department of Buildings, re: Intro 97/98, dated January 30, 1998. (included in "Bill Jacket" for Local Law 11 of 1998, of the NYC City Counsel Archives, obtained for the author by New York Legislative Service, Inc., New York, NY)

[14] From Testimony presented by Deborah B. Beck, Executive Vice President, Real Estate Board of New York before the City Council Housing and Buildings Committee in Support of Intro 97-A, on February 13, 1998. (included in Local Law 11 Bill Jacket).

[15] Local Law 11 of 1998 (the 11th law enacted during the year 1998) was an amendment to the "Building Code" portion of the administrative code of the City of New York, modifying the language contained in administrative code section 27-129.

[16] "Rules and Procedures Relating to the Periodic Inspection of Exterior Walls and Exterior Appurtenances of Buildings," Draft, dated October 6, 1998, as included in the "Notice of Opportunity to Comment on a Proposed Amendment to Rule 32-03 Relating to Governing Periodic Inspection of Exterior Walls and Appurtenances of Buildings," undated. (Copy of document obtained by the author from the NYC Department of Buildings records under the Freedom of Information Law (FOIL).

[17] Draft of Rule 32-03, Section 32-03(b)(2)(iv).

[18] From Testimony by City Council member John D. Sabini, Chair of the City Council's Subcommittee on Landmarks, Public Siting and Maritime Uses. (Copy of document obtained by the author from the NYC DOB records under the FOIL.

[19] Draft of Rule 32-03, Section 32-03(b)(2)(v).

[20] Rule 32-03, as amended, Section 32-03(b)(2)(iv).

[21] Rule 32-03, as amended, part of Section 32-03(b)(2)(v).

[22] Fruchtman, letter to Ms. Lisa Ruffo, dated November 2, 1979.

[23] From Testimony by various speakers at the November 17, 1998 Public Hearing. (Copies of documents obtained by the author from the NYC DOB records under the FOIL.

Section II: Addressing Historic Buildings

Kecia L. Fong and CeCe Louie(1)

Façade Ordinances and Historic Structures – Theoretical and Practical Conservation Issues in Inspection and Repair

Reference: Fong, K. L., and Louie, C.. "**Façade Ordinances and Historic Structures – Theoretical and Practical Conservation Issues in Inspection and Repair**," *Building Façade Maintenance, Repair, and Inspection, ASTM STP 1444*, J. L. Erdly and T. A Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: The primary concern of building façade inspection ordinances is to insure the safety of the public and surrounding property and to establish baseline cycles of inspection, maintenance and repair. While public safety is paramount, what these ordinances do not address are the appropriate means of investigation and remediation with respect to historic structures. Local preservation ordinances acknowledge the primacy of public safety, however, they do not provide direction for appropriate emergency interventions to preserve and protect a building's historic and architectural integrity. The lapse in continuity between façade inspection and preservation ordinances can result in unnecessary and irreparable loss of integrity and value. Work on historic structures poses distinct and often complex philosophical and practical issues requiring thoughtful and creative responses. The inspecting professional must be well informed of historic building practices, contemporary building codes, and established preservation standards and legislation to blend public safety and historic preservation needs effectively. This paper presents the theoretical and practical conservation issues involved in the inspection and repair of historic structures.

Keywords: façade ordinance, historic structure, preservation, inspections, repairs, Secretary of Interior's Standards

Introduction

The inspection, assessment and repair of historic structures requires diverse knowledge ranging from historic building materials and techniques to modern scientific methods of investigation, contemporary building codes, preservation ordinances, national

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standards for treatment of historic properties, and the contextual reasons for a building's significance. This paper presents the many practical and theoretical factors involved in the inspection and repair of historic structures. More importantly, the text identifies the discontinuities between existing ordinances and well established standards for the treatment of historic properties, and it proposes a methodology for inspection and repair that bridges the gap between the ordinances and established standards for treatment. The first half of the paper provides a brief discussion of existing ordinances as they pertain to historic structures, presents definitions of historic structures, and mentions a few of the existing provisions for their protection and repair. The second half of the paper presents a case study of Saint Dominic's Church in San Francisco, CA. The case study illustrates the existing discontinuities, as identified in the first half of the paper, and presents a proposed methodology for assessment, investigation and repair of historic structures that addresses the specific needs of historic structures and is therefore more expansive than existing façade ordinances.

The primary objective of existing façade ordinances is to insure the safety of the public and surrounding property. While the ordinances vary from city to city in their required level of detail with regard to inspecting and reporting, they all seek to establish a baseline cycle of inspection and remediation. In the enforcement and prescription of inspection, repair, and maintenance, the existing ordinances do not differentiate between historic and contemporary structures. The significant differences need to be addressed.

Historic Structures

> ...an historic building is one that gives us a sense of wonder and makes us want to know more about the people and culture that produced it. It has architectural, aesthetic, historic, documentary, archaeological, economic, social and even political and spiritual or symbolic values; but the first impact is always emotional, for it is a symbol of our cultural identity and continuity – a part of our heritage[1]
>
> *-Sir Bernard Feilden*

Criteria for Recognition

Historic structures are identified by established federal, state and local criteria. Federal criteria are stated in the National Register of Historic Places, state criteria are determined by the State Historic Preservation Office (SHPO) of each state, and local criteria are determined by the local municipal agency such as the local historic preservation commission. At each level, the criteria outline the conditions of value, the basis by which sites and structures are evaluated and identified as worthy of recognition and protection. At the federal level, fulfillment of the criteria usually results in inscription into the list of National Register of Historic Places(2). The National Register summarizes an historic structure as one that "possesses integrity of location, design, setting, materials, workmanship, feeling, and association"[2].

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[2] For structures of exceptional national value that represent and function as a common national bond, these are designated as national landmarks. "National Historic Landmark Program," available from: www.cr.nps.gov/nhl/whatis.htm

State and local historic preservation legislation and guidelines are derived from these federal criteria and adapted to incorporate the relative historic place values of each regional entity (state, county, city or town).

Historic structures are more than the mere conglomeration of their constituent material parts. They are imbued with cultural meaning and values. Inappropriate recommendations by the inspecting professional due to a lack of awareness of these special values could result in irreversible damage to the building and its material and cultural values.

Existing Legislation and Review Processes

There are existing provisions for the protection and care of historic structures. These are in the form of legislation and guidelines for treatment that collectively constitute established national standards. Many of these were born of the National Historic Preservation Act 1966 and include:

- National Register of Historic Places,
- Section 106 review process,
- *Secretary of the Interior's Standards for the Treatment of Historic Properties, and*
- The American Institute for the Conservation of Historic and Artistic Works *Code of Ethics* and *Guidelines for Practice.*

The inspecting professional should be informed of pertinent legislation and established standards. These provide qualitative guidelines regarding the design of appropriate treatment strategies. Failure to follow these guidelines can jeopardize a building's historic status, irreparably alter its condition and identified value, and threaten its tax status and funding.

Historic Status and Preservation Legislation

Whenever possible, the inspecting professional should be informed of a building's historic status prior to the field inspection as required by a façade ordinance. If an emergency precludes this preliminary investigation, then the historic status should be determined before the final recommendations are made and the report is submitted.

A structure may be registered or considered eligible for register individually or as part of a larger context, (such as a district), at the federal, state and local levels. At each level, the governing body may exercise its own authority in the form of building codes, ordinances and guidelines in its efforts to protect its regional built heritage. Different regulatory processes may apply to a structure depending upon its level of recognition and the state and/or federal assistance it receives. In general, the strictest codes will apply. These factors can have significant impact upon the processes for rehabilitation and treatment of the building.

Historic status information should be obtainable from the building owner and/or the local historic preservation commission, especially as pertains to local designation(3). It is also advisable to consult the State Historic Preservation Office (SHPO)(4). The SHPO

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[3] Depending on the city, the preservation commission may be named Landmarks Preservation Advisory Board (San Francisco), Landmarks Preservation Commission (New York City), Commission on Chicago Architectural Landmarks (Chicago), etc.

[4] Contact the local governing historic preservation agency (see foot note above) for local historic listing status. At the state level contact the State Historic Preservation Office (SHPO). A list of all SHPOs is provided on the National Park Service website available from: www.cr.nps.gov/nr/shpolist.htm. The SHPO

retains relevant state and national designation information and can often provide more detailed information about individual buildings within nationally registered districts, such as their contributing or non-contributing status. Contributing buildings are integral to the overall value of a district for their individual architectural or historic integrity or their potential to yield important archaeological information. Contributing buildings independently meet the criteria of the National Register. Non-contributing buildings do not enhance the historic, architectural or archaeological values of a district and do not independently meet the criteria of the National Register. Consequently, non-contributing buildings are not subject to the same regulations as contributing buildings and are not eligible for the same tax benefits.

At the national level, if a building is a recognized historic resource and benefits from federal or state assistance, permit, or licensing, the building and all proposed work on it, are potentially subject to a formal review process. This review process is known as Section 106(5). At the state level, each state may administer its own version of a review process, for example, in California this is known as the California Environmental Quality Act (CEQA). Depending upon the involvement and governing power of the local agency, local level ordinances and regulations can be the most stringent of the three levels.

While all agencies recognize the primacy of public and property safety, few have implemented specific measures for precluding unnecessary removal and irreparable alteration of historic fabric. Section 106 refers to such actions as *adverse effects*. Adverse effects are defined as any

> ...undertaking [that] may alter, directly or indirectly, any of the characteristics of a historic property that qualify the property for inclusion in the National Register in a manner that would diminish the integrity of the property's location, design, setting, materials, workmanship, feeling, or association[3].

Adverse effects include but are not limited to:

- Alteration of a property, including restoration, rehabilitation, repair, maintenance, stabilization, hazardous material remediation, and provision of handicapped access, that is not consistent with the Secretary's standards for the treatment of

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should be able to provide National Register listing status. The National Park Service provides access to the list of nationally registered properties, available from: www.cr.nps.gov/nr/research/nris

[5] Section 106 of the National Historic Preservation Act of 1966 (NHPA) requires Federal agencies to take into account the effects of their undertakings on historic properties, and afford the Advisory Council on Historic Preservation (ACHP) a reasonable opportunity to comment. The historic preservation review process mandated by Section 106 is outlined in regulations issued by ACHP. Revised regulations, "Protection of Historic Properties" (36 CFR Part 800), became effective January 11, 2001.

The responsible Federal agency first determines whether it has an undertaking that is a type of activity that could affect historic properties. Historic properties are properties that are included in the National Register of Historic Places or that meet the criteria for the National Register. If so, it must identify the appropriate State Historic Preservation Officer/Tribal Historic Preservation Officer (SHPO/THPO) to consult with during the process. It should also plan to involve the public, and identify other potential consulting parties. If it determines that it has no undertaking, or that its undertaking is a type of activity that has no potential to affect historic properties, the agency has no further Section 106 obligations. Further information on the Section 106 Review process is available from: www.achp.gov/106summary.html historic properties (36 CFR [Code of Federal Regulations] part 68) and applicable guidelines;

- Change of the character of the property's use or of physical features within the property's setting that contribute to its historic significance; and
- Introduction of visual, atmospheric or audible elements that diminish the integrity of the property's significant historic features[4].

In the event of imminent danger or life safety hazards, the Advisory Council on Historic Preservation (ACHP) encourages state and local agencies to develop their own procedures(6).

Inspections

Due to their age, construction, and design, historic structures require close-up, hands-on inspections. Their recognized historical value necessitates a conservative approach to the destruction and removal of original material. Methods of investigation must be thorough and nondestructive when possible.

When an inspection opening is necessary, all precautions should be taken, consistent with safety, to minimize damage to the original fabric. Such methods may include removing already damaged material and/or cutting an opening at existing joints and removing the unit intact. By doing so, the unit can be reinstalled, minimizing unnecessary damage and eliminating the need for patching and repair. This is particularly critical in a material such as terra cotta, where damage to the protective glaze results in accelerated deterioration.

Whenever there is a need to remove original material, whether for an investigative opening or to mitigate an unsafe condition, the existing (as found) condition should be well documented with photographs, drawings, and field notes. The individual pieces should be documented, numbered, and stored, preferably on site, in a protected environment for future reinstallation. The storage location and inventory of materials should be documented and filed both with the building owner and the inspecting official. When the material cannot be immediately reinstalled until a final repair is defined, provide temporary protection to the structure and surrounding materials. In some cases, it may be advisable to stabilize the unsafe condition by installing netting or bracing around the feature and anchoring back to sound substrate. This will prevent the loss of historic fabric, and provide time to perform a more detailed evaluation and devise appropriate repairs to reuse or replicate the feature.

Another purpose of the inspection is to determine the cause of the observed deterioration conditions. Some façade ordinances fail to require determination of the cause of the hazard or observed deterioration. Without this determination, an appropriate long-term solution cannot be designed reliably. Initial emergency stabilization solutions are often temporary "emergency" measures and must be clearly identified as such. These should not be confused or substituted for long-term solutions that are based on a clear understanding of the deterioration mechanisms.

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[6] The Advisory Council on Historic Preservation (ACHP) is an independent Federal agency that promotes the preservation, enhancement, and productive use of our Nation's historic resources, and advises the President and Congress on national historic preservation policy. For more information about the mission and activities of the ACHP go to: www.achp.gov/aboutachp.html

Repair Recommendations.

Guidelines for treatment of historic properties are described in the *Secretary of the Interior's Standards for the Treatment of Historic Properties.* These are the Nation's established standards and guidelines. In this document, levels of treatment are divided into four categories. These are preservation, rehabilitation, restoration, and reconstruction. Each defines a degree of treatment depending upon existing integrity, significance, and potential use. All categories stress:

- minimum intervention,
- maximum retention of original fabric,
- repair before replacement,
- preservation and retention of historic character in materials, design, and spatial relationships,
- thorough documentation of all interventions,
- use of the gentlest means possible for all chemical and physical treatments,
- stabilization *in situ* as a preliminary measure, and
- replacement in kind.

While the Standards do not explicitly advocate reversibility, this is a much advocated concept within conservation. The idea of reversibility is to protect the building or site from irreparable modification by irreversible treatments; to permit future study of the original; and to allow for future treatments which may be more appropriate due to advances in knowledge and technology.

The Standards acknowledge the need to conform to contemporary code requirements but warn,

> ...if hastily or poorly designed, a series of code-required actions may jeopardize a building's materials as well as its historic character. ...modifications to the historic appearance should be minimal[5].

Established conservation principles state clearly that interventions, temporary or otherwise, should cause no damage to the original existing fabric. They should fulfill the requirements of physico-chemical and aesthetic compatibility. They should be reversible whenever possible, should not interfere with future research of the resource, nor prevent the possibility of replacement of the intervention with a newly developed alternative. They should be thoroughly documented and distinguishable from the original upon close inspection, and should not obscure nor alter the inherent historic character nor compromise its integrity(7)

Federal, state and local agencies refer to the Standards as the measure of acceptable practice when reviewing permit applications for work on historic structures, applications for licensing, or funding assistance.

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[7] In the United States, the Secretary of the Interior defines integrity as "...the authenticity of a property's historic identity, evidenced by the survival of physical characteristics that existed during the property's historic or prehistoric period." *Archaeology and Historic Preservation: Secretary of the Interior's Standards and Guidelines* [As Amended and Annotated]. Available from: www.cr.nps.gov/local-law/arch_stnds. In *National Register Bulletin 15* it is defined as, "the unimpaired ability of a property to convey its historical significance." Available from: www.cr.nps.gov/nr/publications/bulletins/nr15_toc.htm

Emergency Repairs

Although cities' building codes define clearly an imminent hazard and who is responsible for its mitigation, they are less clear on how the building official or professional conducting the survey will provide recommendations sensitive to the historic value of features needing emergency removal, stabilization or repair. Often, absent a second opinion on recommended steps, the building feature is removed immediately, or stabilized or repaired in a permanent manner that can be indifferent to its unique nature and function, or special conservation considerations. What is missing in both the façade and landmark ordinances is clear language that both allows for the expeditious removal of unsafe conditions, and provides guidance for retaining as much of the original building fabric as possible and/or use of materials that follow the guidelines of the Secretary of Interior Standards.

The persons responsible for the emergency measures, if lacking experience in architectural preservation or material conservation, may prescribe repairs that create further damage to the structure or building feature, or cause repairs that are functional but aesthetically anathema and potentially non-reversible. In some cases, the repairs can be both damaging to, and aesthetically incompatible with historical aspects of the building. At present, the onus is upon the inspecting professional to be informed of the historic status, and all applicable regulations and standards when recommending remedial actions for hazardous conditions and designing repair recommendations.

Case Study – Saint Dominic's Church

Background

Saint Dominic's Church in San Francisco is a representative case study of the unfavorable effects current façade ordinances can have on an historic structure, and what we propose as an appropriate methodology for the inspection and repair of historic structures. The work at Saint Dominic's did not arise out of a mandatory façade ordinance inspection (San Francisco does not have a façade ordinance), nor is it an officially recognized historic structure, but as with many historic buildings in U.S. cities without façade ordinances, the work on Saint Dominic's Church characterizes the conflict between the need for public safety and special historic needs. Saint Dominic's is also an especially good example of how, even when a building does not possess official historic status, following a sound methodology results in a high standard of care.

Saint Dominic's Church was completed in 1928 by the English architect Arnold S. Constable, who worked with the Seattle firm known as the Beezer Brothers (Figure 1). The Church is a poured-in-place concrete frame structure clad in architectural terra cotta. Aside from its notable English gothic design, it contains many works of art designed or made in Europe, and significant stained glass windows by Charles J. Connick of Boston, Max Ingrand of Paris and Cummings Studios of San Francisco.

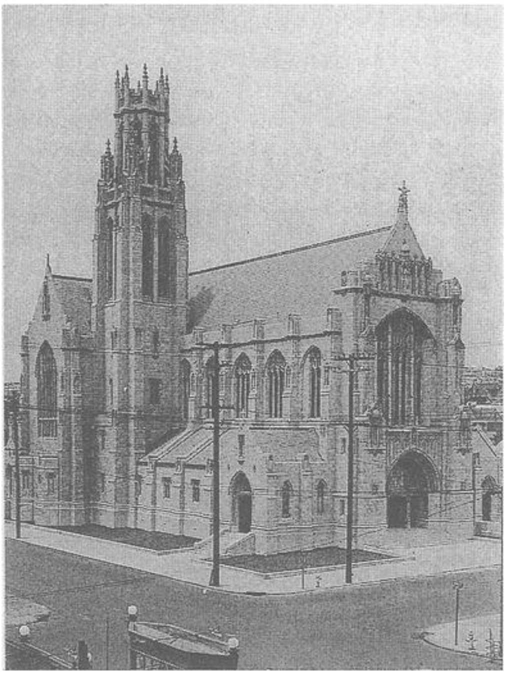

Figure 1 – *Saint Dominic's Church shortly after completion in 1928.*

In the summer of 1989, the San Francisco Landmark Advisory Board began taking action to declare St. Dominic's Church a city landmark. The Church strongly resisted this measure knowing that landmark status would prevent them from implementing a seismic retrofit design located conspicuously on the outside of the structure. The three years of planning that went into the seismic retrofit design was a direct result of the Church's desire to maintain the beauty and integrity of the interior space. Conventional interior seismic schemes would have resulted in bulky shear walls or a series of trusses that would block the stained glass windows or ruin the lofty space. As the architectural critic Allan Temko stated at the time, "That would have virtually destroyed St. Dominic's in order to save it [6]".

Ironically, before the Landmark Advisory Board could move ahead with establishing the Church as a city landmark, the 1989 Loma Prieta Earthquake struck and caused significant damage to the structure. Without further discussion, the Church proceeded with the exterior seismic design. In addition to the installation of the exterior seismic design in the early 1990s, additional work was performed on the Church to address safety hazards and repair damaged and/or deteriorated features. These early 1990 repairs are examples of what can result if façade ordinances are used in their current state on an historic structure without official status; many of which are considered "adverse effects" based on Section 106 of the National Historic Preservation Act of 1966 (NHPA) [7]. They include the following:

- Changes to the defining characteristics of the building,
- Removal of a significant building feature,

- Replacement of deteriorated material with non-matching materials,
- Introduction of visual elements that diminish the property's historic integrity, and
- Failure to address the cause of the problem.

Simpson Gumpertz & Heger's (SGH) work on Saint Dominic's Church arose initially from the Church's concern with interior leaks. Our investigation into the leaks broadened to identifying moisture-related problems at the stained glass windows and the terra cotta masonry. We determined that many of the early 1990's repairs compromised the historic qualities of the church and required further remedial action. The repairs we implemented in 2002 corrected most of these changes using the guidelines of the *Secretary of Interior's Standards for the Treatment of Historic Properties*. The repair work included identifying features and conditions, retaining and preserving, protecting and maintaining, repairing, replacing, and addressing missing historic features.

Investigation and Documentation

Without original drawings of the Church, we reconstructed the elevations by taking measurements during our close-up inspection of the repair areas. We made investigative openings to determine the cause of conditions such as large vertical cracks (that had been repaired several times) by removing whole terra cotta units without damaging adjacent units (Figure 2). This method also provided us with the opportunity to reinstall the original material. Where units were severely cracked or failing, we removed the units in pieces and replaced with material in-kind. In locations where the replacement of material affected special features, we provided a high level of documentation. For example, knowing large portions of the terra cotta at the stain glass windows required replacement, we made a complete and permanent set of record photographs of the windows to serve as a reference for craftspeople restoring the windows. The photo-documentation also provided important insurance information for the Church. We also identified and located on the drawings each terra cotta tracery unit due to the need to disassemble and reassemble the tracery areas to accommodate repairs.

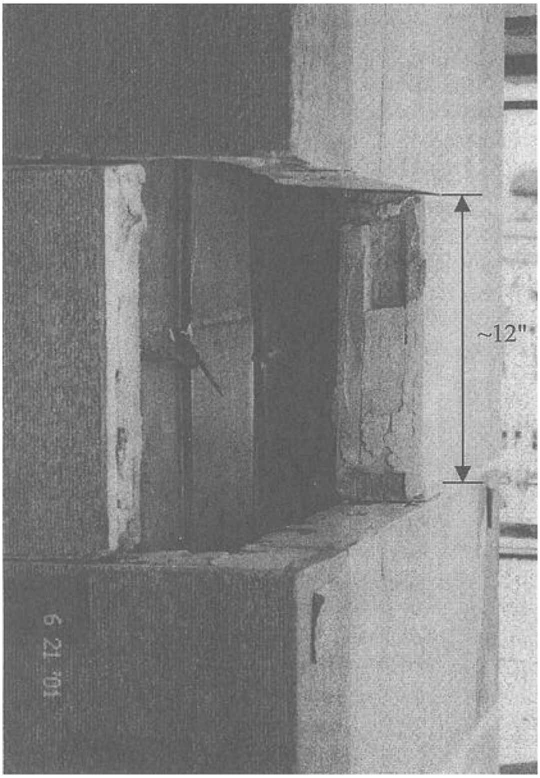

Figure 2 – *Removal of a whole terra cotta unit to reveal substrate conditions*

Repairs

Changes to the Defining Characteristics of the Building - The early 1990s seismic retrofit consisted of installing a series of flying buttresses to the exterior walls. The buttresses were anchored to a new concrete ring beam at the base of the roof and to several caissons extending deeply into the ground. The result significantly altered what was originally a gothic revival design into a high-gothic design (Figure 3). In addition, the buttresses and the roof ring beam were made of concrete; a marked contrast to the granite patternglazed terra cotta cladding. We did not attempt to correct for this change. While the flying buttresses do not meet the established standards for the care of historic structures because they drastically alter the original design, they provide a creative solution utilizing medieval construction concepts to solve modern day seismic requirements. The result is both dramatic and conscious of its architectural forbears. The flying buttresses also allowed the Church to preserve the building's interior space and appearance.

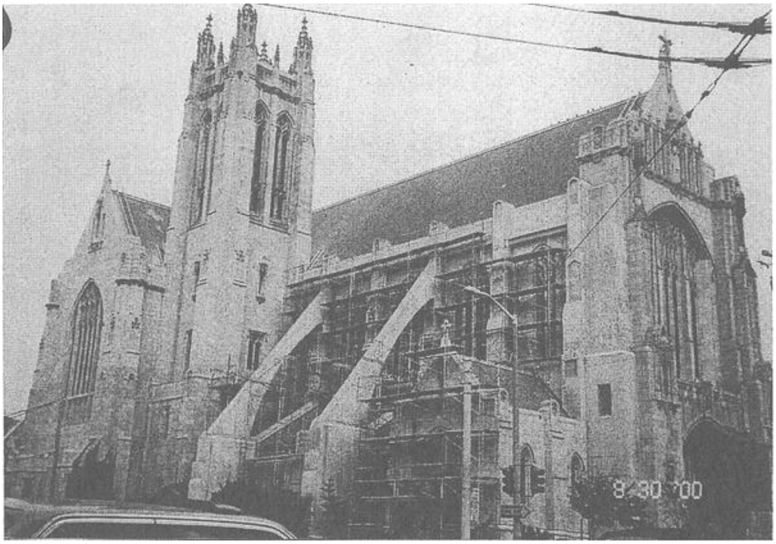

Figure 3 – *Saint Dominic's Church with the flying buttresses and bell tower with no lantern*

Removal of a Significant Building Feature - One of the first areas of the building needing immediate attention after the earthquake was the severely damaged 25-foot tall, ornate terra cotta lantern, located atop the 125-foot, terra cotta bell tower (Figure 1). The engineer-in-charge at the time determined that the lantern's unsafe condition warranted its complete removal. The removal of the lantern resulted in the loss of the bell tower's visual prominence and the apparent loss of original material (Figure 3).

The initial stories from our research of the lantern's history led us to believe the lantern was torn down and sent to a landfill. Later discussions with the Engineer, Architect and Contractor who worked on the post-earthquake repairs indicated the lantern was photo-documented and each terra cotta unit labeled before dismantling began. The Contractor revealed the lantern units were in his warehouse. The possibility of rebuilding the lantern, using the original material and documentation of its deconstruction, was encouraging, for it presented an opportunity to restore the bell tower and the Church to its original design.

Ironically, in the end, the bell tower was seismically retrofitted in the early 1990s with steel bracing but not designed to carry the loads of a rebuilt lantern. The engineer at the time asked the Owner to promise never to consider rebuilding the lantern atop the bell tower. Even with the careful effort to document, record and store the disassembled structure, the lantern was relegated to permanent storage. We are suggesting to the Owner the following three options: 1) reassemble and exhibit all or portions of the lantern on the grounds of their property, 2) construct a new lantern in a lighter material, and 3) strengthen the existing seismic bracing to accommodate rebuilding the lantern with the original material. Constructing a new lantern, or rebuilding the original, is not part of the 2002 work, but will be considered for future work phases.

Replacement of Deteriorated Material With Non-matching Materials – Many severely deteriorated terra cotta units were replaced with precast concrete in the early 1990s and earlier (Figure 4). This occurred at the bell tower mullions and at the stained glass window mullions. In each case, the precast concrete units matched the profile of the original terra cotta but not the terra cotta's color and texture. The replacement with a non-matching material did not by itself cause further deterioration. The overall appearance of the substitute material however, diminished the aesthetic integrity of the building features. Due to the lack of understanding of deterioration mechanisms, the precast concrete units failed (just as the terra cotta units before them had failed) because the underlying steel was not appropriately repaired (see below for further discussion). We repaired the corroding steel and replaced the precast concrete with new terra cotta that matched the profile, color and texture of the original terra cotta.

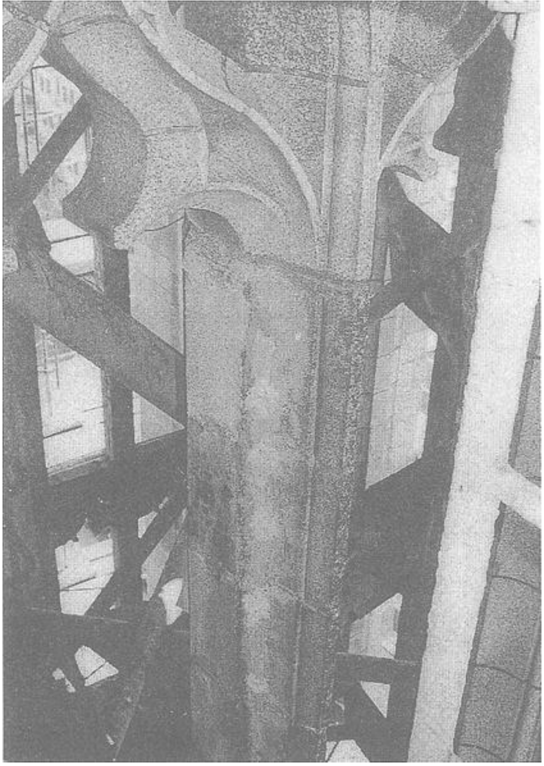

Figure 4 – *Precast concrete used as a replacement material at the bell tower mullions*

Introduction of Visual Elements That Diminish the Property's Historic Integrity - Unlike the lantern, smaller freestanding units survived the earthquake and demolition sentencing. The six-foot tall intermediate finials, cracked in several places, were reassembled in the early 1990s with stainless steel pins set in epoxy, and in some cases, further held together with tie wire (Figure 5). The cobbled-together finials were then anchored to new steel angle bracing which was partially embedded in a new concrete slab. The final outcome was a mixed success. Although some terra cotta was reused, the finials were not assembled with the original units. Parts from other finials (possibly from the lantern) were added, which changed the original design. While the steel angles provided adequate bracing, they were visible from the street. Some of the other means used to repair the finials resulted instead in further damage. For example, stainless steel pins used to connect the terra cotta to the bracing went through the face of the terra cotta, and thereby provided entry points for water infiltration that caused corrosion of the internal steel support. An expansive foam adhesive applied to the cracked surfaces eventually degraded, allowing the cycle of steel corrosion and cracking of the terra cotta to reoccur.

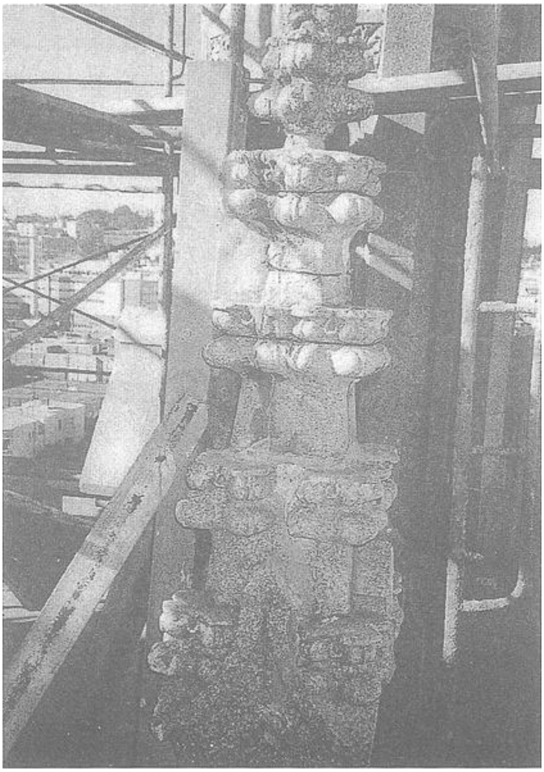

Figure 5 –*Intermediate finial repaired in the early 1990s*

We addressed these issues in 2002 by removing the bracing. We designed and located the new stainless steel bracing inside of the finial pieces and within the tower's structure (Figure 6). We replaced the finial assemblies with new terra cotta to match the original in design, color and texture to restore the original design intent since many of the original terra cotta units were cracked.

Failure to Address the Cause of the Problem - St. Dominic's had no records of when and how repairs were made prior to 1989. By the time we surveyed the bell tower features close-up in 2000, we noted that certain features had been repaired repeatedly but unsuccessfully. Repairs had been made to the bell tower and stained glass mullions, but this early work did not include repointing the mortar joints and repairing areas to prevent water infiltration. Repairs made in the early 1990s addressed the cracked precast concrete and severely cracked terra cotta mullions superficially by injecting epoxy into the cracks but failed to repair the corroded steel. As with the intermediate finials, the repairs lacked a remedy for water infiltration. As a result, the new and existing steel supporting the mullions continued to corrode.

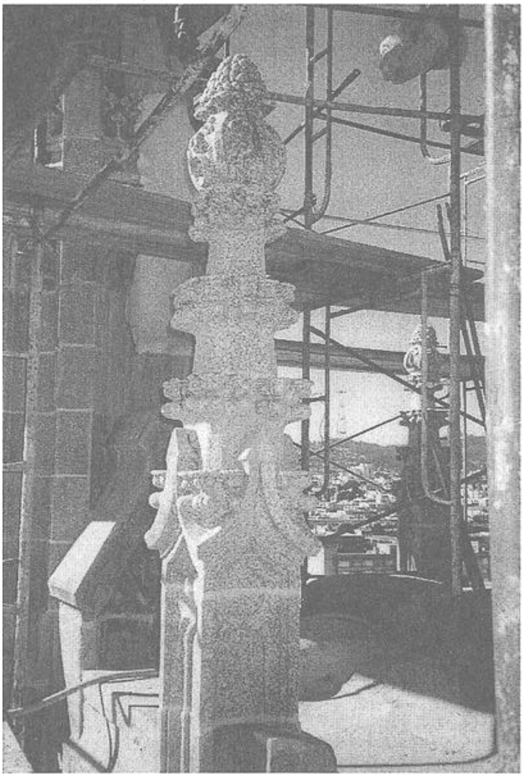

Figure 6 –*Intermediate finial as restored in 2002*

The failure to address the cause of the problem was particularly damaging at the stained glass window mullions, where several cracked mullions collapsed and jeopardized the historic windows (Figure 7). Fortunately, the fallen terra cotta landed on a roof and the stained glass panels remained in place. However, many other stained glass windows were located along walls closer to the streets and sidewalks. It was not only imperative to protect the public, repair deteriorated mullions and prevent future deterioration, but also to prevent loss of the valuable stained glass windows.

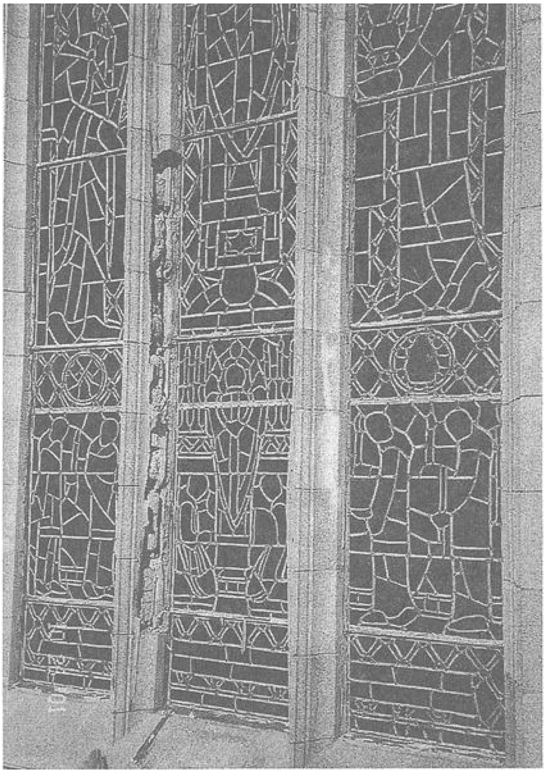

Figure 7 – *Collapsed Stained Glass Window Mullions*

The 2002 repairs included 100% repointing of the mortar joints and removal and replacement of the mullions with new matching terra cotta. The corroded steel was replaced with epoxy-coated rebar at the bell tower mullions and with stainless steel at the stained glass windows. Additional horizontal bracing was installed at the windows to account for wind and seismic loads. The stained glass windows were extensively documented, removed, restored and reinstalled.

The small finials were also repaired in 1989 by pinning together cracked and dislodged portions using stainless steel anchors set in epoxy (Figure 8). Although stainless steel is a durable material, the failure to remove the original corroding steel, the application of paint coatings on wide cracks (in an attempt to prevent water infiltration) and the lack of repointing all contributed to the ultimate loss of the finials. We repaired the small finials in 2002 by removing the corroded steel, designing and installing a new stainless steel support, and replacing the finials with new matching terra cotta.

All repairs designed and implemented in 2002 were based on the need to repair deteriorated and unsafe conditions and to preserve the original design intent of the existing structure. Even though the church was not subject to façade inspection or preservation ordinances, SGH followed a rigorous methodology that integrated the safety concerns of typical façade ordinances with established preservation philosophy.

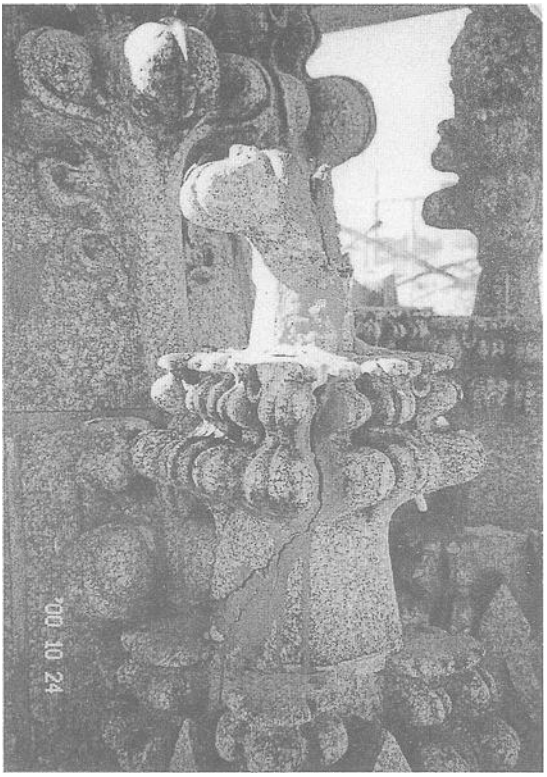

Figure 8 – *Small finial with the original corroded steel left in place during the early 1990s repairs*

We accomplished this by stabilizing unsafe deteriorated conditions; investigating and identifying the causes of the observed conditions; performing detailed documentation; researching original design and construction documents and maintenance histories; designing long-term repair solutions; repairing material before replacing; and replacing with traditional in-kind materials when repair was not feasible. This methodology has helped to preserve the original architectural design and building materials and to prolong the life of the Church building. Yet, this methodology alone is not enough. The support of the Church organization and its regard for the historic, architectural and communal value of the building have been instrumental in pursuing the most appropriate long-term preservation repair solutions. Together, the team is extending the life of the Church in the service of the community and in the interest of San Francisco's architectural heritage.

Conclusion

The inspection and care of historic structures requires diverse and specialized knowledge. This spectrum of knowledge is multi-disciplinary and integrates history, legislation, and a working knowledge of historic and contemporary building technologies and materials. In addition, the inspecting professional must have a clear understanding of deterioration conditions to recognize an unsafe condition.

Existing façade inspection ordinances do not differentiate between buildings of recognized historic and architectural value and buildings without recognized distinction. In this, inspection ordinances fail to fully address the particular requirements related to the assessment and treatment of historic structures. Existing landmark ordinances provide guidance for preserving and protecting the architectural and historic integrity of buildings. Landmark ordinances also acknowledge the primacy of public safety yet they do not provide clear direction for the expeditious removal and remedy of unsafe conditions. In this, landmark ordinances fail to provide sufficient protection to buildings from unnecessary and sometimes irreparable damage. These gaps need to be bridged.

Existing façade ordinances should be amended by expanding the language of the ordinances to accomplish the following:

- Differentiate buildings of recognized historic and architectural value from those without conferred distinction. This difference is recognized by existing legislation, and national, state and local registers of historic landmarks, buildings and districts.
- Direct the inspecting professional to determine the historic status of the building and neighborhood and identify its recognized historic value. The historic status and characteristics of significance may affect treatment recommendations.
- Direct the inspecting professional to established standards, guidelines and resources such as:
    - The *Secretary of the Interior's Standards for the Treatment of Historic Properties,*
    - State Historic Preservation Office (SHPO),
    - Local Landmark Commission or Advisory Board,
    - American Institute for the Conservation of Historic and Artistic Works *Code of Ethics* and *Guidelines for Practice,*
    - National Register of Historic Places,
    - Advisory Council on Historic Preservation - Section 106 Review Process, and
    - National Park Service Preservation Briefs.

Existing landmark ordinances should be amended to address specifically the issues surrounding emergency stabilization and repair. Amendments would include direction to:

- Document the as-found condition, the removal of original material, and all of the new repair interventions;
- Retain as much of the original material in situ as possible;
- Advocate stabilization in situ over unnecessary removal;
- Minimize damage to existing fabric during investigation and repair;
- Replace removed material once the investigation has been completed and a repair solution has been designed (in accordance with building codes and established guidelines for treatment – see above); and
- Preserve and protect the original historic significance and character of the building.

Additional landmark advisory commission resources and landmark ordinance amendments could provide:

- Timelines for redressing temporary repairs and implementing appropriate permanent repairs, which would prevent temporary solutions (such as the temporary removal of original material) from becoming permanent losses by default;
- Repair guidelines and methodology as outlined in established standards; and
- Comprehensive maps and lists of all identified historic districts and individual buildings with coded levels of significance for easy use by inspecting professionals. These interactive databases, such as could be created by Geographic Information Systems (GIS), would be of significant value during emergency situations such as post-earthquake surveys.

Collectively, these modifications would better integrate façade inspection ordinances with the existing body of preservation standards, guidelines and practices. They would provide clearer guidance for the inspecting professional when addressing façade inspection and repair of historic structures for the ultimate safe-keeping and long-term preservation of our shared historic architectural heritage.

[1] Feilden, B.M., *Conservation of Historic Buildings*, Butterworth-Heinemann, Woburn, MA, 1989 p.1.

[2] "National Register of Historic Places; Criteria for Evaluation." available from: www.cr.nps.gov/nr/listing.htm

[3] Section 106, sec. 800.5 (1)

[4] Section 106, sec. 800.5 (2)

[5] Kay D. Weeks and Anne E. Grimmer, *The Secretary of the Interior's Standards for the Treatment of Historic Properties*, 1995, pp. 1.

[6] Temko, Allan. 4 May 1987. "Inspired Rescue of a SF Landmark." *San Francisco Chronicle*

[7] op. cit., See Section 106, 800.5(a)(2).

Michael J. Scheffler[1] and Kenneth M. Itle[1]

New Methods for Designing Restoration Repairs for Historic Building Facades: A Case Study

Reference: Scheffler, M. J., and Itle, K. M., "**New Methods for Designing Restoration Repairs for Historic Building Facades: A Case Study,**" *Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444,* J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: This paper discusses an alternative method of approaching the design, implementation, and documentation of the restoration of monumental stone building facades, as performed at the Wisconsin State Capitol. During the initial design phases it was determined that the appropriate repair for many granite distress conditions could not be determined without close access to the building. Therefore, the State agreed to an approach where individual repairs could be designed by the A/E during construction as access became available. By combining digital photography and computer-aided drafting, it was possible to conduct a detailed survey, design the repairs, and provide repair sketches to the contractor as construction proceeded.

Keywords: Computer-aided Drafting (CAD), digital photography, granite, masonry restoration

Introduction

This paper describes an alternate method for approaching the design, implementation, and documentation of repairs to existing buildings and structures. This approach was used during the restoration of the monumental stone building facade at the Wisconsin State Capitol. This method relies on the extensive use of digital photography and computer-aided drafting to aid in the design of individual stone repairs on-site during construction.

This approach may be appropriate for a wide range of repairs and buildings where a fast turnaround is desired between repair design and implementation, and where precise documentation and cataloging of actual repairs is required. This approach allows for greater control over the quality and implementation of the repair project and minimizes misunderstandings over the scope and intent of the repairs. For the project cited here, the quick turnaround made possible by this new method allowed the design work and construction of repairs to proceed simultaneously without causing delay.

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Building Background

The Wisconsin State Capitol is a monumental neoclassical structure situated in the heart of Madison, Wisconsin, on a hill overlooking Lakes Mendota and Monona. The building is arranged in a Greek cross plan, with four equal, symmetrical wings projecting from a central, domed portion. A grand portico with elaborate sculpture in the pediments is at the end of each wing. Semicircular pavilions with grand staircases are located between the wings. The approaches to the building in the capitol park are defined by granite balustrades, staircases, curbs, benches, and planters. The building was designed by George B. Post and Sons of New York and was constructed from 1906 to 1917. The exterior walls are load-bearing masonry, although the granite-clad dome is supported on a steel frame. The interior floors are supported on steel beams that bear on the exterior masonry walls and interior steel columns.

The exterior of the building, from grade to the top of the dome, is entirely clad in Bethel white granite. The granite units are typically 200 mm to 300 mm (8 to 12 in.) thick, backed up by brick masonry. The average rectangular block has a face approximately 1 400 mm (55 in.) wide by 500 mm (20 in.) high. There are numerous projecting sills, corbels, balustrades, pediments, brackets, carved decorative panels, and sculptural groups across the facade. One part of the recently completed 14-year project to rehabilitate the entire interior and exterior was the restoration of the granite facade.

Based on grade-level surveys and close-up surveys at representative locations, the types of distress that required repair included cracks related to thermal stresses; spalled stone edges; potential spalls; surface deterioration likely caused by a previous acid cleaning; and unsightly or potentially unstable previous repairs using epoxy.

The masonry repairs that were implemented included dutchman repairs to rebuild lost stone profiles; installation of stainless steel helical pins set in grout to anchor cracked portions of stone; blending of broken edges to make small spalls less visually obtrusive; grouting of hairline cracks; and installation of sealant in cracks that experience movement due to thermal stresses. A dutchman repair consists of adding a new piece of like material to fill a gap or replace a missing portion of an original unit. Also included as part of the overall exterior restoration project was repointing of mortar joints and cleaning of stone in order to remove surface deterioration. This paper focuses on the stone masonry repair work.

Access for implementation of repairs included fixed scaffold, suspended scaffolding, and personnel lifts.

Documentation of Distress Conditions

The restoration process began with an initial overall survey of the entire exterior facade. This survey was visual only, performed from grade and flat roof areas using binoculars and a spotting scope. Obvious distress conditions, such as large spalls and cracks in the stone, were recorded on elevation drawings. For example, a large crack, such as that shown in Figure 1, was visible during the initial survey from grade. Unfortunately, other similar cracks, such as that shown in Figure 2, were in shadow or were obscured by general surface dirt. Preliminary quantities were developed from the conditions identified through this limited initial survey. Throughout the project, quantities were tabulated using spreadsheet software. The basic categories of distress were cracking, spall, incipient spall, and prior repair.

The results of the initial overall survey were used to select areas for more detailed study. These areas were selected as representative of the range of distress and architectural detail across the building facade, as well as areas that could be readily accessed using a personnel lift. The detailed or close-up survey covered approximately 10% of the total facade. Also as part of the detailed survey, the width of cracks in the stone masonry was monitored to a precision of 0.0025 mm (0.0001 in.) over a range of ambient temperature change. This monitoring confirmed that the cracking distress was primarily due to unaccommodated stresses from thermal expansion and contraction of the masonry, and that therefore the proposed repairs would be appropriate.

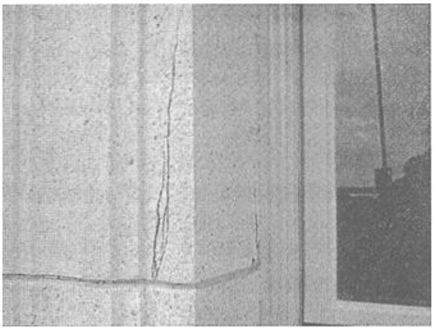


Figure 1 – *Crack in granite window jamb about 400 mm (16 in.) long. Visible from grade.*

Figure 2 – *Crack in granite window header about 500 mm (20 in.) long. Not visible from grade.*

The area of the facade where cracks shown in Figure 1 and Figure 2 were located was not part of the 10% that was surveyed in detail, so these repairs were not specifically called out on the bid documents. Thus, the actual total repair quantities required could not be determined until construction. Only repair quantity estimates could be calculated, based on the detailed survey. The repair quantities were extrapolated over the entire facade, using the preliminary survey as a guide. This new quantity estimate would form the basis of the quantities listed in the bid documents. In the bidding documents, the base bid quantity included 800 helical pins and 205 dutchman repairs, compared to 54 helical pins and 103 dutchman repairs which were specifically indicated on the drawings.

Pre-Design Phase

Because of the significance of the state capitol, the restoration goal was to implement high-quality, long-term repairs, while at the same time using the least intrusive repair methods to preserve as much historic fabric as possible. To guide future maintenance and record accurately current repair activities, the repairs needed to be thoroughly documented.

The precise nature and method of repair for most distress conditions on the facade of the Wisconsin State Capitol could not be determined without close-up, hands-on access to the repair area, which would not become available until actual construction. Although overall categories of distress had been made and repair quantities estimated, each distress condition was unique and required an individual determination of the best repair option. Also, existing dirt accumulation and an outer crust of exfoliated or unsound granite hid many hairline cracks or potential spalls.

Due to these considerations, the State and the A/E agreed to develop an approach where individual repairs were designed by the A/E during construction, as access became available, rather than using a traditional design-bid-build approach.

Design Phase

CAD baseline drawings were prepared based on the original construction documents, as verified by field measurements at selected locations. The repair drawing set included details of typical repair techniques, which were generalized from common distress conditions observed. These details included standard dutchman and helical pin repairs. In areas where the detailed survey had been conducted, the elevation drawings indicated locations where distress had been observed and the repair technique to be used; however, the repair quantities in the bid documents far exceeded the small number of specific repairs indicated on the elevation drawings. The majority of the repair work was thus determined during construction by the A/E. Bids were received on a lump sum basis for the base quantity of each repair type, with add/deduct unit prices also provided for each repair type.

For selecting among the various repair types during construction, a decision-making flow chart was developed. First the size and nature of the observed distress was categorized: either small or large cracks, spalled edges, potential spall with extensive cracking, or failed previous epoxy repair. The appropriate repair was then selected based upon the size, architectural location, or potential safety hazard of the distress. As described above, the types of repairs performed included dutchman repairs, helical pin repairs, blending of stone edges, or filling of cracks with grout or sealant. This flow chart was further refined by sample repairs, which were performed at grade level on the facade. This established practical limits for the size of a single dutchman or the minimum size of the portion of cracked stone that could be successfully pinned.

Construction Phase

A/E personnel conducted a detailed survey during construction, once access to an area of the building was available, by swingstage, fixed scaffolding, or personnel lift. The cleaning process, which was completed first, made the distress locations readily visible, since small hairline cracks or previous epoxy repairs were no longer obscured by dirt or staining. The contrast between cleaned and uncleaned areas of stone is shown in Figure 3. A/E personnel then inspected the cleaning work and performed a close-up survey for stone masonry distress. This survey work was primarily visual, but cracked areas of stone were also sounded with a hammer to identify potentially loose or delaminated areas. The A/E survey also identified the precise locations and extent of

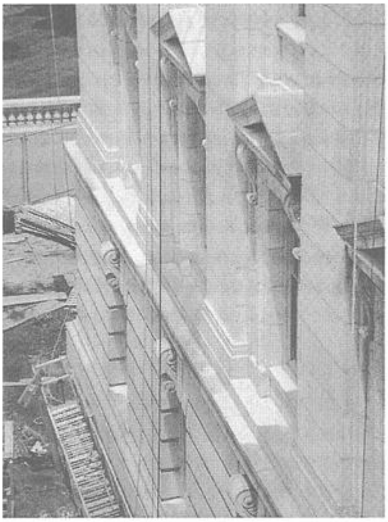


Figure 3 – *The contrast between cleaned and uncleaned areas*

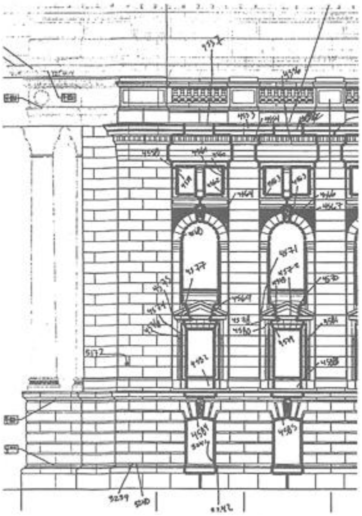


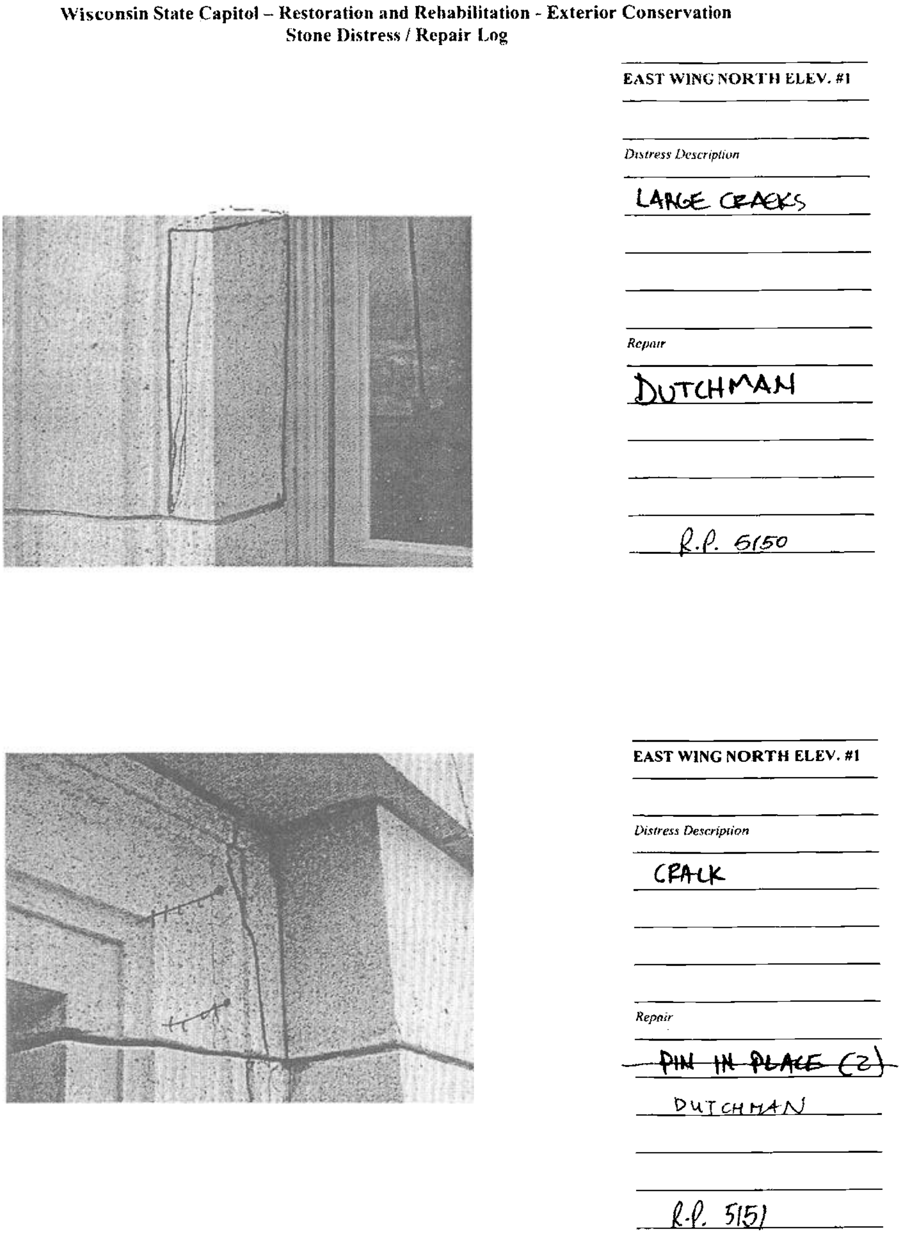
Figure 4 – *Typical partial elevation with distress locations noted*

repointing, and the location for the installation of sealant in joints to accommodate movement. Pointing and sealant work was indicated to workmen with chalk and tape on the face of the building.

Each stone distress location was digitally photographed. The exact location of each photograph was shown on a partial elevation drawing; the file number of the digital photograph automatically assigned by the digital camera was used as the numbering system for tracking repairs. The distress locations noted during the original surveys were also digitally photographed, and locations and types of repairs confirmed. At a swingstage drop on the north elevation of the east wing, approximately 40 individual locations of distress in the stone work were identified in this 220 m^2 (2 400 ft^2) area of the facade. This was typical for the project. Figure 4 shows the partial elevation drawing for this area; Figure 1 and Figure 2 are two of the distress photographs taken in this area.

While the contractor proceeded with remaining cleaning work and with mortar removal prior to repointing, the repair design sheets were prepared by the A/E. This process began when the image from the digital camera was downloaded into a laptop computer, on site. The digital photographs were printed, two per page, as shown in Figure 5. The needed repair was illustrated by hand on each photograph. The repair was based upon the decision-making flow chart described above. The repair sheet highlighted cracks and other distress, the extent of dutchman repairs, or the location and number of helical anchors to be installed. A typical repair sketch sheet is shown in Figure 5.

Some locations of stone distress, such as repair 4559, were designated to receive dutchman repairs, due to the unsoundness of the cracked portion of the stone. At other locations, such as repair 4560, the stone appeared to be generally in sound condition



Wisconsin State Capitol – Restoration and Rehabilitation - Exterior Conservation
Stone Distress / Repair Log

EAST WING NORTH ELEV. #1

Distress Description

LARGE CRACKS

Repair

DUTCHMAN

R.P. 5150

EAST WING NORTH ELEV. #1

Distress Description

CRACK

Repair

~~PIN IN PLACE (2)~~

DUTCHMAN

R.P. 5151

Figure 5 – *Repair sketch sheet for 4559 and 4560 on the north elevation of the east wing*

despite being cracked. These conditions were repaired with stainless steel helical pins. The repair details illustrated on the construction documents were used as a basis for the actual repairs shown on the repair design sheet.

Because of the goal to preserve as much historic fabric as possible, many repairs were designed with instructions that the contractor attempt a pin-in-place repair first. A dutchman repair was an acceptable alternative, in cases where the cracking was too extensive for pinning to be implemented successfully.

A copy of the repair design sheet (Figure 5) was provided to the contractor, along with the overview elevation indicating where each photograph was located (Figure 4). The contractor then proceeded to implement the stone repairs. The dutchman repairs occasionally required further discussion with the A/E; for example, when cracks penetrated into the stone in a different angle than originally anticipated. At times the stone proved to be less stable than expected when attempting a helical pin repair, and broke during the drilling of the pin. The contractor notified the A/E, and a dutchman repair was usually substituted. For example, the cracked portion of the stone at repair 4560 had been held in place only by the intact mortar joint; once the mortar was removed for repointing, it was noted that the stone was actually significantly deteriorated. The A/E quickly determined that a dutchman repair was therefore appropriate.

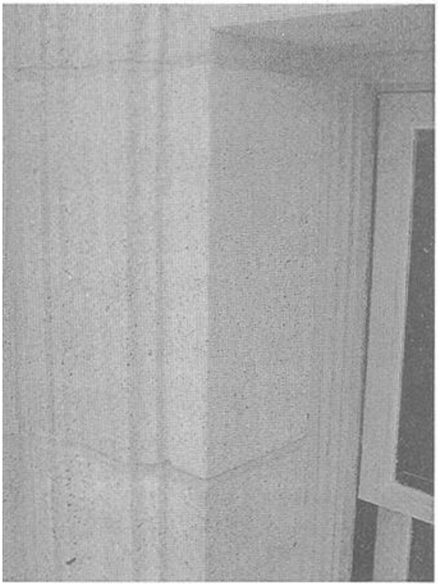


Figure 6 – *Repair photograph 5150*

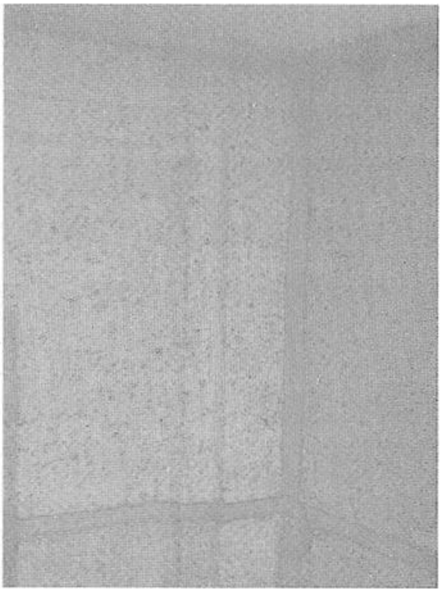


Figure 7 – *Repair photograph 5151*

After completing the repointing, sealant, and stone repairs in an area, the A/E returned to re-photograph the repaired condition from the swingstage or scaffolding. To track the repairs, the repair photograph number was noted on the design sheet, as were any changes made during the work, as shown in Figure 5. The dutchman repair 4559 was re-photographed in photo 5150, provided as Figure 6. Repair 4560 was amended to indicate a dutchman repair and documented in photo 5151, provided as Figure 7. At that time all of the stone repairs were either determined to be complete or identified as requiring additional work. The annotated repair sheets served as the punch list for the project. Once all of the stone repair work and joint repointing had been completed and inspected by the A/E, the contractor could move the swingstage or scaffolding to the next area of the building facade.

The preliminary quantities based upon limited close-up survey and extrapolation over the entire facade proved to be relatively accurate in determining the extent of distress and the project's total cost. Compared to the estimate of 800 helical pins for the base bid, 587 pins were actually installed during construction. Compared to the estimate of 205 dutchman repairs for the base bid, 263 dutchman repairs were actually installed. With the unit prices provided by the contractor as part of the bid, the costs for the added dutchman repairs offset the credit for fewer pins installed, leaving the total project on budget.

Documentation Phase

At the completion of the project, as-built drawings were prepared using CAD. Each repair location was identified with a tag containing four pieces of information: distressed photograph number, repaired photograph number, category of distress, (i.e., crack, spall, potential spall, exfoliation, or previous repair) and category of repair (i.e., helical pin, dutchman, blend edges, grout crack, or seal crack). For example, a portion of the completed as-built drawing for the north side of the east wing is shown in Figure 8. Additional measurements taken during construction were used to correct minor discrepancies in the baseline elevations. This archival documentation was provided to the state archives for future reference, consisting of the as-built CAD drawings printed and on CD-ROM; printouts of each photograph in numerical order and on CD-ROM; the original repair design sheets, organized by building wing and elevation; and daily field reports. The use of the digital photograph file number as the basis for numbering repairs results in all of this information being easily cross-referenced.

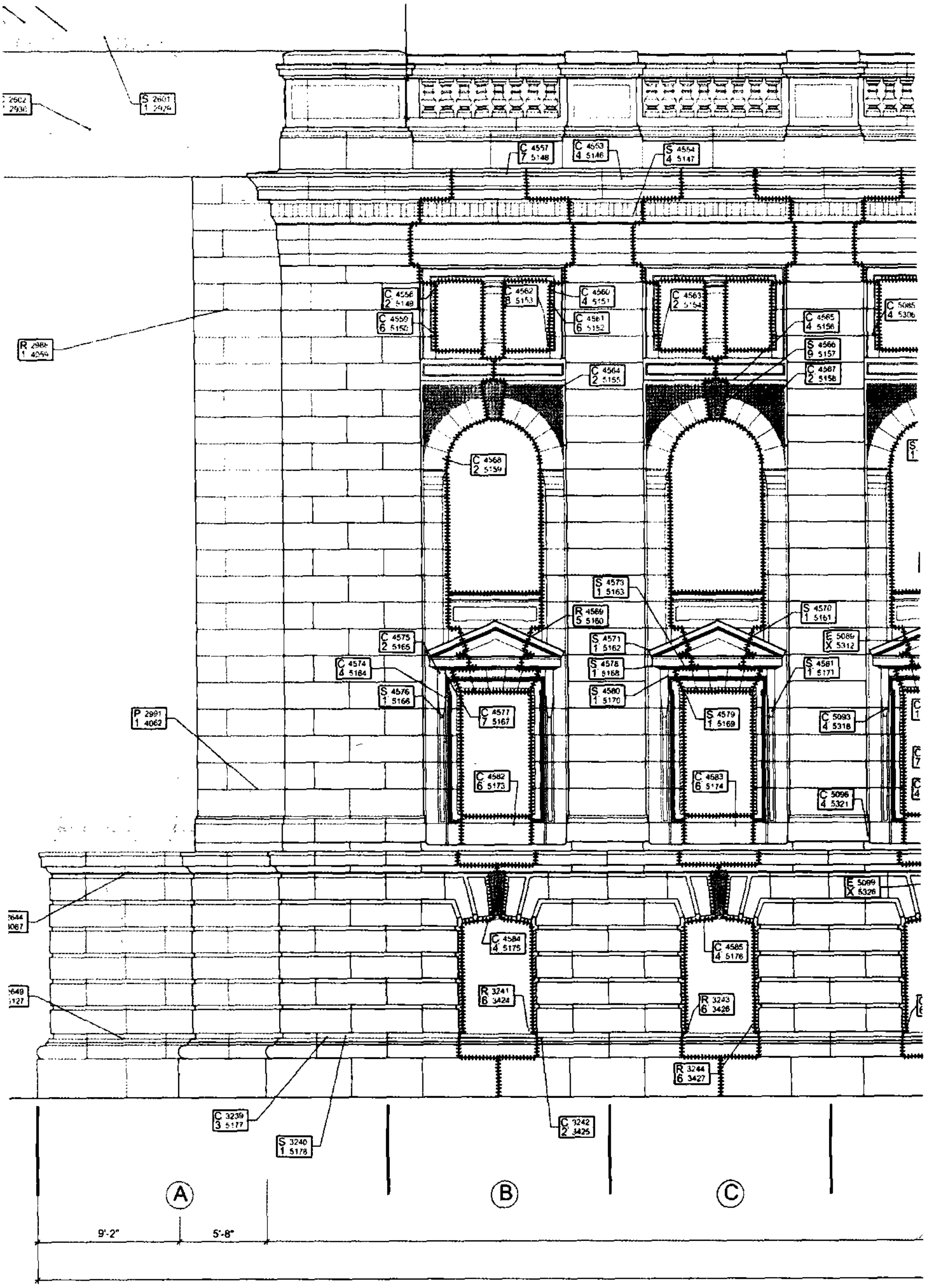


Figure 8 – *As-built drawing for east wing, north elevation*

Conclusion

The on-site design approach pursued for this project fostered a close, collaborative relationship between the A/E and contractor. As the project progressed, the level of craftsmanship among the stone masons increased, and the A/E developed a detailed understanding of the most practical and efficient ways to detail and implement the proposed repairs. This resulted in minimizing the implementation of inappropriate repairs or repairs that would be unnecessarily intrusive to the historic fabric.

This approach required the use of contemporary digital photography technology. With over 5 000 photographs taken over the course of the 1½-year project, the physical logistics of film, developing, collating negatives, tracking prints, etc. would have quickly proved impossible. The digital camera allowed fast turnaround and proved far easier to coordinate and organize as the project progressed. Specific photographs could instantly be retrieved months later, and additional copies could be generated at any moment. In as little as one hour, an unforeseen distress condition could be identified and photographed, the repair could be designed, and a detailed repair sketch could be provided to the contractor for implementation.

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Terra Cotta Facades

Reference: Hoigard, K. R., Mulholland, G. R., and Haukohl, R. C., **"Terra Cotta Facades,"** *Symposium on Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444,* J. L. Erdlv and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: The late nineteenth century saw the rise of terra cotta as a popular building exterior facade material by offering the general appearance of carved stone at a substantial cost saving. Available in a variety of colors and standard shapes, architects could develop complex patterns and designs with off-the-shelf materials supplemented by special pieces fabricated in any shape desired. By the 1950s, changing architectural styles and rising labor costs brought the golden age of terra cotta use to a close.

Many vintage terra cotta-clad buildings have survived the wrecking ball and are today prized for their stylish appearance. Notable survivors include the Wrigley Building and the Reliance Building in Chicago, and the Union Trust Building in Pittsburgh. Architects, engineers, and contractors tasked with the evaluation, maintenance, and repair of these buildings are frequently faced with a daunting task. Failures of terra cotta facades, in some cases resulting in large pieces falling to the street, have been reported in the newspapers of large and small cities alike. The underlying problems can frequently be traced back to long-term deterioration related to original design misconceptions, and ongoing misunderstandings regarding the behavior and construction of terra cotta facades.

This paper will review the history of terra cotta as a facade material. The authors will present information regarding historic terra cotta fabrication and erection detaus, and their manifestation as facade problems today. Sources will include vintage literature on terra cotta construction, as well as the experience gained by the authors while investigating and repairing terra cotta facades.

Keywords: terra cotta, facade, anchor, spall, crack, cladding, deterioration

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History

Terra cotta has been in regular use in the United States since the mid-1850s. Unglazed terra cotta was first used in load-bearing masonry wall construction where complex anchoring systems were not necessary. In the late 1800s, terra cotta units began to be used as fireproofing to encase metal frame constructions [1,2]. The first uses of terra cotta as a facade material were as decorative trim, and this eventually evolved into its use as a complete facade material. Its popularity coincided with the rise of the American skyscraper around the turn of the century. Terra cotta was promoted as durable, impervious to water, and adaptable, which, for the most part, it was. It became popular due to several factors:

- Terra cotta was available in a wide range of colors and textures, and could be produced to imitate the appearance of popular natural stone facades, such as granite, limestone, brownstone, or marble. The resemblance of terra cotta to the natural stones it imitated was so good that the lower two or three floors of a high rise could be constructed of natural stone, while the remaining upper floors could be constructed of terra cotta, without any obvious distinction in appearance. Many building owners are surprised to discover that what they thought was a granite or stone building is actually terra cotta.

- Terra cotta was more economical to produce than natural stone, especially when it came to ornamentation and detail. An extraordinary range of architectural detail and ornamentation was possible with terra cotta, limited only by the ability to create a suitable mold. The cost of molding the clay, and then glazing and firing the blocks, represented a considerable savings compared to carved stone. This was especially true when shapes were repeatedly used in a modular fashion.

- Compared to dimension stone, terra cotta was easier to handle and more quickly set, making it more affordable to install.

- Terra cotta was thought to be relatively durable and permanent, with excellent weathering properties due to the hard surface of the glaze. Thought to be fireproof and waterproof, it was readily adaptable to structures of almost any height.

- Maintenance of the fired and glazed surface was easy; it never needed paint, and periodic washing restored its original appearance.

After World War II, terra cotta fell into disfavor. With the cost of labor rising, it was considered to be too labor intensive to justify its cost. It was also considered to be too ornate to fit changing architectural styles. Terra cotta has been used rather sparingly since. As a testament to terra cotta's durability, many vintage terra cotta buildings are still standing today, most of which are between 60 and 100 years old. Most will continue to be used for many years to come, so long as they are properly maintained and not allowed to deteriorate.

Unfortunately, many terra cotta facades have fallen into disrepair. Original misconceptions regarding the nature and limitations of the material, the behavior of high rise buildings, and lack of adequate maintenance and repair programs, have led to severe deterioration of many terra cotta facades. Numerous incidents of terra cotta pieces falling to the street below have been reported. Building inspection ordinances have been passed in many major cities requiring routine inspections of building facades. As a result, repair programs for terra cotta buildings are now common. This has caused a demand for knowledge of terra cotta systems, much of which has been lost in the last 50 years [1,3].

Material

Terra cotta essentially consists of enriched, molded clay. The term "terra cotta" can be translated from either Latin or Italian as "cooked earth". It can vary widely in color, from reds and browns to whites, depending on the particular blend of clay colors and types used. While its material characteristics are essentially the same as brick, some aspects of terra cotta differ substantially. In particular, the manufacture of terra cotta requires a much higher grade of clay. Partial "vitrification", or "fusion to make glass-like," occurs in the manufacture of terra cotta, which enhances the durability of the body. To achieve this, different materials are added to the original clay which fuse the body into a harder consistency. These ingredients usually range from pure white sand to old pottery and fire bricks finely pulverized, to clay previously burned, otherwise known as "grog" [3,4].

Manufacture

Terra cotta units that are unique, usually highly ornamental pieces, were typically shaped by hand. More often, however, many of the same type of piece would be needed for a particular project, and a mold for the piece would be made. Geometrically simple units could also be extruded from a machine when production demanded it. Glazed terra cotta units for facade applications were usually hollow cast in blocks open at the back, with internal compartments separated by stiffeners, known as "webbing". Webbing significantly strengthens the terra cotta unit without substantially increasing its weight.

The terra cotta units are then fired or "burned" in a kiln for several days. The outer glaze can be accomplished by either applying a "slip glaze," or clay wash; or by brushing or spraying on an aqueous solution of metal salts to the air-dried block before firing. Some metal salts effloresce on the surface of the clay during burning as well. The salts act with the silicates on the surface of the clay and vitrify to a greater degree than within the interior of the unit. This forms the hard glaze on the surface of the terra cotta unit [4,5].

Types

Four general types of terra cotta have been used for American buildings [1]:

Brownstone

Brownstone terra cotta was used primarily in the mid to late 1800s, and was the earliest type of terra cotta used on American buildings. It was hollow cast, unglazed or slip-glazed, and was typically dark red to brown in color. It was most often used in combination with other masonry, or to simulate sandstone, brick, or actual brownstone.

Fireproof Construction

As the construction of high rises increased in America, so did the demand for means to fireproof steel structural members. For a period of time around the turn of the century, unglazed terra cotta blocks were used for "fireproof construction." Unglazed terra cotta was inexpensive, lightweight, and fireproof, and could be easily fit onto structural members. Although an ideal solution for the time, fireproof construction using terra cotta is no longer practiced today.

Ceramic Veneer

Ceramic veneer consists of glazed ceramic tile attached to a backup substrate of clay tile or a grid of metal ties. Ceramic veneer units were not hollow cast, but were solid cast and ribbed on the back. The ribbing was cast in order to back-parge the units onto the substrate. This system was developed during the 1930s and is still used in building construction today.

Glazed Architectural

Glazed architectural terra cotta is the primary focus of this paper, and is what most people think of when they hear the term "terra cotta". It is by far the most prevalent type used for facade work, and was developed and refined throughout the late nineteenth and early twentieth century. Glazed architectural terra cotta units are characterized by being molded or carved with a hollow interior and internal webbing, and being heavily glazed, often in imitation of natural stone.

Construction Details

Terra cotta has historically been used in two different ways, depending on the building system. Early in its history it was used as a part of traditional load-bearing masonry walls in low rise buildings. Later, it was used as a cladding material in high rise buildings. When used with traditional load-bearing masonry walls, the terra cotta units themselves are load-bearing, and do not require a separate anchoring system. When used as cladding material, however, the terra cotta units are not load-bearing, and require anchorage to the building framing system. Generally, it follows that the more complex or elaborate the terra cotta facade, the more complex or elaborate the anchorage system needs to be. This paper deals primarily with the use of terra cotta as a cladding material.

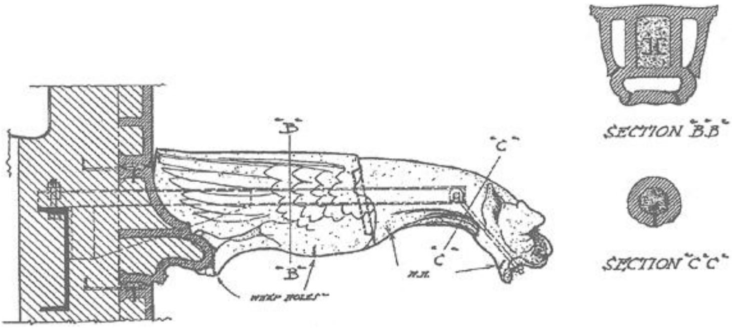

Figure 1 - *Section View of a Gargoyle Ornament [6]*

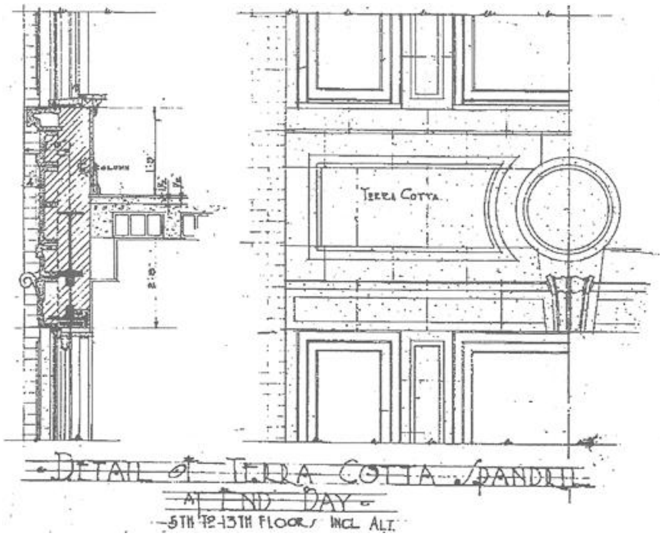

Figure 2 - *Typical Spandrel Detail*

With the many different terra cotta shapes and configurations that were available, it is surprising that there was some degree of standardization among anchorage details. Typical terra cotta units bear

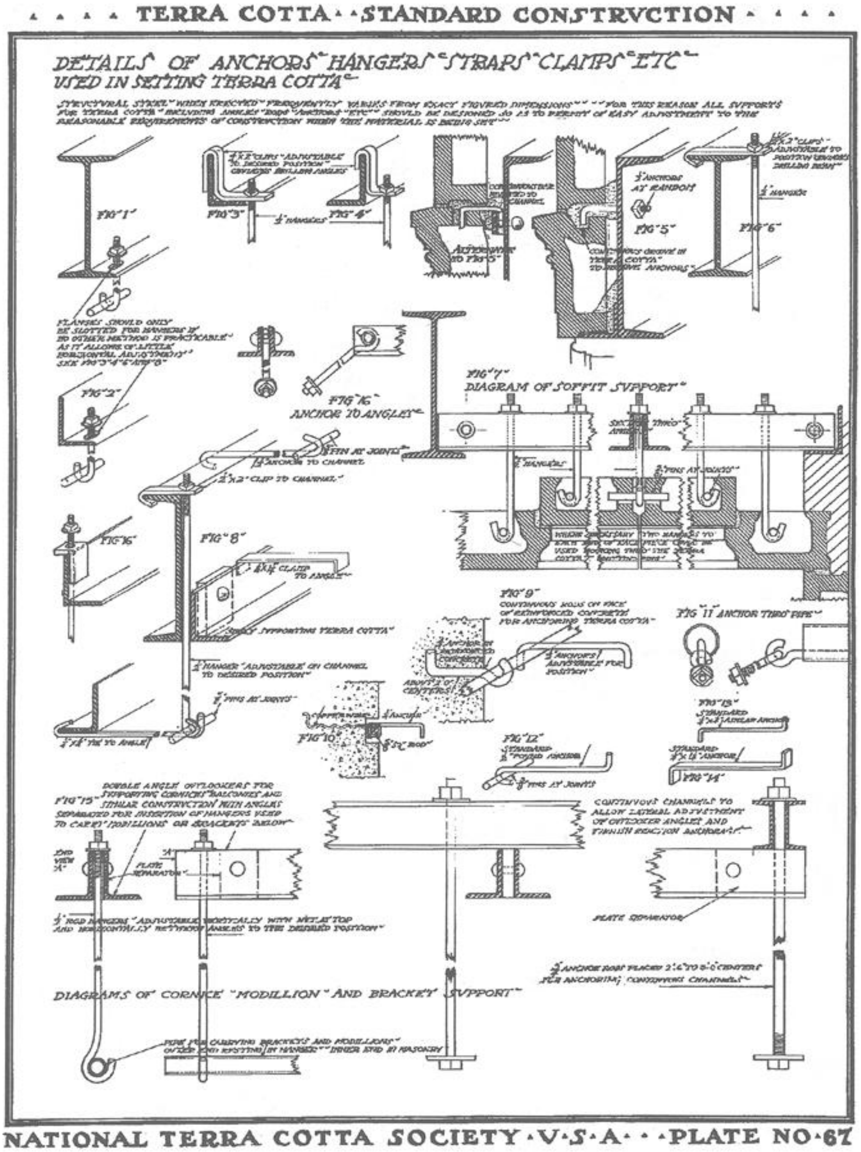

Figure 3 - *Typical Terra Cotta Anchorage Details [6]*

directly on either a steel section or the terra cotta unit below it. They are held in place laterally by steel rods or square stock which hook into holes in the webbing or flanges, and connect to the steel framing members behind the unit. For larger units, horizontal steel rods were inserted through holes in the webbing, and these rods were engaged by steel J-hooks which were bolted to the framing members. For overhead or "hanging" units, such as brackets or modillions beneath cornices, terra cotta generally relies on J-hooks for both vertical and horizontal support. The J-hooks are attached to ancillary steel members that project out from the main building framing. Figures 1 and 2 show typical construction details. Figure 3 shows typical anchorage details [6].

Design Misconceptions

The increase in the use of terra cotta paralleled the increase in the construction of high rise buildings. When these buildings were constructed, the behavior of large buildings and of the terra cotta material itself were not well understood. This led to a number of built-in design problems, such as the omission of flashing, weep holes, and expansion joints. Designs were based on a number of misconceptions:

Misconception #1: Terra cotta glazing is impervious to liquid water

Historically, terra cotta units have been seen as highly waterproof due to their hard and relatively durable glazing. Therefore, measures to handle water entry and water absorption by the terra cotta units, namely flashing, weep holes, and drip edges, were thought to be unnecessary. Although the glazing is relatively impermeable when compared with the surfaces of ordinary masonry or natural stone, it is not a complete moisture barrier. Also, glaze cracking, or "crazing", which is a common occurrence, may breech the glazing and allow moisture to penetrate to the underlying material [3].

Misconception #2: Terra cotta facades are weatherproof

The original designers often did not have a thorough understanding of how the terra cotta units were to perform as a part of the facade system, or as a part of the "building envelope" as a whole. Sometimes only minimal, poorly maintained measures were taken against water entry through the facade system and through interfaces with adjacent building components. Mortar joints, roofing, window and door penetrations, building projections, and other discontinuities in the facade system are all vulnerable points for water entry. At the time of construction, labor was relatively inexpensive. Routine building maintenance, such as tuckpointing, could be performed frequently. When mortar joints or other adjoining components fail, water-related deterioration frequently results. It is common to find terra cotta distress in close proximity to discontinuities in the facade system where sealants or flashings have deteriorated.

Misconception #3: Thermal and moisture expansion not considered

Most terra cotta buildings were built before the concepts of thermal and moisture related expansion and contraction of materials, or "volume change," were fully appreciated. All materials undergo some degree of volume change due to changes in temperature. Some materials, such as concrete, will shrink over time. Clay products, including terra cotta, undergo expansion and contraction due to moisture absorption or release. Different materials may expand and contract at different rates. A terra cotta facade will typically expand and contract to a greater degree than its underlying substrate or structural frame. This effect is relatively negligible for smaller structures, but the cumulative effect on larger, contiguous structures becomes significant. This was not well understood at the time, and therefore many vintage buildings were built without the benefit of stress-relieving details such as expansion joints [2,3].

Distress

A terra cotta facade system, as has been shown, is extremely complex and intricate when compared to other facade types. The mechanisms by which deterioration and failure occur are, likewise, just as complex. However, water is at the root of most deterioration. Whether by its own behavior, by the reactions of building components in its presence, or by its transport of other materials, water is the key element for most deterioration mechanisms. The amount of building component deterioration that takes place is typically in direct proportion to its exposure to water. The entry of water into the facade system needs to be prevented, but at the same time, water that enters the system needs to be allowed to exit. Once deterioration starts to occur, water typically enters the system at an increasing rate, accelerating the process [3].

The following is a list of specific types of distress commonly found on terra cotta clad buildings. It should be noted that, in the authors experience, several different types of distress may be present at the same time, and can have the same or different sources.

Glaze Cracking, or "Crazing"

Glaze cracking, or "crazing," as shown in Figure 4, is characterized by small, numerous, randomly-oriented cracks in the glaze material. It is both a normal and a common phenomenon, and is caused by a difference in material properties between the glaze and the body of the terra cotta unit. The terra cotta unit is in its driest state, and hence its smallest state, immediately after firing. Over time, the body of the terra cotta unit will expand as it absorbs moisture from the air and other sources. This puts the glaze layer into tension, and being a hard, brittle material, it frequently cannot accommodate the expansion of the main body. Under these conditions, the glaze will crack. Crazing in and of itself is not a serious failure, but it can lead to increased water absorption by the terra cotta unit, especially if the cracks penetrate the full depth of the glazing. Increased water absorption by the terra cotta can lead to some of the more severe problems discussed below.

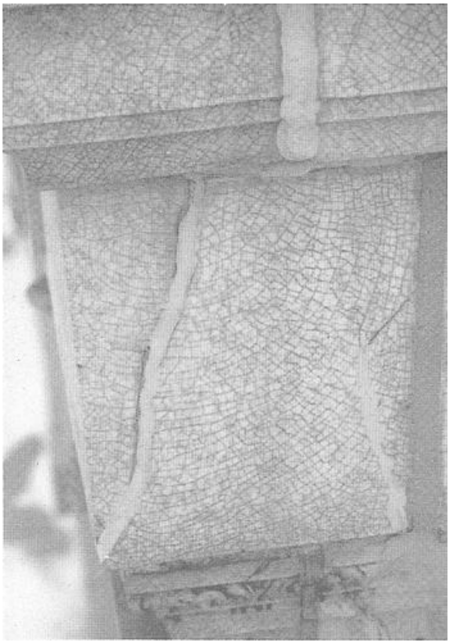

Figure 4 - *Glaze Cracking*

Environmental Attack

Terra cotta glazes are generally durable and have excellent weathering properties, providing a good barrier against most airborne and waterborne environmental chemicals, and resisting the adherence of dirt. Environmental chemicals will typically attack the mortar in the joints more vigorously than the terra cotta glaze, causing the deterioration of the mortar and water infiltration via the joints. Over long periods of time, however, chemical and biological elements can wear away the durability of the facade. Some slip glazes, especially those with relatively high permeability or those with rough-textured surfaces, are especially vulnerable. Long-term exposure to acid rain found in industrial climates can cause a dulling of the glaze [5].

Glaze Spalling

Glaze spalling, as shown in Figure 5, typically first appears as small blisters in the glaze material. The glaze then ruptures and exposes the underlying material. The phenomenon can occur as several individual spalls on the surface of a unit, or as the spalling of the entire glazed surface. There are three main mechanisms by which glaze spalling may occur:

- With typical masonry systems, trapped moisture tends to migrate outward through the masonry units where it will eventually evaporate. Glazed terra cotta, however, has an exterior surface that is relatively impermeable to moisture. Moisture can become trapped behind the glaze, and since water expands when it freezes, this expansion puts pressure on the glaze and on the surrounding terra cotta. This pressure can dislodge sections of the glaze.

- Within all masonry systems, migrating water can pick up and carry water-soluble salts with it. If these salts are deposited on the surface of the masonry, the phenomenon is known as "efflorescence." If the terra cotta glaze is compromised by crazing, or if it is slightly more permeable, moisture may evaporate, but the water-soluble salts are deposited within the boundary layer underneath the glaze. This phenomenon is known as "subflorescence," and the buildup of salt deposits can exert pressure that eventually dislodges sections of the glaze.

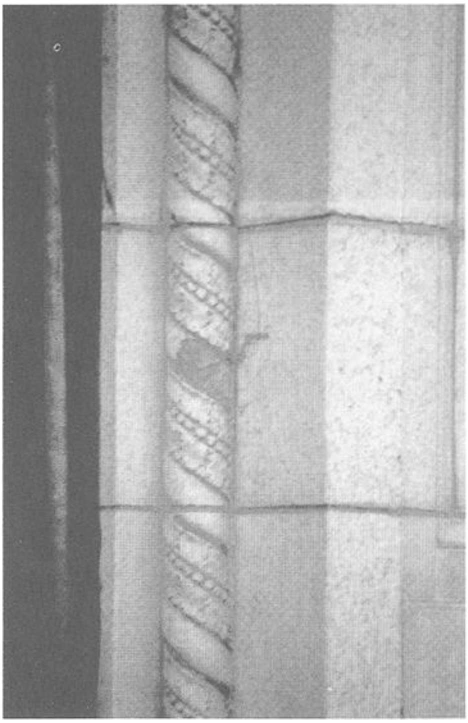

Figure 5 - *Glaze Spall*

- Biological elements, as well, can build up on the boundary layer underneath the glaze. They thrive in porous, moist environments, and if a glaze permits some transmission of sunlight, they will grow and spread. This also generates interior pressure which eventually dislodges sections of the glaze. Glaze spalling from biological growth is commonly found adjacent to mortar joints or previously spalled areas. Biological elements carried with the water can also be deposited on the surface, and their survival depends on the environmental conditions.

Once glaze spalling has started, the amount of water entering the exposed body of the terra cotta increases due to the exposure of the porous body to the elements. The increased water entry accelerates the deterioration of the cladding system [1,2,5].

Mortar Deterioration

The deterioration of mortar joints, as shown in Figure 6 in the form of bond failure, cracking, or erosion, plays a significant role in the deterioration of the overall terra cotta system. Terra cotta facade systems need to "breathe" similar to normal masonry systems, and rely more heavily on the mortar joints for this purpose due to the relative impermeability of the glazed terra cotta units. Deteriorated mortar joints are a primary source of water infiltration into the system, which in turn leads to many of the other problems discussed in this section. The deterioration can be caused by a variety of different mechanisms, ranging from deficiencies in the original mortar installation or mortar mix, to erosion from weathering and attack by air and waterborne chemicals or biological elements. Mortar deterioration is common and is to be expected over time.

Figure 6 - *Mortar Deterioration*

Pointing of mortar joints should be a regular maintenance item for building owners. Lack of ongoing maintenance, combined with long-term exposure to the weather, are mainly responsible for mortar deterioration.

Material Spalling

Material spalling, as shown in Figure 7, is a general category of distress that may have any one of a number of causes. Simply put, it is the loss of portions of the body of the terra cotta unit itself. It is unsightly at a minimum, but it also creates falling hazards and exposes the remainder of the unit, the adjacent units, the metal anchoring system, and other building components, to increased water entry and accelerated deterioration. Material spalling is typically the outward manifestation of other failure mechanisms such as anchor deterioration or compressive stresses from volume change. Repairs are typically difficult and can be very expensive. Nevertheless, it is a phenomenon that must be dealt with as soon as possible, as the resulting problems tend to accelerate.

Material spalling will typically start out as some form of internal or external cracking of the body of the unit, or possibly a delamination of the face. At this stage, the problem can usually be detected by a visual survey or by sounding, and may be dealt with before a spall occurs. Over time, the distress becomes severe enough to dislodge a portion of the unit. As stated above, there are a number of possible causes for material spalling, and each are discussed further on in this section.

Figure 7 - *Material Spall*

Deterioration of Structural Components

Deterioration of underlying structural components supporting the terra cotta units is perhaps the most difficult form of facade deterioration to locate and diagnose. It can also cause the most catastrophic type of failure, as it can undermine the structural integrity of large sections of the facade. In its early stages, outward manifestations of this type of distress may consist of slight displacement of the terra cotta units and bond separation at the mortar joints. This displacement may not be evident upon close-up examination, and observation from afar and from different points of view may be necessary in order to identify large scale facade shifts or bulges. Also, if mortar joints are repointed over time, movement of the terra cotta units can be disguised, appearing as abnormally wide joints. Often, the distress does not become evident until severe cracking and spalling occurs.

This deterioration is primarily due to water that has infiltrated into the system coming in direct contact with the structural components and causing corrosion. Iron or steel hooks and rods near deteriorated mortar joints are especially vulnerable. Corrosion weakens, and in some cases, completely disintegrates metal components, causing the loss of stability of the terra cotta units, as shown in Figure 8.

The very formation of corrosion by-product is a problem in and of itself. Iron or steel components, in contact with water, corrode and form iron oxides. Iron oxides occupy five to ten times the volume of the original iron or steel. This expansive formation of the oxides puts a tremendous amount of pressure on the adjacent terra cotta. This is especially prevalent on shelf angles and lintels, where water can pool and cause corrosion of the steel. The horizontal legs of the angles are usually embedded within mortar joints between terra cotta units, or within C-shaped cavities within the units. The creation of corrosion scale on the horizontal legs of the angles can apply pressure in both the vertical and outward directions. Outward pressure can cause a crack or spall to form within the unit or mortar joint along the toe of the angle, or displace the unit laterally.

Figure 8 - *Corrosion Failure*

Vertical pressure can cause compressive stress to build up between adjacent terra cotta units, resulting in displacement of the units, and cracking and spalling along the unit boundaries.

Compressive / Tensile Stress

A building's structural frame behaves quite differently than the terra cotta facade enclosing it. They are made of two entirely different types of materials, and the frame is not subjected to the same level of thermal and moisture exposure. Temperature changes and moisture absorption lead to expansion and contraction of the terra cotta units, also known as volume change. For shorter buildings, in height or length, the resulting stresses are typically minimal. For taller or longer buildings, however, the consequences are more critical. Structural frames, especially concrete frames, will shorten vertically during the building's lifetime. For concrete frames, most of this shortening will occur early in the building's lifetime due to normal concrete shrinkage, and further shortening can occur over time due to creep. Dead load frame shortening can occur within steel framed buildings early in the building's lifetime.

The terra cotta facade of the building, however, being exposed to the elements, will undergo cyclic volume change to accommodate changes in temperature and moisture. The result is the creation of stress within the terra cotta units, and between the terra cotta and the substrate or structural framing. This can lead to widespread cracking and spalling of the terra cotta facade, and crushing and buckling along the edges of the units. Many terra cotta buildings were built without horizontal or vertical expansion joints to relieve these stresses.

Problems due to volume change of the terra cotta are especially problematic around shelf angles. Terra cotta units that undergo volume change in the vertical direction can cause the terra cotta to partially or completely disengage from the shelf angle. A phenomenon known as "stacking" can also occur, where the gravity load from the units above the shelf angle partially or completely bypass the shelf angle. The units will "stack" up on top of each other, causing crushing and buckling of the units below. In the extreme case, cyclic vertical stresses in conjunction with other stresses can cause the terra cotta to "walk" off the shelf angle.

Volume change in the horizontal direction can cause terra cotta units to crack, buckle, and spall as well. Horizontal stress can cause vertical or step cracking near building corners, and may also cause corner units to be pushed outward. This phenomenon is exacerbated when shelf angles are incomplete around corners, as the combined effects of horizontal stress, shear stresses between the supported and unsupported wall sections, and vertical stacking of the units, cause severe cracking and spalling.

Previous Inappropriate Repairs

Some building owners with terra cotta facades have been able to support excellent, long-term maintenance and repair programs through the years. Unfortunately, repairs to terra cotta facades have frequently included a multitude of inappropriate measures. "Inappropriate" repairs are not limited to aesthetically unpleasant measures. They are

Figure 9 - *Inappropriate Repairs*

more broadly defined as measures taken to maintain or repair a terra cotta facade that attempt to alleviate the immediate symptoms of a problem, but do nothing to correct the underlying problem or distress. Some of these attempts may, in fact, cause more harm than good. Many building owners are either misinformed or constrained by budgetary requirements, and make unfortunate decisions regarding the maintenance of the building's facade. Repair attempts are often made by contractors or building maintenance personnel who have little or no understanding of the behavior of the facade, nor of the consequences of the applied repair measures.

By far the most prevalent inappropriate repair measure encountered by the authors is the application of sealants of various types to cracks and mortar joints. This "caulk and walk" approach, shown in Figure 9, is typically performed in an attempt to prevent water leakage into the building interior. Cracks in the terra cotta units are often indicative of more severe structural problems, and the simple application of sealant to the cracks ignores the root causes of the distress. Sealing over mortar joints prevents the terra cotta system from breathing, and traps water behind the facade. Often, even the original intent to block water entry is not accomplished, due to poor or nonexistent joint preparation, or use of an inappropriate sealant. Removal of this sealant can be problematic due to the difficulty of removing the sealant residue, and it adds expense to the next repair program.

Other inappropriate repairs include brick masonry installed to replace a missing or damaged terra cotta unit, which may not be structurally sound or waterproof; various patching mortars or bituminous materials applied to fill voids may have material properties that are incompatible with the surrounding terra cotta; or attempts to re-anchor terra cotta units that do not properly consider the backup material or members present, resulting in a structurally unsound condition.

Alteration Damage

Alteration damage occurs as a result of the installation of building accessories such as marquees, light fixtures, mechanical equipment, or other architectural ornamentation. Anchoring these items to the building requires the drilling of holes or cutting and removing portions of the terra cotta facade. These holes or openings become sources of water infiltration if measures are not taken to properly waterproof them. They are especially problematic as the anchoring starts to deteriorate, or when these items are removed for subsequent renovation work.

Conclusion

As demonstrated in this paper, terra cotta facade systems and the mechanisms by which deterioration occurs can be either simple or complex, and hence difficult to diagnose and repair. The severity of the deterioration can vary from minimal crazing to widespread spalling and displacement of units. In the authors' experience, many different types of deterioration can occur within the same area on one building. Other times, the nature of these deterioration mechanisms is subtle and not easily recognized with a cursory survey. A thorough investigation that includes close-up visual examination, sounding, penetrative examination, and a review of available drawings, is often necessary to properly understand the problems.

Water is the key. It is at the root of most deterioration, and the amount of deterioration is typically in direct proportion to exposure to water. The entry of water into the facade system needs to be minimized, and water that enters the system needs to be allowed to exit. Once deterioration starts to occur, water typically enters the system at an increasing rate, accelerating the deterioration process.

Building owners must be intimately involved with these issues, as they are being asked to foot the bill for both the investigation and the repair. They may not always be aware that there are typically a range of repair options available. Appropriate repair options can cover a wide range of measures which may not always be aesthetically pleasing, but address the root causes of the problems. The owners need to be presented with enough facts so that they can properly weigh structural integrity, weather-tightness, aesthetics, durability, short-term costs, and long-term costs.

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Emergency Repairs for Historic Facades

Reference: Pulley, D. M., and Robison, E. C., "**Emergency Repairs for Historic Facades,**" *Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444,* J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: Facade inspection and resulting interventions may alter the integrity of historic properties. Therefore, consultants must give careful consideration to historic preservation regulations and guidelines when performing facade inspections and implementing emergency and short-term repairs. Unsafe conditions identified through facade inspection should be remedied without delay to protect the public. However, the accelerated project schedule should not preclude following historic preservation procedures as some interventions may drastically alter a facade's appearance and historic significance, and severely hinder future restoration.

Mandatory historic preservation requirements may include Section 106 review for federally funded or licensed projects and local landmarks commission or architectural review board approval prior to implementing facade repairs. Enforcement levels vary between communities as do the definitions of "emergency," "alteration," and "ordinary maintenance." An industry-wide guideline for emergency repairs to historic facades may safeguard building owners, consultants, and communities against undesirable delays, fines, and losses.

Keywords: historic preservation, facade inspection, emergency repairs, Section 106 review, building permits

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In the United States, The Secretary of the Interior's Standards for the Treatment of Historic Properties set the guidelines for professionals working with historic buildings. Developed from the provisions of the 1964 Venice Charter, the Secretary of the Interior's Standards include the following provisions of special interest to professionals and owners performing facade repairs for historic properties:

- Work needed to stabilize, consolidate and conserve materials and features from the restoration period will be physically and visually compatible, identifiable upon close inspection, and properly documented for future research.
- Distinctive materials, features, finishes, and construction techniques or examples of craftsmanship that characterize the restoration period will be preserved.
- Deteriorated features from the restoration period will be repaired rather than replaced. Where the severity of deterioration requires replacement of a distinctive feature, the new feature will match the old in design, color, texture, and, where possible, materials.
- Replacement of missing features from the restoration period will be substantiated by documentary and physical evidence.

Consultants should give careful consideration to the Secretary's Standards when performing facade inspections and implementing emergency and short-term repairs. Unsafe conditions identified through facade inspection should be remedied without delay to protect the public. However, the accelerated project schedule should not preclude following historic preservation procedures as some interventions may drastically alter a facade's appearance and historic significance.

Public Safety Issues

Deteriorated facades pose a significant threat to public safety in cities across the United States. On May 18, 1984, a pedestrian suffered a fractured skull when she was struck by a 4-foot (1.22 m) long piece of terra cotta that fell seven stories from a historic downtown Chicago building [1]. That same summer, a 75-foot (23 m) section of limestone cornice fell from the then 60-year old Nitschke Building in downtown Columbus, Ohio [2]. The falling limestone injured three people including a city councilman who lost his leg. Many more incidents of falling terra cotta, stone, brick, and glazing facade elements have been reported in other major cities across the United States over the past 30 years [3]. It is probable that many incidents also went unreported.

Local governments, including New York City, Chicago, Detroit, Columbus, and Boston, responded to this crisis by passing facade ordinances in recent years. These local laws require building owners to inspect buildings on a regular basis and mitigate any hazards. The extent and frequency of facade inspections vary between communities, and ASTM has developed a Standard Practice for facade inspections to standardize procedures. Despite the safety hazards unstable facades present, facade ordinances face harsh criticism for being expensive, unreliable, and misused as impetus to demolish historic facade elements [4].

Following the death of a woman hit by falling terra cotta in 1974 Chicago [5], city officials rushed to implement new legislation. Due to an apparent oversight in 1980, however, the facade section of the new building code was omitted. The facade inspection portion of the code wasn't reinstated until 1996 [6].

The political nature of imposing mandatory facade inspections is considerable given the cost to building owners and the varied implications of discovering unsafe conditions. For example, in 1993 a debate ensued between the Memphis, Tennessee, Construction Code Enforcement Office and the Memphis Landmarks Commission over the enforcement of a newly instated "anti-neglect ordinance." Under the public safety ordinance, building owners were given 60 days either to repair or demolish buildings with cited hazards. However, local preservationists argued that the ordinance resulted in the demolition of historic landmarks under the guise of safety and actually encouraged the neglect of salvageable buildings so a judge might order their demolition [7]. Facade inspections pose similar conflicts in cities across the nation as the need for public safety comes in conflict with historic preservation concerns.

Both goals of preserving historically significant buildings and protecting pedestrians and motorists may be largely achieved through proper building maintenance. Effective facade ordinances address short-term and long-term repairs and require the implementation of a maintenance program. Unfortunately, effective maintenance is not performed universally, and consultants performing facade ordinance inspections or a voluntary facade survey for historic buildings may uncover unsafe conditions that require emergency action.

Causes of Deterioration

Specific causes of deterioration are too numerous to describe comprehensively; however, general trends can be identified. Many hazards associated with older facades result from deteriorated and unstable masonry systems. Deferred or improper maintenance may lead to a number of facade failures including pack-rust jacking, metal section loss through corrosion, masonry spalls, masonry cracking, and mortar deterioration. Any of these failures may result in the loss of lateral load and/or gravity load structural support for masonry systems, thereby creating instability.

Uncontrolled water infiltration through failed mortar joints and open cracks within masonry systems is often the root cause of corrosion of metal anchors and support framing (see Figure 1). For the building illustrated, the corrosion resulted in the loss of reliable gravity support and one terra cotta dentil fell from the 29th floor cornice in the middle of the night.

Metal corrosion not only creates a loss of load capacity within a masonry system, it may also induce significant forces within facades due to pack-rust jacking. Corrosion by-products expand and occupy a much larger volume than the original metal. The expansive forces are transferred to surrounding masonry and may induce cracking (see the horizontal cracking at window heads in Figure 2). The installation of elastomeric sealant over masonry cracks in the illustrated building did not alleviate the cause of the corrosion (a leaking roof), so the deterioration continued and the sealant failed. Eventually, portions of the facade were shored and partially dismantled to minimize falling hazards.

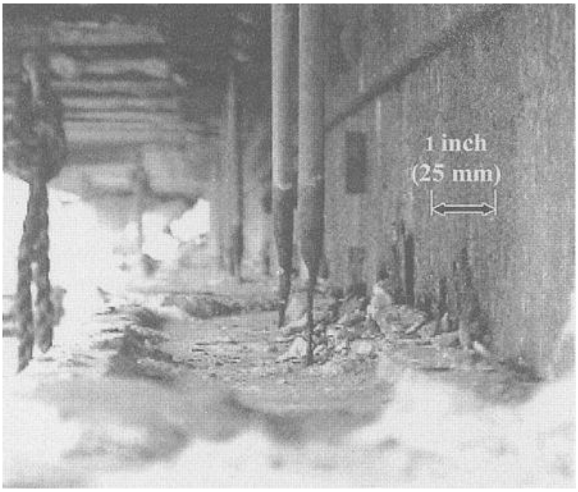

Figure 1 – *Steel Anchor Corrosion within a Terra Cotta Facade*

Figure 2 – *Masonry Cracking Associated with Pack Rust*

Corrosion is not the only failure mode for older facade systems. Masonry distress may result from a number of other mechanisms. For instance, facades lacking expansion joints in large masonry expanses may suffer extensive cracking and potential instability (see the vertical cracks in the brick parapet in Figure 2).

Mortar deterioration may also result in unstable masonry conditions in older buildings. Load bearing masonry relies on mortar for both lateral and gravity support. Significant mortar loss due to erosion or bond line failures and cracking not addressed through a maintenance program may reduce the stability of masonry facade elements significantly.

Masonry spalling and exfoliation may result from structural distress such as pack-rust jacking or reactions to adjacent mortar that is too stiff to allow for normal movement. Spalling and exfoliation may also result from water retention within the body of the masonry that can lead to salt re-crystallization or freeze/thaw damage [8]. Regardless of the varied causes, masonry spalls and exfoliation result in unstable conditions where loosened masonry may fall into the street (see Figure 3).

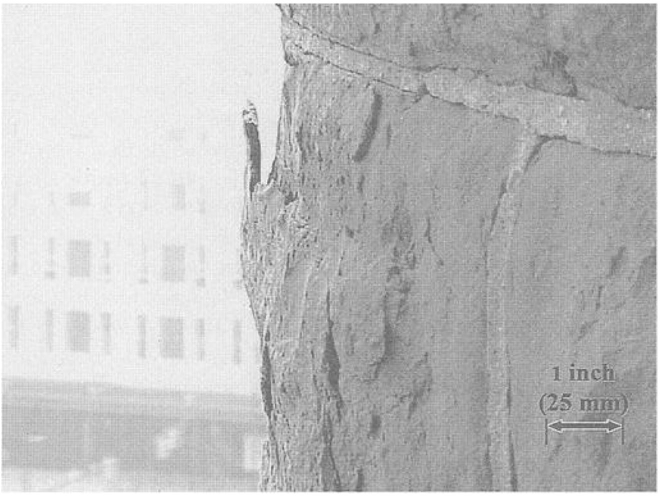

Figure 3 – *Stone Exfoliation on the Tenth Floor of an Historic Building Overlooking a Sidewalk and Street*

Correcting Unsafe Historic Facade Conditions

Correcting unsafe conditions identified through a facade inspection may involve a variety of interventions. Emergency repairs, made prior to implementing a comprehensive remedial design, may range from merely installing sidewalk protection to the removal of facade elements. However, even in emergency conditions, the basic principles of historic preservation apply, and should be considered in the design of emergency repairs.

Stabilization – In performing emergency repairs to correct unsafe conditions, preservation consultants recommend leaving as much original material in place as possible. The accelerated schedule that emergency repairs require often precludes performing an in-depth investigation of the historic significance of a facade prior to implementation. Even materials that appear insignificant at first glance may contribute to the overall historic integrity of a facade. Therefore, stabilization options should be considered first for all facade features in lieu of demolition. For example, cracked masonry features may be stabilized by installing dowels or anchors back to sound substrate in lieu of removing loosened material. Further, the pivotal decision to stabilize materials versus removing them and possibly reinstalling or reconstructing them later may be delayed by simply installing temporary shores or framing, sometimes called cribbing, to enclose or buttress the unstable elements.

Reversibility – When following preservation principles, consultants endeavor to make all emergency interventions reversible. Hundreds of years of architectural history can be irrevocably destroyed in a matter of hours if the future restoration work for a facade is not carefully considered. For example, historic terra cotta elements might be broken or shattered during emergency repair procedures, a non-reversible act that may be avoided given proper work specifications.

Documentation – Despite the rush to remove hazards, the consultant's services should still include documenting and reporting all interventions for the owner's records. Thorough documentation of existing conditions and alterations facilitates the accurate restoration and effective preservation of a historic facade after hazards are mitigated. For example, features that are temporarily removed but are extensively photographed in place and indicated on elevation drawings may be reinstalled or reconstructed accurately in the future. Similarly, repair materials installed on a historic facade and adequately documented, including installation locations, product data provided by the manufacturer, and material performance data, are better maintained. The *Secretary of the Interior's Standards and Guidelines for Architectural and Engineering Documentation: HABS/HAER Standards* provide some guidance on appropriate documentation techniques.

Materials Handling and Storage – Whenever historic elements are removed from a facade, it is a mistake to discard the materials without careful consideration. Whether the materials are reinstalled, used as models for replication, used for materials testing, or used for product testing, the removed materials retain intrinsic value that may be irreplaceable. For example, many restoration products such as consolidants require extensive testing prior to application, and removed materials may provide excellent samples in lieu of extracting more material from the structure. It is often in the owner's best interest to document and store removed historic materials.

Protection – Performing emergency facade repairs may adversely affect the performance of the building envelope. For example, removing dentils from a terra cotta cornice could leave open paths for water infiltration into adjacent facade elements or occupied spaces. Prolonged exposure to water infiltration often caused the facade instability in the first place and could introduce several other problems to the building. Therefore, any emergency repair that may increase water infiltration should also include the design of supplemental water barriers such as temporary flashing or sealant (see Figure 4 for an example of flashing installed by a contractor without the involvement of a consultant). In the interest of future restoration, care must be taken to use sealants that may be removed and that will not stain or leave a residue on the surrounding masonry. Flashings or covers should not trap water and in some instances should be selected so as to not trap water vapor and create condensation. Flashings or protective covers should be fastened into mortar joints instead of masonry units wherever possible. The temporary flashing shown in Figure 4 only meets some of the requirements mentioned above.

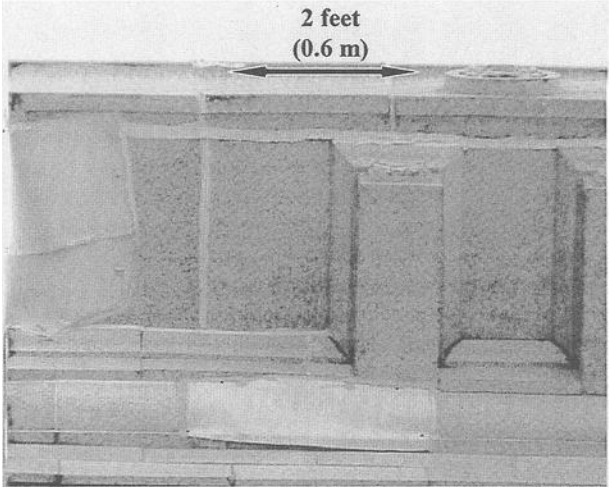

Figure 4 – *Temporary Flashing for the Underside of a Terra Cotta Cornice (Viewed looking up from the street)*

Durability – Funding necessary to implement a comprehensive rehabilitation program may be considerable and unavailable. Short-term emergency interventions may serve indefinitely to stave off disaster. The temporary flashing installed for the building illustrated in Figure 4 has remained in place for over three years while the owner has searched for funds. Therefore, temporary installations must be robust and weather resistant. In addition, consultants must advise owners of the expected service life for such interventions. Emergency installations utilized as long-term solutions will likely lead to further deterioration and potential instability.

Common sense and an understanding of basic historic preservation principles guide the responsible design and implementation of emergency repairs for historic facades. Unfortunately, the most appropriate interventions are not always selected and historic buildings are damaged. Public interest in safeguarding historically significant buildings gave rise to local, state, and federal preservation agency oversight for listed properties. It is in the best interest of the owner and the consultant to understand when mandatory regulations apply in order to avoid penalties and project delays.

Mandatory Historic Preservation Requirements

Local, state, and federal government agencies oversee work performed on historic properties to differing degrees. Enforcement of preservation guidelines usually begins at the local level, and the level of enforcement varies between communities. Federal and state level enforcement typically comes into play when state or federal dollars are used to fund the project or the owner's operations.

Local Preservation Regulations

Most communities either utilize a landmarks commission or an architectural review board to enforce historic preservation guidelines. For example, in Cleveland, Ohio, when an owner applies for a building permit, the Cleveland Landmarks Commission is notified of the project. If the project involves a designated local landmark or a building within a locally designated landmark district, the commission may require a full design review prior to recommending issuance of a building permit. Design review typically requires that the owner present plans, photographs, and renderings for the commission's review, and the owner may be required to alter the design according to preservation guidelines to obtain a permit. Therefore, it is critical that the owner and consultant be advised whether the subject property qualifies for landmark status to avoid unforeseen project delays.

The New York City Landmarks Preservation Commission performs similar services under the Landmark Protection Bill enacted in 1998. The commission enforces compliance with its guidelines through fines up to $5000 and an additional $250 per day. In addition, the commission retains the right to issue Stop Work Orders and may impose criminal penalties ranging from $500 to $15,000 per day depending on the violation [9]. However, Landmarks Preservation Commission enforcement policies include two grace periods to achieve compliance as well as an appeals process.

In addition to review mandated for local landmarks, individual facade easements established between building owners and local governments typically require that all facade alterations be subject to design review by the local review board or commission.

Federal and State Review

Federally designated historic properties are listed on the National Register of Historic Places kept by the National Parks Service. Despite some misconceptions, the National Register listing provides an honorary status that does not restrict the owner. Alterations made to registered properties are not subject to mandatory review, but the listing status of a building may be revoked if inappropriate alterations are implemented. However, any federal revenue in the form of tax credits applied to preservation, rehabilitation, or renovation of listed properties imposes mandatory review for a period of five years after completion of the credited project [10]. Mandatory review may be required for non-registered buildings if they are considered historic. In those instances, the government typically defines a "historic property" as a structure eligible for listing in the National Register of Historic Places.

Mandatory federal historic preservation requirements may include Section 106 review for alterations to a historic facade. Section 106 of the National Historic Preservation Act of 1966 requires that the Advisory Council on Historic Preservation review all federal undertakings including facade repair. Section 106 applies only to "historic properties" with the mission to preserve the nation's heritage. A "federal undertaking" broadly describes projects either funded through federal dollars, such as work performed on HUD properties, or licensed by a federal agency, such as the FCC licensing the installation of a rooftop antenna. Typically, the State Historic Preservation Office administers the review on behalf of the Council.

Emergency Repair Regulations

Emergency Repairs and Building Permits

Given the variety of deterioration that may culminate in an unsafe condition, the impact of performing emergency repairs on a historic facade varies. Individual masonry components, cornice systems, or entire facades may require stabilization and repair. Regardless of the scope of work, review of historic preservation guidelines and regulations is recommended. For example, in Cleveland, Ohio, any activity that can be described as an environmental change requires some level of historic preservation review by the local landmarks commission regardless if it involves re-tooling stones or removing an entire cornice. The Cleveland Landmarks Commission defines an "Environmental Change" as follows:

> *"Environmental Change" means any alteration ["Alteration" means any material change in the external architectural features of any improvement which has been designated a landmark or which is situated in a landmark district, less than demolition, removal or construction of any improvement.], demolition, removal or construction of any property...[11]"*

However, unless an application for a building permit is filed, the Cleveland Landmarks Commission will not be notified of the project at all, let alone require review. Therefore, in performing emergency facade-repairs, it may be critical to understand what types of work require a building permit. For instance, installing temporary shoring and removing facade elements may be treated as ordinary repairs not requiring a permit in one community and may require both a building permit and mandatory preservation review in another.

Most building codes include provisions for what work does not require permits. The 2000 International Building Code (IBC) states in Section 105.2.2:

> *Repairs. Application or notice to the building official is not required for ordinary repairs to structures...Such repairs shall not include the cutting away of any wall, partition or portion thereof, the removal or cutting of any structural beam or load bearing support...or other work affecting public health or general safety [12].*

Unfortunately, the IBC does not define "ordinary repairs" any further. Most local governments do not require building permits for "maintenance" either. The way a community defines these terms largely determines the enforcement of historic preservation guidelines through local building permit review.

In St. Louis County, Missouri, an entire web page is dedicated to "Work that Does Not Require a Building Permit." Included in the list is the following description:

> *25. All repairs (including smoke/fire damage, termite, wind, repairs, etc.) to a building when the building official determines the work is of a minor cosmetic nature and there is no damage or change to any part of the building structure. A field inspection is required to determine the nature of the repair [13].*

This case-by-case approach provides flexibility to the community to prevent insensitive alterations and provides guidance to the consultant that a building inspector should be contacted before facade repairs are implemented. This practice not only safeguards the owner from unexpected permitting delays, it supports the consultant in facilitating local preservation review for historic properties. Therefore, contacting the local building official prior to implementing facade repairs should be considered in any community.

In the event work is performed without the appropriate permits and permissions, the owner will be well served in following industry-wide guidelines for historic preservation. Any oversights may be forgiven if the work in question is implemented within governing regulations. For example, the New York City Landmarks Preservation Commission may legalize work performed without mandatory review and retroactively issue permits if the alterations comply with commission regulations [14].

Emergency Local Preservation Review

In the event that a local building permit is required to perform emergency facade repairs, the local landmarks commission or architectural review board may require design review. Under normal circumstances, the review and permitting process may require one to three months to complete. In emergency situations, however, review is often expedited. For instance, in Cleveland, Ohio, emergency repair review may require less than ten days. Once the proposed repairs are approved, the Cleveland Landmarks Commission will issue a Certificate of Appropriateness certifying their endorsement of the intervention and the building permit is issued. In rare instances, the building department will issue permits even if the commission does not approve the repairs.

Emergency Section 106 Review

The typical Section 106 review may require six months to process all necessary submissions and allow for comments and corrections. Fortunately, governing agencies will often expedite review processes in the interest of safety in emergency situations.

Section 106 defines an "emergency" as "...a disaster or emergency declared by the President, a tribal government, or the Governor of a State...or other immediate threats to life or property"[15]. In the event of an emergency situation such as unsafe conditions identified through facade inspections, alternatives to the normal Section 106 review process may be employed. Alternative procedures include:

> *Notifying the Council, the appropriate SHPO/THPO and any Indian tribe or Native Hawaiian organization that may attach religious and cultural significance to historic properties likely to be affected prior to the undertaking and affording them an opportunity to comment within seven days of notification. If the agency official determines that circumstances do not permit seven days for comment, the agency official shall notify the Council, the SHPO/THPO and the Indian tribe or native Hawaiian organization and invite any comments within the time available*

[16].

Alternative procedures described in Section 800.12 of the Revised Section 106 Regulations may only be employed when the emergency repairs or other emergency undertaking is implemented within 30 days after they are identified. If repairs may be postponed beyond 30 days, full Section 106 review is required.

Emergency Response Time

The exact time and location of a catastrophic facade failure cannot be predicted. Material deterioration, such as steel anchorage corrosion in a terra cotta cornice, usually develops over protracted periods of time, not overnight. Decades of poor maintenance may culminate in a facade failure at any moment regardless of when the hazard is detected. Therefore, determining the appropriate response time for emergency repairs is problematic. Is one hour, one day, one week, one month or one year the most appropriate response time to a threat to public and property?

Any injury incurred after the facade inspection is performed may pose additional risks to the parties involved, thereby lending more urgency to implementing emergency repairs. However, a facade failure is no more likely to occur before or immediately after discovery of a hazard, and basic precautions such as overhead protection should largely prevent injury if implemented in a timely manner. For these reasons, consultants should discourage owners from circumventing proper review under the guise of "emergency repair."

Consultants recommending emergency repairs for historic facades grapple with fulfilling both requirements of eliminating hazards without delay and delaying work to allow for historic preservation review. However, an understanding of mandatory preservation review as it applies to emergency repairs may guide their recommendations.

According to Section 800.12 of the Revised Section 106 Regulations, any repair that is completed after 30 days of discovery is not considered an "emergency repair" [17]. Therefore, 30 days may serve as one measure of appropriate response to an unsafe condition. To maintain a consistent level of service, consultants should consider developing an emergency repair timeline guide similar to that shown in Table 1 and appropriate to the project.

Falling Terra Cotta Case Study

A 50-pound (220 N) terra cotta dentil fell 29 stories in the middle of the night on March 29, 2000 from a cornice in Houston, Texas. No one was injured, but the owner of the property responded to the crisis immediately. A maintenance contractor installed sidewalk scaffolding within 24 hours of the incident. To mitigate unsafe conditions, the contractor immediately removed terra cotta dentils on street-side elevations over the course of one week. The emergency intervention was performed without a city permit or preservation review.

Table 1 - Sample Emergency Response Timeline

Consultant Response	Elapsed Time
Notify owner of unsafe condition	1 hour
Recommend appropriate overhead protection (sidewalk scaffolding, barricades, etc.)	24 hours
Identify potential preservation review requirements	5 days
Perform further documentation as needed	5 days
Develop schematic emergency repair design	5 days
Identify necessary permits	5 days
Submit a Preliminary Repair Report to the owner	5 days
Initiate preservation review (if required)	5 days
Develop initial design documents	10 days
Assist owner in contractor selection	10 days
Complete preservation review (if required)	15 days
Finalize design documents	15 days
Review emergency repair installations	30 days
Submit a Final Repair Report to the Owner	30 days

The owner contacted a consultant to perform engineering services for a limited facade inspection in response to the failure. Engineers performed an up-close review of the terra cotta cornice and supporting steel using a combination of borescope probes and performing limited excavations. The vast majority of the double-angle steel armatures supporting the cornice were found to be severely corroded (see Figure 5). In addition, the steel anchorage for the dentils, a round bar and J-bolt suspended from the armatures, was either failed, missing or severely corroded.

With sidewalk protection in place and many falling hazards removed, a long-term and comprehensive remedial design was implemented to replace missing and corroded steel members. Given the complexity and historic significance of the cornice system, engineers opted to replace the steel armatures without dismantling the terra cotta cornice above. Temporary shoring provided structural support during the reconstruction of the steel framing.

Following typical contract procedure, the contractor was charged with obtaining all necessary building permits. Under the umbrella of "maintenance and repair," no permit was required by the city. The consultant relied upon the contractor to protect the owner against potential fines and delays associated with alterations to the property by following local permit procedures.

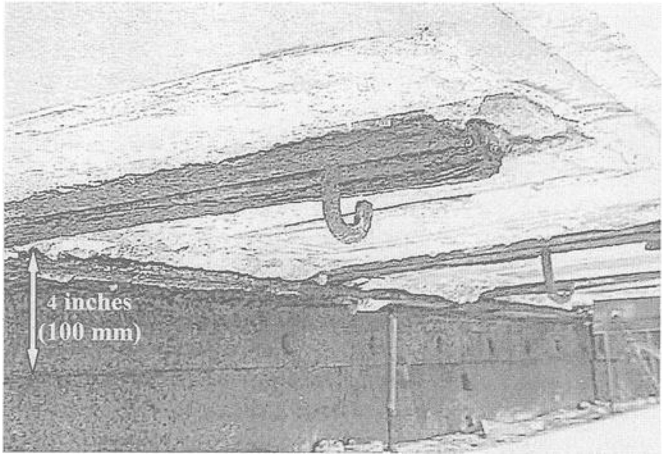

Figure 5 – *Severe Steel Corrosion within a Terra Cotta Cornice*

The repairs were implemented in strict accordance with historic preservation guidelines. Historic terra cotta dentils were reinstalled in their original locations, and the cornice remained intact throughout the rehabilitation. No action was taken against the owner or the consultant for performing the repairs. However, the project raises the issue of the consultant's role in obtaining permits for modifications to historic buildings. In a different community, historic preservation review may be required prior to performing a cornice restoration. Therefore, consultants and their clients benefit from becoming active in the permitting process by asking the following questions prior to implementing repairs:

- Is the building a local landmark?
- Is the building located within a historic district?
- Is the building subject to a facade easement?
- Is the building eligible for listing or already listed on the National Register of Historic Places?
- Does the community require a building permit?
- Does the community require a sidewalk permit?
- Has the local building inspector been contacted?
- Is the building occupied or funded by a federal agency?

Standardizing Emergency Repairs for Historic Facades

Even with good intentions, owners may find themselves in violation of local or federal preservation regulations when performing emergency facade repairs and facing significant fines and delays. An accepted industry guideline for emergency repairs to historic properties would benefit building owners, consultants, and the community by minimizing penalties to owners acting in good faith and minimizing undesirable alterations to historic facades. The development of such a guideline requires consideration of several issues including:

- Definition of "Historic"
- Definition of "Emergency"
- Definition of "Ordinary Repair and Maintenance"
- Definition of "Alteration"
- Definition of "Appropriate Emergency Response Time"
- Minimum Documentation Requirements for Existing Conditions
- Minimum Documentation Requirements for Implemented Emergency Repairs
- Requirements for Consideration of Alternative Interventions
- Requirements for Materials Handling and Storage
- Minimum Documentation Requirements of Owner's Compliance with Historic Preservation Guidelines

To maximize the success of emergency repairs for historic facades, property owners and consultants should be prepared to deal with unsafe conditions prior to beginning an inspection. Consultants should consider their emergency repair services in advance including their role in permitting and design review at the local, state, and federal levels and an appropriate timeline for services. An industry-wide guideline for emergency repairs to historic facades would improve awareness amongst building owners and consultants and help achieve the goals of historic preservation without increasing risks to public safety.

Acknowledgments

Case study information and photographs provided by Engineering Diagnostics, Inc., 6100 Hillcroft, Suite 750, Houston, Texas 77081.

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Section III: Investigation and Data Collection Techniques

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Facade Maintenance: Owner's Techniques for Data Management

Reference: Chadwick, J. J. and McJunkin, J. T., "**Facade Maintenance: Owners' Techniques for Data Management,**" *Building Facade Maintenance, Repair, and Inspection, ASTM STP1444,* J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: The long term success of any building maintenance program is directly influenced by the information available. Facades are particularly challenging since they are not conventionally viewed as regular maintenance items and the components are mistakenly thought of as "permanent." The Standard Practice for Building Data Management ASTM E-2166, gives the building owner a structure on which to arrange all relevant building data to develop factual relationships that will make forecasting, analysis, and budgeting for building maintenance more reliable.

Keywords: facade maintenance, building data, UNIFORMAT, XML, DTD

Introduction

> *At twenty, you have the face you were born with.*
> *At fifty, you have the face you deserve.*

For the average building owner, this observation attributed to Coco Chanel presents a stingingly clear analogy to façade maintenance. While personal care by most definitions is an individual exercise, building maintenance involves a complex orchestration of specialists over time. It is an orchestration where the conductor, musicians, and even the program changes with no predictable basis. In the face of executive transience, and unfortunately persistent personnel migration, the possibility of available and reliable fragments of building history to support immediate decision-making is becoming increasingly remote.

Few large buildings today are constructed as a testament to the business prowess or sentimentalities of the patriarch owner. Although buildings have always been a tool of business, they had somehow lost the burden of cultural icon just as business had lost its interest in the tools of physical production. Consistent with the devolving business proclivities, the Tax Reform Act of 1976 defined Component Depreciation, which recast building consciousness in the business community in terms of "depreciable asset" a term once reserved for process equipment. In short, buildings were emphasized to be just another commodity in the service of the ubiquitous bottom line. Curiously, the guidelines for depreciating the "envelope" component represented a somewhat idealized lifespan of

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that entity - that is, the envelope was indexed to what was thought to be the life of a premium roof, since exterior walls were not understood to "wear out." Funding frequently follows accounting conventions, rather than the actual rate of consumption or wear. Because budgets are typically set to satisfy screaming needs with funds available, it follows that decisions based on accounting rules based on misconceptions of physical science will not adequately support a realistic maintenance program. While it is always easy to blame the accountants, the root problem may lie with the lack of auditable information.

Building facades present unique challenges, even for the alert building owner. The conditions that can precipitate enormous problems are practically speaking, out of sight, and therefore out-of-mind. By the time a presentation of observable distress occurs, concealed damage is significant. Minor leaks that may have been dismissed to an overall nuisance-factor are actively ignored for years. Despite the implied lesson of useful life inherent in the now defunct component depreciation mechanism, money applied to maintaining a building's exterior is not clearly appreciated as having an obvious "payback." New lobby finishes justify a raise in rent. New fire alarms are required by code and insurance. Replacing sealant at the end of its anticipated life definitely lacks glamour. All too frequently, action occurs only as a reaction to prominent failure, particularly in jurisdictions lacking a façade inspection law.

The need for a coherent system to manage building data is enormous, as are the benefits. Historical data, lessons learned, the locations of short-term fixes should not be lost with the retirement of an employee. Engineering observations can provide a critical benchmark –if the report can be found. The ability to cross reference and inter-relate information can make possible more reliable identification of trends for prediction as well as more confident diagnoses. The economic interdependence of systems can be clearly demonstrated to make compelling cases to budget repair or replacement actions. The sum of many ill-fitting windows affects energy consumption; sealant failures can damage primary and secondary structure before interior finishes are affected. These concepts are certainly not new. Many consultants offer building management database services that, for commercial purposes, are specifically designed not to communicate outside of their proprietary system. The view of the data that is available is generally structured by a programmer with accounting advice designed for "high level decision makers." While the executive summary is unarguably necessary, God and the Devil do battle in the details. It is the day-to-day management and individual attention to the viscera of the building that directly contributes to the long-term condition. The problem then is to make building data accessible and available as a useable tool throughout the chain of custody responsible for its upkeep. One should easily be able to research and analyze the history of conditions, cost, or actions taken on any system within a single building, and compare a subject building within a portfolio of buildings, or with buildings of similar use or construction types to a component level of detail.

Two significant deficiencies had stifled this effort - a standardized data structure and a sufficiently robust platform. The Standard Practice for Organizing and Managing Building Data, ASTM E-2166, addresses the problem of data structure. It marries two widely used and accepted organizing systems: UNIFORMAT II ASTM E-1557 and Master Format. Although many popular proprietary estimating systems successfully link the two systems, they have been assembled for the convenience of the practical contemporary estimator and lack the flexibility and definitional precision necessary for data collection, especially in novel or archaic construction types.

UNIFORMAT II reduces a building to essential systems and subordinate functional elements within a three level hierarchy. The systemic and functional relationships are essentially a conceptual model of a building such that a technically inexperienced person can readily understand the basic relationships. The current UNIFORMAT II as a standard is a classification system that evolved from the first elemental classification attributed to the British Ministry of Education following the post WWII school expansion program. This methodology was applied to construction programs in other British Commonwealth countries, such as Canada, and then the United States in the early 1970s. In 1973 the American Institute of Architects undertook to develop an elemental cost estimating format called MASTERCOST. In conjunction with the General Services Administration, a consensus format named UNIFORMAT was produced. Although it was not an official national standard, it did form the basis for any subsequent elemental formats in the United States. In 1989, the American Society for Testing and Materials Subcommittee E06.81 on Building Economics appointed a task group to develop a UNIFORMAT standard. In 1992, the National Institute of Standards and Technology (NIST) issued a special publication [1], in which the name UNIFORMAT II was selected to emphasize that it is an elemental classification system similar to the original UNIFORMAT. The improvements made it more comprehensive, particularly with respect to mechanical systems and sitework. In 1992, the Construction Specifications Institute (CSI) issued an interim edition of UNIFORMAT based on the work in progress of ASTM. CSI also published a practice entitled AFF/180-Preliminary Project Descriptions and Outline Specifications,@ which recommended the use of an elemental project description (specification) based on UNIFORMAT at the schematic design phase. The objective of the classification format was to improve communication and coordination among all parties involved in a project, particularly between the design team and the client. The ASTM standard was approved in 1993 and designated: Standard Classification for Building Elements and Related Sitework - UNIFORMAT II ASTM E-1557-93. In 1996, revisions were made providing a distinctive alphanumeric format for the elements similar to that incorporated by CSI in 1992.

By virtue of its hierarchical structure, UNIFORMAT II offered an organization to logically summarize functional elements. These elements inherently possess details of interest. A method to amplify and focus further on these details in an orderly coherent way is mandatory for the viability of any data structure. The hierarchy should be sufficiently deep to describe the artifact adequately without spiraling into the kind of hair-splitting that yields a burdensome number of levels and sub-sets that dead end in arbitrarily finite categories. Some elemental classification systems pursued ways to wring a narrower focus by creating as many as 12 subordinate tiers. While this exercise may have had some situational merit, the practical application of a 12-tier system with related categories to conventional databases could be numbing. The capacity of such a system to adapt to new or innovative types of construction was limited to the classifier's ability to force the aberrant piece of construction into an existing category. The exercise of forcing an evolution of functional systems to building parts is further rendered futile in the face of an existing, widely accepted classification system for materials.

Master Format groups materials and products into 16 Divisions relative to similarity of material and function, and organizes the Divisions nominally in the order in which that material or product appears in the construction process. For instance, Division 01 is General Requirements, Division 02 is Sitework, Division 03 is Concrete, Division 04 is Masonry, etc. All materials and products are coded with a five digit number, with the leading two digits representing the Division (*Concrete:* **03**120), the next digit designating the broad-scope category (*Concrete Formwork:* 03**1**20) and the last two identifying a narrow scope assignment (*Architectural Cast in Place Concrete Formwork:* 031**20**). Since its introduction in 1963, MasterFormat has been widely accepted and promoted as an industry standard in the United States and Canada. It was first published as part of the ACSI Format for Construction Specifications,@ which evolved as the basis for the AUniform System for Construction Specifications, Data Filing, and Cost Accounting -- Title One Buildings@ published in 1966. Master Format has held the broadest possible acceptance as the filing system for materials and products. Its shortcomings as a method to describe a whole building are obvious to anyone who has had to review a contractor's Application for Payment using a Schedule of Values derived from the Specifications' Table of Contents. Early Division work is artfully front loaded with the hope that the minimal amounts allocated for end-of-project site work, concrete or masonry will be overlooked during the project's honeymoon phase billing. Classification errors in estimating, procurement, and coordination persist due to selective interpretation of broad scope requirements: Are lintels provided by the mason or furnished under Miscellaneous Metals? Access doors are required by mechanical trades, installed by as many as four different finish trades, and furnished under Division 08 (doors). Experience and local custom have largely compensated for many similar tactical difficulties.

ASTM E-2166 bridges these two systems by defining one additional level, the *Type.* A *type* is a kind of user-defined assembly that possesses a unique combination of function and components consistent with, and subordinate to, elements within the third level of the UNIFORMAT II outline. Elements that superficially appear to be similar are constructed with purposeful physical variations in order to accommodate a variety of functional or situational requirements. For instance, some buildings may have façade elements that respond to a base, shaft and crown design. Others may vary by floor level and compass orientation. Some may have an apparently uniform exterior while the backup construction varies wildly to conceal or protect interior activities.

The use of *types* is significant for several reasons. First, UNIFORMAT II does not inherently possess a method to differentiate among the many potential combinations of materials that might be assembled to describe satisfactorily the significant sub-set functions of an element. It would be quite unreasonable to expect to contrive a comprehensive taxonomy of types given the staggering combinations of historic, stylistic, regional, and economic variations that occur in building construction. By setting this level to record the actual functional assembly, an accurate record of the building is maintained. The third level then remains a summary level, enabling comparisons across differing construction techniques or uses. Second, because the type is a sum of Master Format parts, the ability to step into a component level analysis is expedited to the shortest possible path. Third, the application of a type can contribute to support an overall strategy for façade maintenance and component replacement. The capacity to evaluate elements types and components rapidly is useful to identify likely failure points or "weak links" in a given assembly.

When new installations have successfully survived the one-year statutory warranty, one can reasonably expect them to be reliable for the duration of the guarantee period. With the guarantee period complete, we look to what experience would determine to be the *probable* maximum life, or the period of time when a minimum amount of upkeep is required to sustain the desired performance. As the component or system continues to age, the upkeep and maintenance needs continue in theory, to increase. An economic analysis can easily determine the cost-benefit of repair-and-retaining, replacing, or use-in-place-until-failure at the end of its maximum *possible* life. Each choice presents risk. Risk is mitigated with information. The difficulty arises out of selecting an appropriate depth of focus, or establishing a balanced view to state the problem for analysis. With this data structure, a facade can be separated into elements, types and parts to ferret out the instigating problem points. Conscientious application of General Conditions and General Requirements costs will contribute to the body of information from which a decision tree and action plan can be constructed. A reasonably knowledgeable building manager with appropriate engineering advice could implement a scheduled inspection cycle to optimize professional time on site to monitor conditions as well as make a convincing argument to fund a facade account with predictable draw-down events.

The other contribution made by the Standard Practice for Organizing and Managing Building Data is the consideration of joints or the connections between elements as their own typological set. As an outline comprised of functional relationships, UNIFORMAT II tends to focus on each system as a discrete concept. Ultimately, to be functional, the elements must be connected to each other in some way to act as a coherent building. These connections or joints are designed to maintain the integrity of the system by mitigating certain conditions within designed limits. The function of the joint is necessarily more complex than the types being joined. The notion of a "joint" in every elemental system is tacitly presumed to reside in one or the other system and never dealt with directly. An awareness of the joint as a named entity helps to focus attention on its functional criteria. For instance, if a basic function of an exterior wall type is to keep weather out of the building, the joints must additionally accommodate movement, possibly provide galvanic isolation, and present an appearance consistent with an overall architectural vocabulary. The materials used to make joints are frequently unique to the joint and different than the materials comprising the basic types being joined. The useful life and maintenance cycles of many kinds of joints vary sufficiently from the adjacent assemblies to merit scheduled attention.

The Standard Practice for Organizing and Managing Building Data was written with no explicit recommendation for a particular management mechanism. Based on the complexity of the building and the needs of the owner, physical files could be effectively managed under its data schema. Similarly, object-based programming and contemporary databases could be used to manage most buildings on a desk-top personal computer. The unprecedented power of inter-net and intra-net applications, however, provide a degree of resource management and sharing never before thought possible. Shadowing the development of the Standard was the increasing popularity and evolutional improvements of eXtensible Markup Language, or XML.

XML evolved as a subset of Standard Generalized Markup Language (SGML), which is widely used in Europe in the publishing industry to assist in the electronic delivery and publication of text-based documents. As early as 1996 it was clear that SGML was too complex to be handled on the fly by web-based applications. Hypertext Markup Language (HTML), an application of SGML was too limited to handle digital presentations and could only present "images" of numeric tables – that is, the numeric data as-presented could not be manipulated with mathematical operations. XML is becoming the de facto Internet standard for representation of content optimized for Web delivery. It is a meta language for defining an unlimited number of specific markup languages, each of which may contain an unlimited number of tags, hence extensible.

Documents or records encoded to conform with XML have both a logical and physical structure. Logically, they consist of a hierarchy of named elements, which may be likened to fields, with nested elements akin to subfields. Each instance of a document has a single root element to which other elements are subordinate. XML provides for unambiguous identification of complex data structures that can be treated as objects. This is accomplished through the Document Type Definition, or DTD. The DTD declares each of the permitted entities, elements and attributes and the relationships among them, forming a template for the logical structure of associated XML documents. It expresses the hierarchy and granularity of data, allowable attribute values and whether elements are optional, repeatable, etc. DTDs have been used to define a Biosequence Markup Language, as well as ones for astronomy, chemistry, and mathematics. The Standard Practice for Building Data Management could reasonably form the core schema of the DTD for building data.

Using ASTM E-2166 as a style guide, building surveys and consultant reports could be produced in an easily readable organization that could be readily tagged for absorption into a larger data system. Each tagged data element is retrievable at the hierarchical level to which it was set, for "horizontal" comparison as one would expect in a conventional database. The set of attributes that can be associated with the XML data element however, give it an object quality, permitting it to be sorted according to any of the assigned attributes to yield a very fine grained analysis.

This application supports higher level documentation as well as detail analysis, and is intended to accept information on an "as available" basis. This is useful when a large portfolio of properties need to be initially evaluated or when invasive exploration can be deferred. An owner can specify observations to a degree of detail consistent with a directed level of interest. For instance, a binocular inspection with videographic documentation of a set of facades would provide a reasonable amount of baseline data to assign priorities and determine a plan of action. If conditions deserving urgent attention become apparent, they can be noted for subsequent investigation. As additional work is performed and further investigations made, the inventory of activity and degree of detail will eventually populate the component level. This higher level data collection also has obvious application in support of authorities-having-jurisdiction directing many professionals performing triage evaluations after natural disasters.

As the building stock inevitably ages and economic priorities shift, it becomes increasingly important for building owners to possess a lucid understanding of the conditions and costs associated with their buildings. The term " stewardship" found in so many annual reports suggests a level of comprehension often promised but rarely demonstrated. In the face of construction techniques, legal requirements, and business priorities which become more complex daily, it will be the ability to understand and use the data at-hand that will dictate outcomes. Solid data with crisp organization will certainly improve the odds for better decision-making. Indeed, one should not pursue vast plans with half-vast data.

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Hamid Vossoughi, P.E.[1], and Rehan I. Siddiqui[1]

Industrial Rope Access – An Alternate Means for Inspection, Maintenance, and Repair of Building Facades and Structures

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Reference: Vossoughi, H. and Siddiqui, R. I., **"Industrial Rope Access – An Alternate Means for Inspection, Maintenance, and Repair of Building Facades and Structures,"** *Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444,* J. L. Erdley and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: Access is an integral part of inspection, maintenance, and repair of building facades and structures. The criteria for selection of access to building facades is widely varied and, in part, depends on the characteristics of the building (roof, height, set backs, overhangs, etc.), site constraints (traffic, sidewalk, etc.), and economics (time and money).

Some of the most traditional means of access to building facades include boom truck, man lift, swing staging, aerial platform, and scaffolding. Advantages and disadvantages of each type of access should be considered in the selection process. Depending on the complexity of the project, one or more of these types of access may be selected to meet project demands. One technique, which is widely used in Europe and now gaining acceptance in the United States, is Industrial Rope Access (Rope Access). This technique is often a supplementary means of access, and not necessarily a substitute for the traditional means of access to building facades and structures.

This paper presents a general overview on the past, present and future of this technique for inspection, maintenance, and repair of building facades and structures.

Keywords: means of access, Industrial Rope Access, safety, flexibility

Access Solutions in the Age of Facade Ordinances

Maintenance and repair of the exterior facades of buildings are required in order to extend their useful life, minimize the rate of deterioration, and avoid unsafe conditions. Exterior facades are exposed to the elements, but nonetheless, proper maintenance of many buildings is often deferred. The potential threat to public safety brought about by this trend has caused several major cities to enact laws requiring periodic inspection,

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maintenance, and repair of building facades. In cities such as Chicago and New York, with the largest number of aging building stock in the United States, pedestrian sidewalk protection has become a part of the city landscape.

The scope and requirements of these facade ordinances are varied. Some require detailed critical examination of all walls at regular intervals, as well as an annual visual inspection and maintenance program during successive critical examinations. Others require visual inspection of representative portions of exterior facades. Apart from the ordinances, some proactive building owners and asset managers have developed preventive maintenance and repair programs to preserve their investment and extend the useful life of their buildings. However, on some occasions, the threat of deteriorated, distressed, or unsafe conditions may require immediate attention following the inspection, such as emergency stabilization and/or temporary repairs, until a more permanent and long-term repair can be implemented.

Some of the facade ordinances require detailed inspections, and some require a visual representative inspection of the exterior facade. As a result of the inspection, a maintenance and repair program is developed and will probably be implemented. It is likely that the maintenance and repair is spread throughout the entire facade, often making the cost of the access and the logistics of the project more complex and disproportionate to the scope of work. Advantages and disadvantages of each means of access must be evaluated accordingly.

Brief History of Rope Access

In order to surmount the unique challenges of a given project while maintaining the schedule and budget, and considering the risk and reward factors, the construction industry has always been forced to be creative and innovative. Continually, such challenges have given rise to innovations and the application of new techniques to the construction industry.

A classic example of innovation in the construction industry occurred during the construction of the Hoover dam in the early 1930s, where stabilization of the deteriorated walls of the canyon was a major priority prior to the start of the construction. The canyon walls were deteriorated due to the freeze and thaw cycling of built-up water in the canyon crevasses. In that case, the work was successfully accomplished by using skilled miners, known as "high-scalers," to access the sides of the cliffs, utilizing a single rope as a safety line, while hauling their equipment with them. However, because of the primitive equipment and lack of proper safety provisions, there were fatalities among the high-scalers. In the early 1970s, a much safer use of lightweight equipment for descending and ascending on suspended ropes became commonplace in the sport of caving (Figure 1). In the mid to late 1970s, the improved techniques

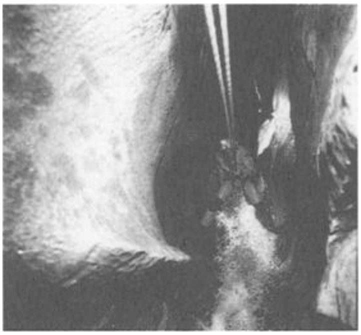

Figure 1 - *Caving*

and equipment were used once again in the construction industry, this time in France, for stabilization and containment of falling hazards on the side of a cliff above a church.

In the early 1980s, the possibilities and advantages of utilizing the mountaineering and caving techniques and equipment were recognized, improved, and became more widely used in the construction industry in Europe, leading to the creation of the profession currently known as "Rope Access." In the mid to late 1990s, the same potentials were realized, and Rope Access started to flourish in the United States.

What is Rope Access?

Rope Access combines specialized technical skills and equipment that originated in mountaineering and caving with further development for industrial use. It provides safe access to buildings and structures by descending and ascending as well as laterally maneuvering by climbing on suspended or tensioned ropes.

In the sports of mountaineering and caving, there is an element of risk associated with the activity. An important difference between these sports and Rope Access is that only calculated and managed risk is acceptable in Rope Access. The main suspension equipment used in Rope Access is double static rope (typically with less than 5% stretch) with independent and redundant anchor points for each rope, specialized descent and ascent equipment, full seat and chest harness (forming a full body harness), and safety helmet. One of the two ropes is the primary (suspension), and the other is the backup (fall arrest) rope. The configuration and arrangement of the harnesses and the equipment (known as the 'rig') is such that the point of suspension is at waist level to maintain a low center of gravity to facilitate a comfortable sitting position to allow for maneuverability. The attachment to the backup line is from a higher point at chest level and holds the body in a safe upright position in the event of a failure of the primary suspension line or in the case of an unconscious personnel. Having the attachments at the front allows for a relatively straightforward rescue of the individual.

Advantages of Rope Access

In comparison with the traditional means of access, the lightweight and flexible nature of Rope Access provides many advantages, as noted below:

- Ease of access – Fast and easy access to structures with minimal equipment requirements;
- Speed of setting up and vacating sites – Rope Access systems can be set up and dismantled quickly, maximizing production, while accommodating project constraints;
- Minimal disruption to adjacent work areas and building operations – Access to structures is independent of site conditions, such as nearby excavations, adjacent buildings, alleyways, train tracks, bridges, bodies of water, etc;
- Flexibility and versatility – Because of the speed and flexibility of this system, project mobilization and demobilization is minimized, thus reducing costs and lead times;

- Security – Most equipment can be removed at the end of each day, reducing the potential for unauthorized access, theft, or vandalism;
- "Hands on" – Rope Access allows close tactile inspections and a more time effective area coverage than other forms of access; and
- Building sensitive – Rope Access has the least visual impact on building structures, and does not damage the building fabric. This is especially valuable when working on historic structures.

Rope Access can be applied to any type of building structure, including mid- to high-rise buildings, historic and ornate buildings, bridges, dams, cliffs, embankments, antennas, masts, chimneys, space frames, and offshore structures. The advantages of this form of access should be evaluated on case by case basis. Due to the history of the Rope Access outside North America, Rope Access is equally weighed in comparison to the traditional forms of access for projects by engineers, architects, contractors, and building owners and managers.

Rope Access Guidelines and Standard Codes of Practice

As the use of Rope Access increased in Europe, companies engaged in Rope Access services, manufacturing, and sales of equipment formed trade organizations and developed guidelines for the use of this technique. The first and most detailed guideline for the use and standardization of this technique was developed and published in the United Kingdom in 1993. A similar movement begun in the United States in 1998 when several companies involved in mountaineering, manufacturing, and equipment sales initiated a trade organization for the Rope Access industry. At the national level, the ASTM International E06 Subcommittee on Rope Access has been preparing the Standard Practice for Industrial Rope Access since 1998. Several drafts of the standard have been reviewed and it will likely be approved in the near future.

Rope Access Personnel and Training

The original Rope Access technicians and personnel were from the sports of mountaineering and caving. As the industry grew, so did the need for training and certification of Rope Access personnel. Tradesmen and professionals were trained in the Rope Access technique to be able to apply their skills at heights more efficiently.

The United Kingdom Rope Access trade association guidelines provide three levels of qualifications for Rope-Access-trained personnel. The qualifications are attained through formal training, experience, and knowledge in the field of Rope Access. The lowest (or the entry level) is Level 1, and the most experienced level is Level 3. During training, there are two instructors to a maximum recommended number of six students per class. The duration of a basic training course (Level 1) is five days and is directed by Level 3 Rope-Access-training personnel. Training includes two days of class work and three days of practice and familiarization with Rope Access equipment and techniques and an examination.

All personnel involved in Rope Access must have the appropriate attitude as well as physical aptitude for such work. They must be physically fit and free from any disability that may prevent them from working safely at heights.

On project sites, a minimum of two Rope Access technicians is required as a safety requirement. Depending on the scope of each project, the recommended number of Rope Access technicians is increased as required. Each qualified person is required to maintain a personal record of the hours and the nature of the Rope Access work carried out. Reassessment of Rope Access personnel is required every three years.

Rope Access Safety Record

The United Kingdom Rope Access trade association maintains the most widely available and documented record for use of Rope Access in the inspection, maintenance, and repair of structures. The available records from 1989 to 1999 indicates that nearly 6,000,000 man hours of work were performed using Rope Access, with a total of 266 incidents. From the total reported incidents, 1 was a major accident, and 25 were reportable under the Reporting of Injuries, Diseases, and Dangerous Occurrences Regulations 1995 (RIDDOR, Approved Code of Practice for the Safe Use of Work Equipment in the United Kingdom). The total rate of the incidents over a period of 11 years in the use of Rope Access is thus 4.6 incidents per 100,000 man hours of work, which is very low. Most of the incidents were minor, none were due to Rope Access techniques, and the majorities were due to falling objects, handling of tools, or blown debris.

Rope Access Case Studies

With a growing awareness and interest, Rope Access techniques are gaining acceptance in the construction industry in the United State and becoming an integral part of any inspection, maintenance, or repair project, joining the ranks of the most traditional means of access, such as boom truck, man lift, swing staging, aerial platform, and scaffolding. Below are some sample projects that highlight the various advantages of Rope Access.

Case Study 1 - Inspection

The scope of work was to perform a detailed condition assessment of the various steep roofs of a historic building with complex geometries, 80-m above the street level. Conventional means of access such as pipe staging would have made the investigation cost prohibitive. Rope Access provided the engineers (Figure 2) ease of access, as well as an economical, flexible, and versatile, hands-on evaluation of the roof structure.

Figure 2 - *Case Study 1*

Figure 3 - *Case Study 2*

Case Study 2 - Inspection

The project budget and requirements imposed a preliminary condition assessment of the exterior facade of an historic building occupying an entire city block, with a 110-m tall bell tower, in a short duration. The Rope Access solution allowed engineers to inspect about 20% of the building facade hands-on (Figure 3) in a period of three days. Using conventional means of access would have taken considerably longer at greater expense, and access to the bell tower would not have been practical.

Case Study 3 – Inspection

Rope Access facilitated a successful hands-on inspection (Figure 4) of defects on the exterior facade of a 185-m tall building. Existing house rigs were not operational, and the mast-climbing scaffolding, installed for the facade repairs, could not be operated due to a city-wide ban on the use of these platforms. Rope Access was also used to perform weld inspections on the radio antenna above the same building, 265-m above the ground, and perform the interior inspection of the 185-m chimney liner in the building. The antenna and chimney inspections were performed over a weekend to minimize down time to the radio antenna and the building's heating system.

Figure 4 – *Case Study 3*

Case Study 4 – Maintenance and Emergency Stabilization

Figure 5 – *Case Study 4*

Initially, hands-on inspection of the exterior facade of a 12-story tall, ornamental and historic building occupying an entire city block was completed in four days utilizing Rope Access. Deteriorated conditions of the exterior facade and potential vibration damage from the adjoining construction activity made the need for emergency stabilization of the facade a necessity. Excavation at two sides of the building, a deep wood-framed cornice and lack of proper tie-backs prevented the use of scaffolding, boom truck, or lifts for the repairs. Rope Access technicians implemented the engineers' emergency stabilization and temporary repairs (Figure 5) (removing loose debris, stitching, netting, and banding displaced and cracked masonry) economically and efficiently.

Case Study 5 - Maintenance and Emergency Stabilization

The building, located in an historic section of the city, is five stories and about 25-m tall with a mansard roof. The exterior walls are load-bearing masonry faced on the exterior with brownstone and brick. During a condition survey of the exterior building envelope, engineers identified loose and incipient spalls in the brownstone on the two street elevations and to the rear of the building. Under direction of the engineers, Rope Access technicians performed a hands-on inspection and removed loose and incipient spalls (Figure 6). Pedestrian protection and debris nets were installed along the street elevations, and the technicians installed debris net over a neighboring roof to protect it from potential falling hazards during the scope of work.

Figure 6 – *Case Study 5*

Case Study 6 - Repair

Rope Access technicians assisted the engineers with a condition assessment of the exterior facade of a museum over a period of four days. The condition assessment of the facade identified loose stone modillions below the vertical joint of the cornice. Because of the manicured grounds, safety, security and visual concerns, the client was adamant that technicians minimize the impact of the emergency repair activities on the museum.

Rope Access technicians removed loose modillions, and the balance of the modillions were anchored to the backup and load tested (Figure 7).

Figure 7 – *Case Study 6*

Figure 8 – *Case Study 7*

Case Study 7 – Repair

The scope of work on a 12-m high sea wall was removal of existing brick masonry repairs and replacement with matching brownstone block (Figure 8). The Rope Access technicians contended with changes in tides of up to 11-m (most of the work site was underwater at high tide), and using Rope Access techniques, the technicians had the ability to work with the rising and falling of the tide, quickly moving from lower to higher locations, and resuming work. All equipment and materials had to be removed from the site and contained in secure storage between shifts.

Conclusions

In comparison to the traditional means of access, the advantages offered by Rope Access can make this technique an attractive and viable option for structural and facade inspection, maintenance, and repairs. Outside of the United States, Rope Access has been accepted as an alternative means of access, with an exemplary safety record, meeting the highest level of safety requirements and standards of the regulatory bodies. In the United States, it is only a matter of time for the construction industry to learn, appreciate, and take advantage of Rope Access.

Kent Diebolt,[1] *James Banta,*[2] *and Charles Corbin*[3]

Direct Digital Input of Façade Survey Data Using Handheld Computing Devices

REFERENCE: Diebolt, K., Banta, J., and Corbin, C., "**Direct Digital Input of Facade Survey Data Using Handheld Computing Devices**" *Building Façade Maintenance, Repair and Inspection ASTM STP 1444,* J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

ABSTRACT: Collection, organization and presentation of existing conditions information for facades on large or complex buildings is a complicated and time-consuming task. In addition, analysis of the output can be difficult to interpret due to the large number of data points collected. This paper discusses various methods of collecting and presenting information derived from façade inspections in an effort to increase understanding between all parties involved in a façade restoration project.

KEYWORDS: survey, façade, inspection, iPAQ, PocketCAD, existing conditions, construction documents, investigation, condition report

Introduction

Information collected during façade surveys is recorded and presented in both graphical and numerical, or "database" formats. Each type of information has its own use, and the two are inextricably connected. For instance, numerical and coded information contained in a database are used to develop estimated construction costs and the graphical data may be used to analyze trends or patterns of deterioration around the building. In the process of annotating façade elevations, building surveyors use symbols to represent the faults found on buildings, and these symbols represent numbers, nouns and verbs (Tufte, E. R., 1990, 1997, 2001).

Review of Past Procedures

In the field, building surveyors typically mark up paper elevations, using shorthand codes for various types of material or structural faults identified. Photographs of fault conditions are keyed to the elevations using a variety of techniques to accurately represent their location on the building. Notes on existing conditions and photo location identifiers are typically superimposed over the drawing or made in the margins. In spite of rigorous training for an entire survey team, the results will vary slightly between individual surveyors, and field notes may be difficult for others to interpret. It is often very difficult for anyone other

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than the technician making the observations to go directly from field notes into CAD, so field notes must be "cleaned up" prior to drafting in CAD by a CAD operator or by the technician(s) who performed the survey.

Base Drawings

Architectural elevations are necessary for accurate notation of façade problems. These drawings should reflect as-built conditions, but in many cases, particularly for historic buildings, elevations are not available. Base drawings may be derived from original drawings in the form of direct copies on paper, using large xerography machines. Original documents may also be scanned and saved as digitized, raster-based files. Simple rectified photography and more sophisticated photogrammetric techniques are especially useful when existing drawings do not reflect as-built conditions, providing the entire building can be photographed, preferably from the approximate mid-point in height. Simple rectified photographs are not scalable, but photogrammetrically derived images are, with additional manipulation.

The raster-based output of scans and rectified photographs are incompatible with vector-based CAD programs. However, a program such as Raster Design (formerly CAD Overlay) from AutoDesk may be used to trace or superimpose vector-based lines over the raster image, deriving CAD-capable files from that information.

Another technology now in frequent use for architectural applications is laser scanning. This technology uses the return time of a reflected laser beam to determine the three-dimensional shapes being scanned. Again, the output is raster-based, and vector-based, "hard-line" drawings are made using proprietary software similar to Raster Design, mentioned above.

In situations where raster-based images such as rectified photographs or scanned documents are used for base drawings, annotations may also be made with a drawing program such as Corel Draw or Adobe Illustrator, rather than AutoCAD.

Documentation of Existing Conditions

In the documentation of existing façade conditions, various forms of data and media must be coordinated. Careful collection and organization of detailed information is essential to the goal of preparing comprehensive construction documents but can be time consuming and difficult to manage depending on the complexity of the project. Photographs and photo numbers must be correlated with their respective locations on the building as well as with notes regarding materials and conditions, and the extent of deteriorated conditions.

In most cases, multiple sets of associated photographs providing supporting documentation are sorted and collated in the office, prior to labeling and insertion into archival sleeves. One problem endemic in a final product that includes a separate volume of photographs keyed to elevations is that while turning to a sequentially numbered photo cited at a specific location on the elevation is straightforward, it is much more difficult to find the location of a photograph on a complex building when perusing the volume of photographs. An associated problem is that photos taken in the field by different technicians are rarely in sequence around the building, so sequentially numbered images in a volume of photos may appear randomly arranged.

The process of field annotation, cleaning-up of field notes and CAD drafting along with photo processing can lead to errors of interpretation and transcription. In addition, these tasks are tedious and time consuming and are often performed by highly skilled personnel who are, in a sense, under-utilized in carrying out such routine and repetitive tasks. Sometimes this is still the most economical approach, although with large-scale projects on complicated structures, management of the graphical, numerical, textual, and supporting data can become overwhelming. Bridge inspectors, for example, are required to take notes on multiple connections on numerous structural members that generate significant amounts of data on large bridges. Bridge inspectors have been grappling with issues of managing complex quantities of data for years.

An added enhancement to the process of documenting existing conditions was the introduction of AutoCAD "attribute tags." Also known as "data blocks," these are essentially database records that become embedded in the CAD file and are identified with specific locations on a building. Fields in the attribute tag or database record contain numerical and/or coded descriptors for existing conditions encountered and may also contain information on the severity or extent of the fault identified at that location. Numerical data in this format is no longer static and may be exported to a database for sorting and manipulation and used as a diagnostic tool as well as for development of take-offs, cost estimates and project phasing options or scenarios. Fault codes may also be reformatted as repair treatment codes for development and implementation of construction documents. Entering the data into attribute tags is generally still a two-step process, though as data collected in the field on paper is transcribed into attribute block format at the office in a desktop computing environment.

Data Management Issues

On large, complex buildings, the sheer density of observations and data points lead to other problems which make existing condition surveys somewhat difficult to use, particularly with multifaceted monumental buildings that may have 6000 or 8000 data points which obscure each other graphically on the drawings. Overlapping conditions combined with the sheer number of data points and the number of drawings required to represent the building at a "plottable" scale may preclude clear analysis and interpretation of patterns of deterioration based on the graphical data.

Especially for complicated monumental buildings, project deliverables at the discovery phase may run into tens of sheets of drawings for each elevation showing different classes of information. Examples of seven different conditions classes on a recent project included Stone Joint Conditions, Stone Patch Conditions, Stone Crack Conditions, Stone Spall Conditions, Soiled Stone Conditions and Cast Iron and Sheet Metal Conditions. The project required complete views of each elevation per building, for a total of 84 sheets of elevations to show all of the conditions classes. Given this situation, the only feasible way to do the analysis and diagnosis of existing conditions was on-screen, using AutoCAD "layouts" to switch selected layers off and on.

Alternative System Model

Like many others who routinely conduct façade surveys, we began to explore better methods for inputting and presenting our graphics and data. This was in 1995, when pen-based, hand-held computers were in their infancy, and there was very little in the way of software to support the notably underpowered hardware available at the time.

As originally conceived, a system integrating data collection in digital format, presentation and interpretation would save technicians time both in the field and in the office and would make condition reports more accessible and more useful for our clients. Ideally, we would be able to input our notes directly into AutoCAD "attribute tag" format, embedding all data relevant to problems at a particular location, in a single database record. Additional data that could be embedded in the attribute tag might be a digital photo, digital video clip, narration or a treatment code. This eliminates the work associated with tagging and locating photographs and allows a person to go directly from a digital volume of photographs to the exact location on the drawing where that specific photograph was taken. Conversely, a photograph at a specific location could be viewed by simply opening the attribute tag associated with that location.

While the notion of what I termed "direct digital input of facade survey data" remained a pipedream, others were making strides, using simple database programs that were becoming available at that time, specifically, running on the Apple Newton.

Other Approaches

IBIIS

A highly evolved iteration of one of these early database programs used for bridge inspections is still available, and was the brainchild of Avanti Shroff of Iffland, Kavenaugh and Waterbury (now Edwards and Kelsey). The program, known as Integrated Bridge Inspection Information System (IBIIS) is a database set up to conform to Pontis bridge inspection protocols. While it is formatted specifically for bridge inspections, one could imagine applying the same approach to façade inspections. With IBIIS, field data is collected utilizing personal digital assistants (PDAs), digital cameras, and video. All information collected is digitized and stored in a multimedia database that differentiates every beam, girder, bearing, etc., on each span of every bridge. The system generates reports automatically, placing deficiencies in three categories: "Reportable," "Needs Attention," and

"Critical." IBIIS is a completely paperless alternative to traditional bridge inspection systems. The database also integrates the inspection process with the maintenance and planning processes. In addition to the data from previous inspections, recent repairs are documented in the database and available to the bridge inspectors.

"Palm" Handheld Computer or Personal Data Assistant (PDA)

Michael Petermann of Wiss Janney Elstner Associates and Dean Koga at Building Conservation Associates, both in New York City, have recently applied similar database-driven systems to the study of building facades, with a few variations. Both use Palm OS handheld computers, running HanDBase and Microsoft Access in a Windows desktop environment. In both cases, elements of a building to be surveyed must each be assigned unique identifying numbers prior to surveying, in order to locate each fault firmly on the building elevation. This identifying number links individual fault incidents to a specific database record for a specific location on the building. Generally, numbered elements are a stone unit or façade panel rather than a region or area on the building, so a perfect understanding of the as-built construction of the building is necessary prior to conducting fieldwork. In addition, it is somewhat difficult to indicate broad rather than specific areas such as open masonry joints or a particular type of soiling.

HanDBase files are readily formatted to allow user-customized, "pull-down" menus containing information about the location of the database record on the building, the fault parameters and associated photo identification number. Conversion to Microsoft Access-compatible files requires some minor in-house software authoring, however.

In both cases, elevations on paper with uniquely numbered individual elements are still taken to the field, for reference and graphical annotation. Markups in CAD are transcribed back in the office in a desktop environment. For a large project at the Empire State Plaza in Albany conducted by Wiss Janney Elstner Associates, approximately 1000 sheets of drawings were taken to the field to record conditions observed on approximately 125 000 marble veneer panels.

Once field data is entered into the database, fault types may be converted to repair codes, with associated unit costs applied to the totals for job costing and phase planning. After drafting into CAD, Building Conservation Associates uses AutoCAD Map to analyze their data through database records linked to locations on the drawings. This GIS (Geographic Information Systems) software allows for database queries to be displayed graphically on the desktop or for plotting onto paper and greatly simplifies generation of reports.

These are very powerful analytical tools, but multi-step inputting processes are still required. A key element that remains unaddressed by this approach is the seamless and direct connection between both graphical and numerical or database components of the data. Furthermore, the data collection system should also accommodate information regarding future repair decisions, treatments, or follow-up surveys.

PocketCAD and the Compaq iPAQ

Several years ago, a software product known as PocketCAD published by Arc Second appeared on the market. Running on hand-held, pen-based computers in a Windows CE environment, this program is fully compatible with AutoCAD and other CAD software programs that are capable of reading files in a ".dwg" format. Using the stylus, any of a number of commonly used drafting "tools" such as lines, points, circles and squares are used to annotate vector-based CAD drawings. Attribute tags containing information about the fault may be directly associated with these specific "icons" and are embedded in the drawing for subsequent extraction and manipulation in a database format.

This system appeared to be the first viable combination of software and hardware allowing the integration of database capabilities and graphical representation of the database information, in the field. The hardware required to run the software in the field is compact, robust in construction and capability and is available at relatively low cost.

Vertical Access technicians first used this program on a project that necessitated making notes on and identifying photograph locations for about 2500 faults on a historic stone building. The system worked well, but entering the data into attribute tag format was clumsy, and while there were some savings of time in the office, fieldwork was slowed down substantially.

The experiment was a success, however, so Arc Second was approached to see if they would write a custom add-on developed specifically for façade surveys. The program as specified would allow for user-customizable drawdown menus that streamlined the data input process and provided automatic behind-the-scenes layer management. Arc Second agreed to undertake the project, for a fee, and we began by developing the specifications for what became known as the Exterior Survey Application or ESA.

When using the ESA, the computer stylus is used to select a drawing tool, causing a drawdown menu to appear, allowing for specification of the criteria associated with that fault location (Figure 1). These criteria are automatically entered into database fields. Four fields are currently available, and hold information on the type of material, class of fault, fault severity or extent, and a photo identification number. The last field may also be used for notes on outlying or unusual conditions (Figure 2).

Upon completion of the menu selections, the menu screen closes and a stylus is used to place the icon on the elevation in a format appropriate to the fault type and extent (Figure 3). The fields completed in the menu-driven database records are stored as attribute tags and are associated with the location on the drawing where the icon has been placed, but are invisibly embedded in the drawing to avoid on-screen clutter.

Figure 1 : Drawing tool pallet

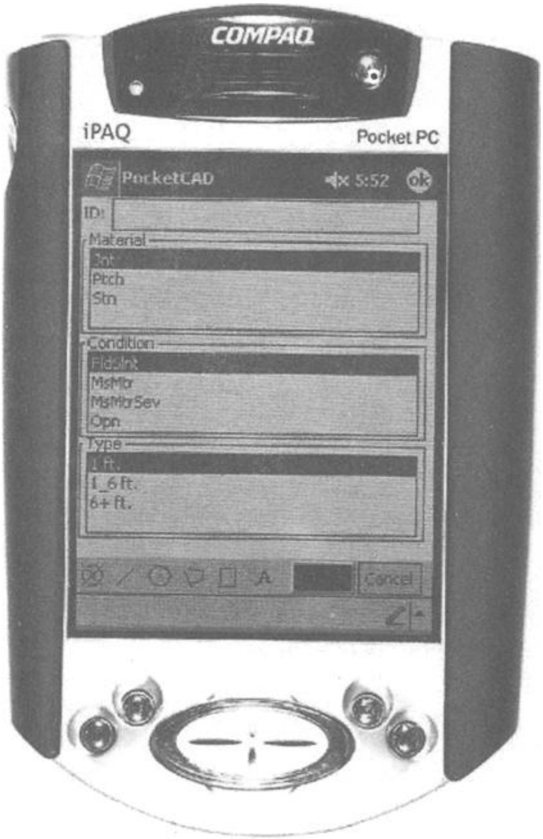

Figure 2 : Drawdown fault code menu

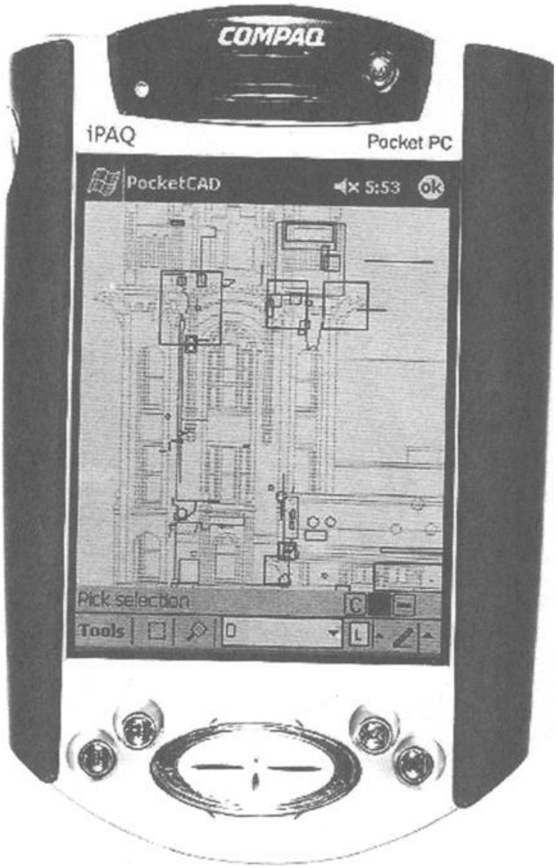

Figure 3 : Graphically annotated elevation

Perhaps the largest benefit of this data input system is that all incidents of each unique permutation of material, fault type, and extent of condition are automatically assigned to a single CAD drawing layer. This allows for the graphical display of selected fault types that may be of particular interest, for example, stone displacement may be looked at independently before adding stone cracks to understand the extent of rusting structural steel that might be addressed in a repair campaign. Within certain limits, this eliminates the necessity of using GIS-type software for visual interpretation and analysis of the data while still allowing the database capabilities of the fault records contained in the attribute tags.

Extraction and layout of the attribute tags on elevations are accomplished later, on the desktop after converting the PocketCAD files to AutoCAD files. Extraction of the data entails revealing the tag on the desktop, dragging the tag to a margin and drafting leaders from the tags to the icons identifying the location of the fault associated with that tag. While office time is still significant, there are several benefits realized, including minimization of interpretive and transcriptive errors combined with automatic layer management.

We had our first opportunity to use the ESA, running on Compaq iPAQ computers on two large buildings in New York, just a few blocks from the World Trade Center site. Our first pass over the building was completed on September 9th, 2001. We re-surveyed the building in the winter of 2001-2002, to determine the extent of damages from the collapse of the World Trade Center Towers. At this time, we had fault code permutations for pre- and post-September 11th conditions resulting in more than 6000 data points distributed between about 600 individual layers.

We have successfully tested the system on several other buildings, but problems associated with file management remain and PocketCAD/Arc Second is no longer supporting the Exterior Survey Application (ESA). As a result, some of the supplemental features that we planned to develop in later iterations of the software are unrealized, including the insertion of hyperlinks to supporting data such as digital photos or video clips and voice narration. At this time, we are still sorting photographs, either by hand or on the computer, for digital images.

Pocket CAD and iPAQ Pros and Cons

Like any emerging application of technology, there are both assets and liabilities associated with adoption of that technology.

Advantages

Because all surveyors using the system are constrained by the same conventions, use of menu-driven databases unquestionably has a distinctly homogenizing effect on the data as recorded between different technicians or surveyors. In addition, transcriptive and interpretive errors in both the graphical and numerical data are minimized as the information flows from the field into the office. Placing data directly into attribute tag format saves significant time over entering that information in the field and transcribing it into AutoCAD in the office. Finally, automatic layer management is an enormous advantage, particularly as it applies to analysis of graphical information, without reliance on unfamiliar CAD plug-ins with GIS capabilities such as AutoDesk Map.

Disadvantages

Like other emerging technologies, significant up-front capital costs are associated with the purchase of new hardware and software. In adopting these technologies, we hope for savings in time and labor, but direct labor costs are applied to individual projects and do not require this sort of capitalization. Costs include iPAQ or equivalent hand-held computers, the PocketCAD program, the cost of the ESA plug-in and a site-based laptop, for file management and backup.

Some of these capital costs are related to ensuring the security of the data while on site, and against the inevitability of electronic equipment failure, theft or damage. To mitigate these possibilities, we back up all data on the laptop to a one-gigabyte microdrive twice a day. The microdrive is carried in the crew leader's pocket or hidden well away in a location remote to the other equipment.

Pre-site CAD time is required to partition the drawings into rational sizes and these must all be installed on the laptop and hand-held computers. Previously, elevations simply needed to be reproduced at an appropriate scale and cut up with scissors or folded to facilitate annotation on site.

In a sense, everything becomes more complicated: there is another layer of battery management required, the data and media are much more ephemeral and everyone on the crew must be facile with both Windows XP and Windows CE operating systems. A number of scenarios can lead to a loss of data due to overwriting of files, making this loss of productivity a real possibility. At this point in time, direct digital input may save time in the field and produce a better product with more and better-organized information, but it does not necessarily simplify the job, especially during the data-gathering process.

The GIS Approach to Façade Condition Mapping

Geographic Information System (GIS) software has been used for a variety of applications relevant to this paper such as the management and inventory of cultural resources, tracking of building maintenance, and mapping of layers of history as in an archaeological site. GIS offers the unique feature of associating numerous data sources with various objects within a map. A variety of data can be linked to but remain hidden from view unless called upon. For example, a map with a river could have a video or audio clip associated with it that is opened up from within the image of the map. Text information, photos and various databases may be linked to map information using GIS much in the same way that a database may be linked with AutoCAD files.

The advantage of GIS technology for facade mapping has great potential. Instead of referring to separate drawings, attribute tags, photos, videotapes, and written reports, all this information is linked together in a relational database using GIS. Furthermore, GIS software has the ability to read and work with AutoCAD files.

So why not use GIS to "map" existing conditions on a building facade? Although it can be done, to our knowledge, GIS has not been used in this way. One limitation is the difficulty in translating GIS data back to a useful format for architects and engineers, namely back to AutoCAD or another familiar CAD program. In our experimental trials converting GIS data back to AutoCAD files, the attribute tags and layer structure established in GIS are not preserved in the conversion back to AutoCAD.

Because AutoCAD is the standard in the architecture, engineering, and construction industries, there is little practical use for GIS unless the client already works with GIS. Users of GIS are generally Departments of Transportation, the National Park Service, or other planning or infrastructure agencies.

Vertical Access is in the process of working out ways to utilize the native, graphics features of AutoCAD along with some required customization to obtain some of the database linking advantages of GIS. Of great potential is AutoCAD's "hyperlink" command that can be used to connect a photo, text file, website, video clip, etc. with any object in a drawing. A website or CD can be the source destination for the hyperlinked photos, video, etc. In either case, in opening the hyperlink, AutoCAD would look to website or local hard drive / network where the digital files are located.

This system could be a very nice way to package the final product of a conditions survey having the advantage of all information being accessible from the AutoCAD drawing and in a digital format (with no paper necessary). However, a limitation to using hyperlinks is that AutoCAD Release 14 and older versions do not support "hot" or active hyperlinks. In other words, it was not until AutoCAD 2000 (released in 1999) that hyperlinks were made "live" so that simply clicking on an object in the drawing opened up embedded information automatically.

In order to streamline this process of data collection in the field, customization or scripting of AutoCAD is required. The ESA program or other field data collection software would have to be modified to accept the file name and location for associated photos along with the attribute tags entered in the field.

Then, in the extraction from PocketCAD to AutoCAD, the custom programming would automatically create the hyperlink between the photo file name and the location of the photo on a website or other location. This would save the step of having to create the hyperlink for each photo, text file, video, etc. separately in AutoCAD.

However, the issue of getting the photo to its hyperlinked "home" remains. How do we shoot a digital image in the field and send it to its hyperlink destination? The digital camera's data could be synched to the on-site laptop after each drop and then uploaded to its website home, or saved and placed on to a CD ROM. Or we can dream a bit bigger and think that there may be a way to link the digital camera with the computer. One model of the Sony Viao laptop and the ViewSonic ViewPad 100 tablet PC both have built-in digital cameras that can also take video clips and have wireless web connection capability. The obvious combination of camera and handheld computer may be of great advantage for the goal of directly linking digital photo and AutoCAD object, but it remains to be tested in the field and may prove to increase survey time required for file manage while on site. Furthermore, there are significant limitations of using these tablet PC models in the field due to insufficient screen brightness and contrast.

Conclusion

In the collection, organization, and presentation of façade survey data, it can be challenging or cumbersome to coordinate the review of drawings, attribute tag information, associated photographs, video, and written reports. However, simultaneous review and synthesis of these different sources of information is necessary for understanding the conditions of a building, diagnosing problems, and preparing treatment or repair solutions.

If we think of façade survey data as various elements representing numbers, nouns, and verbs, we must remember that clutter and confusion in their legibility are failures of information management and presentation, rather than the inherent quality of the information recorded. Not only does the building professional have to collect the data in an efficient and concise manner, but the information must also be presented in a clear and useful format.

The future of façade surveying and the preparation of construction documents appears to be headed towards the reduction of paper. Digital or electronic formats of raw field data, comprehensive conditions reports, construction drawings, and specifications eliminate the need for printing, copying, and shipping, thus reducing production time and cost to both the building professional and client.

In summary, this is a fascinating time to be engaged in the development of systems for direct digital input of façade survey information. The emergence and dissemination of new technologies and the greater power and flexibility of mobile computing is rapidly changing the fields of Architecture, Engineering, and Construction. Hardware and software development is leaping forward as computers become less expensive, more capable, and more robust. In addition, we are seeing the convergence of CAD and GIS capabilities, which will revolutionize the way we analyze and interpret survey data and influence how we prepare construction documents. Furthermore, as these technologies grow, so will the number and type of its users. Although there are still unanswered questions and areas for improvement, the development of handheld computers has made an impact on the work of building professionals by creating more effective means of directly recording, interpreting, and managing façade survey data.

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Seeing and Photographing Your Visual Observations

Reference: Petermann, M. A., **"Seeing and Photographing Your Visual Observations,"** *Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444*, J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: Light can play tricks on your eyes. Deficiencies that are observable at 1:00 pm are not necessarily observable at 3:00 pm. Depending upon the nature of the light, sunlight vs. daylight, an inspector may or may not observe a bowing stone panel. Depending upon the building facade material, its configuration, or texture, light may or may not illuminate a facade deficiency. Ornate facades cast shadows in sunlight. A crack or spall can be lurking in those shadows. The inspector's choice of binoculars, experience, and possibly his psychological disposition may influence whether or not he observes an existing facade deficiency. Recognizing the deficiency may truly be a challenge, but photographing the deficiency may be a greater challenge. The crack or spall that lurks in the shadow on a sunny day may be nearly impossible to photograph. Excessive sunlight that reflects off the facade may wash out a photograph. Automatic cameras typically grab all surrounding light for a photograph but this may be too much or too little depending upon the item to be photographed within the frame. The ability to set your camera manually may be necessary. This paper discusses the nature of light, suggestions for observing building facades, and for recording deficiencies.

Keywords: light, sunlight, daylight, tangential light, borescope, single reflex lens cameras, shade, shadow, facade ordinance

Introduction

In recent years, several well-publicized facade failures have caused considerable property damage, injuries, and in some cases, a loss of life. These tragedies have focused the public's attention on dangers inherent in aging buildings and have given rise to facade ordinances that require critical examinations of building facades. The first step in performing a critical examination is to observe the building's facades. The process of observing a building facade is not defined completely in the existing ordinances. This critical technique is left to the discretion of the inspector. Because the act of visually observing something is considered an everyday event for a majority of us, very little attention has been given to how we look and see as part of a facade inspection.

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Having the correct level of light, an understanding of perception, good visual aids, and photographic equipment is critical in performing a facade inspection. When an inspector visits a building, he typically looks at the building, takes some photographs, returns to the office, and reviews the data to prepare a report. Known deficient conditions that the inspector saw might not be contained in the data. The inspector may be at a loss because he looked at the facade but did not see the condition, or he saw the condition but it did not appear on the photographs.

Light

Light is electromagnetic radiation that is detectable by the human eye. When analyzed as a wave, light is typically defined by the length of its wavelength, which is the color of the light; its amplitude, which is its brightness; and the angle at which it vibrates, which is known as polarization [1]. The retina of the human eye absorbs electromagnetic radiation through receptors known as cones and rods. Rods are prevalent away from the center of the retinal area while cones dominate the central area. Rods are very sensitive to low light levels and movement while cones are active at higher light levels and discern between red, blue, and green wavelengths [2]. While visually scanning a facade, good peripheral vision will assist in drawing the inspector's attention to unusual conditions.

For the purpose of facade inspections, there are two basic types of light that render a building facade–daylight and sunlight. Daylight can also be referred to as indirect sunlight. A facade rendered in daylight is simply a facade with uniform low light intensity casting few shadows. By contrast, a facade rendered in sunlight is a facade illuminated with direct sunlight. If the facade is articulated with varying planes or features, such as ornate masonry, clearly defined dark shadows will be cast in sunlight.

When the sunlight is at a very low angle of incidence to the facade, it is then considered tangential to the facade. For the purpose of facade inspections, the author proposes referring to this type of light as tangential light–light that is nearly parallel to the facade. Facades are illuminated by daylight and sunlight at varying times throughout the year due to changes in the sun's altitude–the angular distance of the sun's path above the earth's horizon (solar elevation). In the northern hemisphere, the summer months typically provide sunlight on all sides of a subject building. During winter months, only three facades of a typical four-facade building will receive sunlight. The varying positions of the sun and the duration of sunlight that any building facade may receive can be predicted using a sunpath diagram (Figure 1) [3]. For example, sunlight striking a building on 21 June that is located at 43 degrees north latitude will have an altitude of approximately 73 degrees above the horizon. Sunrise will be approximately 121 degrees east of south while sunset will be approximately 121 degrees west of south. The sun's path will produce an arc across the sky slightly greater than 240 degrees. As a result, all sides of the subject building will receive sunlight sometime during the day. A true north facade will receive sunlight for two hours in the morning and two hours in the evening and the angle at which it strikes the facade will be no more than 35 degress. By comparison, only three of the four subject building facades will receive sunlight on 20 December when the sun's altitude is only 25 degrees above the horizon and its arc is approximately 114 degrees (57 degrees east and 57 degrees west of south).

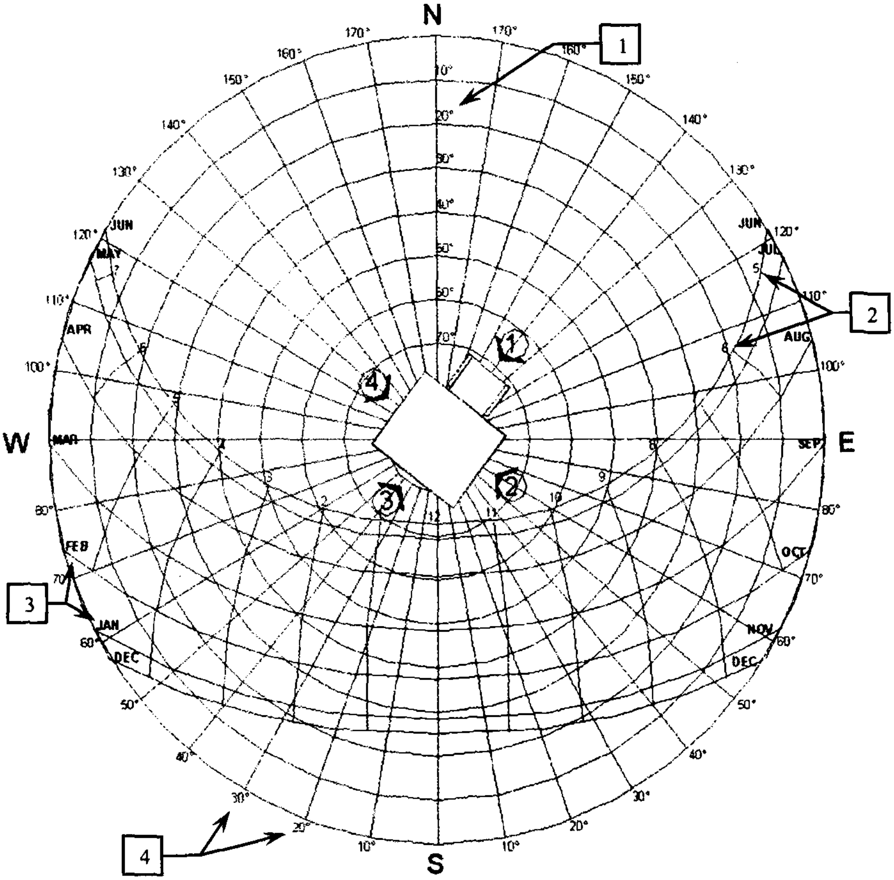

Figure 1 - *A sunpath diagram with a building footprint at 43° north latitude. Note that between April and August, the sun rises and sets north of east and west respectively, meaning that north facades will be in sunlight for a couple hours each day. Variables/contours of a sunpath diagram are as follows: 1. Solar Elevation; 2. Hours in a day 3.Sun's Path on 21st of each month; 4. Azimuth (angle of sunrise and sunset from south)*

Perception and Gestalt Theory in Psychology

Finding observable deficiencies on a building facade will not only depend upon the type of light and the ability of the inspector's cones and rods, but it will also depend upon how the inspector perceives the visual information. Perception, as defined by Henry Gleitman in his book Psychology, is how we come to apprehend the objects and events in the external reality around us [4]. How do we come to see not just a rough, rectangular, gray object, but perceive a concrete panel [5]? Gestalt theory, as founded by Max Wertheimer, tells us that forms are not perceived by summing individual stimuli (rough + rectangular + gray), but that the object is experienced as a whole. We perceive certain relationships among the individual stimuli and recognize a pattern we have seen before [6]. The inspector's perception of a facade deficiency and of the pattern it may form will require some level of experience to recognize it. Experience and psychological disposition will further influence the inspector's perception (Figure 2).

Figure 2 - *Ambiguous woman is a well-known study performed by psychologists on perception (Gleitman). Our experience will influence whether or not we see the young or old woman.*

Another concept to consider in Gestalt theory, which describes how perceptions are organized, is *proximity*. Proximity in Gestalt theory states that "the closer two figures are to each other (proximity) the more they will tend to be grouped together perceptually" [7]. It should be noted that proximity is not limited to a spatial relationship but may also occur in time [8]. Two cracks in a brick masonry veneer 10 ft apart may be related as deficiencies that occur at a repetitive condition, such as a shelf angle, however they may or may not be related in time. One crack may have caused the other. It should also be noted that in order to observe spatial proximity, one must be at sufficient distance between the two conditions. In order to determine time proximity, one may need to consult building staff for reports or analyze/test the deficiency area for information on age, such as carbonation depth of concrete or debris deposits within cracked surfaces.

Another way in which deficiencies are grouped is known in Gestalt theory as *good continuation,* which implies that "our visual system seems to 'prefer' contours that continue smoothly along their original course" [9]. As noted by Gleitman, "good continuation is a powerful organizational factor that will often prevail even when pitted against prior experience" [10] (Figure 3). Prior experience may inform you that there are potential delaminated surfaces with sedimentary stone that is face-bedded on a facade; however, the natural appearance and/or sculpting of the stone may camouflage the deficiency (Figure 4).

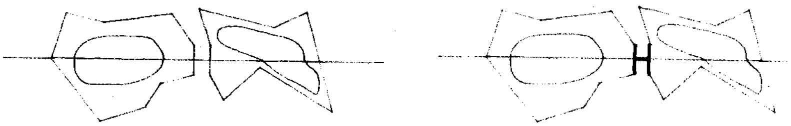

Figure 3 - *Even though the letter H is well known, good continuation within an image will make it difficult to identify items such as the letter H.*

Binoculars

Figure 4 - *Good continuation of the lines in this sedimentary stone camouflage separations in the stone's layers.*

Most visual inspections begin with an overall observation of the building facades and are conducted using binoculars. Binoculars vary in quality and can hinder or assist in making observations. As stated in many consumer buying guides, binoculars are typically identified by three numbers: the magnification, the aperture opening (lens diameter), and the field of view. The author typically uses 7x 35 mm, 8.6 degrees for observations less than 60 m (200 ft) away. The magnification simply means how much larger or closer the image will appear through the binoculars as compared to the image without binoculars. It is typically expressed with an "x" after it indicating multiplication, in the author's case, 7x. It should be noted that higher magnification for a given aperture diameter typically results in less brightness and angular field of view. In addition, greater magnification requires the inspector to hold the binoculars extremely steady to observe details. A tripod may be required at greater magnifications for observations greater than 60 m (200 ft) away. The aperture opening is the diameter of the objective, in the author's case 35 mm. It should be noted that the larger the aperture, the more light that comes into the binoculars. More light will assist in making the image brighter and clearer. The field of view is typically expressed in angular form but can also be expressed linearly across the image. The field of view is typically rated at 914 m (1000 yd), with one degree of arc representing 16 m (52.5 ft). In the author's case, 8.6 degrees represents an image width of 136 m (451 ft) when viewed from 914 m (1000 yd). The larger the angular degree, the greater the image width viewed in the binoculars. This may help locate an observation so that it can be noted on an elevation drawing [11].

There are many more features to binoculars with which each inspector should become familiar prior to selecting his or her binocular of choice. These features include light transmission, phase correction, and image stabilization. For light transmission, 90% or better should be sought. This means that more than 90% of the light that enters the binocular travels to your eye. This is achieved in quality binoculars by coatings applied to the lenses and prisms to reduce reflection. Phase correction is a feature that forces light that has been split into two out-of-phase beams to reconnect into the same phase prior to reaching your eye resulting in a better quality image. Image stabilization is recent technology now employed by manufacturers to limit image shake in high-powered handheld binoculars for observations greater than 60 m (200 ft) away. This is achieved through gyroscopes that detect motion, a computer chip that calculates displacement, and a movable lens that maintains image control. Image-stabilized binoculars require batteries and are expensive compared with traditional binoculars. The July 2002 cost of an 18x 50 mm, 3.7 degrees image-stabilized binocular was approximately $1100 [12].

Cost and technology aside, a very important factor to consider when selecting a binocular is feel and weight. Binoculars should comfortably fit the user's hands and eyes. Weight should be minimal because the user will be holding the binocular for hours per day while observing facades at various angles. The author's binocular, at 680 g (24 oz), weighs approximately half the weight of the image-stabilized binocular.

Cameras

In addition to binoculars, the author recommends using SLR (single-lens reflex) cameras with both automatic and manual settings. While the development of digital photography may eventually make SLRs obsolete, current digital technology does not match the flexibility and quality of an SLR camera when taking the photographs necessary for a facade inspection. In addition, some courts of law may not accept digital photographs because of the possibility of manipulation.

SLR cameras function like the human eye; they have a lens, an aperture opening controlled by a diaphragm–similar to the pupil and iris–and the film is like the retina. Decreasing the amount of light to which the film is exposed by decreasing the size of the aperture opening will result in photographs with greater focus and depth of field. It should also be noted that decreasing the aperture opening will require decreasing the shutter speed so that enough light can pass through the restricted opening to record the image. A slower shutter speed may require a tripod in order to hold the camera steady while photographing. It is also recommended that on sunny days (in the summer) slower speed film be used and on darker days (in the winter) a higher speed film be used. The author typically uses 400-speed film in the winter and 100-speed film in the summer. The 400-speed film will produce grainy photographs as compared to 100-speed film.

Most new SLR cameras come with an automatic setting, which permits the user to point and shoot. In many cases this will be satisfactory; however, the inspector needs to realize that when the frame of the picture contains a shadow on a sunny day, deficiencies in the shade will most likely not appear. The inspector will have to switch to a manual setting on the camera. Using a zoom lens and focusing only on objects in the shade, the inspector can measure the recommended shutter speed and aperture setting with the light meter in his SLR camera while on the automatic setting. He can then zoom out of the image, change his camera to manual, and input the setting recommended for the shade. Most SLR's also have an automatic exposure, known as an AE Lock button, that allows you to zoom out and recompose while maintaining the zoomed exposure setting. The bright sunny area of the photo will be washed out, while the shaded area should be the right exposure. The inspector should consider shooting the same picture two more times, once each with a higher and lower aperture setting.

Photography utilized for facade inspections typically include zoom photography, standard/wide angle photography, and close-up photography (magnifying filters and borescope photography). The author uses a 100-300 mm zoom lens for buildings less than 60 m (200 ft) in height. Higher-powered zoom lenses are also available; however, the higher the zoom, the steadier the camera must be held and the more light that is required. A tripod, shutter release, and higher speed film may be required for zoom photography over 300 mm. For buildings taller than 60 m (200 ft), a camera attached to a spotter scope is used where necessary.

Tall buildings, especially in dense metropolitan areas, are a challenge to capture in a single photo. The author typically uses a 28-80 mm lens for standard photos. A tall building can be captured on multiple pictures and then spliced together. As an alternate, the author also uses a 15 mm wide angle lens to capture buildings more than 150 m (500 ft) in height in a single photo. This typically results in distortion at the perimeter of the photo, which is commonly referred to as "fisheye". Some computer applications, such as Adobe Photoshop, have perspective correction features but such features do not correct fisheye distortion. Fisheye type lenses are good for providing context such as an overall picture of a subject building facade or if there are several deficiencies spread across a wide area.

During detailed observations, a macro lens and/or magnifying filter may be used to permit close-up photography and an enlarged image of the deficiency being recorded. Such photographs tend to emphasize a condition of the deficiency such as the layer of rust on a hanger support for terra cotta (Figure 5).

Another type of close-up photography that can be used is borescope photography. Borescopes are devices that typically contain a fiber optic shaft or cable and an adapter for a SLR camera. This type of device permits one to view into cavity wall systems by drilling a small hole, typically less than 13 mm (1/2 in.), and inserting the shaft or cable. Newer devices are available consisting of fiberoptic cables connected to video recording devices. This permits one to film the interior portion of the wall cavity. The film is then converted to a video computer file and various images can be captured by a desktop computer to illustrate a point (Figure 6). The advantage to filming the cavity is that it gives context for the deficiencies that are found.

Figure 5 - *This photo was taken with magnifying filters. Note the multiple rust layers of the hanger support.*

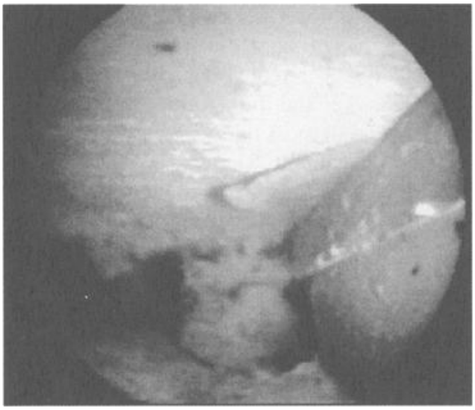

Figure 6 - *A borescopic photograph of a joint condition between two stone panels. The object at right in the photograph is a plastic shim that was left in place during construction and is now causing increased stress in the stone panels.*

Performing a Facade Inspection

Preparation

Prior to inspection day, the inspector should obtain a plan of the building with a true north reference arrow so that the position of the building relative to the sun can be established. If a plan is not available, then a compass can be used. Once the position of the building's footprint is established and the latitude is known, then a sun path diagram can be used to determine the duration of sunlight each facade will receive during the day of inspection. The inspection of each facade can now be timed to follow the sun.

In addition to anticipating the path of the sun, existing drawings should be obtained and reviewed so that one can have a mental image of the exterior wall details while seeking and observing the facade deficiencies. Information gathered from the construction drawings may provide some direction in searching for specific types of deficiencies, such as the possible relationship of cracked or spalled masonry to embedded steel components. If detail drawings are not available, then documentation from historical publications may be useful. Ultimately, probing of the facade may be required to establish typical existing details. However, such probing is usually deferred until after the initial visual survey and scaffold inspection.

Elevation drawings or enlarged copies of elevation photographs, also known as background drawings, should be prepared prior to inspection so that individual deficiencies can be noted on the drawings/photographs. Recording the deficient conditions on the background drawings/photographs will permit easier recognition of any possible patterns of failure (Figure 7).

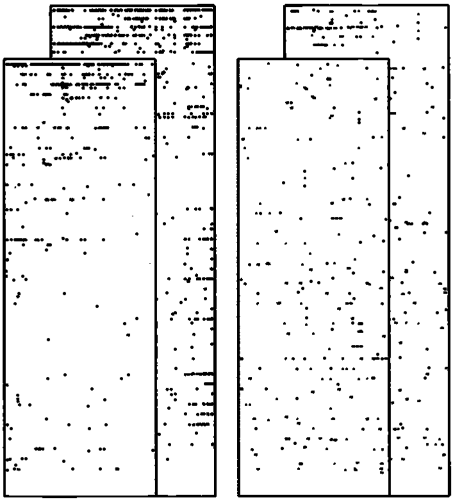

Figure 7 - *Identifying deficiencies on separate layers in Autocad permits selective printing in order to see patterns or proximity of deficiencies. The dots on the image at left show an increased density of cracks along the top and right side of the building, while the spalls shown in the image at right are generally more sporadic.*

Observations

Start observing early in the morning to take advantage of available light, especially in the summer time when sunrise is from the northeast. Note the time and position of the sun. If a shadow is cast from a corner of the building, estimate the angle of the shadow relative to the plane of the facade. Keep in mind that the earth rotates 15 degrees per hour so if the shadow edge is approximately 30 degrees off the plane of the facade, it will be 2 hours before the sunlight is tangential to the facade (Figure 8).

Time the observations accordingly. Plan and pace the inspection so that the facade can be seen under the best conditions. Prior to scanning a specific facade, walk the perimeter of the building noting possible vantage points and overall conditions. Take overall photographs of each facade.

Select a facade bathed in sunlight to start with. Assume a position at a horizontal distance that permits viewing the facade at approximately a 45 degree angle or less (Figure 9). Using the binoculars, scan the facade vertically and horizontally. Do not ignore objects seen peripherally; attention should be given to such objects. Be mindful of the wall details and Gestalt rules, such as proximity and good continuance (potential for camouflage), while observing. Question the observations, do not become a recording device. The observed deficiencies are symptoms of a failure. Experience and attention to these items will assist in identifying a cause of failure. However, be cautious not to jump to conclusions based solely upon previous experience. Use various symbols to note each deficiency on the elevation drawings. Photograph each unsafe condition and at least one representative photograph of all other conditions. Note the roll of film and frame number next to the observation. While framing each photo, note if there are any shadow lines that may prohibit proper recording of the deficiency. Adjust the camera manually, if necessary, to obtain the exposure needed to photograph the deficiency (Figure 10).

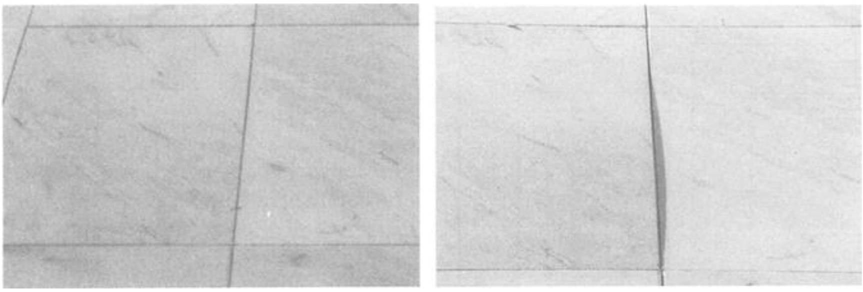

Figure 8 - *These photographs depict the same two panels on a north facade at different days and different times. The photograph at left was taken during daylight while the photograph at right was taken during tangential light.*

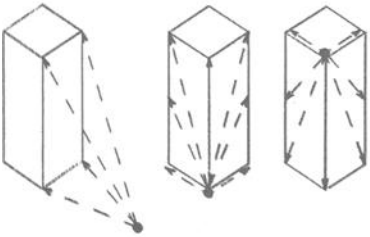

Figure 9 - *Various positions for observing a facade with binoculars.*
Illustration by Nathan P. Walker

Figure 10 - *Strong shadows cast by a railing obscures observation of the cracked concrete curb.*

After recording observations at a distance, assume a position at the base of the building and scan vertically up the facade from various points along the base using the binoculars. Carefully scan for out-of-plane movement. Note that this typically requires constant adjustment of the focal length while scanning up the facade from the base of the building. Photograph and note out-of-plane movement on the background drawings/photographs. Repeat the same procedure of scanning the facade from the roof and other vantage points. While at the roof, also scan horizontally along the parapet to observe for displacement (Figure 11).

Continue monitoring the time to ensure that the other facades are observed when tangential light first washes across them. Keep in mind that tangential light may only last approximately 20 minutes at each facade since the earth will rotate 5 degrees in 20 minutes. Depending upon the severity of the deficiency, some conditions may cast shadows for a longer period of time. The inspector may need to quickly photograph each condition and wait to record notes after tangential light has passed.

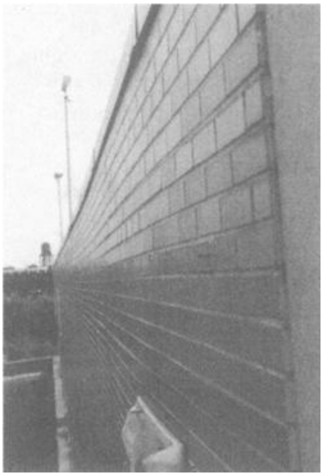

Figure 11 - *An undulating parapet was observed only when looking on edge as depicted in this photograph.*

Limitations and Close-up Inspections

Although plenty of information can be obtained through keen visual observations, close-up inspection via scaffolding is required because of the limitations of distant observations. Information gathered from distant visual observations, existing drawing review, and available history from building staff should be considered to select the best location for a close-up survey via scaffolding.

Some of the same techniques employed to perform the distant visual survey can be used during a scaffold inspection. In New York City, the author typically uses binoculars during scaffold inspections to view horizontally across a facade while moving vertically down the facade. Depending upon scaffold location, one may have the opportunity to view other facades from multiple angles. In addition to these techniques, macro, magnified, and borescope photography should be considered.

Macro and magnified photography communicates the fine details of an observation. Some SLR cameras come with an automatic close-up setting, which permits detail photography similar to a macro lens. The close-up option on the author's camera permits an image to be focused from 20 cm (8 in.) away using an 80 mm zoom lens. As an alternative, the author also uses magnifying filters that permit 7 times magnification of an image at a 15 cm (6 in.) focal length, see figure 5.

Use fiberoptic borescopes during detailed close-up inspections to view inside wall cavities. This will permit the inspector to observe concealed connectors in a cavity wall system. Assistance from a contractor is typically required for scaffold access and to drill and patch the necessary probe holes, usually less than 13 mm (1/2 in.) in diameter. It is always good practice to photograph or film the insertion point of the borescope so that orientation can be maintained. Some borescopes come with a mark in the lens so that the viewer can maintain orientation.

Conclusion

Seeing and photographing observations of facade deficiencies can be a challenge. Since the acts of observing and photographing are considered simple everyday events, extra consideration should be given to how and what is seen, and how it is photographed. The visual tools used and possible psychological disposition can assist or hinder an inspector. Some of the factors that can hinder the inspector include not having the right light, insufficient experience, being clouded by too much experience, being physically too cold or hot, and deficient cameras or binoculars. Some of the factors that can assist the inspector are being mindful of the task at hand, questioning the observations, knowing when there is sufficient and appropriate light, being mentally prepared, and utilizing good, functioning equipment.

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Integrating Advance Evaluation Techniques with Terra Cotta Examinations

REFERENCE: Gentry, T. A., and Davis, A. G., **"Integrating Advance Evaluation Techniques with Terra Cotta Examinations,"** *Building Façade Maintenance, Repair, and Inspection, ASTM STP 1444,* J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

ABSTRACT: Fundamental evaluation techniques for terra cotta facades, which include at-arms-length visual review and mechanical sounding, are unsurpassed in determining the existing condition of individual terra cotta units. However, extrapolating the information gained from these techniques to determine the overall condition of assemblies and potential problems is problematic at best and often speculative in nature. Advance evaluation techniques, including impulse response and fiberscope optical survey, can readily identify conditions that do not present themselves to fundamental techniques. These conditions include; excessive stress due to stacking effect at shelf angles, improperly supported corner units, and corrosion of J-hooks, rods, anchors and ties. When this type of additional information is integrated with the information from fundamental techniques the evaluation evolves from being an inventory of existing conditions to a comprehensive understanding of façade performance. This paper will address some of the special techniques that are available for evaluating the nature and extent of distress in terra cotta cladding systems during the performance of a field investigation. Techniques discussed include the use of optical borescopes, in-situ measurement of stresses and impulse response testing.

KEYWORDS: Terra cotta, borescope, in-situ strain relief testing, impulse response testing

Introduction

Terra cotta cladding systems on historic buildings are characterized by the formation of distress relatively early in the life of the building, and continuing throughout the service life of the building. These problems stem from the original design of the terra cotta system and lack of maintenance during its service life. Terra cotta was thought to be a nearly maintenance-free cladding material, but time has proven this not to be the case.

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Traditional field investigations of existing terra cotta cladding systems include the following steps:

1. Performance of a close-up, hands-on visual inspection of the cladding system from swing stage platforms. This task includes documenting of areas of distress and deterioration in the terra cotta system and severely distressed units that will require replacement.
2. Performance of mechanical sounding of terra cotta units on the building. This is the most common nondestructive method of evaluation for attempting to determine if there are problems beneath the surface of the terra cotta relating to anchorage.
3. Using metal detection equipment to look for embedded metal anchorage devices. This is also a common technique that is used to determine if corrosion of the embedded metal anchors and supports, such as steel shelf angles, is causing the distress that is visible on the surface of the terra cotta.
4. Cutting inspection openings into the cladding systems. Inspection openings are commonly used in investigations. These inspection openings allow the investigator to diagnose problems more fully and accurately that are visible on the exterior surface of the terra cotta. The inspection openings are usually made in the regions of observed distress in order to examine embedded metal anchorages, backup materials, and main structural elements on which the terra cotta units are supported.

Advanced evaluation techniques are available to help the investigator delve deeper into the causes and extent of the observed distress in a terra cotta clad buildings. These include:

1. Borescope inspections of metal anchorages hidden within terra cotta units.
2. In-situ strain relief testing to assess stress levels in continuously constructed assemblies of terra cotta such as column and mullion covers.
3. Impulse response testing to assess stress levels in continuously constructed assemblies of terra cotta such as column and mullion covers.

These advanced evaluation techniques are the focus of this paper.

Borescope Inspections

Fiber optic borescopes are used by investigators to allow for visual inspection of anchors hidden inside the voids of terra cotta units. This method is most commonly used for evaluation of anchors that are suspected to be corroded, which are typically located in lintels, dentils and other units that are suspended from metal J-hooks and horizontal rod assemblies. See FIG. 1- *View through Borescope of J-hook without Rod.* Corrosion of the horizontal rods and the J-hooks often leads to cracking distress and displacement of the suspended units, or portions thereof. This type of distress can be life-threatening as pieces or whole units of terra cotta can become dislodged and fall from the building.

The diagnosis of corrosion damage to the suspension system can be readily confirmed in most cases through use of an optical borescope. Two basic types of equipment are available and are different only in the type of arm that is used with the equipment, straight shaft and flexible shafts. See FIG. 2 – *Straight Shaft Borescope.* With both types small holes are drilled through the underside of the unit to provide access for the borescope. Drilling holes through the underside of the unit is essentially nondestructive, because those holes can be used as drain holes for any moisture that may collect in the units. Such weep holes do not normally exist, and are frequently specified to be installed as part of a remedial program, to allow for proper drainage of the units.

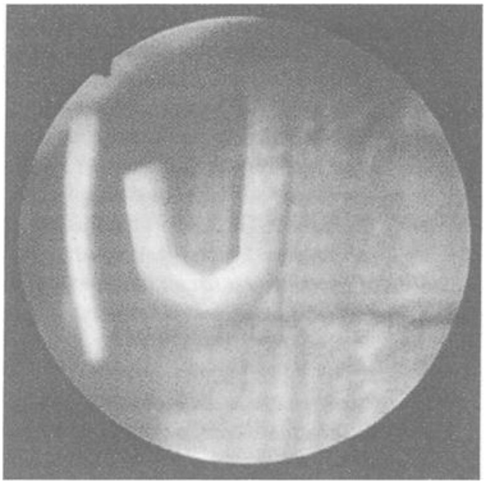

FIG. 1—*View through Borescope of J-hook without Rod.*

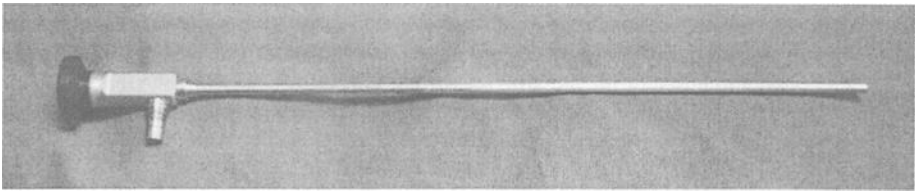

FIG. 2—*Straight Shaft Borescope.*

While optical borescopes are helpful in confirming the diagnosis of steel corrosion they have limited uses due to the presence of backup masonry materials that obscure the available views through the borescope.

In-situ Strain Relief Testing

Recent projects to evaluate the distress in terra cotta cladding systems on historic buildings have incorporated the direct measurement of existing (residual) stress measurements in selected elements of those systems.

One significant reason for determining the magnitude of stresses that may exist in terra cotta cladding is the possible presence of vertical splitting or cracking distress, usually associated with high vertical compressive stresses in the cladding materials. These high vertical stresses can exist for any combination of factors, including, but not limited to the following.

1. Corrosion of the horizontal leg of steel shelf angles in the cladding system;
2. Corrosion of small diameter metal anchors used to attach the terra cotta units to the face of the building;
3. Volumetric expansion of the terra cotta due to moisture absorption or in response to thermal variations; and
4. Vertical stacking of terra cotta units over significant heights, resulting in high compressive stresses in the units in the lower portions of the cladding systems.

These stresses may vary in magnitude and orientation, depending on a number of factors, but not necessarily limited to, the following.

1. Age of the building.
2. Height of the building.
3. Architectural interrelationships between column covers, mullions, and spandrel beam covers, water tables, etc.
4. Type of building frame (i.e. – concrete, steel, etc).
5. Degree of exposure to the environment, particularly direct sunlight
6. Degree of exposure to direct precipitation runoff down the building face.
7. Composition of the backup materials behind the terra cotta cladding, and the degree of attachment of the terra cotta to that backup.
8. Extent and timing of previous repair efforts, that may have involved removing and replacing units, or introducing horizontal relief joints by saw cuts or other means. These repair efforts may have relieved stresses in the terra cotta, without treating how those stresses originated in the first place. This can partially or fully mask the problem.
9. Type of mortar used.
10. Presence, location, and frequencies of steel shelf angles in the façade.
11. Physical material properties of the terra cotta and the backup masonry.

Investigations of terra cotta cladding systems by others have included mounting electrical resistance strain gauges on the surface of the terra cotta element in the region where it is wished to determine the existing compressive stresses in the cladding system.

The gauges are mounted directly on the glaze of the terra cotta unit to be tested. Initial base readings are taken, and a masonry saw is used to cut a square or rectangle completely through the face shell of the terra cotta around the gauge. When the cut is completed, a second set of strain gauge readings are made. This approach is based upon the principle that there is a pre-existing (residual) strain (and thus stress) in the terra cotta at the test point, and the change in reading of the strain gauge that occurs where these cuts are made will reflect that this residual strain has been relieved. The problem with this approach is that changes in strain are only measured on the outer surface of the terra cotta. These readings can be misleading and not representative of the strain and stress through the body of the terra cotta unit. The stresses in terra cotta cladding vary with depth into the element.

Terra cotta does not normally consist of a tile of shallow thickness only, but more often comprises a hollow unit with a face shell usually 1 to 1.5 inches (25 to 38 mm) thick, backed by vertical and horizontal webs or flanges, and depending upon the causes of those stresses, the stresses in such a unit will not necessarily travel down the face shell of the unit. Compressive stresses as the result of corrosion of a shelf angle supporting the unit sometimes will be concentrated on the face shell, as the outer portion of the shelf angel is corroding. However, compressive stresses set up as a result of restrained volume changes in a terra cotta clad column cover may be more evenly distributed through the depth of the unit. This is also probably the case where compressive stresses occur in the terra cotta cladding as a result of differential movement between the cladding and the adjacent building frame. The range of values for these types of stresses varies greatly and no absolute values can be given.

The degree to which the cladding is attached to the backup material also can greatly affect how much stress is present in the terra cotta elements. In most structures a fair-to-good bond exists between the back-up materials and the terra cotta units. Frequently, the terra cotta has common brick back-up extending into it, together with voids that are partially or nearly totally filled with grout. These back-up material intrusions into the voids of the terra cotta facilitate significant transfer or sharing of vertical stresses between the terra cotta and the back-up materials.

In some situations terra cotta units were treated as a veneer material, with little or no back-up material behind it. In these cases the units are fastened to the building with small diameter metal ties that were intended to prevent the walls from overturning out of the vertical plane of the wall. The weight of the terra cotta units is supported by the steel shelf angles, upon which they are constructed. This is similar to brick veneer construction in common use today.

Terra cotta veneer is most commonly found in building facades at the frame of the building and is typically attached to the backup masonry encasing the frame. There are cases where there are no backup materials between the back of the terra cotta units and the face of the steel or concrete-encased columns. The vertical stresses that buildup in these thin veneers of terra cotta are resisted by the full body of the units rather than just the outer face shell.

This situation frequently results on large terra cotta column covers where the vertical residual stresses in the veneer terra cotta units located directly in front of the columns are grossly different than the vertical residual stresses in the adjacent fully backed-up terra cotta units on either side of the columns. Values for these stresses vary greatly. On the

Wrigley Building, Chicago, Illinois, a maximum value of 1,900 psi (134 Kg/cm2) was recorded. See FIG. 3 - *Wrigley Building, Chicago, IL.* Vertical cracks frequently occur in the interface between the "veneer" terra cotta and the adjacent, brick back-up units. The residual stress conditions in these adjacent regions can be dramatically different. In many cases, a vertical tearing effect can be seen along these crack planes.

Another factor affecting the magnitude and location of vertical compressive stresses existing in terra cotta cladding units is the location and size of the steel shelf angles supporting the cladding. If these shelf angles are set too far back into the wall, this can contribute to a vertical stacking effect in the terra cotta. In these cases the stacked weight of the units can create a vertical force that bridges around the shelf angles and travels down the outer face shell of the cladding. The stiffness of the horizontal leg of the shelf angle also plays an important role in where the stresses from the supported terra cotta units travel down the wall.

FIG. 3—*Wrigley Building, Chicago, IL.*

If the shelf angles were under-designed, or if the outstanding leg has been weakened by corrosion, those affected shelf angles may deflect under load and become only partially effective or useless. Deflection of the shelf angles can also cause stress concentrations to occur on the outer regions of the terra cotta units below, thus altering the stress paths.

In some cases weak or weakened shelf angles can actually be bent upwards or downwards by vertical volumetric expansion of the terra cotta units.

Remedial repair programs can also significantly alter the locations and paths of vertical stresses in terra cotta cladding systems. The unfortunately common practice of pointing horizontal joints below shelf angles full with mortar will render those original relief joints ineffective. Vertical loads will pass downward in the terra cotta across such joints, rather than transfer into the shelf angles, as originally intended. Pointing mortar joints with a pointing mortar having a higher compressive strength than the original mortar can also cause stress concentrations. This is particularly true in situations where only the outer inch or less of the mortar joints is raked or ground and pointed, or only part of the length of a horizontal mortar join is pointed.

It should be recognized that stresses in long vertical terra cotta elements frequently do not have a near vertical orientation. Restraint provided by integrally constructed spandrel beam covers to long vertically oriented terra cotta clad column covers and mullions can cause the stresses within the column covers to change direction radically in the vicinity of the spandrel beam covers.

It is suggested that the testing method described below provides a more complete picture of the residual stresses existing within the depth of the terra cotta units. In this approach the strain gauges are not located on the outer surface, but rather inside the outer face shell of the terra cotta unit. This method also eliminates the uncertainty of how much stress relief actually takes place during the test. If the test procedure described earlier of saw cutting a rectangle or square around the strain gauge does not also include complete physical removal of the piece of terra cotta surrounded by the four saw cuts, then complete stress relief does not necessarily occur because of the bond to the backup materials. The test method outlined below includes complete removal of the entire terra cotta element containing the strain gauges, resulting in total stress relief in that element.

For this paper an investigation of residual stress distribution in the terra cotta cladding system of the Wrigley Building is referenced. The original designers of that building incorporated full height terra cotta clad mullions between windows, and full height column covers around the building perimeter. Both these element types exhibited excessive cracking in a vertical direction over the height of the elements. Other investigators surmised that the cracking was a result of the vertical stacking effect; i.e., the weight of the terra cotta accumulating over the height of the structure allowed compressive stresses to exceed the splitting tensile strength of the terra cotta units. This theory was not deemed plausible since the vertical cracking was not concentrated in the lower floors of the building. As a result, additional testing was proposed to determine the causes of the vertical cracking.

In-situ stress relief tests were performed on two terra cotta clad column covers and included the following steps:

1. A 3-inch (75 mm) diameter core was drilled and removed from the center of the terra cotta unit in the middle of the area under study. This core was drilled completely through the face shell of the terra cotta unit, which was approximately 1.25 inches (30 mm) thick. The core was retained for subsequent laboratory testing. See FIG. 4 - *In-situ Stress Relief Test Cofiguration.*
2. Four one-inch (25 mm) long electrical resistance strain gauges were mounted on the freshly exposed clay surface within the depth of the face shell at the hole location. These gauges were aligned with their longitudinal axes parallel with the plane of the exposed terra cotta face shell, and at a depth at one half the thickness of the face shell. Each of the gauges was aligned around the perimeter of the core hole at 90-degree intervals (at 90, 180, 270, and 360 degrees, with 360 degrees at the top of the core hole).
3. Initial base readings were taken on each of the four gauges around the perimeter of the core hole.
4. A larger hole was cut by masonry saw through the face shell of the terra cotta surrounding the core hole, approximately 8 inches (200 mm) square. The square piece so generated was then removed completely from the face of the location of the core hole. Any residual stresses existing in the area of the terra cotta cladding under investigation would be relieved in the vicinity of the core hole and the strain gauges.
5. A second set of strain gauge readings was then taken.

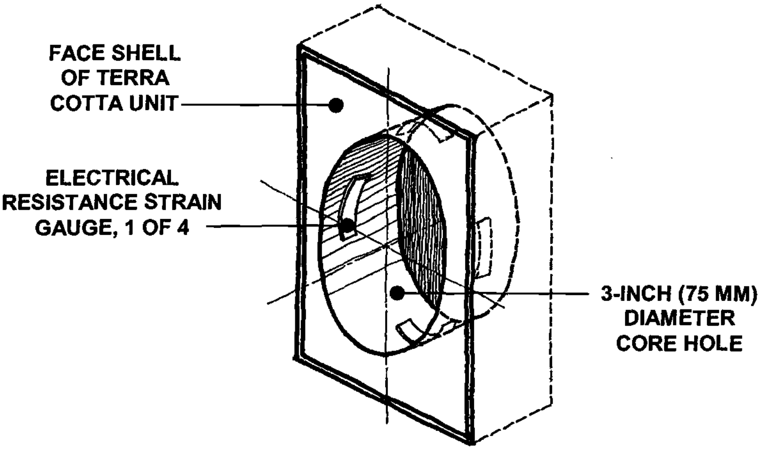

FIG. 4 — *In-situ Stress Relief Test Configuration.*

This method measures the strain around the perimeter of the core hole caused by stress relief through the saw cutting procedure. A certain degree of ovalization (essentially microscopic in magnitude) of the hole will occur when the square block is cut out and removed from the structure. The difference between the horizontal and the vertical gauge readings is an indication of the vertical-to-horizontal stress ratio. If the modulus of elasticity (E) and the Poisson's ratio (v) of the terra cotta is known, then the radial and circumferential (hoop) stresses can be calculated, as they are related to the respective change in the hoop strain.

Impulse Response Testing

During the 1980s and 1990s, nondestructive stress wave methods for structural testing were developed to include tests such as Impact-Echo (I-E) [1] and Spectral Analysis of Surface Waves (SASW) [2]. These methods provide a much-needed way to nondestructively identify and locate defects within concrete plate structures such as slabs and walls. In spite of progress in the acceptance of these methods, a major drawback in their use has been the slow collection of data from large structures. Nondestructive testing claims to be fast and economical, while producing as large of a test coverage of the structure as possible. It has become apparent that I-E and SASW stress wave testing of large structures such as the terra cotta clad buildings described in this article can be time consuming and relatively expensive if full evaluation of the structure is to be achieved. The Impulse Response (IRS) test [3] has been developed to fill this gap by providing a fast test method with full structure coverage in a relatively short time, in order to identify those anomalous areas of the structure that require more intensive investigation using other tools, and to select optimal locations for coring and sampling. In this way, evaluation costs and time have been considerably reduced, while maximizing the amount of information obtained.

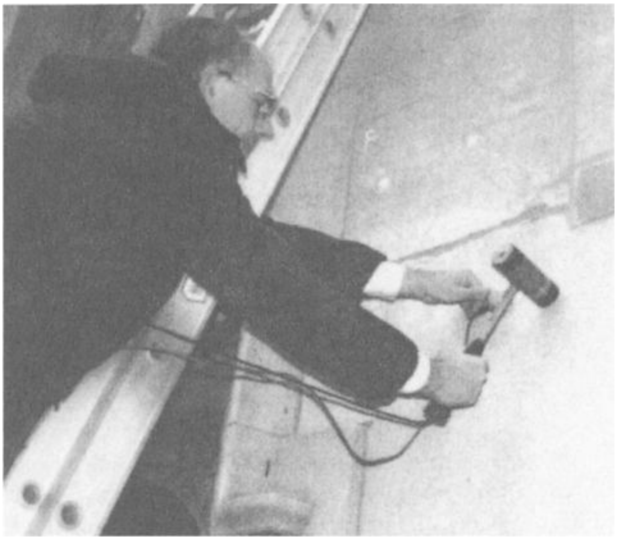

FIG. 5— *Impulse Response Testing.*

Another reason for selecting the IRS test is that the latter uses a compressive stress impact approximately 100 times that of the I-E and SASW tests. This greater stress input means that the tested unit responds to the IRS hammer impact in a bending mode over a very much lower frequency range (0-1 kHz terra cotta units), as opposed to the reflective mode of the I-E and SASW tests. The response of the unit can be used to estimate the relative locked-in (residual) stresses in each element, as described hereafter.

The IRS equipment consists of a hammer with a load cell mounted on the striking head, and a geophone (velocity transducer), both linked to a data acquisition unit in a portable computer [3]. See FIG. 6 - *Schematic of Impulse Response Testing Method.*

Each IRS test is performed by striking the terra cotta unit in a central location, common for all units where testing is conducted. The response of the terra cotta unit and its support mechanism to the hammer blow is measured with a geophone, which is placed approximately 6 inches (150 mm) from the point of impact on each of the terra cotta units.

Impulse Response tests were performed on vertical terra cotta clad column covers and vertical terra cotta clad mullions on the Wrigley Building. These elements were selected in recognition of the visual presence of vertical cracking distress. The tests were conducted from the 4th to the 13th floor because the vertical column and mullion elements were continuous over this vertical distance, and were also representative of the similar elements around the entire perimeter of the building where the problem of vertical splitting or cracking of the column covers and mullions existed.

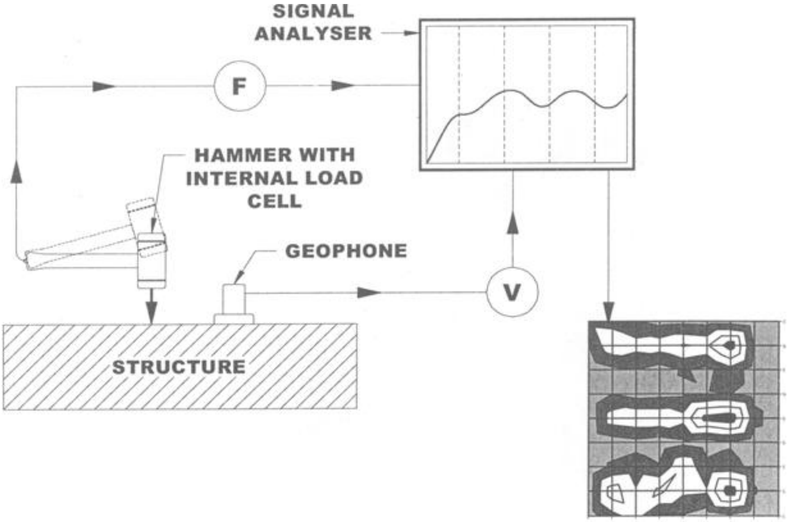

FIG. 6 —*Schematic of Impulse Response Testing Method.*

The velocity, v, measured by the geophone divided by the force, F imparted by the hammer is known as the Mobility (v/F) of the unit tested. Each test response is plotted as a frequency spectrum of Mobility over the range 0 to 800 Hz. Features such as debonding of the unit from its supporting backup masonry and non-visible internal delamination or splitting within the terra cotta unit can be detected in this manner.

This value is a function of the dimensions of the terra cotta unit under test, as well as the quality of the support for that unit, including the locked-in stresses that may exist within that unit. The slope of the portion of the Mobility plot below 0.1 kHz defines the compliance or flexibility of the area around the test point for a normalized force input. The inverse of the compliance is the dynamic stiffness, K_d of the structural element at the test point. This can be expressed as:

Stiffness *f [terra cotta quality, unit thickness, unit support condition]*

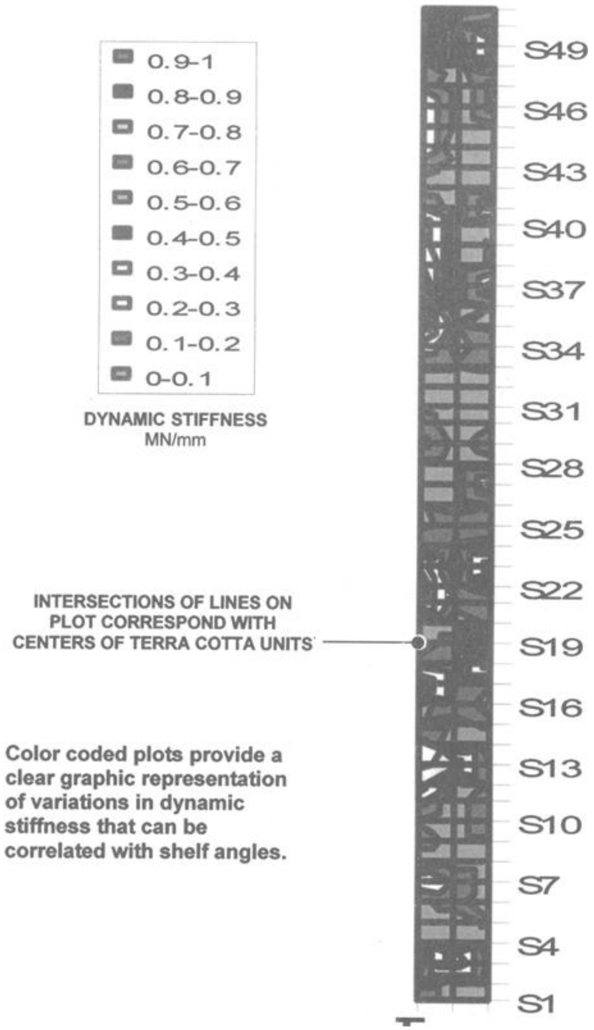

FIG. 7 – *Plot of Dynamic Stiffness verses Position of Terra Cotta Units of Column Cover for Wrigley Building*

The residual stress distribution in the tested terra cotta clad columns and mullions were determined from the IRS tests on each column/mullion from the 4th to the 13th floor. One test was performed on each terra cotta unit in each element over the entire height. There was an average of 11 units per floor on each element.

The dynamic stiffness recorded for each terra cotta unit was plotted versus the position or height of the unit from the fourth floor upward. See FIG. 7 – *Plot of Dynamic Stiffness verses Position of Terra Cotta Units of Column Cover for Wrigley Building.* The residual stress measurements made in this manner indicate very strong concentrations of high compressive stress directly above and below the position of the steel shelf angles supporting the terra cotta at each floor line. The stresses clearly tapered off between the locations of the shelf angles. The IRS results for the mullions show less uniformity (with high concentrations of compressive stress at the 6th to 7th floor levels, and at the 12th to 13th floor levels). It is probable that the corrosion of the steel shelf angles was greatest at these two levels in that particular element. Of particular significance is the higher level of distress observed in the mullions. This would translate into lower stress levels, because the formation of distress will act to relieve stresses in the terracotta units; once the distress appears, the stresses are partially to completely relieved. Therefore it is to be expected that the measured stresses would be higher in the units where cracking distress was less frequent. These are precisely the findings of this study.

The results from in-situ strain relief test were combined with these results to determine the actual distress that existed in the column cover and the mullion terra cotta elements, was caused by localized corrosion of the steel shelf angles located at each floor line over the height of these nine-story high elements. The vertical stacking of the total terra cotta weight over the height of the building clearly did not cause the distress. The most dramatic finding of this study was that the weight of the terra cotta cladding was being supported more by its attachment to the backup materials than by the steel shelf angles. This was more evident on the terra cotta mullions than on the much larger column covers.

It was further discovered through this testing that the magnitude of the compressive stresses in the terra cotta column cover was much lower than would be expected for vertical stacking, which would be a function of both the location of the unit in the height of the building and the weight of the individual units. However, these test results do indicate that corrosion of tightly embedded steel shelf angle with no relief joint beneath can result in relatively high, localized compression stresses within the terra cotta;

certainly high enough to cause vertical splitting of the terra cotta units above and below those elements.

Conclusion

A terra cotta façade inspection that integrates advanced evaluation techniques, such as borescope inspections, in-situ strain relief testing and impulse response testing, with fundamental evaluation techniques provides an understanding of how the terra cotta cladding is performing as a system. Mechanisms and locations of failures can be identified before they result in catastrophic failures. Without these advanced techniques little more than the immediate condition of individual terra cotta units can be ascertained without relying on speculation.

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Unique Considerations for Stone Facade Inspection and Assessment

REFERENCE: Farmer, M.C., "***Unique Considerations for Stone Facade Inspection and Assessment,***" *Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444,* J. Erdly and T.A. Schwartz, Ed(s)., ASTM International, West Conshohocken, PA, 2004.

ABSTRACT: Natural stone has been used in construction of buildings and structures for centuries. However, over the years stone construction methods have changed and evolved dramatically. Designers are moving away from empirically designed, load-bearing masonry systems, to more highly engineered thin stone cladding systems.

This evolution has changed not only the way we build with stone, but also the methods by which we evaluate in-service performance of a façade, as well as how we maintain these structures and preserve them for the future. This natural material demands unique inspection methods, a broad knowledge of construction, and an understanding of how various building materials work together.

This paper attempts to describe some current techniques used to inspect and assess stone façade performance. It addresses common, and some uncommon, considerations systems when performing an evaluation of stone façades for both load-bearing walls and stone cladding, ways to identify the wall system in use, and key elements in stone façades that require special attention. Other methods available to engineers and architects to enhance our evaluative capabilities are also highlighted. Lastly, this paper discusses the availability and use of ASTM standards addressing topics related to stone inspection, assessment, and maintenance.

KEYWORDS: Stone, Inspection, Assessment, Load-Bearing, Cladding

Introduction

Natural stone is certainly one of, if not the oldest, of building materials. From the earliest cultures to our modern civilization, stone structures have played an important role in our society by becoming places we live, worship, and work. Due to its relative permanence, durability, and timeless beauty, stone has been used throughout history for monuments and tributes to recognize significant individuals or events in our past. Over the years, stone has been used in many different applications, most principally as structural supports and wall enclosures. Our focus will be limited to the use of stone in wall systems. Façades incorporating load bearing walls will be discussed, as will the use of stone as wall cladding.

Stone is a unique natural material that has many fine characteristics. It is attractive, durable, and resistant to water penetration, easily shaped and carved, as well as widely

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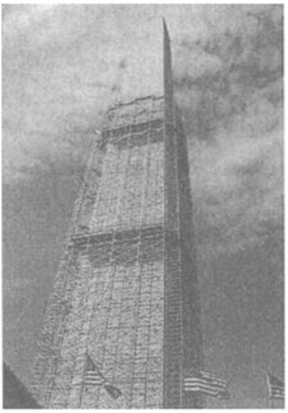
available. However, not all stone is suitable for construction. Many stone varieties are too soft to resist environmental forces, or too weak to withstand stresses induced in a structure. As a natural material, one must also consider that fissures, cracks, inclusions of foreign matter, or other naturally occurring defects that may be present and weaken stone for its particular application. Therefore, selection of the stone for construction is the first critical step in understanding and evaluating stone structures. Stone used for construction of exterior wall systems usually include granites, limestones, marbles, quartz-based stones (sandstone, brownstone, etc.), and slates in more limited applications, although even within these classifications there are varieties inappropriate for these applications.

There is an expectation of permanence with all building structures. Once constructed, our attention as building professionals rightly focuses on preservation and maintenance. Due to the wide variety of ways in which stone is used, its range of characteristics and physical properties, as well as the labor intensive processes used to fabricate, erect and integrate stone in our modern buildings, our inspection practices and maintenance programs must reflect this dynamic and unique material. This paper discusses some of the key considerations for stone façade inspection and assessment, in an effort to better prepare the reader interested in a safe, stable, and attractive stone building throughout its service life.



The Washington Monument is an example of a load bearing structure.

Evolution of Stone Façades from Load-Bearing Structure to Supported Cladding

To understand the performance and behavior of stone façades, one must first look back at the evolution of stone construction. The earliest structures constructed of stone were designed empirically, and utilized the high compressive strength that stone offers. However, heights were limited by the weights of the earliest load bearing structures, forcing the base of walls to be more massive to support the self weight of the stone above. As our understanding of structural behavior and physical properties increased, and the cost of stone began to rise, other masonry materials were combined with a stone facing. These composite walls cost less by reducing the amount of stone used, but still provide a load-bearing wall system capable of supporting it and the applied loads. Both pure load-bearing stone walls and those of composite construction can still be found in use today. However, the change from solid stone walls to stone-faced walls was the beginning of the change to the more modern wall construction techniques that we see in newer construction, primarily what we know today as stone cladding.

Broken granite cladding panel.

Stone cladding systems by definition carry only self-weight of the material and lateral loads due to wind exposure, transferring them into the primary structural components. Within cladding systems, some are partially load bearing, in that stone units may be stacked, with the combined weight of several stones transferred to the primary structure at consistent intervals such as floor lines. Other cladding systems rely on each stone to be supported individually, with anchors provided to take lateral and gravity load from each stone. This development offers numerous advantages including more efficient use of both cladding and primary structural materials, reduced cost, and greater opportunity for innovative design.

Stone cladding systems have many advantages over load bearing stone masonry for modern construction, and are favored for most of today's new façades. But by increasing the efficiency of stone use, much of the inherent redundancy and durability found with more massive load bearing systems are lost. Provided the systems are properly designed, analyzed, and tested, stone cladding systems offer a durable and cost effective approach to stone construction.

Identifying Load Bearing Stone Masonry versus Stone Cladding

Perhaps the most fundamental determination that is required to assess stone façade is whether it is load bearing. One significant clue is the age of the structure – there is a greater likelihood of finding load bearing masonry or load bearing cladding in older buildings than in more modern construction. By examining the interior surface of the wall system at unfinished spaces, the investigator can determine whether the stone is continuous through the wall or of composite construction where a different material is incorporated into the backup. Stone units used in load bearing stone masonry also tend to have smaller face dimensions than cladding panels due to weight and handling limitations, and solid lintels over wall openings.

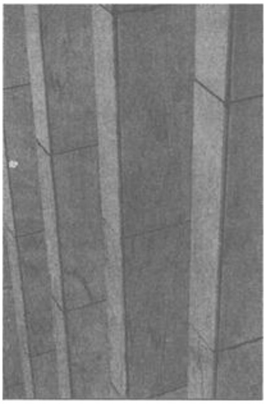
Joint material also can be a clue to the construction methods. A non-load bearing cladding will usually contain sealant, particularly those on more flexible back-up systems, such as light gage metal framing. Load bearing cladding almost exclusively uses mortar, except where the mortar has been removed and replaced with the sealant. The presence of expansion joints in a stone façade with mortar joints will usually also indicate a cladding system; however, they are usually not found in non-load bearing cladding systems with sealant-filled joints. An exception to this observation is load bearing and composite stone wall systems that also use mortar at joints. Expansion joints are rare in these systems partially due to a lack of understanding of the volume changes, and partially because the mortar used in older construction was weaker, and more capable of absorbing the volume changes without damaging the stone.

Probably the most obvious indication of a stone cladding wall system is the presence of other primary structural elements, such as pilasters, columns, spandrel beams, or thickened slab edges.

Unique Considerations for Stone Façades

Principal Properties

Load bearing wall systems are principally under compression. Bearing capacity is also critical to carry the high gravity loads without localized failure. Provided the walls are plumb, lateral stability is rarely a concern due to the stout proportions that the high gravity loads require. As with other



Cracked slender cladding panels.

masonry wall systems, Stone masonry has some ability to redistribute loads if there is a localized failure in bearing or compression.

The most critical property of the stone used on a stone cladding system is flexural strength. Since there is no accumulation of external vertical load, and each cladding panel (or group of panels) is independently supported, the aspect ratio of the stone can be more slender. The only external loading a cladding unit typically experiences is from wind acting perpendicular to the span of the panel, which induces bending stresses.

Primary Structure

A stone cladding system must rely on the primary structure to handle service loads adequately. If the primary structure is under-designed, the cladding may suffer from excessive structural displacement, unintended load transfer, or other failures or deficiencies compromising the load-carrying elements. Though it is rare that cladding concerns are a result of primary structural deficiencies, a comprehensive investigation should not rule out this possibility prematurely.


Unintentionally loaded limestone cladding.

Back-Up Materials

Besides the stone itself, composite walls and cladding systems include backup materials. For composite construction, the back-up material is almost always unit masonry such as brick, terra cotta tile, or concrete masonry. In some cases, less expensive or smaller stone units are used. For stone cladding systems the back-up material is crucial since it transfers the weight of the stone cladding, as well as applied lateral loads, back to the primary structure. Common back-up materials for stone cladding include concrete, precast concrete panels, brick or concrete unit masonry, steel truss systems and light gage metal framing.

When evaluating a stone cladding system, it is essential to identify the back-up system and understand its potential effects on the performance of the cladding. Though by no means exhaustive, some general considerations for various back-up systems follow.


Granite panels installed against concrete with mortar.

Cast-in-Place Concrete - Concrete is stable, easily flashed, highly resistant to water penetration, takes anchor fasteners well, and is sufficiently rigid to support large stone units. Concrete does shrink and creep over time, so joints widths are critical to avoid stone-to-stone contact or stress concentrations from anchors or shims that may remain in the joints. Tolerances with a concrete back-up are also sometimes difficult to control, requiring changes in anchor types or excessive shimming.

Concrete Masonry Units - Also very stable and sufficiently rigid to support large stone pieces when properly anchored to the primary structure, concrete masonry units are easily flashed but requires damp-proofing or another cavity lining material to limit water infiltration. Anchor fasteners have a lower capacity in hollow units, and different anchors are usually required depending upon where the anchor falls. Fully grouted cells are often used to improve resistance to water penetration and improve anchor fastener pull-out and placement flexibility. Shrinkage and creep of the block must also be considered when detailing joints between stone units.

Precast Concrete - Very stable and sufficiently rigid to support large stone units, precast concrete is also dimensionally more stable than cast-in-place concrete. Stone panel erection and placement position can vary widely, impacting stone tolerances and anchor selection.

Stone-Faced Precast Concrete - These systems involve casting the concrete directly over the stone facing panels and its anchors, creating a large wall panel that may contain several stone cladding panels. Isolation between the stone and the concrete is critical to prevent bond – if there is bond, some stone types may fracture due to differential movements between the materials over time. Anchor placement, frequency, and embedment are also significant considerations; the author's experience suggests that this system typically is under-designed. Once fabricated, erection tolerances are critical to maintain consistent joint widths and alignment with adjacent wall panels. There is no flexibility to relocate individual stone panels or adjust their positions after installation. There is also no drainage cavity or flashing; trapped moisture behind the stone can lead to staining, discoloration, water leakage, and joint material failure.

Marble-faced concrete.

Steel Truss Framing - Similar to stone-faced precast concrete, this option allows for large wall panels to be created by attaching several stone panels to a single truss element then erecting the truss. The stiffness of the truss members supporting the stone must be sufficiently rigid to prevent cracking. Overall the truss is more flexible than other back up systems, and flashing is difficult due to the irregular nature of the steel members forming the truss.

Granite panels attached to steel truss.

Light Gage Metal Framing - Very popular due to its relatively low cost and speed of construction. With this type of framing, it is difficult to develop sufficient rigidity and limit deflection to support the stone units adequately without cracking; therefore, it is better suited to smaller stone units. Anchor placement is limited and controlled by the framing member placement. The system is relatively easy to flash, but does require a cavity liner to guard against water infiltration.

Stone Anchorage

Anchors are typically used to transfer loads between stone elements in a stone wall system. Load bearing applications rely on anchors such as cramps and dowels to secure stones to each other horizontally and vertically.

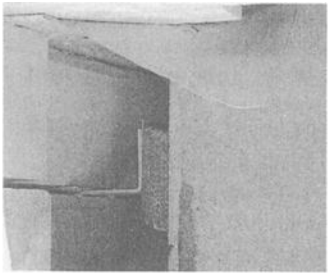
Stone panel failure at anchor.

The anchors themselves, as well as the means by which they contact the stone, are critical parts of any stone inspection or assessment.

Stone cladding anchors must be designed to transfer external loads from the stone, as well as the load of the stone itself, back to the primary structure. There are many standard and customized anchor configurations to every situation. The size, frequency, and distance between anchor points are all critical factors in stone performance. The method of load transfer from the stone to the anchor, such as holes, kerfs, dowels, tabs, must be properly designed with adequate safety factors and fabricated from suitable material. Stainless steel is recommended for all elements in contact with stone to limit the potential for corrosive failure. The transfer of load from the anchor to the back-up structure also cannot be ignored, and is often the source of anchor failure. ASTM Standard Guide for Selection, Design, and Installation of Exterior Dimension Stone Anchors and Anchoring Systems (C1242-02), is an excellent source of information on the various anchor types in use and recommended practices. Literature from the stone industry sources such as the Indiana Limestone Institute [1], the Marble Institute of America [2], and the National Building Granite Quarries Association [3], and are also very helpful.

Shims

Shims are constantly used to adjust and correct the position of the stone relative to other stone units and/or control points, as well as maintain adequate joint spacing. Shims used today can be stainless steel or a hard plastic. Wood and lead wedges and shims were often uses in older construction. Only shims made of metal or plastic are recommended in modern construction due to their dimensional stability under load.

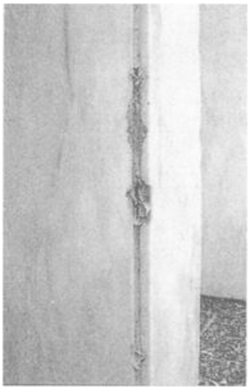


Shims used to extend reach of anchor.

Joint Clearances

Stone is modular in nature, with wall construction or cladding consisting of stone cut or shaped to fit together. Joints exist where the individual units meet, and these joints must be filled to maximize a weather tight exterior. Load bearing stone walls or cladding systems that require some vertical load transfer, are usually filled with a cementitious and/or lime-based mortar. Some load bearing wall systems and cladding can use sealant as a more impermeable weather barrier, and accomplish load transfer through setting mortar or solid shims.

In cladding systems with independently supported panels, proper clearances at all joints must be maintained to allow volume changes and avoid unintended stone-to-stone contact. Joint widths should be designed to accommodate anticipated expansion and contraction, building movements, shortening of concrete frame primary and back-up structures, fabrication and construction tolerances, and the minimum requirements

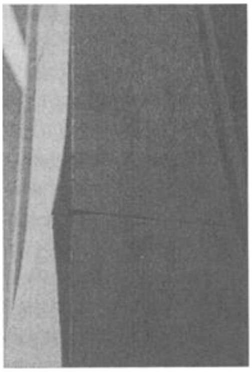


Failed sealants at cladding joint.

for the joints filling material to perform well. More often stresses develop at locations where anchors or shims inadvertently fill the joint space and prevent adequate clearance.

Expansion Joints

As with all materials, stone expands and contracts as a result of thermal changes. To accommodate these movements, stone cladding that utilizes mortar as the joint filler must also utilize clear, unobstructed, compressible expansion joints at an appropriate spacing. Depending upon the design of the system, expansion joints may be needed both horizontally and vertically. The expansion joint size must also take into account the accumulation of these effects over the distance between expansion joints. Complete compression or crushing of joint materials is a strong indication of unanticipated stone movements, improperly designed joints, or anchor failure. Stone damage or a lack of distortion in the joint material may indicate obstructed soft joints. A non-load bearing stone cladding system does not require expansion joints if the joints are unobstructed and filled with sealant.



Failure of cladding due to unaccomodated movement

Setting Mortar

Stones requiring load transfer can be placed using setting mortar. The mortar provides a uniform distribution of the load from one stone to the next. To control and maintain a uniform joint thickness, shims are bedded in the mortar, then removed after the mortar has set. Rapid setting mortars that are often used contain components that can adversely affect its durability in an exterior environment, or expand and damage the stone. If setting mortar is present, it should be examined carefully and potentially tested for adverse components, particularly if stone distress is evident.

Expansive mortar used to set stone.

Interface Conditions

When examining a load bearing stone façade, it is important to note how the stone meets other façade elements, such as windows and doors, as well as the ground. The interface conditions between materials are susceptible to many concerns, like water leakage, accommodation of expansion/contraction, anchorage to the stone, and mechanical damage. Therefore, these locations will usually require more maintenance, and the detailing is critical to ensure continued serviceability.

Common interface condition between stone and curtainwall framing.

Common Stone Deficiencies

Though stone is considered one of the most durable construction materials, it too can suffer from mechanical damage from fabrication or erection, environmental exposure, and unintended internal stresses. Of paramount importance is public safety, and a careful examination of stone integrity is the primary focus of a thorough investigation. Stone damage can occur in the form of spalls, cracks, erosion, and exfoliation or scaling. Provided these conditions can be stabilized for safety purposes, these conditions may not be critical structurally in a load bearing wall system if the damaged material is a small percentage of the overall stone, and the remaining load transfer mechanisms are adequate.

In contrast, due to the thinness of cladding panels, any reduction in stone section from a crack, spall, or excessive erosion or exfoliation can lead to a reduction in anchor capacity, loss of an anchor in its entirety, and eventual instability of the cladding panel. Cladding systems should be inspected on a frequent basis and any deficiencies exist that could compromise the cladding integrity should be addressed immediately.

Failure of an anchor at a kerf.

Cracks and Spalls - Crack and spalls tend to occur as a result of stress concentrations or mechanical damage. Stress concentrations can be caused by, among other things, unanticipated or unaccommodated building movements, improperly sized or placed shims, anchor corrosion, or a combination of these factors. Any damage of this nature should be examined carefully; however, the best course of action in a load bearing stone wall system may be a relatively minor repair to simply minimize further damage.

The causes of cracks and spalls in cladding systems are similar to those for load bearing masonry (see discussion above). However, the same crack or spall that would prompt a minor repair in a load bearing system may require total panel replacement in a cladding system.

Erosion - Erosion is a natural process for stone and should be expected to be commensurate with the age of the subject façade. The natural erosion process can be accelerated as a result of environmental forces, atmospheric pollutants, and improper cleaning methods. Exposure to weather, proximity to pollution sources, and historical data on cleaning methods are all important to understand when evaluating stone weathering and erosion. The orientation of the bedding planes or "layers" within sedimentary stone can also have a great effect on the rate of erosion. If bedding planes are exposed and oriented vertically in the wall system, erosion is usually more rapid and severe than if the bedding planes are horizontal, perpendicular to the direction of water as it runs down the façade. If the stone type contains deep veins, seams, silt layers, or other pockets of lower density material, erosion is usually more pronounced, washing out the looser, less compressed material in the softer layers. This process can create cracks and allow moisture to penetrate more deeply into the stone. This in turn can result in accelerated erosion, water leakage, or other damage related to excessive water entry.

Erosion due to weathering.

Exfoliation due to excessive water entry.

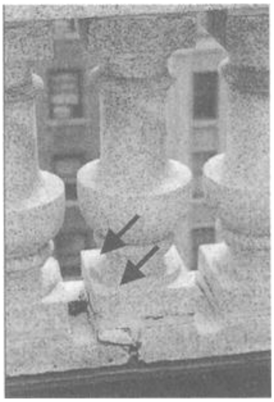
Exfoliation - Exfoliation sometimes referred to as "scaling" is a condition where outer layers of the stone material separate from the body of the material and become unstable. This often is a result of bedding plane orientation where the bedding planes are oriented parallel to the exposed face of the stone. Water and pollutants are absorbed into the outer bedding planes, reducing bond at these zones of weaker material. In colder climates the expansion of trapped water during freeze thaw cycling can also be a source of exfoliation, as can exposure to deicing chemicals In softer sedimentary stones chemical consolidants or binders are often used to limit or slow erosion damage; the outer consolidated "crust" of stone can become unstable by creating an artificially weakened plane behind the consolidated material. The consolidated material can also trap moisture and salts that can adversely affect stone integrity. Therefore, bedding plane orientation, environmental conditions, and prior repair/stabilization methods are also critical factors to assess stone conditions.

Stone cladding panels are inherently more sensitive to erosion and exfoliation due to the reduced thickness. Loss of any cross section can result in a relatively high reduction in strength. Erosion can also be an indication of strength loss of weaker material. That could be inadequate to support service loading. Panels showing elevated amounts of erosion, exfoliation, excessive surface voids, pronounced veining, or open bedding planes should be carefully examined for structural defects.

Displacement - Displacement of stone units within a load bearing masonry façade is usually an indication of uncontrolled movement or disruption in the wall itself, perhaps due to expansion of a corroded metal element, or trapped water freezing within joints, cracks or other voids within the wall system. In load bearing façades, displacement is usually not sufficient to cause instability due to their large bearing surfaces and the tendency for load to redistribute as needed. But displacement can be an indication of a serious problem and, at a minimum, provides an opportunity for increased water entry that can adversely affect stone performance or cause other problems.

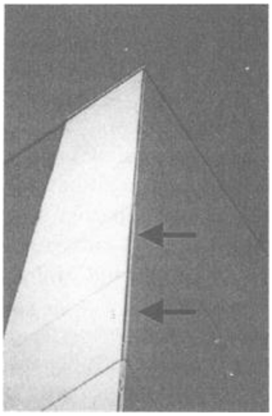
Displacement of stone cladding panels can be a more serious issue since there is no inherent stability through load transfer. If the displacement is recent, it can be an indication of an unstable condition such as unaccommodated panel movements, a failed anchor, or loss of stone integrity. Some apparent displacements actually can be the result of stone panel installation outside accepted tolerances. Though unsightly, the panel may be perfectly stable and secure.

Corrosion Damage - As mentioned earlier, anchor corrosion, or corrosion of other metal elements embedded in the façade can be a leading cause of spalling and cracking in load bearing stone façades. The earliest buildings used mild steel or wrought iron anchors and shims to hold the stone



Corrosion damage.

units together, align them vertically, and level them. The anchors were usually set tightly into holes in the stone with setting mortar or shims. Leveling shims are typically compressed between stone units. Over time, the metal breaks down in the presence of moisture and oxygen, forming iron oxide, or rust scale. The scale takes up significantly more volume than the original base metal, thereby applying pressure on the surfaces it contacts. If the stresses are greater than the capacity of the stone unit to restrain it, a crack or spall develops. The formed crack or spall, if left untreated, allows more moisture and air to reach the steel elements, feeding a cycle of further corrosion and subsequent damage.



Bowing of marble cladding.

Bowing - With certain types of stone and anchorage conditions, cladding panels can bow or dish over time. This deformation can be an indication of strength loss, restraint of expansion. Panels with high aspect ratios and long spans with only edge support are most prone to this phenomenon. If bowing is evident, material sampling and verification of physical properties is recommended.

Soiling/Staining - Soiling and staining patterns can provide clues to uncontrolled water sources such as gutter leaks but also can lead to locations where rain water settles. These locations should be noted and inspected carefully, since the additional water exposure can increase the likelihood of damage to the stone. Some façade features that can hold or retain water include windowsills, spandrel closures, water tables, cornices, and other projecting elements. If properly designed, these should be sloped and/or be protected by flashing to limit collection of water on these surfaces, and have functioning drip edge to divert collected water away from the façade.

For further information on cleaning soiled or stained stone, refer to ASTM Standard Guide for Cleaning of Exterior Dimension Stone, Vertical and Horizontal Surfaces, New or Existing (C1515-01).

Soiling and staining of stone features.

Efflorescence - Efflorescence is a white deposit on the surface of the mortar or stone, formed by certain salts within the stone or mortar dissolving into solution, flowing to the exterior, then re-solidifying on the surface as the moisture dissipates. Usually the efflorescence is periodically cleaned and will eventually stop returning as all the accessible soluble salts leach out. The presence of efflorescence in older buildings can indicate that some condition is allowing water to reach further into the stone or mortar, or recent repairs have introduced new salts that have not yet leached out.

Water Infiltration

Wall systems are intended to protect interior spaces from environmental effects. One of the most serious is water leakage, since it can result in not only expensive interior repairs, but also be the source of health concerns by promoting mold growth. Load bearing walls are principally considered a barrier-type system that rely on the water tightness of the exterior and the wall mass to repel atmospheric moisture or absorb the amounts that do enter the wall system prior to it reaching the interior. The integrity of the mortar joints, solidity of the collar joint, and wall thickness are the three most important factors in limiting water penetration. The joints between sections of projecting elements such as water tables and cornices are particularly critical since water is retained longer on their "horizontal" surfaces. The thickness of a wall system is obviously a given and cannot be easily changed. Therefore, it is important to examine the interior sides of wall systems for evidence of water leakage. If a water leakage problem exists, the joint material conditions must be evaluated carefully, in addition to other potential sources.

With the exception of stone-faced precast concrete panels, stone cladding systems are constructed as cavity walls, which provide an excellent opportunity to manage water infiltration through the cladding prior to it reaching the building interior. Even with a cavity, many cladding systems are designed to perform as a barrier, assuming water will not enter through the outer "skin" formed by the stone and the joint material. Unlike the load bearing "barrier" wall, a barrier cladding system does not have the stone mass to slow the water entry once it passes the outer surface. As a result, maintenance of joint materials to limit water infiltration is even more critical with stone cladding. The redundancy offered by a cavity flashing relieves some of the maintenance concerns about keeping the exterior "skin" watertight. When evaluating an existing flashing system, it is important to study the interface between the flashing and the stone anchors – often the flashing is compromised by penetrations resulting from anchor placement.

Inspection and Assessment Methods for Stone Façades In-Situ

Many of the methods for assessing stone façades are similar to those followed in other façade investigation. Though not unique to stone façades, they are restated in an effort to provide a complete array of the techniques and methods commonly used.

Document Review

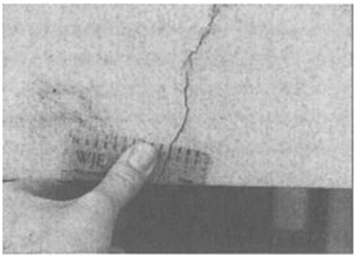
At a minimum, original design drawings, shop drawings, and specifications should be reviewed, if available. Repair/renovation/restoration contract documents and specifications where appropriate, along with consultant reports, due diligence documentation, and maintenance records should be examined.

Visual Observations

Every investigation begins with a thorough examination of the visible portions of the stone façade. Observations should be made from as many locations as possible that afford an opportunity to view the façade. These usually include street level, the roof, balconies, setbacks, or interior windows with views of the building exterior, and from adjacent buildings and their roofs. Documentation of observations in the form of notes, photographs, sketches, and annotations on elevation drawings is helpful in recalling critical conditions and identifying patterns of distress.

Close-Range Inspections

Based on the findings of the visual survey, locations that are representative of the conditions observed or locations of unique features should be selected for closer range examination, either by ladder, personnel lift, fixed or suspended scaffolding.



Crack measurement using a width gauge.

Inspection of Interior Finishes

Inspecting the interior of a stone façade is important to identify any evidence of uncontrolled water penetration. In older structures, cracks in finishes applied directly to the interior side of a load bearing wall may also indicate where movement in the façade has occurred. Removal of finishes and inspection of the wall system directly may also allow for examination of otherwise concealed anchorage and stone conditions.

Inspection Openings/Removal of Stone Material

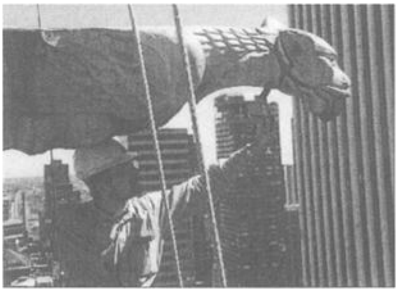
If the observed conditions warrant it, there is no better way to examine anchor and stone conditions than to remove stone panels and look at them directly. Back-up materials, flashing, are also readily visible. Assistance from a qualified stone mason is required to remove and reinstall the stone units. In some cases, existing anchors may need to be modified or new anchors installed to secure the removed piece. If material is needed for physical testing (see discussion below), this original piece can be replaced with a matching one from attic stock or one fabricated for the purpose.

Joint Probes

Much can be learned in an inspection simply by probing the joints between stone units. The presence of shims can be detected, joint clearance verified, an often the type of anchors can be identified.

Sounding

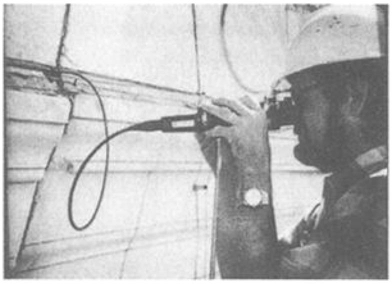
Stone suspected of containing internal defects or concealed spalls, can be sounded by tapping with a soft mallet to detect underlying delamination or changes in thickness of material. Sounding can also be used to determine if the facing is bonded to the concrete in stone-faced precast concrete panel systems. With a little practice and some inspection probes to calibrate the sounds, this technique can be a reliable tool for evaluation.



Sounding for internal defects.

Borescopic Examination

Where removal of stone is not possible, a borescope can be used to examine the concealed side of the stone unit, though its application is limited to cladding systems with a cavity space behind the stone. A borescope is a fiber optic instrument that, when inserted through a small diameter hole (usually less than ½ inch), can be manipulated to observe various conditions within a cavity, such as anchor and stone integrity. It is particularly helpful as a diagnostic tool where a problem with the stone is suspected but instability is not certain. There are a wide range of borescopes available; one can be a great asset to anyone performing inspections of stone façades, from building engineering staff to consultants.



Borescope examination lamination.

Metal Detection Methods

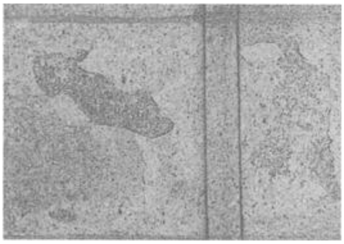
If questions arise about the presence or frequency of anchors, or if there is a need to correlate stone damage to anchor or shim locations, metal detection can be used. As with the borescope, there is a wide variety of metal detection equipment available. It is important to use one that can detect stainless steel, and other non-magnetic metals because they are commonly used in stone façades.

Additional Evaluation Methods for Stone Façades

Structural Analysis

Depending on the condition of the stone façade, a thorough inspection or comprehensive assessment may need to include some level of structural analysis, such as a quick check of compressive stresses in a load bearing stone façade, flexural stresses in a solid lintel, or anchor engagement in a stone cladding panel. Lateral and gravity load criteria can be estimated from past experience, original design documents, or obtained from the construction divisions of local municipalities. If physical properties have not been obtained through testing or documented in the construction records, ASTM provides minimum physical properties for the major stone types that can be used as a starting point. However, one must always be sensitive to unique stone types that have atypical properties, are more difficult to install, or that possess particularly low strength properties. Travertine is a good example of a stone type that is frequently used, but whose durability and physical properties are highly variable. Recommended safety factors that are consistent with the level of variability in physical properties expected for a given stone type are available in ASTM C1242-02, as well as from other industry sources[7].

A structural review should also include a preliminary analysis of the stone anchors used, including both the attachments to the stone as well as to the back up structure. Note that the appropriate safety factors may be different and will depend upon the material to which the anchor is engaged. Highly critical connection points, such as full and partial kerfs and dowel holes, should be analyzed separately.

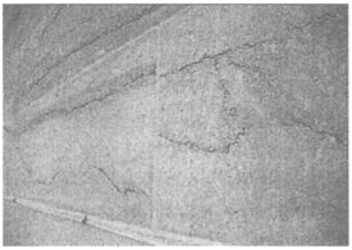


Scaling of granite.

Material Studies

If the stone material appears to be unusually deteriorated, from unusual cracking to unidentifiable stains, material studies can be helpful. Materials testing can also be used to solve problems with materials associated with the stone, such as setting mortar, and joint materials. By comparing material that is performing well with some that is not, the cause of the observed deficiencies can be isolated.

Material studies for stone can be grouped into three general categories: Petrographic examination, chemical analyses, and physical testing. Petrographic studies examine the physical structure of the materials at a microscopic level; for example, petrography can identify the presence of certain minerals in the stone, or examine cracks to determine to estimate their age and source.



Deep veining of marble.

Chemical testing, such as X-ray diffraction and solubility studies, can reveal information about the actual composition of the stone system components, identify coatings, or determine if a setting mortar is expansive.

Perhaps most important for stone itself is physical testing. It is used to estimate physical properties of the stone to assess the potential for strength loss, to predict future performance in service, or measure physical properties to verify assumptions made during structural analysis. Typical properties that are determined through testing include compressive strength, flexural strength, modulus of rupture, tensile strength, and absorption.

Samples of the stone and other materials in question are needed to conduct the testing. It is important to review the test sample requirements prior to visiting the site for an inspection, so that a sufficient quantity of material can be collected.

Anchorage Testing

Depending upon the type of stone deficiencies observed, the capacity of the stone anchorage system might require verification. With some anchor types, testing can be performed in-situ, using relatively simple hardware, such as force scales or load cells. Individual anchors may be isolated or combined into a group for testing depending upon the capacity of the testing equipment. Anchor engagement into the stone and into the back-up material can also be tested separately to confirm which controls the in-service capacity of the anchor assembly. For more complex anchorage configurations that cannot be readily modeled in-situ, laboratory testing can be performed with the actual materials to simulate in service behavior. The ASTM Standard Test Method for Strength of Individual Stone Anchorage in Dimension Stone (C1354-96) may be applicable, depending on the anchor system being examined. Bear in mind that this type of testing is destructive and will require replacement of the panel sections tested.

Recommended Assessment and Inspection Intervals

Frequent inspections and evaluations are recommended to ensure that any deficiencies or maintenance needs are identified prior to the conditions becoming severe or a safety concern. In fact, many municipalities now require periodic inspection of all façades to encourage building owners to be proactive with needed maintenance and repairs. Although highly dependent upon the stone façade type, the age of the structure, and the level of maintenance performed, the author usually recommends semi-annual visual inspections consistent with the change of seasons, and more detailed close range inspections on a biannual basis. If a problem or deficiency is known to exist, then more frequent monitoring may be warranted.

Additional Resource for Inspection and Maintenance

ASTM has recently published a guide to inspecting and maintaining stone masonry façades, called ASTM Standard Guide for Assessment and Maintenance of Exterior Dimension Stone Masonry Wall and Façade (C1496-01). This document includes discussion of typical conditions observed and recommended corrective actions to be taken. It also includes a checklist for those responsible for façade inspection and maintenance to assist in prioritizing problems and determining when expert assistance is advised.

Summary

As a natural material, stone used in building façades has a unique appeal that cannot be found in other man-made materials. But the very characteristics that are appealing make it a somewhat unpredictable material, subject to naturally occurring variations, flaws and inconsistencies. For this reason the building profession including owners, facility maintenance personnel, design professionals and stone consultants, will continue to construct buildings of stone, and also continue to vigorously inspect and maintain them to ensure their long term performance and the safety of the public. It the author's hope that this paper has informed the reader of some of the important factors and unique considerations associated with the task of inspecting and assessing stone façades.

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[1] *Indiana Limestone Handbook*, 20th Edition, Indiana Limestone Institute of America, Inc., Bedford, IN, 1998, pp. 17-19, 21, 50, 52-55, 57-, 75-78.

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Section IV: Material and Repair Techniques

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Facade Inspections a Must for Both New and Old Buildings - A Case Study on Two High Rise Structures

Reference: McGinley, W. M. and Ernest, C. L., "**Facade Inspections a Must for Both New and Old Buildings – A Case Study on Two High Rise Structures,**" *Building Facade Maintenance, Repair and Inspection, ASTM STP 1444*, J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: To the untrained eye of a nonprofessional and even to the trained eye of many professionals, it is not uncommon for casual observance of the facade of a high rise structure to suggest all is well. However, one must look deeper to uncover obscure telltale signs that may reveal underlying problems of great significance. Two case studies illustrate that what is not readily obvious may be critical to public safety in facade inspection. These two studies include a 20 story building built in the mid 1920s with a steel frame and masonry cladding that had developed vertical cracking extending up the corners and a 29 story building barely a decade old which signaled distress when a panel of brick masonry veneer fell to the sidewalk below.

Keywords: building, facade, inspection, cladding, masonry, failures, case studies

Introduction

It is not uncommon for brief observance of the facade of a high-rise structure to suggest all is well. Even for many professionals who are familiar with the investigation of masonry veneer building facades, significant and serious problems may not be obvious at first glance. It is critical that these systems are examined in detail so that the sometimes obscure telltale signs that reveal underlying problems of great significance are not overlooked. The two case studies discussed in the following paper illustrate that sometimes what is not readily obvious may be critical to public safety in facade inspection.

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The Nissan Building

The Nissan Wagon Works, which dated from the early days of Old Salem (a historic Moravian community in Winston-Salem, North Carolina), began construction in 1926 on the Nissan Building in downtown Winston-Salem. When completed in 1927 its 18 stories with 2 additional floors of mechanical penthouses was the tallest building in North Carolina.

The building is U-shaped with six outside corners with the "U" facing northward (see Figure 1). The facade of the building on all but the south face consists of a brick masonry veneer from the 4th to the 14th floors, a limestone facade from grade to the 4th floor and limestone from the 14th floor to the roof level. The south facade of brick masonry veneer extends to the full height of the building. Both the brick and limestone facades are backed up with 8 inches (203 mm) of clay tile. The brick masonry facades were constructed continuously over the full height of the building with no horizontal shelf angles or relief joints, as was a common construction practice of the time. The clay tile back up was constructed supported on each floor and fitted tightly to the floor above. A combination of masonry headers and lightweight corrugated metal ties secured the brick to the clay tile back-up.

Figure 1 - *Nissan Building Elevation*

The structural system for this building consists of steel columns, beams and girders and a concrete joist floor system. All interior beams and girders are encased in concrete. The perimeter beams and girders were also intended to be encased in concrete, but the encasement was incomplete because the exterior face of the beams was not easily accessed in the construction process. The floor reinforcing steel typically just hooks over the outside of the top beam flange of perimeter beams and the beam was in-filled with brick as the exterior brick masonry was constructed. The steel columns were not encased in concrete but were tightly wrapped with masonry. The clay tile was used to surround the interior side of the columns and the exterior of the columns were in-filled with brick as the exterior brick masonry veneer was constructed. The original steel columns were built up using four steel angles and a web plate that were riveted together. Beams and girders are rolled shapes. Perimeter beams were eccentrically connected to the columns at the outside face using shear plates or angle seats riveted to the column flanges.

By 1991, five of the six building corners had cracked generally over the full height of the brick masonry with some cracking extending into the limestone facade below the brick (see Figure 2). The building owner did not recognize the distress as a potential structural concern but rather as an aesthetic nuisance. He had for a number of years paid contractors to fill the cracks with sealant "to keep the water out of the building" and to make the cracks disappear (see Figure 3).

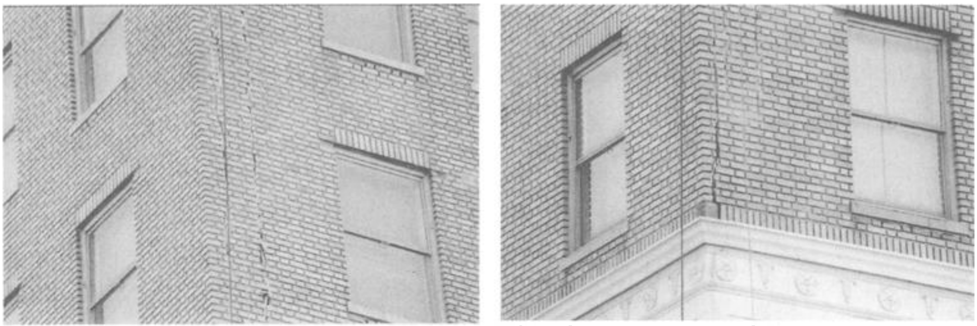


Figure 2 - *Corner Cracking in Brick and Limestone Facade*

Figure 3 - *Sealed Brick Cracks at the Building Corners*

When the latest sealant contractor removed sealant from the lower portion of one of the cracks, he noted water streaming from the wall interior and significant outward displacement of adjacent brick masonry. He wisely alerted the building owner to the potential for major facade and structural deterioration.

In response to the alert provided by the contractor, a facade inspection was commissioned. This investigation involved using a swing stage to access the wall face and to saw probe holes in strategic locations within the brick veneer and limestone, particularly near the cracked corners of the building. Severe corrosion of steel members at the corners was discovered (see Figure 4). Corrosion was found at all four columns along the north wall between the 4th and 14th floors. However, corrosion of the perimeter beams appeared to diminish significantly away from the corners. A surprisingly small amount of corrosion of the steel columns and beams was found below the fourth floor, behind the limestone facade. Much less severe corrosion was also found on the steel column within the fifth corner which exhibited cracking on the southwest corner of the building.

The perimeter columns and beams away from the corners were not found to be experiencing any significant corrosion. This lack of deterioration was probably due to the fact that the brick masonry was not cracked at these locations, and due to the presence of a cavity behind the brick veneer away from the corner sections.

Figure 4 - *Typical Corrosion of the Steel Columns at the Building Corners*

The severity of the corrosion on the four north face columns was sufficient to convince the building owner to strip the masonry from the exterior of these columns in order to permit a detailed examination of these areas. Severe corrosion was found to be present over nearly the full height between the 4th and 14th floors, on all four columns. In fact, just above the 12th floor, one of the columns had experienced such severe corrosion that the outer flange was reduced in width and the 5/16" (7.9 mm) steel angle leg was reduced to an irregular knife edge (see Figure 5). Subsequent measurements and calculations found the column cross-section at this location to be reduced by about 20%. Since this column supported 6 floors, and in response to the observed wide spread corrosion, the thickness of sound steel in all of the exposed columns and beams was systematically and extensively measured using ultrasonic techniques.

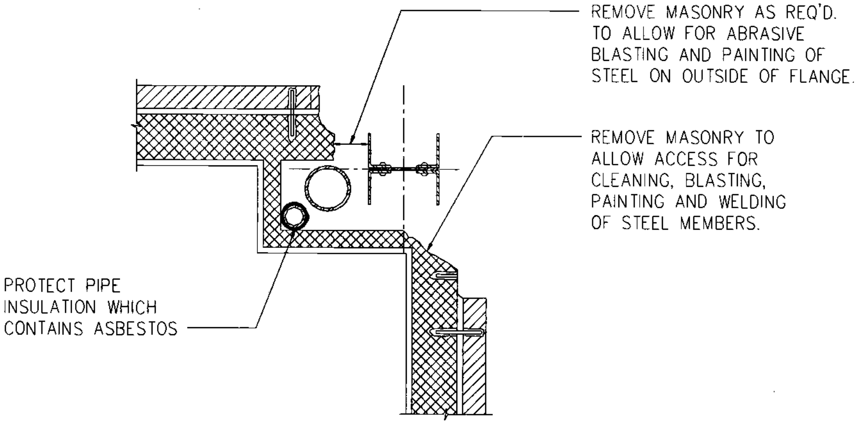
As a result of the investigation and observed corrosion, there was a significant concern about safety. This building was occupied as an office building and the four columns on the north facade abut a busy public sidewalk. The corrosion had severely weakened the steel perimeter elements and the adequacy of these elements had to be addressed.

Figure 5 - *Severe Corrosion of the Steel Column Flanges*

The original construction drawings provided the column schedule and the loads on each column line, at each level. The schedule did not however provide section sizes but simply listed the weight per foot of the columns rather than the components used to build it up. Utilizing the weights on the drawings and the field measurements it was possible to determine the angles and plates used to fabricate the columns and to calculate both the corroded and un-corroded column section properties. The typical column cross-section is shown in the figure below (see Figure 6).

To evaluate the safety of the steel columns and beams, loads were estimated for the four corner columns for three load cases: (1) Dead Load, (2) Dead load + a 10 psf (0.479 kN/m) Live Load and (3) Dead Load + a 50 psf (2.39 kN/m) Live Load + a 30 psf (1.44 kN/m) Interior Partition Load. The live load for Load Case 2 was believed to be an approximation of the actual load present on the column at the time of the investigation while Load Case 3 was the code prescribed live load and partition load as defined in the North Carolina Building Code [*1*] in effect at the time of the investigation. The calculated total loads exceeded those listed on the drawings by about 20%. The calculated dead load exceeded the dead load on the drawings by about the weight of the limestone above the 14th level, and it is speculated that the original designer omitted this load in error.

The trade name "Carnegie" was found embossed on one of the steel members, so the 1921 Carnegie Steel Manual was consulted during the analysis. It is interesting to note that this document predated the first American Institute of Steel Construction (AISC) Manual published in 1927. It was determined that the steel likely had a yield of 27.5 ksi (189.6 MPa) and was probably designed for a maximum compressive stress of 13 to 15 ksi (89.6 to 103.4 MPa). At each critical location, for each of the measured cross sections, the factor of safety, FS_b, against "Critical Euler Buckling" and against yielding due to "Compression plus Eccentric Bending" , FS_{y1}, were calculated for the DL + 10 psf (0.479 kN/m) LL load case for the four corner columns. The factor of safety against yielding, FS_{y2}, under full compressive load (no bending) was also calculated for the full design load, DL + 50 psf (2.39 kN/m) LL + 30 psf (1.44 kN/m) Interior Partition Load. FS_b ranged from 2.53 to 8.22, FS_{y1} ranged from 1.25 to 1.76 and FS_{y2} was 1.08 at its worst location. As a result of this analysis, it was concluded that it was safe to repair the columns without shoring the building or removing occupants. As a precaution the building owner removed all storage loads from the corner bays.



TYPICAL DEMOLITION AT CORNER

Figure - 6 *Existing Column Cross Section* \

To strengthen the columns and bring the building structural capacity up to current code required load levels, both concrete encasement and structural steel reinforcement were considered. However, any concrete encasement would have to extend to the foundation, which required working through space occupied by a bank on the first level and would have required significant disruption in corner office space of the nearly fully occupied building. The steel reinforcement strategy, on the other hand, would require significant welding, with the fire hazards that go with this activity. In addition, it was determined that load sharing with the existing steel would not be possible since the full design loads would place the existing steel 15% to 20% beyond its allowable stress limit. Thus it was decided that the remedial section would be designed to support all the loads and the existing steel section would be ignored. This strategy significantly reduced the amount of welding required.

After lengthy discussion with the building owner it was determined that the masonry wrapping around the corner columns could be removed from the exterior corner and new steel column and beam sections could be added from the exterior. This remediation scheme had the advantage of producing minimal interior disruption in the occupied building and allowed avoidance of the lower floors near the bank because little corrosion was present in this location. The additional steel was designed to support 100% of the critical design load.

The scope of repairs required to strengthen the steel columns consisted of removing all masonry around the column to permit access, sandblast all exposed surfaces of the existing steel to a near white finish (see Figure 7), taping all areas where welding would occur, painting all existing steel with an organic zinc rich paint and welding steel shapes in place over the full height of each of the four corner columns. Levels 4 through 10 were strengthened by adding three 5" x 4" x ¾" (127 x 102 x 19 mm) angles and an angle of two plates built-up to permit access for final welding on the interior side of the column. Levels 10 through 14 were strengthened by adding three 4" x 4" x ½ " (102 x 102 x 13 mm) angles and an angle of two built-up plates to permit access for final welding. All steel was grade 50. Angles and plates were located as shown in Figure 8.

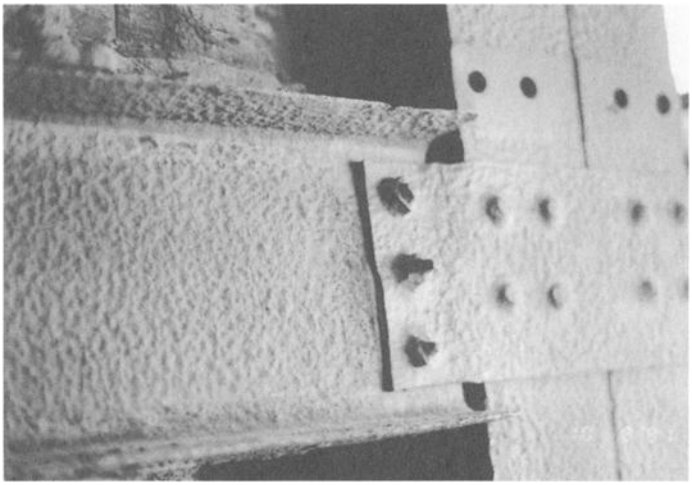


Figure 7 - *Prepared Existing Steel Column and Beam*

Four unique problems had to be addressed during the design to ensure all load paths were maintained or redirected to the added steel supports and to ensure that all stresses were within allowable limits. First, at a number of locations the rivets interfered with installation of the new angles and plates. This was addressed in two ways. Since the same size angles were used at several consecutive floors the angles were sized based on the stress conditions at the bottom of the grouping, i.e. levels 4 and 10. At all other floors the added reinforcing was somewhat larger than necessary. Where the angles were larger than necessary the angles were typically notched to pass by the rivets. The reduced section was still sufficient to maintain the stress within an allowable level. Where necessary at the lower levels and at other locations where notching removed too much steel, the added element was reinforced with a plate to strengthen the section.

The second problem area involved the transition splices between added members. Full penetration welding was utilized where splices occurred between ends of angles. Where angles changed sizes just above the 10th floor level, a tapered transition splice was devised to permit loads to be transferred while avoiding stress increasing eccentricities.

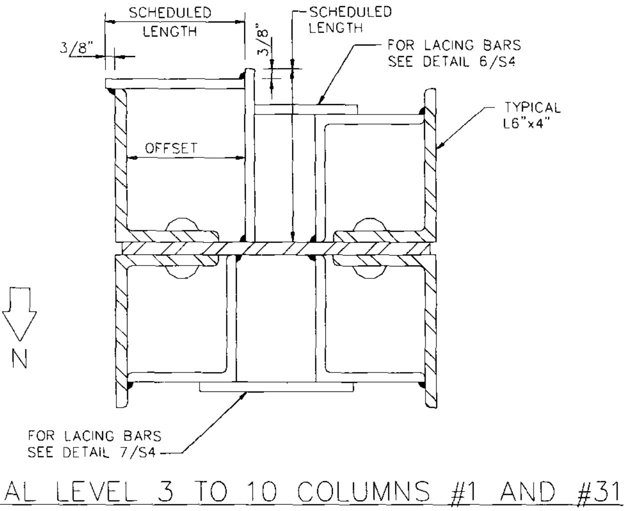


Figure 8 - *Section Detail for the Repaired Columns*

The third area of concern was the beam-to-column connections. In most areas it was necessary to transfer all beam loads to the added steel shapes by providing new connections. Where the existing beams framed parallel to the flanges of the columns they were connected to the column flange with a shear plate connected to the web of the beam. Where the existing beams framed perpendicular to the flanges of the columns they were aligned approximately with the outer edge of the flange and were supported upon an angle seat connected to the face of the flange. A seat connection was designed to transfer the beam load into the new vertical angle reinforcing where existing shear plates were present. A shear plate was designed to transfer the beam load into the new vertical angle reinforcing where existing angle seats were present.

Lastly, the strengthening of spandrel steel beams that were corroded had to be addressed. The brick veneer had only been removed at the corner columns and had not been removed from along the spandrel beams. At locations where evidence of corrosion extended behind brick along the spandrel beams, the brick veneer and brick infill between the top and bottom flange was removed until corrosion was sufficiently reduced to require no repairs. As brick was removed the masonry above was supported using screw jacks placed intermittently along the opening. Beams were then sandblasted, taped and painted, and a reinforcing web plate was welded in place. Where necessary, the beam flanges were strengthened with plates and the brick veneer was reinstalled.

The closure of each corner was designed to ensure that cracking and water infiltration would not occur in the repaired corners. The masonry backup wall was not reinstalled around the exterior face of the corner steel columns. Steel shelf angles were installed at each floor level in these areas. Gypsum sheathing was installed on the exterior of the column with a vapor barrier installed, which extended some distance out onto the exterior face of the clay tile and concrete masonry unit back-up wall and down over metal flashing with drip edge extensions installed upon the shelf angles. Rigid insulation was installed on the exterior of the vapor barrier to prevent condensation on the interior steel surfaces. The brick veneer was installed with horizontal relief joints below the shelf angles and vertical expansion joints to isolate the corner masonry from the adjacent continuous brick facade. Silicone sealant was used in all relief and expansion joints, along with weeps at the shelf angle levels. Figure 9 shows a section of the completed corner repair and Figure 10 shows the repaired column corner after work had been competed.

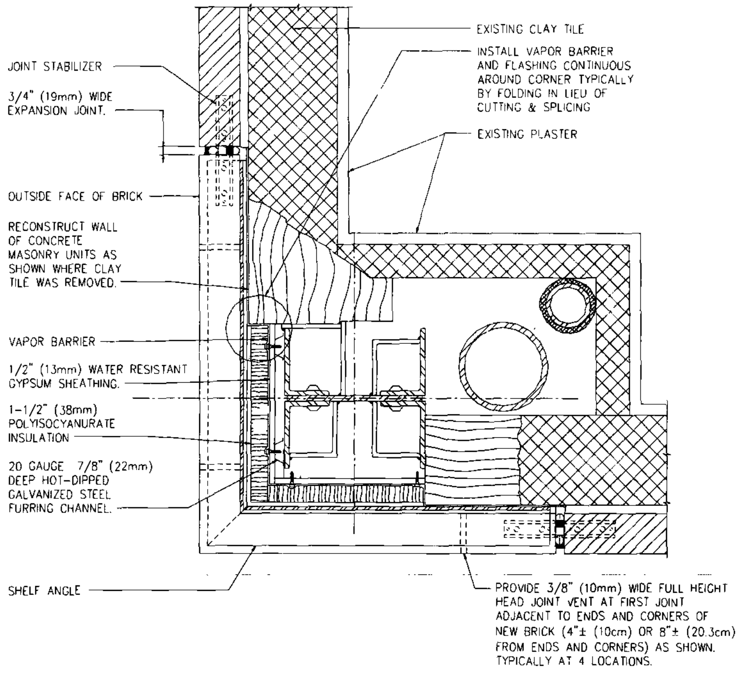


Figure 9 - *Typical Strengthened Corner Column Section*

Approximately one year following the completion of these repairs, the remaining cracked corner on the south face was stripped and repaired. Although outwardly exhibiting far less masonry cracking than the other four repaired corners, the most severe corrosion was found in the column on this corner (see Figure 11). As a result of these findings it was concluded that severe structural deterioration can truly be hidden behind minimal surface defects.

Figure 10 - *Completed Corner Repair*

Figure 11 – *Severe Column Flange Corrosion on Southwest Corner*

Harbor Court Condominium Tower

A 28 story steel framed condominium /office tower in the Inner Harbor area of Baltimore Maryland (Figure 12), came under scrutiny when a section of its brick veneer cracked, dropped off the building and landed on a public sidewalk. This building was designed and built in the mid 1980s and was only about a decade old when the brick veneer failure occurred. Luckily the failure occurred in the early morning hours and no one was around.

This event did, however, prompt an extensive investigation of the building and this investigation found a number of deficiencies in the building brick veneer/steel stud facade system and the steel shelf angle/steel beam supports. These deficiencies included many of conditions often found with this type of system built between the 1960s through 1980s *[2- 4]* such as:

1. Lack of adequate flashing, both at the shelf angles and windows, resulting in water damage to the wall system and the building interior;
2. Lack of properly functioning horizontal expansion joints under the shelf angle supports at the floor levels that caused bowing, cracking and localized spalling of the brick veneer;
3. Lack of and improperly installed veneer ties; and
4. Localized inadequate vertical, in-plane, and horizontal out-of-plane support of the brick veneer, particularly at the corners of the building.

The last item is of particular interest because these deficiencies were not obvious until a detailed investigation of these areas was conducted, including a close-up examination of the brick, vertical supports and backing systems, a plan review and a detailed structural analysis.

Examining the plan sections of a typical corner section (Figure 13) indicates that the brick veneer forms essentially a column cover at this location, with the vertical control joints at approximately 3 ft (.9 m) from each side. In addition, the presence of horizontal expansion joints at the floor levels was intended to separate each level of veneer and thus allow the brick sections at each corner to move independently. There were vertical steel studs placed near the edges of the column flanges, spanning between the floor slabs, that were sheathed on the exterior with exterior grade gypsum wall board. The brick veneer was attached to only the two studs near the column flanges using adjustable veneer ties. There were no veneer ties attached directly to the column. The adjustable ties used in the construction of this building used a "V" wire tie and a backing clip that was fastened to the steel stud using two screws. There was no backing plate between the wire and gypsum sheathing and this resulted in the wire tie bearing directly on the relatively low bearing strength gypsum sheathing.

This configuration of stud, tie and brick veneer was analyzed for performance under wind and dead loadings. The finite element model of the single story of the brick veneer column cover is shown below (Figure 14). The plate brick veneer elements were subjected to a critical suction loading of approximately 50 psf (2.39 kN/m^2) applied perpendicular to their face (as per American Society of Civil Engineering (ASCE), *Minimum Design Loads for Buildings and Other Structures*, ASCE Standard 7-93 American Society of Civil Engineers, Reston, Virginia, 1993).

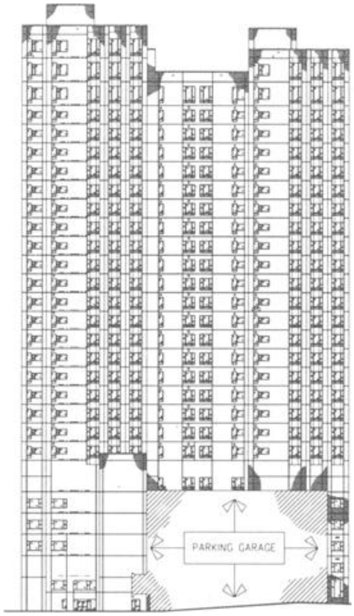


Figure 12 - *East Elevation of the Harbor Court Condominium Tower*

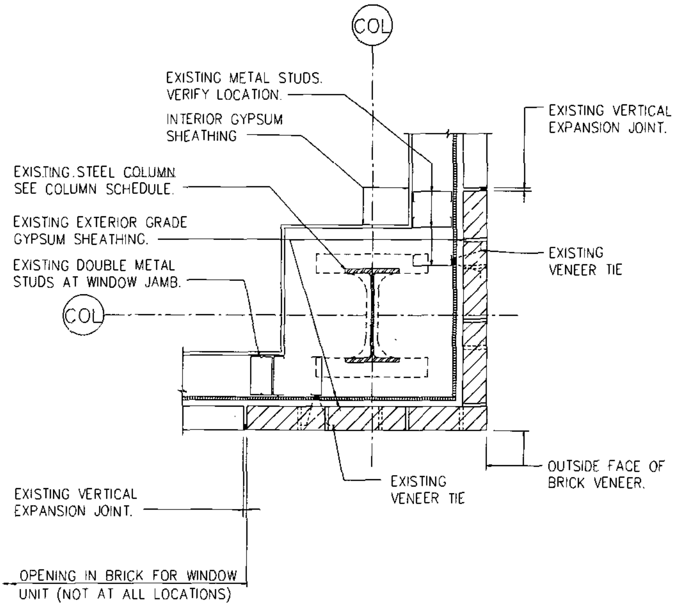


Figure 13 - *Plan Section-Typical Corner of the Harbor Court Condominium*

Figure 14 - *Finite Element Model of Corner Section of the Brick Veneer and Stud Supports*

Because the veneer ties have little or no strength in the plane of the brick, their resistance in this direction was ignored. It is fairly obvious from examination of the model that the brick veneer section is unstable under the out–of-plane wind loads. There are only two points of restraint in the out-of-plane direction and three are required for a stable system. The ties and the studs at these locations were also inadequate to resist the wind loading applied to them even if the localized bearing failure of the ties and instability was ignored. It was therefore determined that significant strengthening of the out-of-plane support system was required at the corners of the tower. It should be noted that a wind load study was commissioned for this building and the code defined wind loads were found to be only slightly conservative in the upper sections of the building.

This instability of the corner sections of the veneer was not obvious during the initial review of the building because there was no significant distress observed in most of the brick in these areas. It was only after intrusive investigations turned up a number of poorly fastened shelf angles that additional scrutiny was applied to these corner sections of brick and exposed the support deficiencies.

It is likely that the lack of distress of the brick veneer corner sections can be attributed to the absence of the design level wind loads thus far in the life of the building, the clamping forces/friction developed at the under side of the shelf angles due to the restrained brick expansion combined with nonfunctional expansion joints and frame shortening, and the so called "structural caulking (or some combination thereof). It is disturbing to note that these clamping/friction forces were to be relieved as part of the remediation of the brick distress produced by the other deficiencies in the building and this would have weakened these areas further if the lack of support had not been discovered.

Understanding of how the building systems function under loading, knowledge of where potential problems may exist and a systematic site investigation is very important in the evaluation of these facade systems since even careful review of the Harbor Court building plans may not have disclosed the deficiencies in the corner areas. There were notes on the plans that indicated that the studs should be spaced at no more than 16" (406 mm) centers and this implied that they should be located around the column, even though in most cases the 6" Stud (152 mm) would not fit around the column section. As can be seen from examination of the section (Figure 15) the remediation effort added an additional support point on each veneer face, attached directly to the column section. This out-of-plane support was located near the mid height of the sections and near the floor levels (Figure 15).

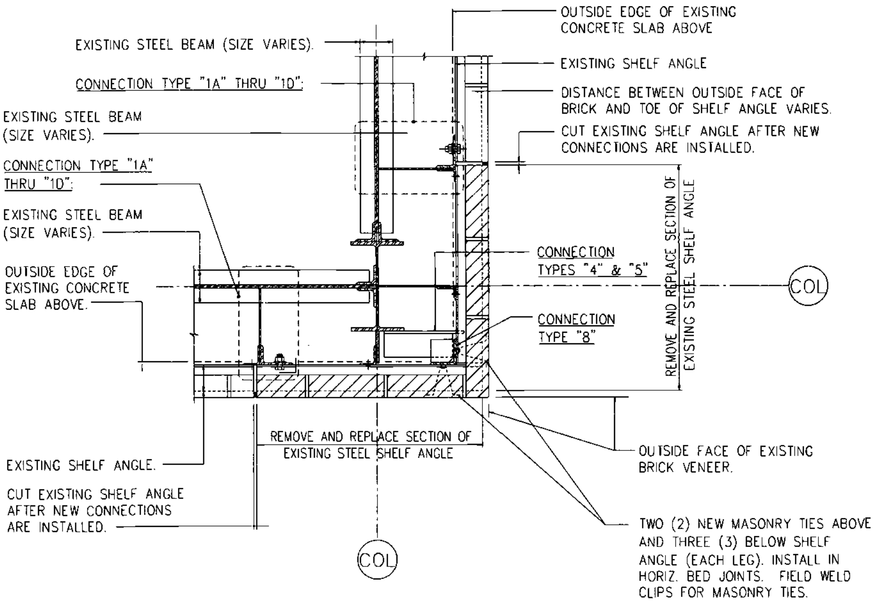


Figure 15 - *Plan Section of Corner Area Near Floor Level*

Careful examination of the section above (Figure 15) also shows vertical support of the brick veneer involves the use of steel shelf angles attached to spandrel beams and the steel column section with shear plates. Site inspection showed that in the corner sections, the steel angles consistently extended across the vertical expansion joints to a shear plate support on the beam. Further, these examinations showed that, in most cases, the steel angle cantilevered from the shear plate attached to the column and did not extend to the corner of the brick.

The finite element model shown in Figure 14 was analyzed for the dead load of the brick and it was found that these support configurations were far too flexible. In addition, the continuity of the steel shelf angle through the vertical expansion joints coupled with this flexibility may have caused some of the observed brick cracking and spalling. The remediation strategy required the steel shelf angle at the expansion joint to be cut and to provide an additional bolted shear plate connection for the corner angle beside the vertical expansion joint.

The vertical finite element analysis also showed that the "conventional" analysis procedure typically used for the design of the shelf angle is flawed. This type of design often is done by assuming that the shelf angle acts as a beam in simple flexure spanning between supports, in this case the shear plates. However, the eccentric load of the veneer on the angle places significant torsion on the torsionally weak angle. The brick is quite stiff relative to the flexible angle and appears to span between the more rigid sections of angle near the shear plates. This suggests that the shelf angle could be sized to support brick while being laid up (potentially with shores) and detailed to allow the brick to span between these support areas when cured (without shores).

Summary and Conclusions

The two case studies discussed in this paper illustrate that in the inspection of masonry veneer facade systems it is sometimes not readily obvious where significant system deficiencies critical to the performance of the building and to public safety may exist. Even for many professionals who are familiar with the investigation of masonry veneer building facades, significant and serious problems may not be obvious at first glance. The first of the two case studies showed that simple brick cracking may hide significant and dangerous corrosion of the building structural supports. The second of the two case studies showed that even though there was no obvious outward masonry distress, significant structural deficiencies may still exist. It is critical that professionals knowledgeable in the performance of masonry facades examine these systems in detail so that the sometimes obscure telltale signs that reveal underlying problems of great significance are not overlooked.

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Benjamin Lavon, P.E.[1]

How Deteriorated Can Marble Facades Get? Investigation and Design of Repairs

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REFERENCE Lavon, B., **"How Deteriorated Can Marble Facades Get? Investigation and Design of Repairs,"** *Building Facade Maintenance, Repair and Inspection*, *ASTM STP 1444*, J.L. Erdly and T.A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

ABSTRACT: The existing condition of the exterior marble panel facade of a high-rise building in midtown Manhattan were unsafe. A number of marble faced panels had bulged out and were ready to pop out and fall down to the crowded street below. These deteriorated and unsafe conditions necessitated a facade investigation, close examinations, field testing of marble, and analyses of the deteriorated marble panels. The causes for the failure were determined in order to design effective repairs.

KEYWORDS: facades, marble panels, hi-rise, deterioration, unsafe conditions, field testing, repair work design

Introduction

Concerns regarding the deteriorated condition of the exterior marble panel facade of a high-rise building in midtown Manhattan were raised after a number of marble faced panels had bulged out and were ready to pop out and fall down to the crowded street below.

Due to the urgency of these conditions, a facade investigation and repair program was conducted by Feld, Kaminetzky & Cohen, P.C. (FKC) on behalf of the Owner. Close examinations and field-testing of the failed marble panels were performed from hanging scaffolds, and analyses of the marble panels took place. In order to minimize disruption to the facility and street traffic below, it was necessary to determine the causes for the failure and design effective repairs, in a very short time. Our findings, conclusions, recommendations, and lessons learned from this investigation are presented herewith.

FKC's scope of work was as follows:

a. Review existing construction documents, including architectural and structural drawings.

b. Visit the site to investigate the existing facade conditions.

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c. Form hands-on scaffold drop observations at selected areas; perform stress relief tests and probes of the bulged marble panels; perform boroscope observations.
d. Select physical samples to be tested by a laboratory.
e. Report on the findings, and submit our conclusions and recommendations.
f. Design repairs.
g. Administer construction.

Site Location

The building is a highly visible office building and an advertising icon facility that is located in midtown Manhattan, in the Time Square area. Several facades carry huge advertising signs and displays. The streets around the building are subject to heavy pedestrian and vehicular traffic and therefore public safety is of extreme importance.

Type of Construction

The structure is 23 stories high (340 ft., or 113 m, from street level to top of the roof parapet) with four cellar levels below the street level. The structural frame is constructed of concrete encased structural steel beams and columns. The exterior walls are constructed of precast reinforced concrete panels compositely cast with marble facing. The marble facing is 7/8 in. thick ($2.^2$ cm).

Facade Investigation

Visual Inspection

The facade elevations were inspected from the street level by the naked eye at lower levels and by using binoculars and telephoto lenses at upper levels.

Scaffolds were used for close-up inspections, stress relief tests, and probes at selected failed bulged marble panels.

The facades were 23 stories high and consisted of precast concrete panels cast with marble veneer facing. A typical precast panel measured approximately 12 ft. (4 m) high by 9 ft. (3 m) wide, faced with twelve 3 ft. (1 m) square by 7/8 in. ($2.^2$ cm) thick marble panels. Joint fillers, 3/16 in. (5 mm) wide and 3/8 in. (10 mm) wide, including rubber gaskets, with a channel shape cross section, were provided around all the marble panels. The marble facing was bonded to the precast concrete panel and attached to it by means of stainless steel pins that engaged only half of the marble panel thickness.

A number of the marble panels had some local previous repairs done in the past. The previously repaired panels included broken corners (that had been repaired), replaced pieces of marble (dutchman), patches, etc.

A number of marble panels exhibited bulging out from the vertical plane by as much as approximately ½ in. (1.2 cm) to 1 in. (2.5 cm). Additionally, there were many irregularities in the marble panel facades, such as misalignment of the panels in the vertical and horizontal directions.

Since several marble panels were severely bulged out and loose, it was necessary to install sidewalk bridges along the affected facades for protection purposes.

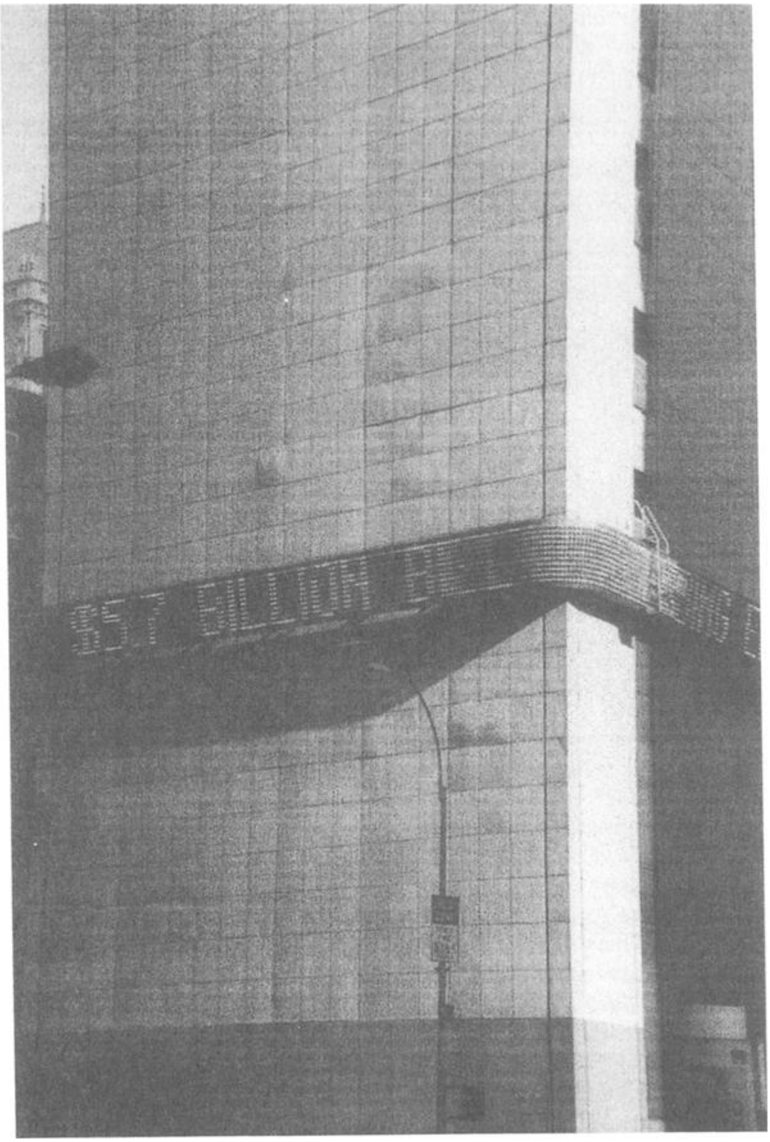

FIG. 1

Boroscope Observations

The boroscope is a fiberoptic device that enables an observer to view inside an existing cavity through bore holes that are drilled for this purpose. The drilled holes are 5/8 in. (1.5 cm) in diameter. Several boroscope observations were performed by FKC staff in order to determine the as-built details and conditions of the inner face of the precast panels, their integrity, and their connections to the structure. The as-built details were subsequently compared with the available design details.

It was observed that the typical precast panels were connected to the structure with a total of five clip angles, each. Two top and three bottom clip angles connected welded plates, embedded in the concrete precast panel, to the structural steel spandrel beams. The observed conditions at the connections indicated no evidence of structural distress, corrosion, or water damage.

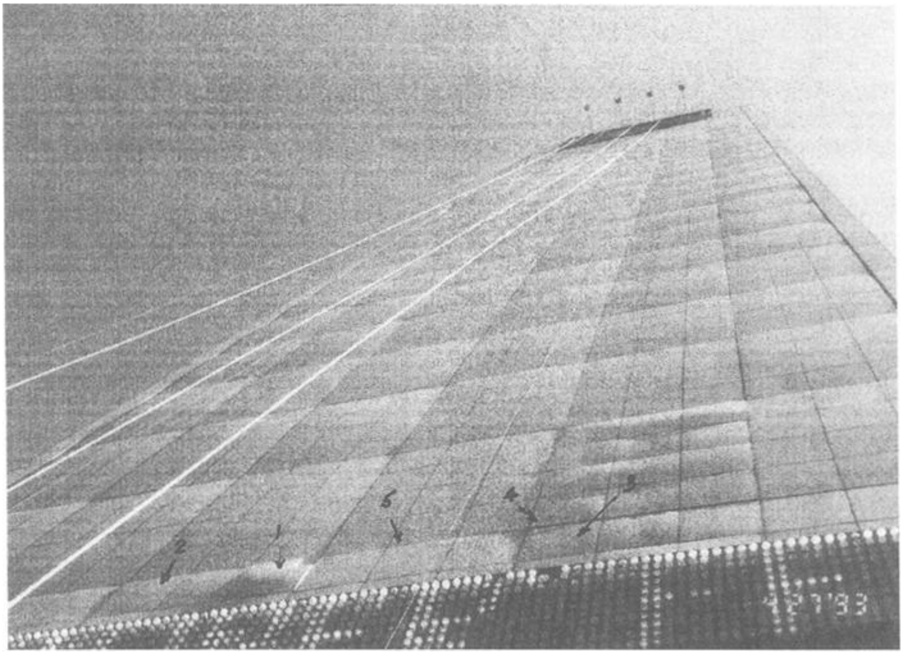

FIG. 2

Close Range Scaffold Observations, Probes, Stress Relief Tests

Close-up examinations of selected marble panels were performed at several elevations. The examinations included exploratory probes, removal of portions of and whole marble panels, stress relief test, and close-range panel inspection from hanging scaffolds. The probe cuts revealed that the marble facing had deteriorated, delaminated and separated from the back-up precast concrete panels. Additionally, behind the rubber gasket joint filler, there was an existing mortar joint between the marble panels, approximately 3/16 in. (.5 cm) wide by 1/4 in. (.6 cm) thick, i.e., approximately one quarter of the panel thickness.

A stress relief test of a selected severely bulged panel, 1 in. (2.5 cm) bulge, was conducted with the use of field-mounted strain gauges. The stress relief tests performed revealed that the panel was stressed in a cylindrical bending mode where the compression axis was at the inner face. The width of the mortar layer between the marble panels was approximately 1/4 in. (.6 cm), i.e. roughly one quarter of the panel thickness. As a result of this bending compression, the outer average strain was 150 microstrains. For an assumed modulus of elasticity of 2 000 ksi (14 000 MPa), as expected for this type of material, the equivalent calculated tested stress was 50 psi (350 kPa). Additionally, the stress relief tests revealed that the tested panel was stressed in a cylindrical bendind mode. It was determined that the nature of the strains and the stresses measured during these tests were consistent with bending related rather than compression of the said marble panel. The causes for the bending mechanism of the marble panel were associated with the difference in the physical properties of the deteriorated exterior surface versus the protected interior surface of the said marble panel. The exterior surface of the panel had weathered, deteriorated, and lost its strength and modulus of elasticity due to the harsh environment it had been subjected to. According to investigations and tests, by FKC over the years, physical properties of marble will deteriorate substantially when exposed to the elements. Test results, as indicated by Dov Kaminetzky [1] show that exposed marble panels may loose in average 80% of their original flexural strength and 50% of their modulus of elasticity. Furthermore, when the said tested panel was removed it was revealed that the marble material had actually deteriorated to such an extent that the marble piece was crumbling at the touch of the fingers.

The existing stainless steel pins anchoring the marble facing to the precast concrete panel were disengaged and no longer attached to solid marble material, and were actually loose and hanging "in the air." The marble facing was debonded of the precast concrete panel back-up. What prevented this marble panel from falling off to the crowded street below was only the glued-on continuous rubber gasket joint filler. Since this condition was judged to be hazardous, the entire marble unit was subsequently removed, and the area was made temporarily weatherproof.

Several other probed panels, with measured bulging of up to ½ in. (1.2 cm), were probed and inspected. They exhibited marble panel material that appeared to be in relatively good condition. Some rusting of stainless steel pins was observed. However, no loss of stainless pin anchorage was noted. The marble panels were originally bonded to the back-up precast panels. The continuous rubber gasket around the probed panels appeared generally to be in good condition. However, at a few locations, the rubber gaskets were found to be deteriorated and in need of attention.

Additionally, several physical samples of the marble panel pieces, removed during the probing operations, were forwarded to a laboratory for petrographic examination and physical testing. The petrographic and laboratory test results confirmed the severe deterioration of bulged marble panels over 5/8 in. (1.5 cm), due to exposure to adverse weather conditions. Test results of moderately bulged panels, less than ½ in. (12 mm), indicated no substantial loss of physical properties.

Based on our investigation, we have categorized the marble panel deterioration and bulging as follows:

1. Bulged panels from 5/8 in. (1.5 cm) to 1 in. (2.5 cm): This condition was unsafe and required the complete replacement of these panels.
2. Bulged panels from 3/8 in. (1 cm) to 5/8 in. (1.5 cm): This condition required repairs, as noted.
3. Panels with bulges of less than 1/4 in. (6 mm) were judged to be acceptable. No repairs were needed at that time.

Refer to Figures 1 to 7 for selected photographs of the marble panel conditions, probing, and repair details.

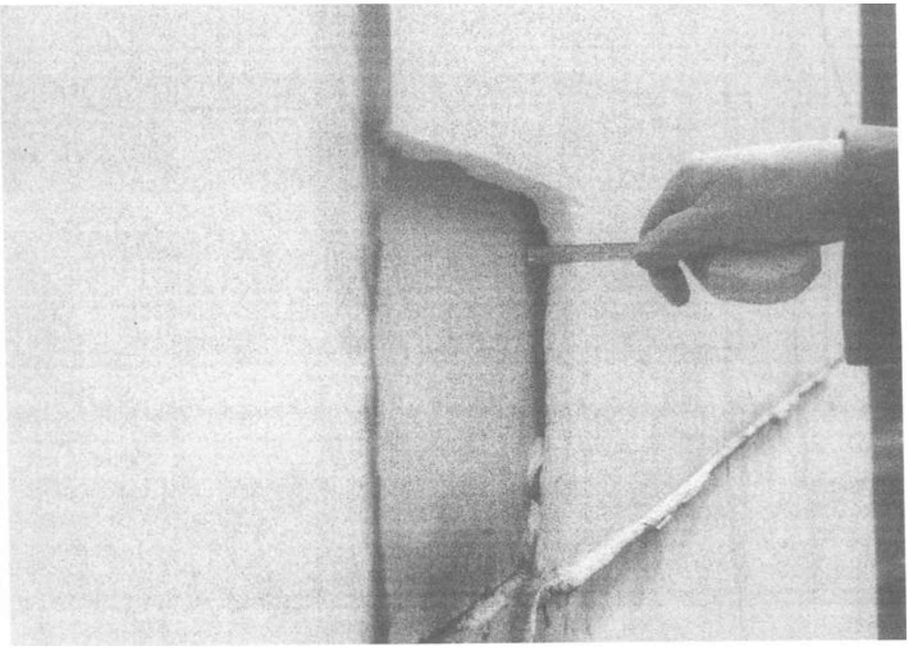

FIG. 3

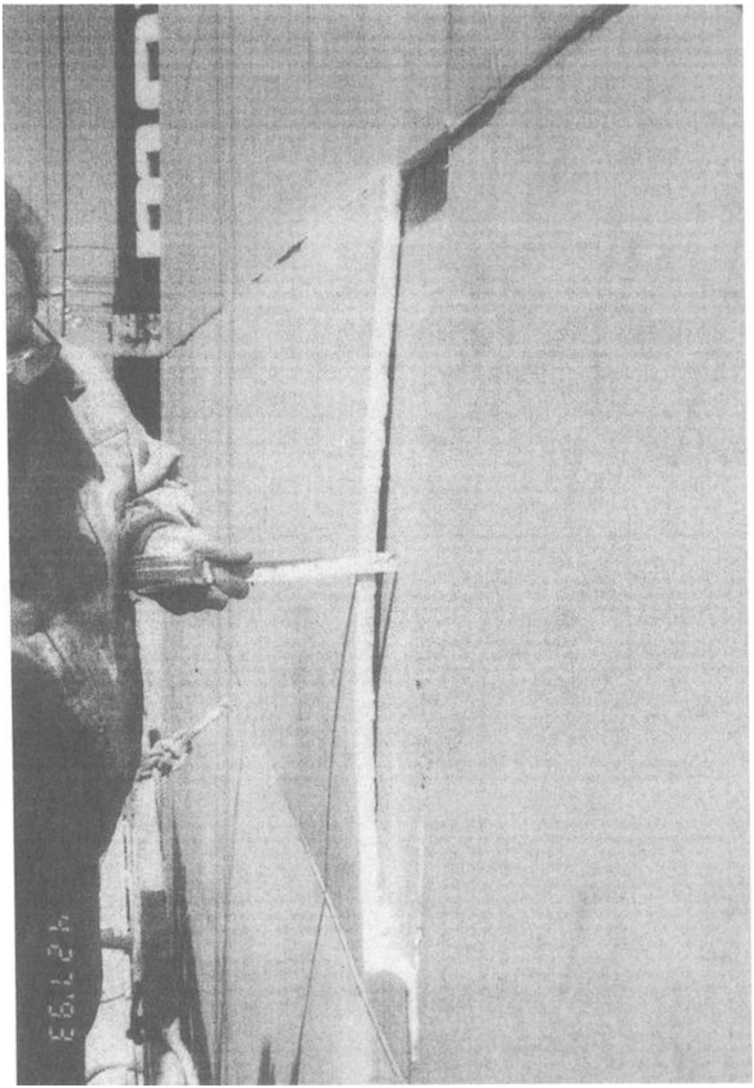

FIG. 4

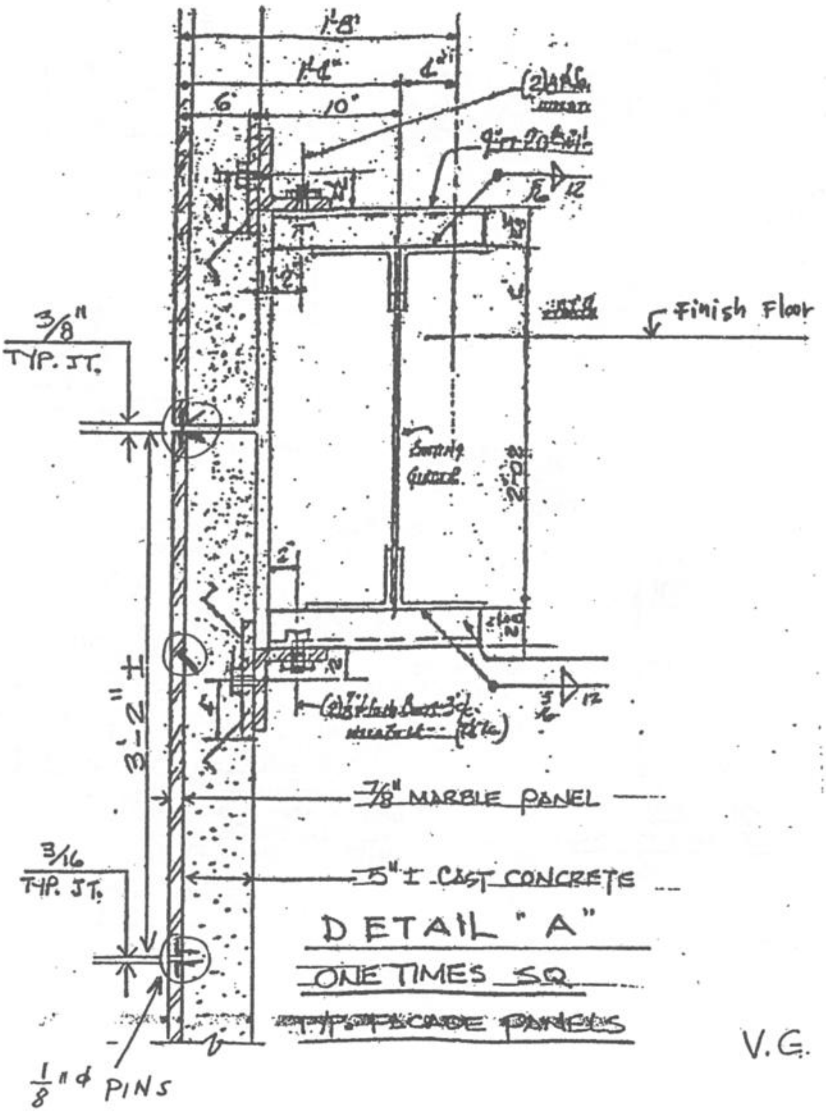
1'-8"
1'-4"
4"
6"
10"
Finish Floor
3/8" TYP. JT.
Existing Girder
3'-2" ±
7/8" MARBLE PANEL
5" ± CAST CONCRETE
3/16 TYP. JT.
DETAIL "A"
ONE TIMES SQ
TYP. FACADE PANELS
V.G.
1/8"φ PINS

FIG. 5

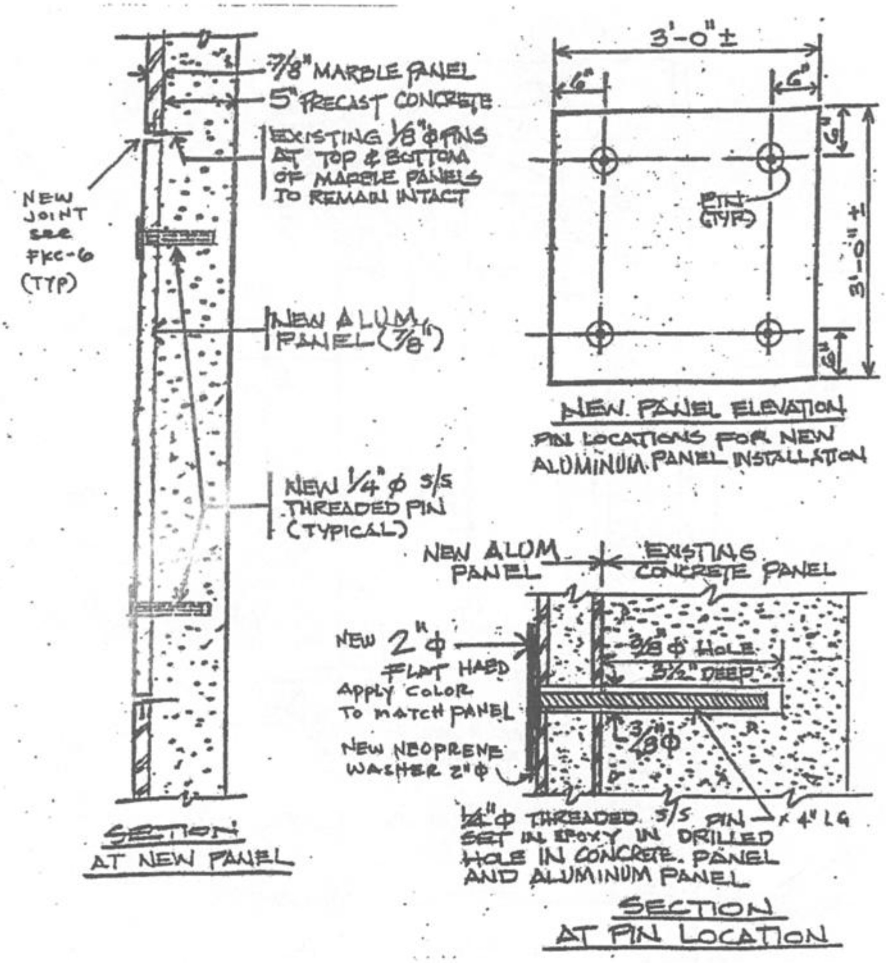
7/8" MARBLE PANEL
5" PRECAST CONCRETE
EXISTING 1/8"φ PINS AT TOP & BOTTOM OF MARBLE PANELS TO REMAIN INTACT
NEW JOINT see FKC-6 (TYP)
NEW ALUM. PANEL (7/8")
NEW 1/4"φ S/S THREADED PIN (TYPICAL)
SECTION AT NEW PANEL
3'-0"±
6"
6"
PIN (TYP)
3'-0"±
6"
6"
NEW PANEL ELEVATION
PIN LOCATIONS FOR NEW ALUMINUM PANEL INSTALLATION
NEW ALUM PANEL
EXISTING CONCRETE PANEL
NEW 2"φ FLAT HEAD APPLY COLOR TO MATCH PANEL
NEW NEOPRENE WASHER 2"φ
3/8"φ HOLE
3 1/2" DEEP
3/8"φ
1/4"φ THREADED S/S PIN × 4" LG SET IN EPOXY IN DRILLED HOLE IN CONCRETE. PANEL AND ALUMINUM PANEL
SECTION AT PIN LOCATION

FIG. 6

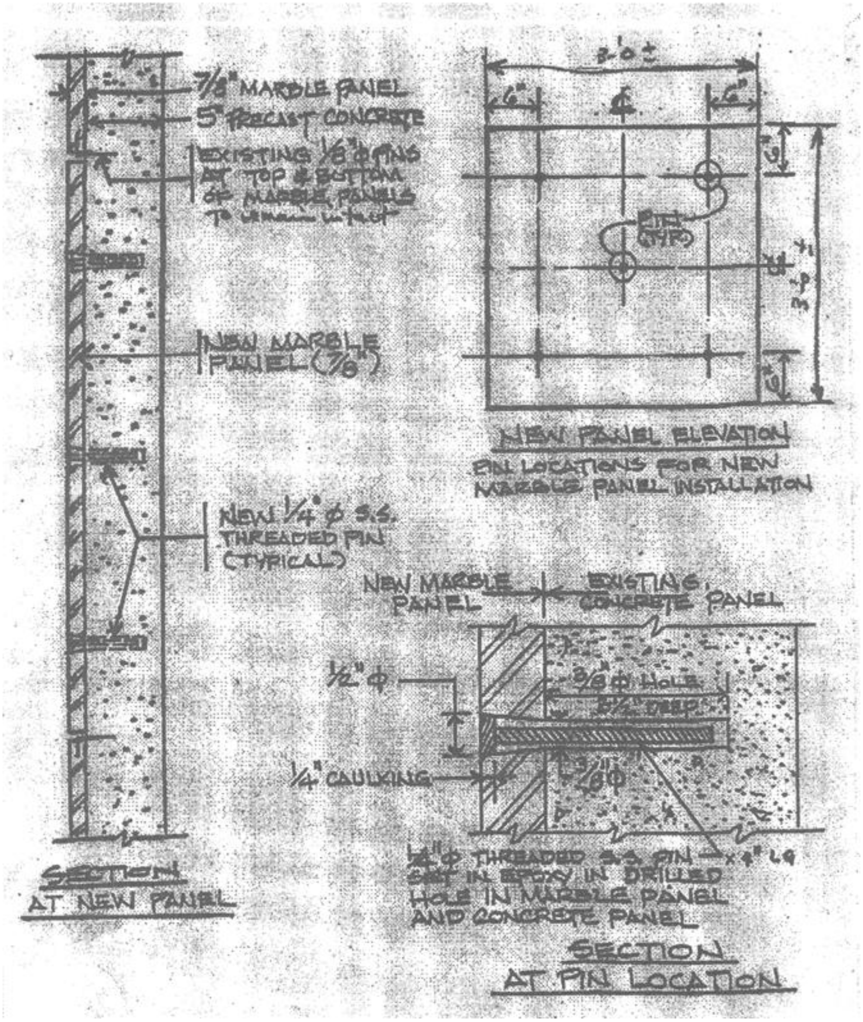

FIG. 7

Conclusions and Recommendations

1. The causes for the marble panel deterioration were the severe exposure conditions of the marble panels to the harsh environmental elements in New York City. As a result, a number of bulged marble panels had deteriorated to such a degree that they crumbled at the touch of the fingers. The marble panel bulging was up to 1 in. (2.5 cm).

2. The joint fillers around the facade panels, consisted of rubber gaskets, which had performed well and allowed the marble units to move freely during temperature changes. Thus, the reported bulging, damage and deterioration of the marble panels was not related to temperature changes.
3. The severely bulged marble panels from 5/8 in. (1.5 cm) to 1 in. (2.5 cm) were unsafe and required immediate replacement. FKC recommended and developed replacement details for these conditions using two alternates: new marble panel placement, and new metal panel placement.
4. The moderately bulged marble panels, from 3/8 in. (1 cm) to 5/8 in. (1.5 cm) required repairs. Effective repair details were recommended and developed for the various types of needed repairs, such as installation of new pins and partial marble replacement.

Lessons Learned

1. The original pin mechanical attachment method of the 7/8 in. (2.2 cm) thick marble panels to the back-up precast concrete panels limited the extent of the pin anchorage to engage only half of the full marble panel thickness. Thus, the pin anchorage was very sensitive to the marble panel physical condition changes and deterioration. Subsequently, the extremely bulged and deteriorated marble panels became debonded, loose, disengaged, and were ready to fall off.
2. In order to correct the pin anchorage deficiency, FKC has designed an improved pin attachment method that engaged the entire marble panel thickness.
3. When designing new marble panels:
    a. The designers should select marble materials that have compatible physical properties with those of the back-up walls, such as coefficient of thermal expansion and modulus of elasticity. Compatible materials will minimize the potential for debonding.
    b. When it is not possible to ensure that the physical properties of the marble panels are compatible with those of the back-up walls, prevent bonding of the marble panels to the back-up walls. Attach the marble panels by mechanical means only, such as pins.
    c. Test selected marble panel samples in order to confirm that the marble panel properties meet the design requirements.

References

[1] Kaminetzky D., "Design and Construction Failures: Lessons from Forensic Investigations," McGraw-Hill, New York, NY, 1991.

Timothy T. Taylor,[1] and Frederick M. Hueston[2]

STONE FACADE INSPECTION OF 1776 F STREET

Reference: Taylor, T.T., and Hueston, F.M., "**Stone Facade Inspection of 1776 F Street,**" *Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444*, J.L. Erdly and T.A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

ABSTRACT: This paper will address the inspection of an exterior stone facade for a commercial office building. The existing building was constructed during 1970 and 1971 with a cast-in-place concrete structural frame, which is clad with bonded travertine-faced precast concrete panels, mechanically anchored travertine perimeter column covers, and discontinuous strip windows. Travertine-faced precast concrete panels directly support the windows. The inspection procedures included the acquisition of existing drawings, document review, formulation of visual interior and exterior inspection procedures for cladding components, and physical testing recommendation development. Physical testing techniques included a laboratory examination of the weathered travertine; petrographic, chloride ion, and air void analysis of concrete cores taken from existing precast panels; and the locating of reinforcing bars within the existing precast panels. While the presence of widespread systemic failures of the stone cladding system are revealed, the inspections indicated that the precast panels are serviceable.

Background

The District of Columbia currently lacks facade inspection laws ensuring public health and safety and the protection of adjacent properties. As in many other jurisdictions, building owners having such concerns have turned to the design community for advice in the matters of building facade inspection, maintenance and repair. The subject of this paper began by a building owner engaging the authors to develop cleaning methods and specifications for its travertine-clad building. When the author's observed the fragile nature of the existing cladding and noted pieces of cladding lying at the base of the building's perimeter, the owner expanded the cleaning program into an exterior wall inspection program. This eventually resulted in a full recladding of the building facade.

---

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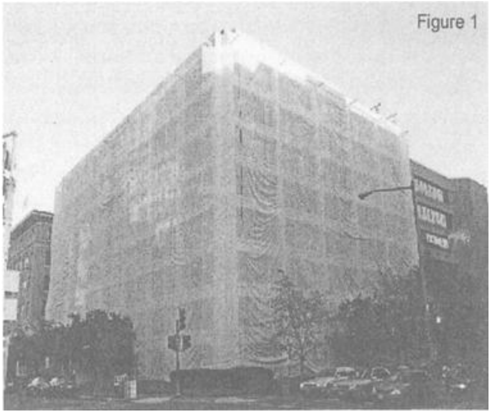
Data Acquisition



The building is an eight-story, free-standing structure located on the corner of a city block at a busy street intersection (Figure 1 shown with shrouding). Stone repairs had been performed to the facades, as evidenced by the presence of dutchmen that can be seen from street level. The building shows no evidence of structural frame deformation in the facade components, broken glass, or severe staining from the street level. On the interior side of the exterior walls, and at the ceiling plane immediately above the exterior window openings, we found no water damage.

The facade is primarily composed of combination hairpin anchored and concrete-bonded travertine-faced precast concrete panels attached to the building frame perimeter spandrel beams, mechanically anchored travertine cladding to concrete frame perimeter columns, and discontinuous strip windows (Figure 2). Unfortunately, no project specifications, except those that existed on the drawings, nor as-built drawings, are available.

These observations and records showed how the existing building facade was designed, and to a certain point how it was performing. Street-level observations were not sufficient and the historic records were too incomplete to pinpoint the cause(s) for the weakened and falling stone and what would be needed to remediate it.

Facade Inspections

Visual Inspection Preparations:

Using the original building elevation drawings, we developed an identifier system that divided the facade into small repeating areas. This identifier system included the site plan compass direction of each facade, the swing stage drop, and the floor elevation as well as individual stone piece numbers.

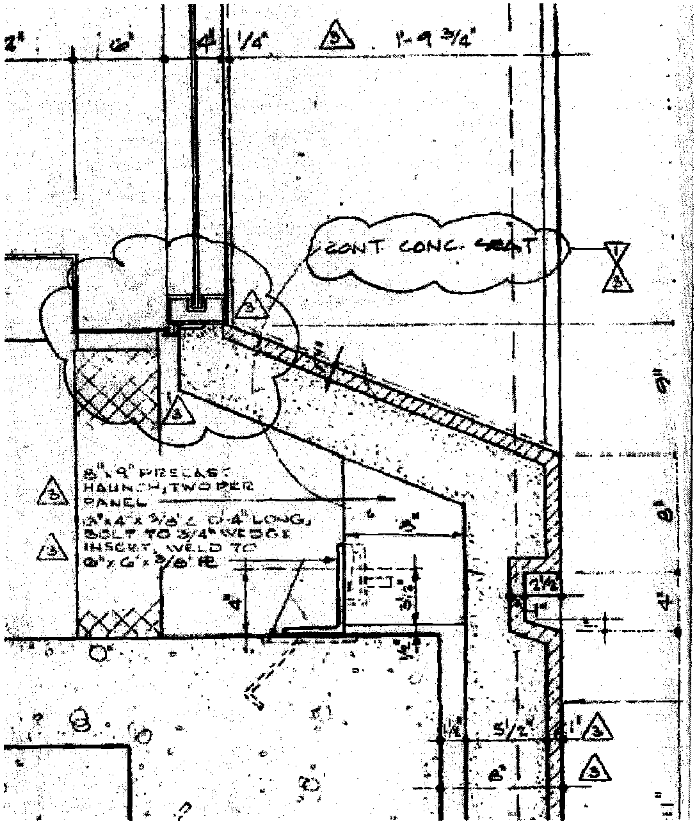


Figure 2 - *Typical Existing Spandrel Condition*

Visual Inspections:

We performed visual inspections using visual, hands-on, and close-up inspection techniques. An acoustic impact tester (rubber tipped hammer) was the primary instrument used for surface inspection. We used five-way knives and borescopes for inspecting concealed cavity conditions. We observed the following conditions:

[1] Most of the exterior wall joints had aged, but not failing, joint sealers. We observed that some had received a skim coat of sealant. Most notably, window sill to existing precast concrete joints lacked sealant or grout. We observed no pointing mortars.

[2] We tested each stone panel for soundness and performed a visual inspection

(Figure 4). Weathering of the travertine had resulted in etching and dulling of the stone surface. Many of the travertine voids had been filled with a cementitious material. Shadow lines were developing in the stone from obvious formation of calcite at cracks. Approximately 27% of the stone panels at the spandrel units were cracked. Other typical exposed stone cladding deficiencies were spalled edges (Figure 5), loose and missing stone, cracking at hairpin anchor locations, panel displacement at column covers, green algae and dust build-up in unfilled travertine voids, efflorescence, and iron staining.


Figure 4

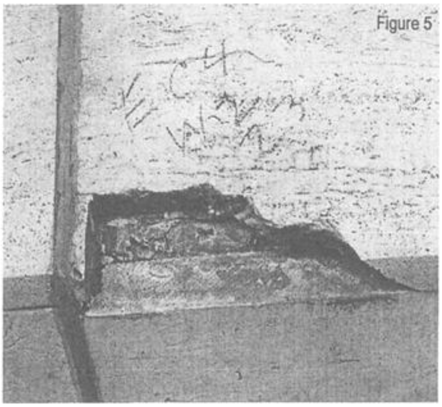


Figure 5

[3] The stone is 25.4 mm (1 in.) thick.

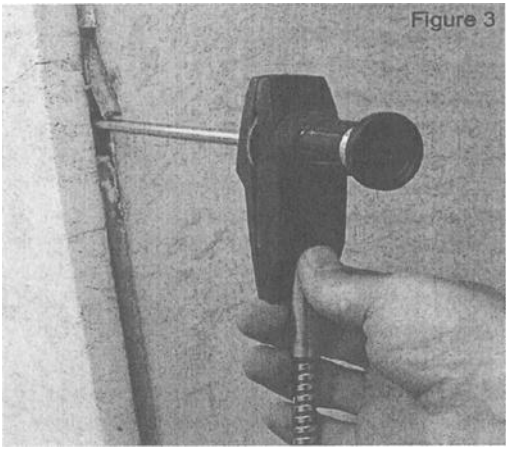
[4] We identified two methods of stone attachment: (1) a conventional set stone veneer system utilizing non-corrosive metal strap anchors primarily at the column covers and (2) a bonded stone veneer precast concrete panel system utilizing hairpin style, toe-in type, non-corrosive wire anchors at the sloping sill, vertical spandrel and soffit panel exposures. The metal strap anchors at the column covers were bedded in the stone veneer and spaced from the back-up concrete column framing with a gypsum-based molding plaster. The stone-veneer to precast-concrete interface appeared to be wet cast as it had no visual form of bond coat or intermediate adhesive and the stone could be easily detached from the precast concrete. We also found a thin, discontinuous, orange-tinted film on the underside of the stone.

[5] We found rock wool type, unfaced, insulation in the cavities between the stone veneer and the concrete columns. We saw no evidence of fungal growth or water damage. We did not find flashing in the wall system.

Physical Testing/Probing:

We used the following methods to reveal the underlying construction.

[1] Cavity examination using an Olympus Borescope (Figure 3).



[2] Laboratory examination of travertine samples.

[3] Field and laboratory examination of precast concrete panels limited to a typical one-story bay. Examination and testing included documenting panel reinforcement, connection to the building structural frame, verification of panel cross-section dimensions, petrographic examination, compression testing, air content testing, and chloride ion content testing of core samples. We used a Profometer 4 to determine the size, shape and symmetry of the embedded precast panel reinforcement.

Travertine Examination:

Sample Preparation: We prepared three thin sections 27 mm x 46 mm, 30 microns thick, sliced perpendicular to the external face of the travertine, from the travertine specimens. We took one section through a specimen where there was a thin layer (<1 mm) of mortar, a second section through a (<0.08 mm) wide crack, and a third section through a crack that contained corrosion products. We used blue-dye epoxy to preserve delicate textures and minerals.

Results of Petrographic Analysis: The travertine consisted almost entirely of calcite. Cementitious material, a gypsiferrous repair material composed of crushed quartz sand, marble dust, and gypsum cement, filled the travertine voids. Weathering had caused visual intergranular disintegration of some of the calcite crystals at the edges of unfilled travertine voids, etching of the gypsum cement void fill material, gypsum cement induced microcracking of the stone by expansion, and microcracking due to freeze-thaw action of water. We did not find bonding agent residues. We analyzed the orange tinted film with a scanning electron microscope (SEM) and identified it as a complex steel corrosion product. Chemically, it consisted of a combination of iron oxide, gypsum (hydrated calcium sulfate), some intergrown thaumasite (hydrated calcium-silico-sulfo-carbonate), and ettringite (hydrated calcium aluminum sulfate). Oxidized fragments of steel dust from the wear of saws and grinders used in slabbing and backgauging the travertine probably was the source of the iron. Our petrographer, Stone Products Consultants (SPC) of Ashland, Massachusetts, attributed the source of the gypsum to being either a byproduct of advanced sulfate attack in the concrete or as a precipitated salt from the dissolution of gypsiferrous materials. According to SPC thaumasite normally forms in concrete that has severe sulfate attack but they reported that it more likely came from Type I Portland cement commonly used in the manufacture of the precast concrete products. Ettringite is a normal byproduct of cement hydration. The gypsiferrous repair material dissolved during weathering of the building panels and seeped to the concrete-to-stone interface by migratory water. The combination of the gypsum formation at the bond line and freeze thaw action broke whatever bond the stone had to the precast concrete.

Precast Concrete Panel Examination:

Petrographic Analysis of Concrete: We took two 75 mm (2-15/16 in.) diameter cores from the existing precast concrete panels and tested them in accordance with ASTM Standard C856-95 AStandard Practice for Petrographic Examination of Hardened Concrete@. The testing included a macroscopic analysis of the cement paste and aggregate as well as a microscopic analysis of a cut/polished section. We documented the depth of carbonation using a phenolphthalein indicator solution applied on a freshly cut and polished surface of the concrete samples. We estimated water/cement ratio based on our analysis of a thin section of the concrete under an Olympus BH-2 polarizing microscope. We summarized our findings as follows:

> The coarse aggregate was comprised of 9.5 mm (3/8 in.) maximum sized crushed gabbro that was subjectively characterized, but not measured, as being fairly well graded with good overall uniform distribution.
>
> Pozzolanic admixtures were not present in either concrete sample.
>
> Slump was estimated to have been between 127 mm and 178 mm (5 in. and 7 in.) based on the estimated water/cement ratio.

Paste/aggregate bond was judged to be fair to poor based on our visual observations of the bond with matrix; peripheral cracks inside the borders of aggregate grains; internal cracks.

The depth of carbonation ranged from 13 mm up to 19 mm (2 in. up to 3/4 in.).

Some microcracking, parallel to the outer surface, existed in the outer 5 mm of the sample, with few microcracks, perpendicular to the inner surface observed in the inner 6 mm of the sample.

The water/cement ratio of the cores ranged between 0.52 and 0.57 with approximately 6-8% unhydrated cement particles.

Based on the above findings, we judged the overall concrete quality fair, due to the concrete being placed at a medium to high slump with a high water/cement ratio.

Compression Strength Testing: We took four 75 mm (2-15/16 in.) diameter cores from the existing precast concrete panels and tested them in accordance with ASTM C42/C42M-99, AStandard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete@. The compression test results of the cores ranged from 40.6 MPa (5,889 psi) to 55.67 MPa (8,075 psi) with an average of 45.88 MPa (6,654 psi). Compression strength in excess of 34.47 MPa (5,000 psi) is a common specification for new architectural precast concrete elements.

Air Content Testing: We conducted tests using ASTM C457-98, AStandard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete@. We cut the cores perpendicular to the horizontal plane of the concrete as placed and then polished them prior to testing. The percentage of entrapped air was between 1.5 % and 0.9 %, the percentage of entrained air was 1.3 % to 0.3 %, resulting in a total air void content of 1.8 % to 1.2 % between samples.

Chloride Ion Content Testing: We tested three 10 gram pulverized concrete samples in accordance with AASHTO Sampling and Testing for Chloride- Ion in Concrete and Concrete Raw Materials (T260) - Procedure C. Results of all three samples showed chloride ion content of less than 80 ppm. Chloride ion levels in excess of 400 ppm cause corrosion of embedded steel reinforcement.

Precast Panel Connection Points: Each panel was connected to the building frame through the use of unequal leg steel angles, two at the top and two at the bottom (See Figure 2). The angles were bolted to the panels and then welded to embed plates cast into the top and the soffit (underside) of the perimeter spandrel beam located below and above the panels. Cast in haunches, located immediately adjacent to the top angle connections, transferred the gravity loads of the panels to the top of the spandrel beam. The flanges of the top angles were welded full length to the embed plates on three sides. The flanges of the bottom angles were welded to the embed plates the full length of one side and half of the front side. The angles and the bolts appeared to be in good condition with very little evidence of rusting or deterioration. However, two of the bottom angles had been cut and rewelded. We observed a crack and a spall on the bottom of one of the panels adjacent to one of the lifting hooks for the panel but saw no structural damage to the panel. We did not see any evidence of water penetration in the cavity beneath the sloping precast panel faces.

Location of Reinforcement: Using a Profometer 4 we mapped the location of the embedded reinforcing at the outboard (stone faced side) sloping, vertical, and horizontal (soffit) face of each panel inspected. In addition, we mapped the inboard (cavity side) of the sloping and vertical face of each panel. The location of the spandrel panel prohibited mapping of the inboard horizontal face of the panels. One layer of 100 mm x 100 mm (4 in. x 4 in.) reinforcement was located in each panel=s sloping and horizontal face, two layers of 100 mm x 100 mm (4 in. x 4 in.) reinforcement were in each panel=s vertical face. The approximate concrete cover for the reinforcement was 38 mm (1-1/2 in.) in each (outboard and inboard) face. The stone cladding had been previously removed from the outboard sloping and vertical faces of one of the precast panels making visual observation and localized pocket removal of the precast matrix possible. We observed smooth-surfaced welded wire fabric reinforcement at three locations in one of the exposed, stone formed, vertical precast concrete panel faces. Pocket removal of the concrete matrix at two locations in the vertical and sloping faces exposed the vertical and horizontal reinforcement, which had an approximate diameter of 5.7 mm (0.225 in.) (W4 wire).

Findings and Recommendations

The following summarizes our findings that formed the basis of our remedial recommendations:

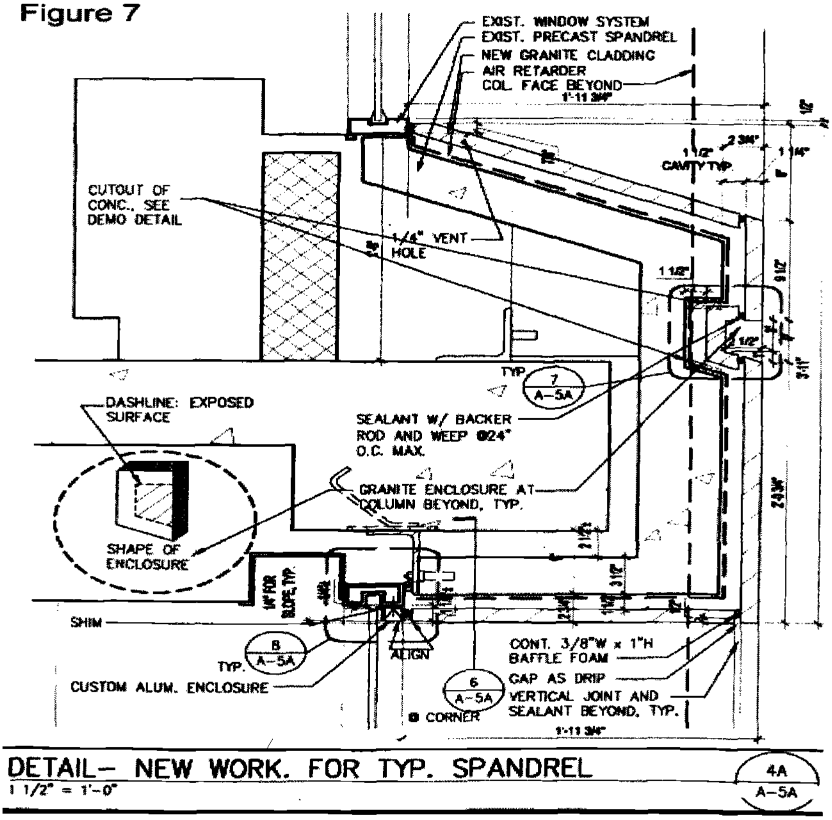
> Weathering of the travertine resulted in etching and dulling its surface, intergranular disintegration of unfilled travertine voids, etching the surface of gypsiferrous repair material, expansion of gypsiferrous repair material with consequent microcracking of the travertine, and other microcracking of the travertine related to freezing water.
>
> The joint sealants in the facade were found to be in generally acceptable condition; however, the age of the material had exceeded its normal life span. Window sill to existing precast concrete joints lacked sealant.
>
> The precast concrete substrates supporting the existing travertine cladding were in fair condition with regard to moisture and freeze thaw resistance. The panel

displacement at the column covers was caused by gypsum based molding plaster that had swelled through the absorption of water over time. The existing precast concrete substrates were contaminated with substances that were found to be detrimental to the travertine=s durability and the travertine-to-precast bond capability.

Analysis of the results of the visual inspection and physical testing of the precast panels indicated that precast panels could support the existing window wall system and a new granite cladding system. This permitted full tenant occupancy during recladding operations.

Based on the above, we recommended the following remedial actions to the Owner which were all subsequently accepted and employed. A typical spandrel condition of the recladding scheme is illustrated in Figure 7.

Figure 7



Based on the widespread, systemic nature of the weathering and cladding deficiencies, and the absence of technology to arrest the advancement of the weathering and cladding deficiencies, we recommended the complete removal of the travertine cladding and its replacement with a more weather resistant granite cladding. The proposed stone thickness was recommended to be 30 mm (1-1/4 in.) thick and of a color and matte finished texture similar to the profiles, color and texture of the existing travertine. This recommendation satisfies many of the criteria that would later be demanded by the District of Columbia=s Fine Arts Commission.

Stone-to-stone joints should be sized to accommodate fabrication and installation tolerances of the selected cladding while allowing for calculated thermal movements in the stone and the proper use of high-performance joint sealants for long-term durability. Joint sealant preconstruction compatibility and adhesion testing programs should be provided and enforced to minimize the potential for stone staining and adhesion failures.

A stone anchorage system, utilizing stainless steel fasteners and strap anchors, permitting vertical and lateral adjustment of each stone panel should be employed. Three dimensional adjustability would be critical as there would be a need to fasten new stone panels over existing substrates bearing a high probability for flatness variability. Adjustability offers the additional advantage of a vented, wept, and baffled pressure equalization chamber (drainage cavity) between the unexposed back faces of the granite cladding and the air retarder indicated in figure 7.

Use a cold fluid applied air retarder system with integral flashings that span joints and form watertight terminations to overcome the fair conditioned and contaminated substrates. This system would be compatible with, and adherent to, the existing concrete substrates, possess self-healing properties that seal around the new granite anchor penetrations, and allow the transmission of water vapor providing a future option of insulating the building without having an inaccessible vapor barrier forward of the insulation plane.

Require mock-ups and sample installations for the stone recladding. These aid the construction team in understanding the design intent of the recladding program and afford the opportunity to proof test alternative stone erection means and methods.

The owner should engage full-time, on-site, independent testing and inspection agency to monitor the contractor=s quality control.

James C. LaBelle [1]

Façade Repair Examples in the Midwest: Cracking, Twisting and Falling

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Reference: LaBelle, J. C., "**Façade Repair Examples in the Midwest: Cracking, Twisting and Falling,**" *Building Façade Maintenance, Repair and Inspection, ASTM STP 1444,* J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: Case studies of the investigations and repairs of façades of three buildings in the Midwestern United States are discussed. Strength and water penetration issues were identified. The years of initial investigation of the deficient façades range from 1992 to 2000. Building characteristics include:

1) 45+ stories; mid-1980s; glass and aluminum curtainwall on upper portion of building; steel framing.
2) 30 stories; mid-1980s; brick veneer on shelf angles, individual windows; primarily reinforced and post-tensioned concrete framing.
3) 22 stories; 1930; limestone panels on shelf angles; steel framing.

Deficiencies and repairs included:

Building One: Substantial leaking occurred at many locations in the glass-and-aluminum curtainwall. Initial repair attempts disclosed the existence of some cracked screws, which secured the glass retainers. Repairs included additional sealing, installation of missing end dams, and installation of a different type of screw to avoid hydrogen-assisted stress-corrosion cracking of screws.

Building Two: Extensive leaking occurred, and cracked mortar joints existed at a recurring architectural feature. These items led to extensive investigation (by two firms) and repair. Other significant deficiencies were uncovered as veneer was removed and analyses were made, including errors regarding torsional stiffness and attachment of the shelf angles. Construction errors had been made in field-modifying the angles. Field changes, to permit erection, were needed because of a shop drawing error in the connection detail. Repairs included installation of new flashings (EPDM in lieu of stainless steel) and angle reinforcing.

Building Three: Several limestone panels (units) fell to the sidewalk from the tenth story. Investigation revealed cracks in some of the limestone units, severe rusting of some shelf angles and the lack of a functioning soft joint below some angles. Repairs included installation of repair anchors and soft joints.

Keywords: façade, hydrogen-assisted stress-corrosion cracking, limestone panel, shelf angle, water penetration

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Three case histories of extensive repairs of deficient cladding systems (façades) for mid-rise to high rise buildings are presented in this article. The deficiencies that needed to be addressed included strength concerns and rainwater penetration problems. All of these repairs were undertaken in the last eleven years. The initial motivation to investigate and repair was primarily leaks for the first two buildings, but safety concerns for the third building.

Building One

The first project is a high-rise residential building, in the Minneapolis-St. Paul metropolitan area, which is about 45 stories tall. It is a steel framed building constructed in the mid–1980s. It was learned that a repair contractor had previously been engaged to attempt to solve widespread, substantial leak problems in the glass-and-aluminum-framing curtainwall. In at least one location, the leaking was so consistent and plentiful that a system of tubes was installed to collect the penetrating rainwater and irrigate plants with it!

This is a stick system (i.e., horizontal and vertical aluminum frame members were each installed piece-by-piece on the building) with rubber gaskets around the edges of the glass infill. This system is used on the upper portion of the building, while masonry cladding is used below the stick system.

Investigation (No. 1)

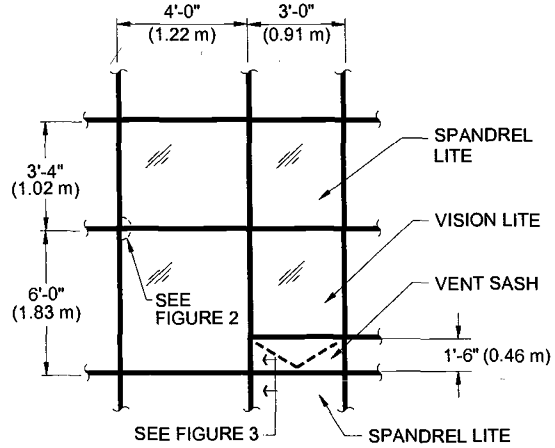

Figure 1 – *Building One: representative area of façade*

The curtainwall uses external aluminum retainers (pressure "plates"), secured by screws, on all four edges of each glass lite. Vision lites vary but most are about 3' (0.9 m) to 4' (1.2 m) wide by 6' (1.8 m) high. Figure 1 shows a typical vision and spandrel lite arrangement, with an inswing hopper type of vent window in one vision area. Log sheets, created by a repair contractor during preliminary repair attempts, recorded that, during their work, heads broke off of some of the screws that held the retainers. These breaks were detected as some retainers were removed and then replaced.

During a visit to the site, the writer witnessed five fasteners being removed from a horizontal retainer at one area. In this instance, which eventually proved to be one of the worst areas, three of the screws either broke during unscrewing or were already broken. The outer piece of each broken screw included the head plus two to six threads. Thus after discussion with the owner, the scope of the investigation and repairs was enlarged to include the screw problem.

As a routine precaution, temporary canopies were installed above the sidewalks, adjacent to the building, because of the investigation and repair activities, and to provide some protection in the event of a portion of the façade falling.

Laboratory study of the five screws revealed that all of these drilling screws ("self-drillers") were zinc plated, carbon steel and hardened to a Rockwell hardness of C52 to C55 at the "case" (thin annular zone including the exterior surface) and C45 in the screw's core near mid-radius. They were #12 screws, which are 0.216" (5.5 mm) nominal in major diameter, and threaded (14 threads per inch, which is a 1.8 mm spacing), except at the drill point, for their full length of 1.5" (38 mm). At least one of the screws had rust on some of the fracture surface area, as observed immediately upon removal, indicating that cracking existed prior to the removal of that screw. The pressure plates had pre-fabricated holes but the drilling screws created their own hole and threads

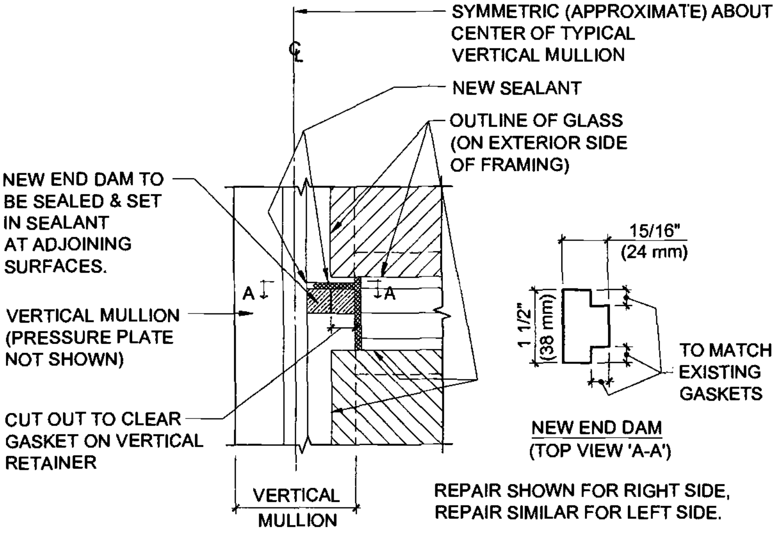

Figure 2 – *Building One: end dam*

in the frame member. The screw hardness was high in the threads (both in the case and core) well beyond the point and lead portion of the thread. This hardness made the screws, when tightened, susceptible to Hydrogen-Assisted Stress-Corrosion Cracking (HASCC), which is one of several deleterious phenomena that involve hydrogen's effect on hardened, high strength steel fasteners. Reference [1] provides brief descriptions of these phenomena. The metallurgical phenomenon of HASCC is discussed in more detail in [2].

In brief, HASCC is a phenomenon in which sufficiently hard steel (greater then Rockwell C35) behaves in a brittle manner and is susceptible to cracking if it is in tension, in contact with aluminum and in contact with moisture. Microscopic crevices in the surface of the screw are thought to attract hydrogen atoms. These atoms are present because the contact of dissimilar metals results in the breakdown of water molecules into hydrogen and oxygen. The crevices and hydrogen atoms act as stress raisers and the screw tension drives a crack, starting at one or more crevices. Of course, as the crack affects a progressively greater area, the remaining uncracked cross-section area becomes progressively smaller. Thus the stress increases on the uncracked area until gross failure occurs.

It is interesting to note that the typical screws (including failed screws) had a safety factor of at least 10, for wind load, against normal tensile failure of the screw. This is the ratio of the screw's tensile strength (without stress corrosion effects) to its design load, which arises from the outward design wind pressure acting on the glass infills.

Water testing was conducted at a one-story portion in the upper part of the façade. At the vision area, water testing was conducted both with and without an intentional air pressure difference. At the spandrel area, no intentional air pressure differential was imposed during the water test. Especially with an air pressure difference, the leaking prior to repairs was substantial. It was learned that drain holes (weeps) in the retainers were contributing to the uncontrolled water penetration because they were insufficient in

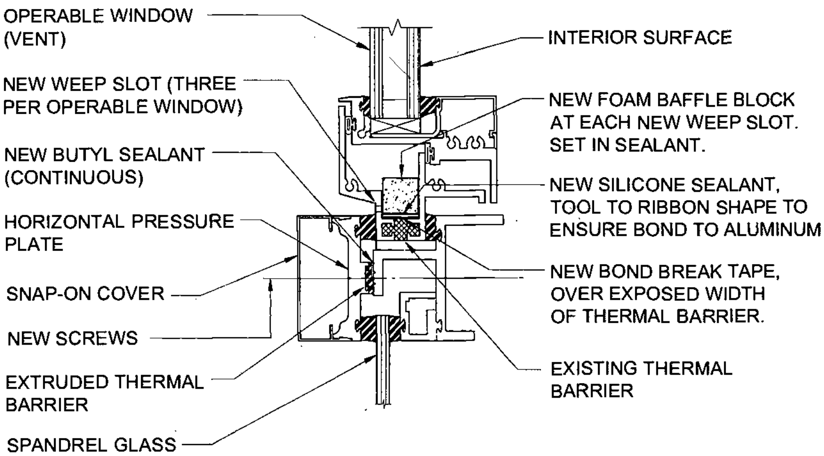

Figure 3 – *Building One: section through horizontal members*

size, shape, and quantity per glass lite. Some locations had setting blocks (rubber pieces, which support the glass weight) secured with sealant and no weep between setting blocks. Water could not flow past the blocks, which trapped the water in the central portion of the sill trough. The sill is the framing at the base of the glass lite. Water was not being drained rapidly enough from the sill cavity to avoid overflowing to the interior (e.g., pooling on exposed interior surfaces of the sills). For the test area, elongating the round holes to horizontal slots and adding a weep slot between setting blocks substantially reduced, to very little or to nothing, depending on location, the quantity of water which penetrated to the interior.

Another significant item was the omission of rubber end (zone) dams at many, but not all, of the intersections of vertical and horizontal members (Figures 1 and 2). This occurred especially at the upper portion of the glass curtainwall. This omission allowed water to run down the verticals and penetrate at a lower elevation. Another contributor to water leaks was the fact that the sills of some vent window frames were found to have experienced dry shrinkage of the ends of the urethane thermal barrier ("break"). This created a small gap at part of the mitered sill-to-jamb joint and allowed some water to leak in. During testing, these gaps were sealed and that proved effective at enabling the sill trough to hold water when the weep openings were temporarily blocked.

Repairs (No.1)

Remediation of the screw problem was accomplished by using a tapping screw made of stainless steel, instead of high carbon steel. One of the austenitic alloys that comprise the American Iron and Steel Institute's 300 series was used. The next larger diameter, which is 1/4" (6.3 mm), was employed in lieu of the original #12 (5.5 mm) screw. The larger screw diameter permitted re-use of most of the existing holes, after removal of existing screws and drilling of the correct diameter pilot hole. The remnants of certain of the original screws – those that were broken or broke when unscrewing was attempted – were removed by using pliers after the pressure plate was taken off.

Repairs to address water leaks included creating nominal 1/2" (13mm) by 1/4" (6mm) weep slots (versus round holes) in all horizontal retainers, installing new rubber dams in all areas where they were missing, and applying sealant to each dam (Figure 2). Sealant was also applied to the contact surface between the pressure plate's thermal barrier and the exterior side of the horizontal frame member. This was to make the sill trough watertight below the weeps (Figure 3). The gaps due to "shrink-back" (dry shrinkage), at the ends of the poured-in-place, urethane thermal barrier in the sill of the vent window frames, were repaired using a ribbon of silicone sealant for the entire length of each sill. Bond-breaker tape was employed to prevent adhesion between the urethane and the sealant. The sealant was extended vertically 2" (50 mm) at each jamb. The ribbon of sealant was bonded to the aluminum section on each side of the thermal barrier, forming a water-resistant cover over the thermal barrier. This repair was based on the method given in Reference [3], which is published by the American Architectural Manufacturers Association.

Building Two

The second project is an office building, about 30 stories in height, in the Midwestern United States. This building primarily has a concrete frame, with both mild and post-tensioned reinforcing. Construction was finished in the late 1980s. The façade system consists primarily of individual windows surrounded by clay brick veneer panels (Figure 4). There are also some areas with precast concrete panels. Cavity wall construction was used throughout.

Investigation (No. 2)

The writer began his investigation in 1992. Water leaks had occurred in many areas of the façade. In addition, cracked mortar joints were observed at many locations that had a typical architectural feature, usually within a region extending several courses above the shelf angle. This feature, termed an external point, is a vertical strip of brick masonry, which is triangular in cross-section and protrudes from the wall (Figure 5). The writer's firm was asked to assist in the façade investigation and design of structural repairs. The remediation project involved the services of two consulting firms. One firm, which was engaged first, was primarily concerned with addressing the water penetration aspects and issuing most of the construction documents for the remediation. The writer's firm, engaged later, was primarily responsible for structural aspects of the investigation and repairs, and for most of the inspection program, which addressed water-

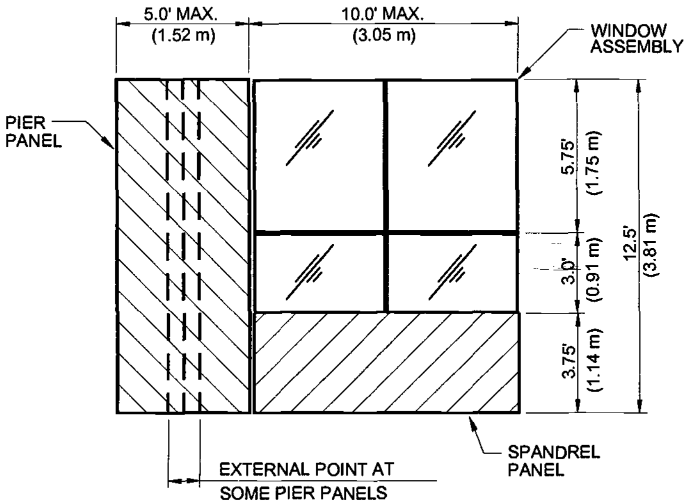

Figure 4 – *Building Two: panels and window*

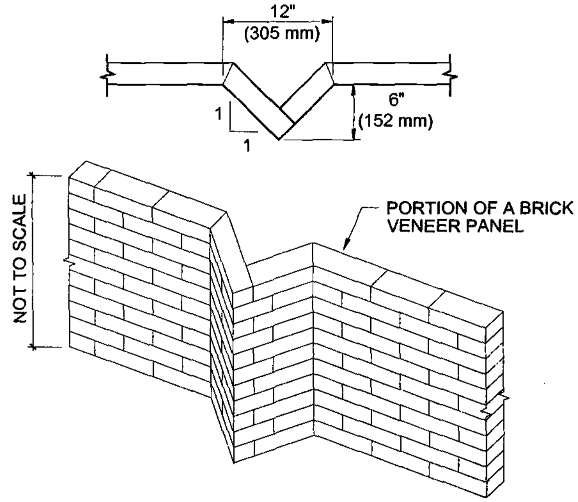

Figure 5 – *Building Two: external point*

penetration control and structural repairs. The first firm had already planned the basic aspects of the trial ("experimental") remediation, of water penetration problems, which was to be conducted at a story that allowed for relatively easy access.

Investigation and trial repairs were conducted at one story before beginning the several years-long project of fixing the brick façade's support system and flashing at all stories. It was determined that removing six courses of bricks from the bottom of each panel, for its full width, was needed. This opening size would lessen the number of lap joints needed in the new flashing. It would also provide enough vertical room for the new structural items and the vertical leg of the new flashing which covered those items. For temporary support of the upper part of the panel, one or two special brackets, depending on panel width, were installed through appropriate openings and anchored to the concrete frame (Figure 6).

The water leaks proved to be caused primarily by deficiencies in the as-installed flashing. Lap joints between the pieces of stainless steel flashing had been made with sealant and the installed quality of the sealant joints was erratic. The flashing also did not protrude past the outer face of the façade. Instead, the flashing was cut so that the outer edge was inboard of the external face of the veneer (and of the outer edge of the shelf angle) and was covered by the horizontal joint sealant. In many locations water drained from the wall cavity through deficient lap joints, and then through the gaps between ends of adjoining shelf angles (typically above a window jamb). Other leaks were traced to the omission of vertical stainless steel flashing at joints between the precast panels and brick panels. Also, most of the sealant joints between precast panels had a single sealant bead, rather than a double bead (separated by drainage space), as called for in the original design.

The structural aspects included excessive torsional flexibility in the shelf angles, especially at the special portion (consisting of a welded, triangular horizontal plate with

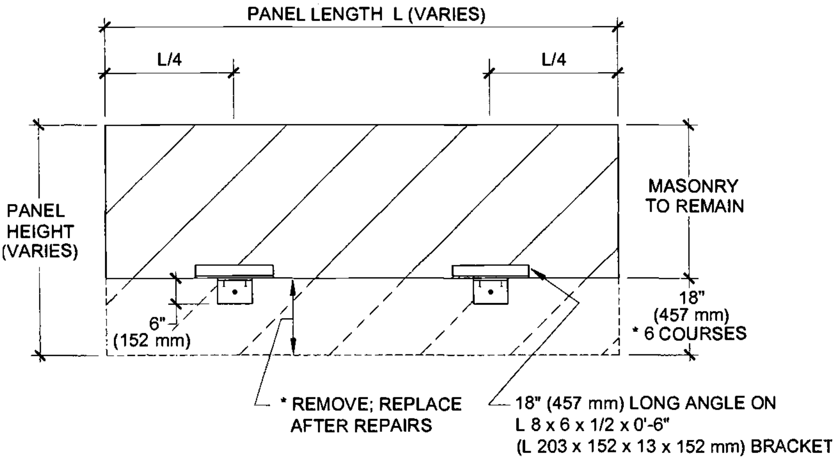

Figure 6 – *Building Two: temporary support*

welded vertical stiffener) used to support the "external point" feature. Many finite element analyses were made to evaluate both existing conditions and repair options. In addition, due to a dimension error on shop drawings, the shelf angles could not be properly positioned with respect to the cast-in-place inserts (with vertical slots) and subsequently mounted with the wedge-head bolts (Figure 7). The typical field modification made during original construction, to enable attaching the angles, was an enlargement (vertically and occasionally horizontally) of the horizontal slots in the

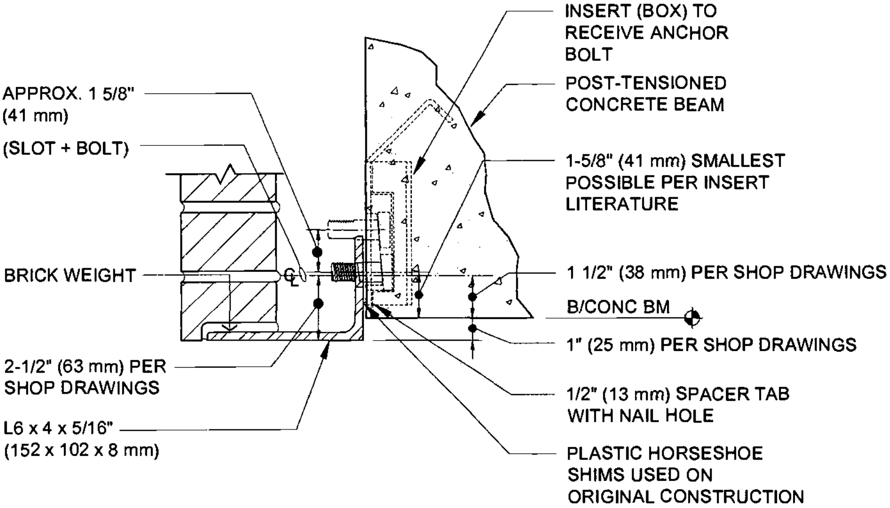

Figure 7 – *Building Two: existing anchor (flashing not shown)*

vertical leg of the shelf angle (Figure 8). The remaining net width of steel above the bolt was generally much less than in the original design. In some situations, the net width of steel above the anchor bolt was found to be insufficient.

At some of the shimmed connections, between the angles and the concrete spandrel beam, the nut was not fully engaged due to insufficient bolt length. In a few locations, normal tolerance on maximum cavity depth was exceeded and the thicknesses of shim stacks were excessive – up to 2" (50 mm) thick was measured.

It was also found that corkboard strips had been used typically as spacers to set the bottom of the shelf angles at about the width of a sealant joint, 5/8" (16 mm), above the brick panel below. Laboratory testing confirmed that this corkboard was too stiff to remain permanently in a "soft" joint. This became a concern in part because the bolts in the inserts would loosen, due to the wedge shape of the bolt heads (Figure 7), if sufficient upward force were applied to the bottom of the shelf angles. Upward force on the angles could be (and in some cases was) generated due to the presence of the corkboard, which prevented the joint's width from contracting freely. Because of the corkboard's effect on the joint, axial compression force arose in the panels from various sources. These sources included column shortening, spandrel beam differential deflection and increased panel height caused by brick expansion due to moisture absorption and/or an increase in temperature.

It was determined from the investigation that it was necessary to remediate certain typical aspects of the entire brick panel system, particularly the gravity supports and the flashing.

Repairs (No. 2)

Repairs generally, but not always, were done starting at the top of the building and proceeding down. To fix the problem of uncontrolled penetration of rainwater, an EPDM (rubber, "ethylene-propylene-diene-monomer") flashing was selected. This material was

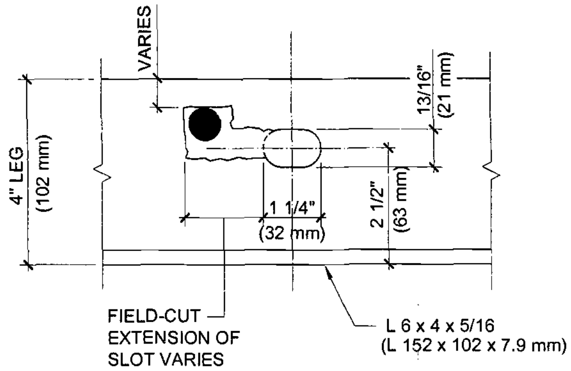

Figure 8 – *Building Two: shelf angle at anchor slot*

relatively easy to install. Because the full length of the shelf angles was accessible due to the method of temporary support used for the masonry panels, the number of EPDM joints was minimized. The outer edge of the new flashing projected beyond the panel face. The projection was trimmed to an appropriate drip-lip dimension. New sealant, with foam backer rod, was installed flush with the exterior surface of the wall. The bottom of the sealant bead adhered to the top of a brick panel or window-frame head-receptor, both of which are immediately below the shelf angle and flashing. The top of the sealant bead adhered to the underside of the new flashing. The old stainless flashing was re-used to serve as a housing to cover the nuts on the attachment bolts and parts of other structural items (e.g., clamps), and thus to provide a smooth surface for most of the EPDM contact area.

The new flashing system and necessary structural modifications were designed to fit together as compatible systems. The flashing's vertical leg was tall enough to cover the new structural items. New structural reinforcement items included:

- clamp "plates" (cut from angles) at appropriate spacing (further apart at spandrel panels and closer together at the pier panels) between existing anchors, with one anchor bolt per clamp plate, to reduce the torsional flexibility of the shelf angles by restraining the angle's vertical leg (Figure 9);
- hanger "bars" (also cut from an angle), with an expansion anchor bolt into the concrete and an ordinary bolt into a tapped hole in the shelf angle's vertical leg, at deficient slots and/or inadequately installed original anchors; and
- diagonal bracket (tension strap welded to a plate, and anchors), at each brick point, that was attached to the outer portion of the stiffener for the shelf angle's triangular extension. The bracket's top was connected to the concrete framing.

The corkboard spacers were removed to create a true soft joint, which had only foam backer rod and sealant.

The repair program generally involved three phases of inspection:

a) Initial observations after demolition, including removal of the stainless steel flashing.

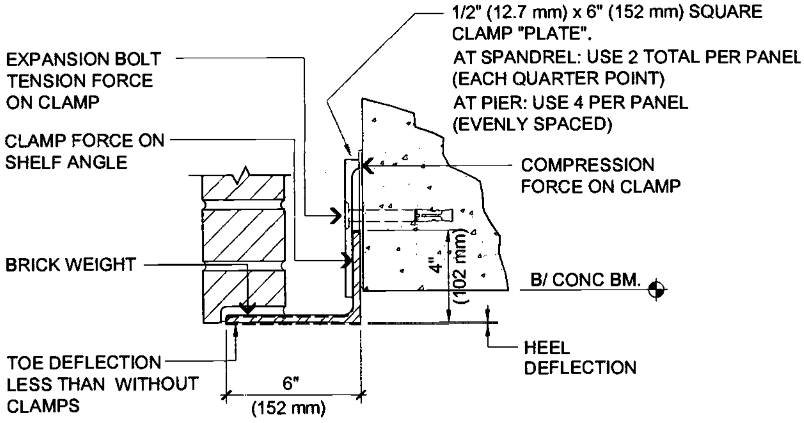

Figure 9 – *Building Two: shelf angle plus clamp (flashing not shown)*

A location for each new structural item was marked on the galvanized shelf angles. Most of the items were the same for each of several shelf conditions but special situations arose often.

b) Inspection of the installed new structural items (e.g., clamps, hangers, diagonal brackets).

c) Inspection of the installed EPDM flashing, including lap joints and sealing of the termination bar at the top of the flashing's vertical leg, prior to re-laying bricks.

Special forms were prepared so that very thorough inspection records could be created in the field. These records also kept track of the quantities of extra items (e.g., additional hangers and clamp plates) that could be determined only when a given area was opened up. The contractor was then reimbursed for the extra items, based on unit prices in the base contract, by means of a change order.

Some field-testing was conducted, either during the investigation or during the repair process, to verify the:

- Adequacy of strength of the dovetail anchors (brick ties) installed in the embedded slots;
- Strength of a selected brick panel using an external chamber to create outward pressure; and
- Adequacy of the as-repaired resistance to water penetration, using large chambers (one at each of two areas) that were custom built to accommodate spray racks and resist the specified air pressure.

Laboratory testing determined the following:

- Bond strength of the horizontal joints in a masonry specimen (made with new mortar and bricks) using the wrench bond method,
- Compressive strength of new brick units and of new mortar, and
- Compressive strength of masonry specimens (new bricks laid in new mortar).

Various other items were addressed in the project but most of the major ones have been presented.

Building Three

The third façade is part of a 22 story steel-framed building located in the Midwest. It was constructed about 1930. The cladding consists primarily of limestone panels (units) and individual double-hung, steel frame windows (Figures 10 and 11). The limestone panels are supported on shelf angles at each story. Typically, each shelf angle at "pier" strips between windows carries the weight of five limestone rectangular panels, which are arranged vertically in stack bond. The mortar joints are about 1/8" (3 mm) thick between panels. The stone thickness varies between 3" (76 mm) for the majority of each stone unit and up to 5" (127 mm) at some edge regions. The panels are about 5' (1.5 m) wide by 2' (0.6 m) high. The shelf angles are uncoated (current condition) steel. It is not known if the shelf angles were originally coated. The top edge of the top stone unit in each story is shaped so that a vertical lip covers the outer edge of the horizontal leg of the shelf angle for the course above.

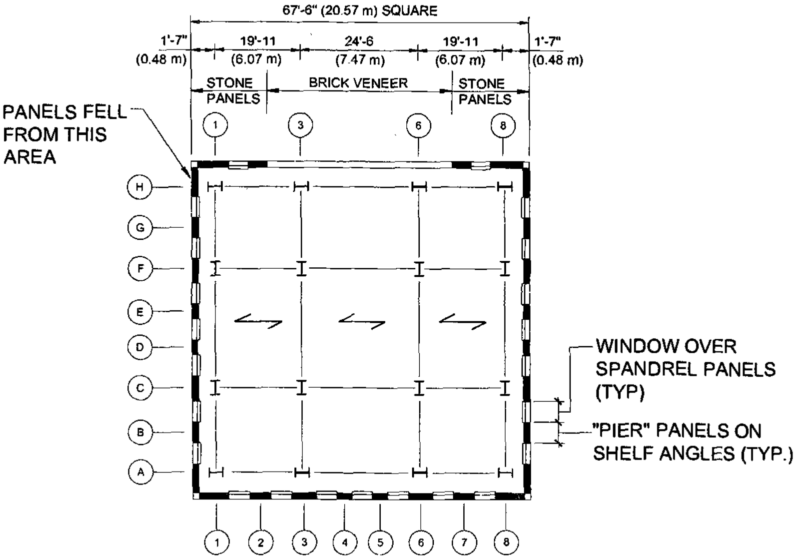

Figure 10 – *Building Three: plan view*

Investigation (No. 3)

Four stone panels (all of the lowest three and most of a fourth) fell from a corner "pier" of the tenth story in the autumn of 2000. Fortunately, no one was injured. The entire street in front of the building was barricaded after the panels fell. These units hit a public sidewalk on a calm, sunny day. Each panel weighed, by calculation, between 500 and 600 pounds (2.2 and 2.7 kN). The writer observed two cracked portions of the top panel (unit #5 in Figure 11) hanging from a horizontal sealant joint. The cracks were essentially vertical and had been caulked at some time in the past. A remnant of panel #4 supported the third (corner) portion of panel #5 above. These pieces were pried loose by workers, using a wrecking bar while standing in the cage of a telescoping boom lift. The pieces were allowed to fall to the walk, because the pieces were too heavy to safely load into the lift.

Both drawings and building observations were used to investigate the accident's cause. Observations revealed that the lower four of the typical stone panels were each secured by two steel strap ("zee" or "cee" shaped) anchors, at the top, to resist outward wind. These straps are 1/8" (3.2 mm) thick by 1 1/4" (32 mm) wide and were set in a clay-brick masonry back-up wall with the exterior ends fitted into kerfs (slots) in the stone. Some stone units had spalled at the anchor, either at the panel's exterior side (observed at several locations away from the failure area) or at the panel's interior side. Such an interior-side spall occurred in at least one of the failed stone units. Steel pins, 5/8" (16 mm) in diameter, had been used to secure the top edge of the top unit to the shelf

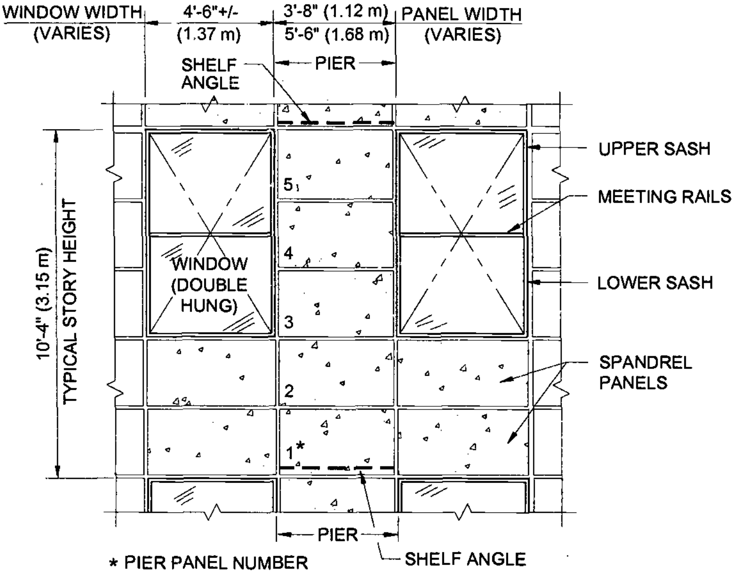

Figure 11 – *Building Three: typical portion of façade*

angle. Mortar was found at some locations between the bottom of the angle and the top of the stone below. This created a hard joint and it meant that axial (vertical) pressure could increase, with time, in the stone units as the shelf angles rusted and the rust tried to lift the bottom panel. Rust (iron oxide) has a volume that is about five to seven times the volume of the steel from which it is formed by oxidation. Rust has sufficient stiffness and strength to exert significant pressure. Substantial layers of rust were found on some of the angles. No flashings were found in the façade system.

At one anchor, a "cone" of brick masonry was pulled out of the brick back-up behind one of the fallen panels. At another anchor, the interior side of the connected panels failed, leaving two stone fragments and the anchor attached to the back-up wall. At the tenth story area behind the panels that failed, "excess" mortar was found that acted as an irregularly spaced and sized set of "inward load" supports (back-up bed) for the panels. In addition, no pins were found between panels at the failed area. Thus, bond and/or friction were considered to be the sole means of supporting the bottom of each typical panel against outward loads (e.g., wind), prior to repairs.

At both the fallen panels and the other panels, there was very little space (cavity), if any, between the stone panels and the brick back-up.

A significant number of other panels were found to be in the process of failing, as evidenced by varying degrees of outward tipping (out of plumb) of the panels. One tipped panel was first detected with binocular observation from a nearby parking structure. It was one of the most severely tipped, at least of the panels that did not fall, and was found to be tipped by about 1.1" (28 mm) when a close-up examination was made via an adjoining window. The tipped panels are thought to have failed at one or both anchors, but no panels were intentionally removed for examination. This theory, however, receives some support from the existence of a retained spall observed at an anchor at the area of the fallen panels. The most likely failure mode was limestone fracture initiating at the kerf, and progressing to cause a spall at the back (hidden) side of the panel. Water entering the kerf and freezing, over many years, is a likely mechanism to cause stone fracture. In addition, another possible mechanism is expansion force due to rusting of the steel strap within the kerf. Further, it appears fairly likely that cycles of freezing of water in the narrow cavity, between brick back-up and stone panel, caused an outward force on some panels resulting in outward movement (after anchor failure) in a ratchet-like manner.

Repairs (No. 3)

This façade repair was primarily a structural fix, so that panels would be better supported. Thus, remediation efforts were intended to limit the shelf angle's vertical load in each story to the weight of that story's panels and to compensate for possible deficiencies in existing anchoring. Also, the repair program called for replacement of any shelf angles whose thickness had been reduced to below a calculated thickness limit.

To limit the vertical load resisted by the panels, true soft joints were needed below all shelf angles. To provide soft joints, the stone lip was removed along with any mortar, and backer rod and sealant were installed.

To address likely deficiencies in the existing anchoring, the use of special repair anchors was considered. These anchors are of the double expansion type and consist of brass and stainless steel components. The inner anchor is installed in the brick back-up, via the hole drilled through the stone and into the brick back-up. Next, the connecting rod is threaded into the expanded anchor and the outer anchor is expanded after its outermost edge is set slightly inboard of the exterior face of the stone. Sealant, of a color similar to the limestone, is then applied to fill the shallow hole and cover the end of the anchor. Sand may be applied to the uncured sealant to simulate the surface texture of the surrounding stone.

Pull-out tests of the proposed repair anchors were conducted at various locations to verify adequacy, both in the clay brick masonry and in the limestone panels. After testing, which produced very good results, four repair anchors (two in the upper half and two in the lower half) were installed, as a high priority action, at each stone panel that had tipped outward more than 1/4" (6 mm). Anchoring of all remaining "pier" panels (those arranged in vertical bands) was recommended for subsequent remedial work.

Conclusions

Some lessons that might be learned from these case studies include:

1) The value of hiring an independent inspector to observe, in detail and at many locations, curtainwall construction as it progresses. Items such as missing parts,

substantial field modifications and other potentially significant ways of not conforming to drawings and specifications can and should be detected during construction.

2) Together with lesson #1, photographs (and perhaps some video records) of a façade as it is being constructed might help to record what will ultimately become "hidden conditions." Maybe this would tend to counter the "out-of-sight, out of mind" approach that unfortunately has sometimes been adopted for façade construction.
3) Considering, for example, the use of inappropriate screws (Building One), the presence of overly stiff corkboard in a "soft" joint (Building Two), and the presence of mortar in a "soft" joint (Building Three), it is appropriate to question why these errors occurred. The writer suggests that those involved in design and construction of façades should constructively question the validity and appropriateness of current "usual practices" so as to reduce the occurrence of possible serious problems after façades are in service.
4) Once again, the value of a critical examination for older masonry façades (Building Three) has been shown. Had such an inspection been performed substantially prior to the stone panels falling, it is likely that the event could have been precluded. The examination of remaining panels fortunately provided a warning that some of them were significantly out-of-plumb and that some of these were also possibly being subjected to added axial load.

It is hoped that a requirement for critical examinations of façades, especially of mid and high rise buildings, becomes the norm throughout the United States and other countries.

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Thomas A. Schwartz, P.E.[1]

Glass Facade Assessment

Reference: Schwartz, T.A., **"Glass Facade Assessment,"** *Symposium on Building Facade Maintenance, Repair, and Inspection, ASTM 1444,* J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: Glass is the most widely used building facade material. No other material matches its optical clarity, durability, economy, and ability to control light and heat transfer. Despite these attractive properties, the inherent brittleness and potentially dangerous fracture characteristics of glass will continue to challenge cladding designers and building owners. Since glass makes up such a substantial portion of contemporary building facades, the economic consequences of poor glass performance are significant.

This paper presents a brief review of glass in facade architecture, its strengths and weaknesses, and in situ condition assessment techniques. Issues addressed in this paper include glass surface assessment, evaluating glass breakage characteristics, and insulating glass unit durability and performance.

Keywords: annealed, fully tempered, heat-strengthened, laminated, hermetic seal, interlayer, etch, GASP

Introduction

This paper generally addresses non-invasive glass condition assessment methods, i.e., those methods of assessment not requiring disassembly of glazing or removal of glass. Glass breakage assessment is an exception, since definitive determination of the cause of glass breakage requires removal of the glass to analyze the fracture surfaces.

1. Glass in Building Facades

From sand, soda, and lime come sparkling cathedrals, buildings that disappear into their surroundings, and indoors that feel outdoors. Glass in architecture represents simultaneously the simplicity and complexity of the contemporary building facade. Formed of cheap abundant natural materials, glass is a cost-effective barrier, and sometimes filter, of inside and outside environments that has performance attributes and durability unmatched by alternative materials.

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[1] President, Simpson Gumpertz & Heger Inc., Waltham, MA 02453.

Consider the following glass performance attributes:

- highly transparent to visible light
- impermeable to water and air penetration
- strength and stiffness to allow use in thin sheets
- resistant to weathering and UV degradation
- dimensionally stable
- formable and moldable
- rigid at building service temperatures
- resistant to atmospheric chemicals
- receptive to decorative and opacifying coatings
- receptive to heat tempering for added resistance to loads

With the addition of minerals and surface coatings, glass adds the following features:

- absorption and reflection of solar radiation to limit heat gain to buildings
- reflection of visible light to create interesting aesthetics
- reflection of infrared radiation to limit heat gain or to reduce heat loss from buildings
- ability to change color and opacity under the control of electrical current

And all this from materials found at the beach.

1.1 Types of Glass Used in Building Facades

The following are the four most common types of glass used in the exterior of buildings:

- annealed
- fully tempered
- heat-strengthened
- laminated

The characteristics of these types are summarized in Table 1.

Table 1 – ***Common Types of Glass Used in Building Facades***

Glass Type and Description	Advantages	Disadvantages
Annealed (A) – Annealed glass is cooled slowly from its molten state to avoid the introduction of significant residual stress; it is the most common architectural form.	• Excellent surface flatness and uniform reflective qualities • Can be cut to size after fabrication • Inexpensive	• Does not resist significant mechanical, thermal, or impact loads • Breaks into dangerous, sharp fragments

Glass Type and Description	Advantages	Disadvantages
Fully Tempered (FT) – Glass that has been rapidly cooled from its molten state to introduce compressive stress at its surface. Has about four times the strength of annealed glass. Fully tempered glass can conform to "safety glazing" requirements.	• Extra strength resists breakage due to high winds, impacts, and thermal stress • If fracture does occur, fracture fragments are small and much less dangerous than those of annealed glass	• Heat treatment imparts surface waviness • Subject to spontaneous breakage from nickel sulfide impurities in the glass • More expensive than annealed glass • Fracture fragments tend to fall out of the opening • Cannot be cut after heat treatment
Heat-Strengthened (HS) – Similar to fully tempered glass, but the cooling rate is controlled to impart a lower level of surface compressive stress. Has about two times the strength of annealed glass. HS glass is the most common glass used in high-heat architectural applications, such as spandrel glass.	• Extra strength resists breakage due to high winds, impacts, and thermal stress, but to a lesser extent than FT glass • Not prone to NiS impurity breakage if residual stress properly controlled • Tends to stay in the opening after breakage, but can break like annealed glass	• Heat treatment imparts surface waviness • More expensive than annealed glass (Comparable to FT) • Cannot be cut after heat treatment
Laminated (Lami) – Laminated glass consists of two or more sheets of glass bonded together with a clear plastic interlayer. It is commonly used in skylights and in areas prone to missile impacts. It can be fabricated to meet safety glazing requirements.	• Provides post-breakage fall-out protection • Interlayer can block UV – used in museums • Interlayer attenuates some vibration frequencies – used in airports	• Plastic interlayer can lose adhesion to glass if subject to certain chemicals or prolonged water contact • Size limitations for lamination • More expensive due to multiple production steps

2. Gathering of Background Information for Glass Facade Assessment

The following section on the gathering of background information is brief since the collection of useful documentation of past performance of glass is not much different from similar activities associated with other facade materials. ASCE 30-00 "Guideline for Condition Assessment of the Building Envelope," provides guidance for collecting information on the past performance of facade materials.

Significant in situ glass conditions are generally revealed through a review of historical building records. Monolithic, non-reflective glass generally weathers very well. If it is cleaned regularly, using non-abrasive, mild cleaning solutions, the glass will generally provide undiminished long-term performance. The same is not necessarily true for insulating and laminated glass. Both products can degrade for reasons unrelated to maintenance methods.

The first step in assessing glass conditions and durability, therefore, is to pay attention to the past performance results. Records that should be investigated include the following:

- occupant complaints
- routine maintenance records, e.g., cleaning frequency and methods, means of access, glazing sealant replacement, etc.
- breakage records
- glass replacement records
- original plans and specifications

2.1 Review of Plans and Specifications

A review of construction documents is important because it can alert the condition assessor to glazing conditions that are likely to cause performance problems with glass and that are concealed from view in a non-invasive investigation.

2.1.1 Architectural Plans

Architectural plans are generally too generic to provide useful information in a condition assessment. Shop drawings generally define the glazing conditions in sufficient detail to allow a meaningful assessment. Issues to evaluate through shop drawing review include the following:

Table 2 – *Shop Drawing Review Guide*

Component/Checklist	Comments
Setting Blocks • Sufficient in height to prevent "wicking" of water to bottom of insulating or laminated glass? • Configured to allow drainage to weep holes? • Sufficient indentation hardness to prevent loss of thickness? • Chemically compatible with IG seals?	• Watch out for rubber setting blocks supported on metal channels ("chairs"); metal chairs can slide, causing loss of glass support, glass shifting, and glass-to-metal contact. • Water can wick to the underside of glass if setting blocks are < 3/16 in. thick. • Setting blocks must not obstruct weep path.
Anti-Walk Pads • Properly sized and positioned to prevent glass-to-frame contact during installation and in service?	• Anti-walk pads must be secured mechanically, adhesively, or through friction to remain in position throughout the service life of the glass.
Glazing Seals • Properly configured to accommodate differential movement between glass and frame and to shed water? • Appropriate materials for longevity? • Chemical compatibility of IG unit and glazing seals, especially heal beads in contact with IG seals?	• Wet glazing generally provides a higher level of watertightness but presents difficulty in reglazing. • Dry gaskets offer ease of re-glazing; gaskets without vulcanized corners will tend to develop gaps in corners, if gasket length is not properly oversized.
Glazing Stops • Opportunity for loosening? • Position relative to weep system? • Probability of allowing drainage to the interior?	• Interior glazing stops set at the level of the water in the glazing pocket tend to allow leakage to the interior.
Weep System • Does the system drain within each glazed opening, or does it use verticals as downspouts? • Size and spacing sufficient to prevent blockage and allow drainage around setting blocks? • Consequences of dirt and debris in drainage tracks?	• Framing systems that utilize vertical members as downspouts for drainage of vertically stacked glass tend to leak. • Weeps less than 5/16 in. in smallest dimension tend to become blocked and restrict water drainage.

2.1.2 Specifications

The contract specifications and material approvals will provide glass quality requirements and glass types. If the glass has surface blemishes, for example, the glass specification will define the acceptable level of scratches and other surface defects at the time of manufacture. If the glass is fully tempered, the assessor should verify that spontaneous breakage has not been occurring. If the building has a history of spontaneous glass fracture, further investigation is required into the probability that impurities in the glass batch are causing the breakage.

3. Condition Assessment of Architectural Glass

The in situ visual assessment of architectural glass should include the following aspects of glass performance:

- Surface quality and uniformity
- Coating uniformity and durability
- Insulating glass hermetic seal quality and durability
- Laminated glass interlayer bond integrity
- Assessment of appropriate glass type
- Analysis of fractures, if any

3.1 Assessment of Surface Quality and Uniformity

Most architectural glass is manufactured to conform to ASTM C1036. Among other things, C1036 defines the surface quality of glass.

3.1.1 Surface Scratches

Tables 3, 4, 5, and 6 below show the allowable size, intensity, and spacing of surface scratches and blemishes. These tables are reprinted from ASTM C1036-01[2].

Table 3 – ***Allowable Point Blemish Size and Distribution For Cut Size Qualities Thickness 6.0 mm (1/4 in.) or Less*** [A]

Blemish size mm (in.) [B, C, D]	Q1 Quality 1	Q2 Quality 2	Q3 Quality 3	Q4 Quality 4
< 0.50 (0.02)	Allowed [E]	Allowed [E]	Allowed	Allowed
≥ 0.50 < 0.80 ≥ (0.02) < (0.03)	Allowed with a minimum separation of 1,500 mm (60 in.) [F]	Allowed with a minimum separation of 600 mm (24 in.) [F]	Allowed	Allowed
≥ 0.80 < 1.20 ≥ (0.03) < (0.05)	None allowed	Allowed with a minimum separation of 1,200 mm (48 in.) [F]	Allowed	Allowed

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[2] Reprinted with permission of ASTM

Blemish size mm (in.) [B, C, D]	Q1 Quality 1	Q2 Quality 2	Q3 Quality 3	Q4 Quality 4
$\geq 1.20 < 1.50$ $\geq (0.05) < (0.06)$	None allowed	Allowed with a minimum separation of 1,500 mm (60 in.) [F]	Allowed with a minimum separation of 600 mm (24 in.) [F]	Allowed
$\geq 1.50 < 2.00$ $\geq (0.06) < (0.08)$	None allowed	None allowed	Allowed with a minimum separation of 600 mm (24 in.) [F]	Allowed
$\geq 2.00 < 2.50$ $\geq (0.08) < (0.10)$	None allowed	None allowed	None allowed	Allowed with a minimum separation of 600 mm (24 in.) [F]
≥ 2.5 $\geq (0.10)$	None allowed	None allowed	None allowed	None allowed

[A] Glass thicker than 6.0 mm (1/4 in.) and less than or equal to 12.0 mm (1/2 in.) may contain proportionally more and larger blemishes. Table 3 does not apply to glass thicker than 12.0 mm (1/2 in.). Allowable blemishes for glass thicker than 12.0 mm (1/2 in.) shall be determined by agreement between the buyer and the seller.

[B] See 6.1.2 for detection of point blemishes.

[C] See 6.1.5 for measurement of point blemishes.

[D] For Q1 and Q2 only, the blemish size includes associated distortion (See 6.1.5).

[E] Provided that normally non-detectable blemishes do not form a cluster that is detectable at 1,800 mm (6 ft)

[F] See 6.1.6 for minimum blemish separation.

Table 4 – *Allowable Point Blemish Size and Distribution for Stock Sheet Qualities Thickness 6.0 mm (1/4 in.) or Less*[A]

Glass Area	Point Blemishes Allowed
If glass area $< 7\ m^2\ (75\ ft^2)$	One rejectable point blemish allowed
If glass area $\geq 7\ m^2\ (75\ ft^2)$ but $< 14\ m^2\ (150\ ft^2)$	Two rejectable point blemishes allowed
If glass area $\geq 14\ m^2\ (150\ ft^2)$	Three rejectable point blemishes allowed

[A] Follow the appropriate requirements in Table 3. Note these additional details for Stock Sheet quality requirements (including minimum separation requirements).

Table 5 – *Allowable Linear Blemish Size and Distribution for Cut Size and Stock Sheet Qualities Thicknesses 6.0 mm (1/4 in.) or Less*[A]

Linear Blemish Size[B] Intensity	Length	Q1 Quality 1 Distribution	Q2 Quality 2 Distribution	Q3 Quality 3 Distribution	Q4 Quality 4 Distribution
Faint	$\leq$ 75mm (3 in.)	Allowed with a minimum separation of 1,500 mm (60 in.)	Allowed with a minimum separation of 1,200 mm (48 in.)	Allowed	Allowed
Faint	> 75 mm (3 in.)	None allowed	None allowed	Allowed	Allowed

Linear Blemish Size[B] Intensity Length	Q1 Quality 1 Distribution	Q2 Quality 2 Distribution	Q3 Quality 3 Distribution	Q4 Quality 4 Distribution
Light ≤ 75 mm (3 in.)	None allowed	Allowed with a minimum separation of 1,200 mm (48 in.)	Allowed	Allowed
Light > 75 mm (3 in.)	None allowed	None allowed	Allowed	Allowed
Medium ≤ 75 mm (3 in.)	None allowed	None allowed	Allowed with a minimum separation of 600 mm (24 in.)	Allowed
Medium > 75 mm (3 in.)	None allowed	None allowed	None allowed	Allowed
Heavy ≤ 150 mm (6 in.)	None allowed	None allowed	None allowed	Allowed with a minimum separation of 600 mm (24 in.)
Heavy >150 mm (6 in.)	None allowed	None allowed	None allowed	None allowed

[A] Glass thicker than 6.0 mm (1/4 in.) and less than or equal to 12.0 mm (1/2 in.) may contain proportionally more and longer blemishes. Table 5 does not apply to glass thicker than 12.0 mm (1/2 in.). Allowable blemishes for glass thicker than 12.0 mm (1/2 in.) shall be determined by agreement between the buyer and the seller.

[B] See 6.1.5 for detection of linear blemishes

Table 6 – *Blemish Intensity Chart*

Detection Distance	Blemish Intensity
Over 3.3 m (132 in.)	Heavy
3.3 m (132 in.) to 1.01 m (40 in.)	Medium
1 m (39 in.) to 0.2 m (8 in.)	Light
Less than 0.2 m (8 in.)	Faint

If surface scratches are significant, this condition is generally a subject of occupant or owner complaint and can be discovered by inquiry. Seriously damaged glass is obvious, even if the glass is dirty, but a meaningful in situ assessment requires glass cleaning. Conventional, non-aggressive cleaning using mild detergent, a soft cloth or sheepskin applicator, and a rubber squeegee wipe is generally sufficient to reveal significant surface damage. A thorough assessment of surface blemishes, however, requires a more extensive cleaning, possibly including polishing with a pumice paste, to reveal all damage for visual assessment per ASTM C1036. Refer to PPG publication TD-107 for a detailed discussion of glass cleaning methods (available at http://www.ppg.com).

Surface scratches are not just an aesthetic issue; they diminish glass strength. The degree to which the scratches degrade strength depends on the depth of the scratches and the sharpness of the tips of the scratches. Both of these factors are impractical to assess in the field. When faced with the task of determining replacement needs resulting from strength degradation due to surface damage, the practical approach is to locate the significant scratches (i.e., "white" scratches that can be detected with a fingernail) on elevation drawings of each lite. Overlay each elevation drawing with an outline of stress contours derived from the design wind load, pick a low stress contour line that will produce a low probability of breakage (about 2,000 psi for damaged glass), and replace all glass with significant scratches within that stress contour.

3.1.2 Image Distortion

Glass reflects and transmits light. Both reflection and transmission are affected by waviness in the glass surface.

Significant glass surface undulations are common in heat-treated glass. Float glass is very flat, but the heating and quenching associated in the heat strengthening and tempering of glass impart waviness.

The industry standard for heat-treated flat glass, ASTM C1048, stipulates that localized warp shall not exceed 1/16 in. (1.6 mm) over any 12-in. (300-mm) span. Glass complying with this standard can still exhibit noticeable and sometimes objectionable distortion in reflected and transmitted images.

Reflective coatings on glass accentuate distortion of reflected images. The effect is minimized by orienting the wave in adjacent lites of glass in a consistent direction. Distortion of transmitted light through heat-treated glass is most noticeable when the viewer's angle to the glass is acute. Such a viewing condition is common in large expanses of heat-treated glass used in viewing stands, such as race tracks and sports arenas.

3.1.3 Chemical Attack/Etching of Glass Surfaces

Compounds shed by building facades can etch glass or become very tightly bound to the glass surface such that normal cleaning methods do not remove them. For example, alkalis that leach from masonry, especially new masonry, can etch the glass surface if left on the glass for a prolonged period. Metal oxides shed from rusting steel can become so well bonded to glass that only polishing with abrasive paste (a very costly process) can remove the "stain."

Some of the chemicals used to clean building facades can also etch glass. Some masonry facade cleaners contain hydrofluoric acid, an acid particularly aggressive on glass. Such cleaners can be carried by wind to adjacent and nearby buildings. If those buildings contain glass with a reflective coating on the outside (i.e., #1 surface), even a minute amount of the chemical deposited on the glass can pit the glass surface deeply enough to penetrate the reflective coating and produce a noticeable scar. The reflective glass on all four sides of a twenty-five-story building in Richmond, Virginia, was pitted by HF acid used to clean the facade of a building across the street, despite precautions, including the use of shrouds to confine the spray-applied cleaner and avoiding work on windy days. The chemical was so aggressive that glass on the opposite side of the building from the cleaning operation was pitted by minute amounts of wind-blown acid. This type of damage is most noticeable when viewing transmitted light through the glass. Groups of pits give the appearance of a smudge on the surface of the glass, even though the individual pits are too small to be noticed by the naked eye.

3.2 Assessment of Coating Uniformity and Durability

Reflective coatings deposited on the glass surface by vapor deposition are generally quite uniform in thickness, color, and reflected image intensity. Coatings that are sprayed onto glass, which is sometimes done in pyrolitic (i.e., applied to heated glass) coating processes tend to be less uniform in color and reflectivity as a result of thickness variations (Photo 1). Some of the metals and metal oxides used to create reflective coatings on glass can react with atmospheric gases and, in so doing, change their reflective characteristics.

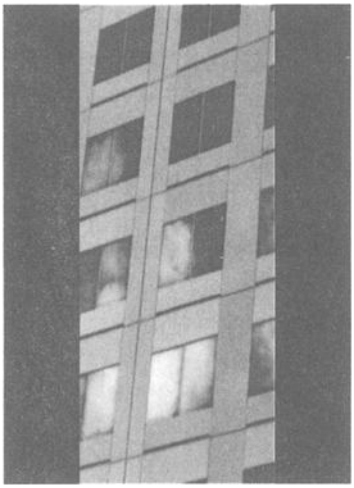

Photo 1 – *Dark edges of reflective glass are the result of thin coating.*

Pyrolytically applied reflective coatings are integral with the glass surface and resist color changes, but vapor deposited coatings on the #2 or #3 surface of IG units are vulnerable. (The four glass surfaces of a two-lite insulating glass unit are numbered from 1 to 4, starting with the surface exposed to the outside of the building). Changes in the color of reflected light produce facades that have a mottled appearance when viewed from a distance (Photo 2).

A mottled appearance can be an indication of moisture ingress into IG units because moisture in the unit can contribute to chemical changes in the reflective coating. A mottled appearance can also show the extent of glass replacement, since replaced units often do not match the original units in color. The mottled appearance of a reflective insulating glass facade is, therefore, cause for further investigation of the insulating glass units as discussed below.

Photo 2 – *Mottled appearance results from coating color changes*

3.3 Insulating Glass (IG) Hermetic Seal Quality and Durability

Properly cared for, a sheet of glass maintains its performance characteristics for the life of the building. The same, however, is not necessarily true for the form of glass used in most buildings today, i.e., insulating glass.

IG units are composed of two or more lites of glass separated by a hermetically sealed air space. This configuration approximately doubles the thermal resistance of the glass compared to a single sheet. The energy efficiency of an IG unit can be further improved with the addition of tints and coatings in and on the glass and with the addition of low-conductivity gases in the air space (e.g., Argon). The key to durable IG unit performance is the longevity of the hermetic seal between the lites of glass and the ability of the desiccant within the IG unit to adsorb the moisture that does enter.

The spacer that maintains the distance between the lites of glass that make up the IG unit contains desiccant to dehydrate the sealed air space. Desiccants are generally so effective that the air inside an IG unit will have a dew/frost point in the range of -90°F or less at the time of manufacture. The units will continue to provide the intended performance until the internal dew point of the air space reaches normal operating temperatures, at which time condensation will form in the air space and eventually lead to permanent fogging through etching of the glass.

Given the significant expense and disruption associated with reglazing a building, it is critical that a facade assessment address the past and probable future performance of the insulating glass. That assessment should include the following:

- review of window shop drawings to verify that glazing is properly designed with respect to glass setting and drainage of the glazing pocket

- review of past building maintenance records and interview of maintenance personnel to determine frequency and magnitude of seal failures, if any
- exterior visual survey of glass to detect color differences between IG units that may indicate coating deterioration as a result of moisture ingress into unit
- interior visual survey of glass to look for fogged units and to make note of the spacer date stamps

3.3.1 Field Investigation of IG Unit Condition and Longevity

A spot survey of spacer date stamps is important in the assessment of past and probable future performance of IG units. Units dated after the date of construction are replacement units. The occurrence of a significant number of replacement units with date stamps extending over a period of time indicates a systemic problem with the glazing system or IG units. The reasons for unit replacement should be investigated to determine if the replacement was necessitated by glass fracture or by IG unit seal failure. In either case, the cause should be determined. See the discussion below for guidance on glass fracture evaluation.

If significant replacement has occurred due to seal failure, two questions need to be answered: What is the cause? What is the likelihood of future failure of both the original and replacement units? Probable cause can sometimes be determined through non-invasive means (e.g., the glazing pocket traps water against the edge seal of the IG unit and degrades the hermetic seal), but more often, the investigation into cause requires glass removal and perhaps disassembly of the IG unit.

Photo 3 – *Dew/Frost Point test apparatus in use.*

The determination of future performance of IG units that have not visibly fogged can be evaluated through the use of ASTM E576, "Standard Test Method for Frost Point of Sealed Insulating Glass Units in a Vertical Position." This test involves the application of a cold plate to the surface of the glass to lower the temperature of the glass surfaces facing the sealed air space (Photo 3). The point at which condensation or frost appears on the glass surface is the dew or frost point of the air in the unit. Frost points that are approaching -20°F or 0°F (-29°C to -18°C) indicate that the unit is nearing failure, i.e., the point at which visible moisture within the sealed air space occurs at in-service temperatures. As a field test, this method is only approximate because the temperature of the desiccant cannot be maintained at a constant temperature of 70°F to 80°F (21°C to 27°C) for twenty-four hours prior to the test. The desiccant temperature is important because the amount of moisture that the desiccant can hold is highly dependent on its temperature. For this reason, we recommend that these tests be run when the glass temperature is in the 70°F to 80°F (21°C to 27°C) temperature range and when the glass is not in direct sunlight.

3.4 Laminated Glass Interlayer Bond

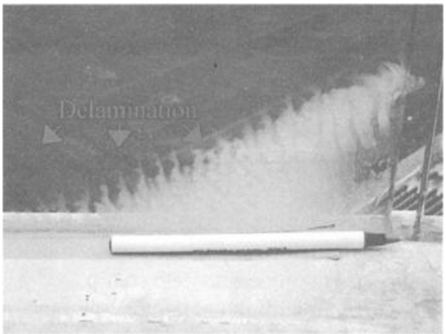

Photo 4 – *Delamination of laminated glass handrail panel.*

The plastic interlayer that bonds the sheets of glass together and retains the fracture fragments in the event of breakage will debond if the edge of the glass is subjected to certain chemicals or if the edge is exposed to prolonged water contact. The debonding generally appears as fingers emanating from the edge of the glass (Photo 4).

3.5 **Assessment of Appropriate Glass Type**

Certain types of glass are needed in specific building locations for safety reasons. For example, overhead glazing should be laminated to prevent fall-out should breakage occur; doors and sidelites need to be safety glazing to help prevent injury in the event of an impact. Some of these assessments of glass type can be made with non-destructive visual tools.

Laminated glass can be easily identified if one of the edges is exposed because the interlayer is of sufficient thickness to see with the unaided eye. If all the edges are concealed, the laminated glass can sometimes be detected using a match or laser penlight to create a reflection off of the glass surfaces. If the glass is monolithic, the light will produce two images due to the reflection off of the front and back surfaces. If the glass is a two-lite laminate, the light will produce four images, representing the four surfaces of the two sheets in the laminated glass.

Heat-treated glass can be detected using polarizing light filters. The retardation of light waves caused by the internal stresses in the glass is visible when viewed with polarized light. This phenomenon is apparent in the tempered rear lites of automobiles when viewed under certain natural daylighting conditions or when viewed with polarizing sunglasses. Alternating light and dark areas depict the stress pattern. A quantitative assessment of residual surface stress in the glass can be made using a Grazing Angle Surface Polarimeter (GASP) instrument. The use of the GASP instrument is described in ASTM C1279, "Test Method for Nondestructive Photoelastic Measurement of Edge and Surface Stresses in Annealed, Heat-Strengthened, and Fully Tempered Flat Glass." This is useful in determining if the glass is heat-treated to the proper level to comply with heat-strengthened or fully tempered glass specifications.

3.6 Glass Breakage Assessment

Glass is a brittle material, meaning that it does not yield prior to fracturing. When glass breaks, it converts almost all of its strain energy into surface energy, so the number of fracture lines and the surface roughness of the fracture surfaces created tell a great deal about the strain and stress state at the time of fracture. For example, the number of fracture lines created, the orientation of those lines, and the markings on the fracture surfaces reveal the stress direction, magnitude, and type of stress. Analysis of the fracture origin is the critical task since it discloses the conditions at the time of fracture initiation.

3.6.1 Fracture Surface Morphology

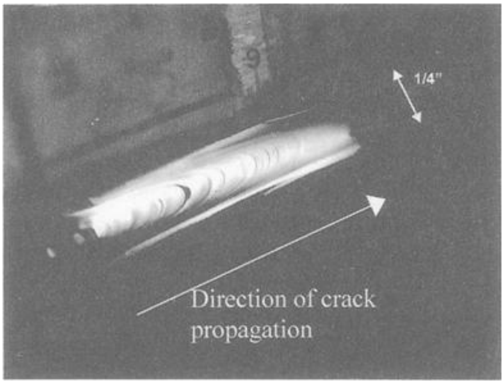

Photo 5 – *Rib marks show direction of crack.*

Propagating fractures in glass leave marks that describe the crack direction. These undulations in the fracture surface are concave toward the fracture origin (Photo 5).

These rib marks are used to trace the fractures back to the point of origin. At the point of origin, the following features describe the type and magnitude of stress that caused the break:

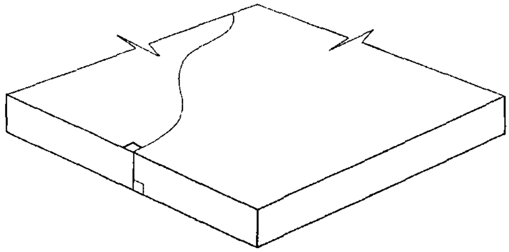

Figure 1 – *Thermal Crack Orientation to Edge of Glass*

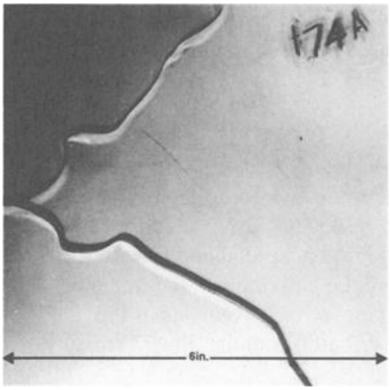

Photo 6 – *Thermal Crack Pattern.*

Crack Orientation – A crack that is perpendicular to both the glass edge and the glass surfaces is a thermal fracture that results from a temperature difference between the center and edge of the glass (Figure 1; Photo 6). Thermal stresses resulting from a center-to-edge temperature difference are nearly uniform across the thickness of the glass and create tension along the glass edge. Since cracks run perpendicular to the principle stress direction, the cracks are perpendicular to the edge and surfaces of the glass.

Conversely, a crack that is at an angle to the edge is generally the result of mechanical stress (e.g., bending and/or torsion). The fracture may or may not be perpendicular to the glass surfaces, depending on the type of mechanical loading.

- Number of Cracks – Since glass converts strain energy into surface energy, the number of cracks emanating from the origin is a rough measure of the stress and strain in the glass at the time of breakage. A single crack denotes a lower stress break.

- Fracture Surface Marks – The fracture surface at the point of origin reveals a great deal about the stress type and magnitude at the time of fracture initiation. This analysis is best made using a 10x magnification loupe. The fracture surface immediately adjacent the fracture origin is smooth and is called the "mirror." The narrow band surrounding the mirror is called the "mist" region, and the rough surface surrounding the mist is called the "hackle" region. The distance from the origin to the point of mist onset is called the "mirror radius" and it is inversely proportional to the stress magnitude that initiated the fracture according to the following formula for typical soda-lime-silicate float glass:

$$\sigma_f = \frac{1{,}950}{\sqrt{r}}$$

where, r = radius of the mirror in inches
σ_f = fracture stress in pounds per in.^{2}

The shape of the mirror describes the type of stress. Thermal stress mirrors (Photo 7) are generally fully enclosed by mist and hackle due to the uniformity of the tensile stress across the thickness of the glass. Bending stress mirrors (Photo 8) are open toward the mid-plane of the glass because the tensile stress is highest at the surface and diminishes to zero at the neutral plane in the middle of the glass thickness.

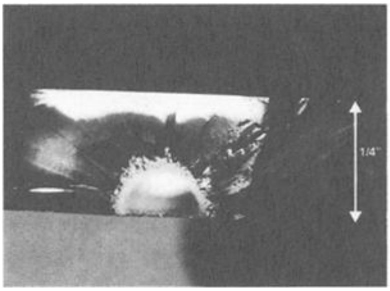

Photo 7 – *Fracture origin resulting from thermal stress.*

Photo 8 – *Fracture origin resulting from bending stress.*

When the stress at the time of fracture is very low (e.g., less than 1,500 psi (10.4 Mpa)) the mirror is unbounded by mist or hackle and the fracture surface is clear. Such fractures indicate significant edge or surface damage that concentrated the tensile stress in the plate sufficiently to cause a fracture. In such cases, the faint rib marks can be used to trace the fracture back to the point of origin where the surface or edge flaw exists.

4. Summary

Glass is a major component of most building facades. A thorough, non-destructive evaluation of existing glass conditions and future performance prospects is an important component of an assessment of the value of a building asset and the safety of the public. Such an assessment includes a thorough evaluation of the records of past performance and design documents to the extent obtainable, and utilization of the tools available to assess surface quality, coating quality, insulating and laminated glass durability, and the causes of performance failures, including glass breakage, insulating glass fogging, and delamination of laminated glass.

George I. Taylor, PE, SE[1] and Paul E. Gaudette[1]

Concrete Facades: Investigation and Repair Project Approaches

Reference: Taylor, G. I., PE, SE, and Gaudette, P. E. "**Concrete Facades: Investigation and Repair Project Approaches,**" *Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444*, J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: This paper presents an overall project approach for the investigation and repair of concrete building facades. Types of distress typically encountered in concrete facades, such as spalling, cracking, and delamination, are described, including a review of the causes of distress and deterioration and their structural significance. Distress of facade elements, such as architectural reveals and balcony railings, is also discussed. Attention is given to specific techniques for assessing existing conditions, materials sampling, and testing. The advantages and disadvantages of complete and representative facade surveys are discussed as a means of generating information on which to base drawings and specifications. Recent building facade and maintenance ordinances are reviewed as they relate to concrete building facades.

Included in this paper are options for repair to address characteristic distress conditions. The advantages of performing trial repairs prior to the implementation of full-scale repairs are reviewed. Finally, methods of quality control and field observation are described for a typical project.

Keywords: concrete, facades, investigation, repair, maintenance, facade ordinances

Introduction

An overall approach to the investigation and repair of concrete building facades requires an understanding of the materials that comprise reinforced concrete, a clear understanding of the type and causes of deterioration and distress, and an understanding of the type of structure. The concrete elements of the facade may include expressed structural elements such as columns, slab edges, beams, window lintels and sills. Other special architectural features may include cornices, balconies, reveals, and ornamental expressions. Types of distress can include cracking, spalling and delaminations, and others. The type of structure may be residential, commercial, industrial, or institutional;

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the building may be one story tall or it may be a sixty story high rise. In addition, the building may be a historic structure requiring special attention and specific guidelines for its repair, and a more stringent review and approval process. Once a clear understanding of all of the above are obtained, then appropriate repairs can be developed for the conditions encountered, drawings and specifications prepared, a repair contractor selected, and repairs performed.

Need for Investigation

The need for an investigation and subsequent repair of a concrete facade usually arises from one or more of the following events:

1. Visual observation of deterioration and distress on the exterior facade, balcony, or other facade element is made. This deterioration and distress can be as severe as large sections of concrete falling from the structure, or may consist of minor spalling, or cracking of concrete elements.
2. The building owner has a maintenance program whereby the building or elements are inspected periodically. During this periodic inspection the building owner is alerted to potential problems on the facade.
3. The building owner may be cited by the governing municipality regarding conditions on the facade that require repair. In most of these cases a citation is issued demanding that the building owner repair the condition. Usually at this point removal of loose concrete, some short-term stabilization, or repair work is required.
4. Most recently, with the advent of building facade maintenance ordinances enacted by municipalities such as Chicago, New York, and other cities, the building owner is required to perform periodic inspections on the facade. The frequency of the inspections and facade elements to be inspected varies from city to city. If distress or deterioration is encountered during the inspections then the building owner may be required to investigate the causes of distress and develop repairs.

City of Chicago Facade Ordinance

The current City of Chicago Facade Ordinance [1][2] requires building owners buildings greater than to submit an "ongoing inspection repair program report" or a "critical examination report" on the condition of the exterior walls. The ordinance applies to buildings that are 80 feet or more in height above grade. These reports are to be prepared by licensed professionals (a licensed architect or structural engineer), or be prepared under their supervision. An ongoing inspection and repair program is required bi-annually, while the critical examination program is required every four years. The ongoing inspection is based on visual surveys of the facade. These inspections are typically performed from street level with the use of binoculars or from adjacent roofs. The critical inspection requires that the licensed professional perform a hands-on or close-up inspection of the facade from suspended scaffolding (swingstage).

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[2] Visit www.cityofchicago.org for the latest ordinance. Also visit www.facade-ordinance.com for links to facade ordinances of other cities.

In the facade report to the City of Chicago, the professional is to indicate the condition of the facade and the existence of "unsafe or imminently hazardous conditions," "safe with repair conditions," or "safe conditions." "Unsafe and imminently hazardous conditions" are defined as conditions that have no reliable means of structural support and that are dangerous to people or property. If "unsafe and imminently hazardous conditions" are encountered during the facade inspection under either program, these conditions are to be reported immediately to City of Chicago officials. The owner is required to repair or stabilize the unsafe and imminently hazardous condition promptly. For concrete facades, these unsafe and imminently hazardous conditions usually involve concrete portions of the facade that can be dislodged and typically consist of spalling and/or loose concrete.

Stabilization procedures for imminently hazardous conditions commonly involve installing a sidewalk canopy protection and retaining the services of a contractor to remove loose concrete that is considered a hazard until permanent repairs can be performed. A time frame for the structure to be repaired is also required by City of Chicago officials. A separate investigation into the causes of these hazardous conditions is required to be performed by the licensed professional in order to develop appropriate repairs.

Distress Condition Investigations

Approach to the Investigation

To determine the cause(s) of concrete deterioration and distress, the architect/engineer should first gain an understanding of the building structure, maintenance history, and previous repair work on the facade, if any. This can be achieved by performing a document review of available drawings, previous reports, and maintenance records, and by interviewing the building engineer and maintenance staff. Important information to be retrieved from the document review phase includes the age and type of the structure; information about architectural features such as reveals, balconies, the use of precast panels, and reinforcing details of the structural elements, and information about past repairs. For example, the age of the structure may give an indication of the type of distress to expect. In newer structures, less than ten years old, concrete deterioration and distress at such early age may be an indication of significant future problems. These early age problems may be associated with material problems, construction detailing, as-built construction, or structural problems. For older structures, usually greater than 20 years old, the type of distress is typically related to corrosion of embedded reinforcing steel in the concrete elements.

Once the architect/engineer has performed the document review, a condition assessment or survey of the structure can be performed. During the condition assessment, the professional inspects and documents the distress observed on the facade. Ideally, the condition assessment will include investigation of representative conditions and include at least some close-up inspections of the facade from a swingstage. Generally, one inspection drop or a minimum of 25% of the surface area per each elevation, is suggested because conditions may vary depending on the exposure of the particular elevation. However, prior to selecting the locations for close-up inspections, the overall visual inspection from grade should be performed to identify representative areas for the close-up inspection. The close-up inspection drops should be selected where the different types of distress conditions can be documented. For example, columns, slab edges, shear walls, and balconies may all reveal different types and levels of deterioration. In some cases, the entire facade may have to be inspected close-up. Such is the case for the close-up inspections required by the City of Chicago facade ordinance where the majority of the facade is required to be inspected every four years.

During a close-up inspection, if conditions require it, the architect/engineer may recommend that the owner retain a contractor to remove loose and unsound concrete from the facade or that protective canopies or barricades be installed. The removal of loose concrete should be done immediately. One of the advantages of an extensive close-up investigation is that it provides more in-depth information for the development and design of repairs and a more accurate estimate of repair quantities than can generally be determined from a limited close-up survey or a visual survey from grade.

The extent of detail required for a facade inspection should be evaluated independently for each particular facade and depending upon existing conditions and upon client requirements. Of course, the more limited the condition assessment, the more probable that hidden conditions may not be uncovered.

Types of Facade Distress

Concrete building facade deterioration is generally related to three principal causes: corrosion of the embedded steel, deterioration of the concrete itself, and structural deficiencies. Distress in concrete facades is manifested by the following conditions: cracking of the concrete, delaminations within the concrete, and spalling. Refer to Figures 1 through 4 for examples of concrete distress.

Cracking in the facade concrete is frequently caused by restrained shrinkage of the concrete, concrete material deficiencies, or structural loadings.[3] Delamination cracking is usually caused by the corrosion of embedded reinforcing steel. Cracks related to restrained shrinkage can be expected in concrete elements. Depending on the width of the cracks and their location, these cracks should be addressed in a repair program. If left untreated these cracks may provide an avenue for moisture to reach embedded reinforcing steel and begin the corrosion process.

Delaminations are planar cracks within a concrete member that typically occur at the level of corroding embedded reinforcing steel. Corrosion of the embedded steel reinforcement takes place when the normally protective alkaline environment (pH greater than 12.5) [2] of concrete is disrupted by the presence of chloride ions or by carbonation of the concrete to the level of the reinforcement. Carbonation is a natural process whereby carbon dioxide in the air reacts with the calcium hydroxide in the concrete to form calcium carbonate and water. The occurrence of carbonation at the level of reinforcement disrupts the normally protective concrete environment by lowering the pH (pH below 9). Carbonation, along with conditions of available moisture and oxygen, can initiate a corrosion reaction. Because the volume of

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[3] Causes, Evaluation and Repair of Cracks in Concrete Structures, ACI Committee 224, ACI 224.1 R-93.

corrosion byproducts (rust) is greater than the volume of the steel from which they are formed, the corrosion generates expansive forces within the concrete. After a period of time these forces exceed the tensile strength of the concrete and cause cracking that forms delamination planes. If allowed to continue, the corrosion will eventually lead to an area of delaminated concrete breaking away from the substrate (concrete spalling) and significant loss of reinforcing bar cross section.

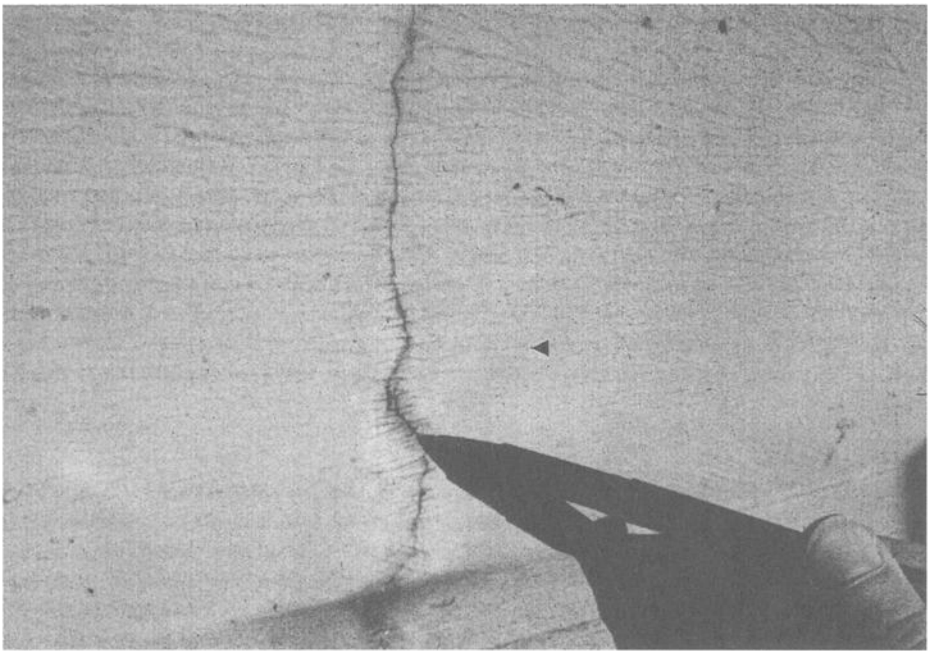


FIG. 1 -*View of cracking distress in slab edge.*

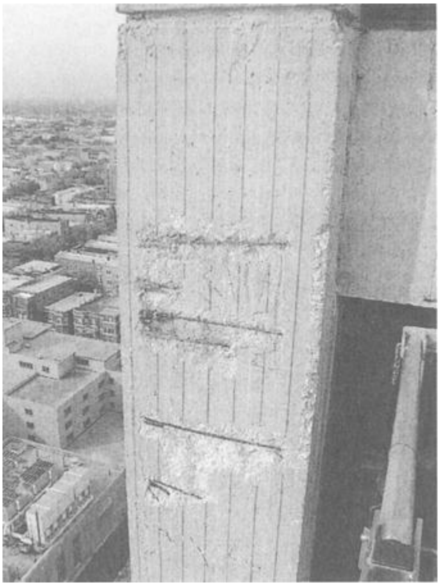


FIG. 2 - *Example of concrete spalling distress in a building column.*

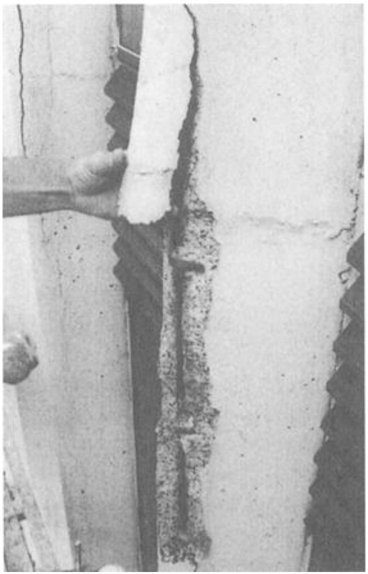


FIG. 3 - *Example of severe concrete spalling and cracking distress at a building column*

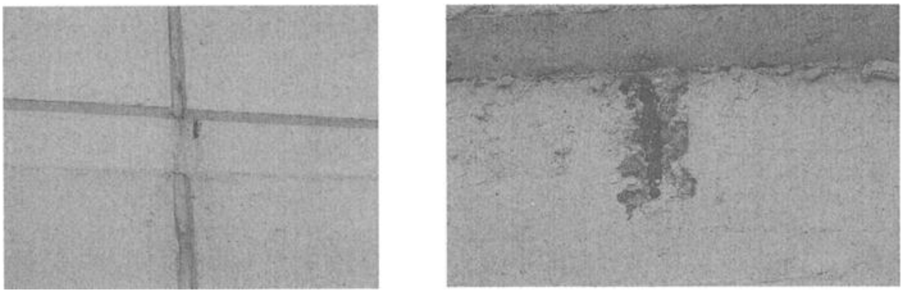


FIG. 4 - *View of distress caused by exposed reinforcing at slab edge at shear wall.*

In older structures and on balconies in temperate and cold climates, damage due to the cyclic freezing and thawing action may be experienced. Symptoms of freeze-thaw damage are usually observed as severe cracking of the concrete surface and degradation and erosion of the paste. An example is shown in Figure 5.

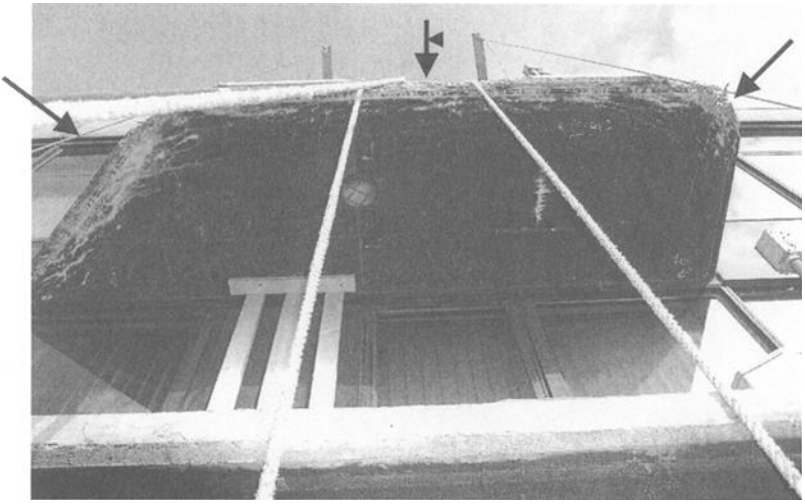


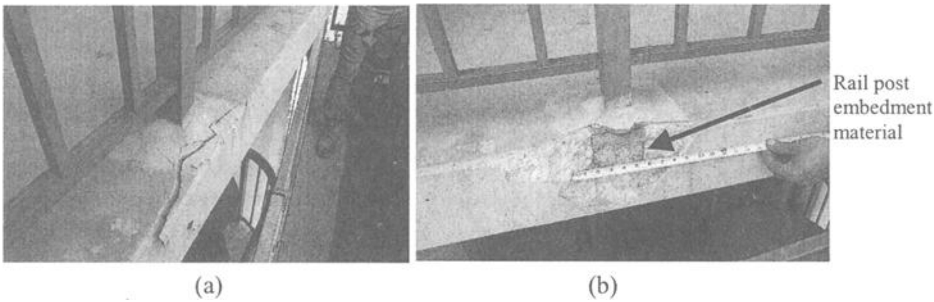
FIG. 5 - *Severe freeze-thaw damage on a concrete balcony slab in a building. The arrow indicates surface degradation at edge of balcony. During the repair project this entire balcony slab had to be reconstructed.*

Distress may also be related to material or structural deficiency problems. In the case of structural deficiencies, the structural element may be receiving more load than it was originally designed to support and therefore may develop cracking. This cracking may allow moisture to reach the embedded reinforcing steel, which then corrodes. The structural cause of this type of distress should be fully investigated and understood in order to develop appropriate repairs. Other examples of distress are restrained shrinkage cracking, construction defects such as misplacement of reinforcing bars which may cause structural deficiencies, improper cover, improperly placed or consolidated concrete, and construction deficiencies.

Concrete distress may occur at previously repaired locations or adjacent to previously repaired locations. Failure of previously repaired areas may be caused by improper surface preparation or improper repair installations, the use of inappropriate original or repair materials, or the repair may have reached its design life expectancy. The authors' experience with the repair of concrete structures is that repairs last about 7 to 15 years depending upon the original concrete characteristics, repair procedures and materials, exposure to the elements and protection. Maintenance of the protection system i.e. architectural coatings, sealing of cracks, and other measures will lengthen the life of the repairs. Often, distress developing at previous repairs in the facade is an indication of improperly prepared repair areas.

Some types of concrete distress are related to specific architectural features or construction details. At reveals or other architectural features, distress may be related to insufficient concrete cover over the reinforcing steel as shown in Figure 4. These problems are particularly common at reveals because of the difficulty in placing the reinforcing steel in the wall in a consistent plane; when the wall steps back at the reveal, the concrete cover is physically reduced.

During investigations of concrete balconies, concrete distress is often observed on the top of the balcony surface or along the balcony edges. This distress may be caused by the embedded railing element, reinforcing steel along the edge, or the material used to set the balcony rail posts. In some instances where distress has occurred around a rail post, a gypsum-based material can be traced to the cracking distress. In an exterior application or in the presence of moisture, this gypsum-based material will degrade and also expand. This expansive force will cause the concrete around the rail post to spall. An example is show in Figure 6.



(a) (b)

FIG. 6 - *Concrete cracking and delamination observed at a balcony edge (a). When the spall was removed, as shown in figure (b), no reinforcing steel was observed behind the spall. In this case the cause of the distress was the use of an inappropriate balcony rail post embedment material.*

On the balcony surface, distress may be related to floor coverings on the balcony. Some coverings such as carpets or certain types of tile may trap moisture within the concrete, leading to corrosion of the embedded reinforcing steel. Coverings can also conceal existing distress on the balcony concrete top surface.

The slope of the balcony surface should be checked to determine if the balcony collects or ponds water. Without a membrane, balcony slabs will be exposed to significantly more moisture. In most cases, the sliding door, window, or other element of the building facade is supported on the balcony slab by a curb that was cast at a different time than the balcony slab. The curb must be carefully inspected for watertightness at its joint with the balcony slab.

Distress and deterioration of concrete surfaces may be caused by many conditions. Therefore, it is very important that the repairs to the facade address the causes of distress and not only the symptoms. If the conditions that cause the distress are not identified and controlled, the repairs will not be very durable. For example, in the case of some balcony railing embedments, the cause of the distress is related to the use of improper embedment material containing gypsum. If all of the gypsum is not removed during the repair, this distress condition will continue to occur.

Other items to note during the condition survey are whether the concrete has been treated with an architectural coating or a sealer, or of it is untreated concrete. If a new coating is to be applied, the condition of the concrete or coating substrate should be evaluated to determine the level of surface preparation that will be required for the proper bond of the protective coating.

Methods of Investigation

Sounding techniques are typically used to detect unsound concrete surfaces. One common technique is detecting delaminations by impacting the concrete surfaces with a hammer as shown in Figure 7. When the hammer strikes the concrete surface, a distinct hollow sound is heard if delaminations exist. If significant areas of delaminations are detected during the investigation, this is typically an indication that substantial reinforcement corrosion has occurred, although actual reinforcement section loss may be minimal.

Other investigation techniques include the use of nondestructive and invasive methods. Nondestructive methods include the use of pachometers (metal detectors) to detect existing reinforcing and impact echo equipment to locate unsound areas of concrete.

Invasive methods include creating inspection openings in the concrete. Inspection openings provide an understanding of underlying conditions and may be used to assess previous repairs. Inspection openings also allow the architect/engineer to confirm the existence of reinforcing steel, depth of embedded reinforcing steel, size of reinforcing steel, extent of breakout or growth of the patch, and amount of section loss in the reinforcing steel. The extent of surface preparation in the previous repair work

FIG. 7 - *Sound testing of concrete with a hammer to detect delaminations.*

can be determined, as well as whether the reinforcing steel was sandblasted, or if supplemental steel was added. This information is important in determining repair quantities if previous repairs are determined to have a limited short remaining service life.

Removal of concrete samples for laboratory analysis is an important tool in obtaining an understanding of causes of the distress. Laboratory testing on concrete samples provides information about potential causes of deterioration and distress, confirms observations made during the field investigation, determines previous repair materials, and also generates information for use in designing repairs. Types of laboratory testing that are typically performed on the concrete samples are petrographic evaluations following ASTM Practice for Petrographic Examination of Hardened Concrete (C 856). ASTM C 856 provides an indication of general aggregate identification, carbonation depth, cement content estimates, and chloride content. Other tests such: as ASTM Practice for Microscopical Determination of Air Void Content and Parameters of the Air Void System in Hardened Concrete (C 547) and ASTM Test Method for Compressive Strength of Cylindrical Concrete Specimens (C 39) concrete compressive tests also provide useful data. The results of laboratory analysis are important in the design of repairs to a facade. For example, if laboratory analysis indicates that the concrete has high chloride levels, then it is especially important that the repairs include a protection system for the concrete to reduce moisture penetration into the concrete. This is due to the fact that chlorides in the presence of moisture and oxygen will corrode embedded reinforcing steel.

The petrographic evaluation can be used to identify aggregate materials that may be reacting with other components of the concrete, causing deterioration. Figure 8 shows a precast concrete column cap cover from a high rise structure. The column cap was exhibiting cracking distress. The laboratory analysis using methods outlined in ASTM C 856 determined that the distress was primarily caused by a reaction of glass aggregates used in the concrete reacting with the alkalies in the cement (alkali-silica reactivity)[4]. A petrographic examination can also provide information for compositional analysis of the existing concrete that can assist in developing a repair mix design to match the existing concrete. This may be the case with repairs to historic or architecturally significant uncoated concrete structure.

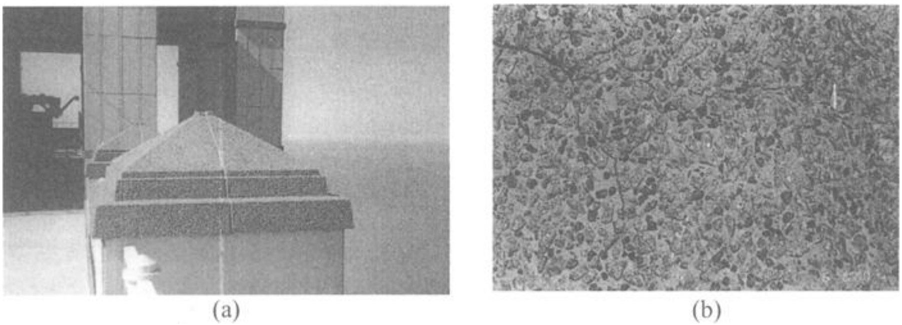

(a) (b)

FIG. 8 - *Figure (a) shows a column capital cover with extensive cracking distress. Figure (b) shows a close-up of observed cracking which was determined by laboratory analysis to be caused by alkali-silica reaction.*

Repair Approaches and Implementation of Repairs

After reviewing the results of the laboratory analysis and condition assessment, conceptual repairs for the concrete elements can be developed. The appropriate repair for the structure will depend largely on the type of structure and the intent of the repairs. In the context of ordinance inspections, the intent of repairs is to address and prevent "imminent hazards." To address such conditions, it is sometimes necessary to designate repairs for immediate implementation. These repairs may include immediate removal of designated concrete pieces or stabilization of selected elements of the facade. Following removal or stabilization repairs, short-term or long-term repairs are implemented. Prioritization of repairs should address the existing distress conditions, the owner's requirements, and budget limitations for the project. The size and cost of the repair project may require that the work be performed in phases.

When the building is architecturally significant, or a historic or landmark structure, the repair design should be sympathetic to the existing facade as well as meet

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[4] Alkali silica reactivity - The reaction between alkalies in portland cement and certain siliceous aggregates used in the concrete.

performance requirements. [3,4] In many cases, the repairs must match the appearance of the original concrete. If the facade elements consist of uncoated exposed concrete or an exposed aggregate finish, the concrete repairs should be developed to match the existing concrete in color, finish, and texture. Even if the original concrete mix is known, trial repairs should still be performed due to the variability of the cement and aggregates and to match the existing appearance. To determine the existing appearance of the concrete, a cleaning study should be undertaken to determine the appearance of the original existing concrete. If the facade is to be protected with an architectural coating, then color matching of the repair concrete with the existing concrete is not as critical. When new repairs are required to match the existing concrete color, several trial repair mixes and sufficient curing time may be needed to develop the appropriate mix. Additionally, various finishing methods may need to be included in the trial samples.

An example of an appropriate repair approach based upon laboratory analysis is the repair of the distress at balcony railing posts. Usually, the deterioration is the result of one of two conditions. If the distress is related to a gypsum-based embedment material used to the set the railings, then the repairs will likely need to be implemented at all of the balconies where this material was used. On the other hand, if the distress is related just to the balcony railing itself, such as corrosion of the railing or of reinforcing steel near the railing post, then repairs can be performed only at specific areas of observed distress.

Bidding Approaches

Concrete facade repair projects are often bid using the unit price bid approach due to the difficulty of accurately establishing repair quantities in advance of the work. If a lump sum approach is selected, the contractor may conservatively assume higher repair quantities. It is our experience that the unit price approach based upon estimated repair quantities, where accurate, works well with concrete repair projects. The final project cost is determined based on the final repair quantities. The architect/engineer estimates concrete repair quantities based on previous experience and results from the condition survey.

One of the drawbacks to the unit price approach is that the true final repair cost is not known until the completion of the project. One way to reduce the level of this uncertainty is to conduct a trial repair at a representative section of the facade so that the work can be evaluated. This allows for adjustment of the repair estimate based upon the trial repair work. The trial repair also provides an opportunity for the architect/engineer to modify the repair procedures and techniques to adapt to actual as-built conditions. Performing trial repairs in a representative area gives the owner and architect/engineer an opportunity to evaluate the contractor's work, evaluate how the repair project may disrupt the building operations, provide more accurate repair quantities, and make other adjustments accordingly for full scale repairs.

Quality Control

Involvement of the architect/engineer is essential in the implementation of repairs. The level of architect/engineer involvement during the repairs is dependent in part on the nature and complexity of the repairs. While the implementation of quality control measures is the responsibility of the contractor, involvement of the architect/engineer to review work in progress for conformance with the construction documents is also needed. In addition, the architect/engineer should be involved in review of submittals and shop drawings, mix designs, and test results, and should participate in evaluation of conditions as they are uncovered.

To maintain consistency during the project, periodic testing of materials is recommended. Generally the testing will include measurement of slump ASTM Test Method for Slump of Portland Cement Concrete and air content (C 143), ASTM Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method (C 231) in the field and compressive strength, ASTM Test Method for Compressive Strength of Cylindrical Concrete Specimens (C 39) in the laboratory.

Mock-ups of the repairs are also an important quality control measure. Mock-ups should be performed in areas that are fully accessible and unobtrusive. Additionally, a mock-up should be performed for each type of repairs that is going to be performed on the facade. When approved, this mock-up will become the standard for the project. We generally recommend that testing of repair materials in trial batches and establishment of testing criteria be completed prior to the work on the building.

Conclusion

Investigation and repair of concrete building facades begins with an understanding of reinforced concrete, a clear understanding of the type and causes of deterioration and distress present, and an understanding of the type of structure. In order to develop an understanding of the existing condition of the facade, the investigation should include a condition survey of the facade. The condition survey should include sounding tests to locate deteriorated and delaminated concrete, inspection openings to observe as-built construction, material sampling, and laboratory analysis.

The results of the investigation are important in the design and development of repair options for the concrete facade. Investigation findings that indicate a higher rate of concrete deterioration may lend more weight to more aggressive repair options and protection systems.

The overall approach to investigation and repair of facades commonly includes phasing of the repair process. Repair projects are divided into phases that include investigation, repair documents, bidding, and construction. The authors recommend the inclusion of an additional phase of trial repairs in the repair documents phase or to be implemented in the construction phase. The trial phase can offer opportunities to verify and modify repair designs, monitor constructability, and verify repair quantity estimates.

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Facade Ordinances and Temporary Stabilization Techniques for Historic Masonry Facades

Reference: Gabby, B., and Vossoughi, H., "**Facade Ordinances and Temporary Stabilization for Historic Masonry Facades,**" *Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444,* J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: To the general public, the image of buildings, especially those with historic significance, is often one of permanency. Yet, those involved in building design and construction and, to some extent, building owners, understand the relative nature of this word. Building owners and managers must continually maintain their building envelopes so that function, durability, and safety can be achieved. This is increasingly important as building stocks get older. The rise in the number of cities adopting facade inspection ordinances indicates that maintenance is often differed.

Several cities in the United States have enacted facade inspection ordinances that apply to all types of buildings. These ordinances are largely in response to the declining health of historic masonry buildings. These buildings often exhibit conditions that require immediate attention to remediate potentially "unsafe" conditions. This paper briefly describes historic masonry construction, explains architectural details and materials that are often problematic, identifies unsafe conditions, discusses temporary stabilization techniques, and explores issues of owner responsibility in pursuing the discovery of potential unsafe conditions until final repairs are made.

Keywords: historic masonry facades, temporary stabilization, facade ordinances

Introduction

All building owners and managers have to juggle the competing demands of operating costs and maximizing the value of their assets. Understandably, the primary concern of most building owners and managers is occupancy and providing attractive service to their tenants. Owners of historic masonry buildings, however, must pay particular attention to their facades due to the age of these buildings and inherent problems in the original construction and detailing that are now manifesting themselves.

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With the advent of facade ordinances in New York, NY, Chicago, IL, Boston, MA, Detroit, MI, and Columbus, OH, building owners in these cities are required by law to have their facades inspected on a routine basis. Inspections of historic masonry facades often reveal conditions that require immediate attention. Various remedial approaches are available, ranging from temporary to long-term. Long-term repairs require time to evaluate and select remedial alternatives and often require significant expenditures. Temporary stabilization techniques are often implemented to mitigate hazardous conditions, reduce the rate of further damage resulting from deteriorated conditions, and to "buy time."

The following discussions attempt to place historic masonry buildings in an appropriate architectural context, explains typical facade problems and their consequences, discusses hazardous conditions, focuses on temporary stabilization techniques, and explores issues of ownership expectations and responsibility while prudent decisions can be made on long-term repairs.

Overview of Historic Masonry Facades

Masonry has been used for thousands of years to construct buildings that vary in size and historical significance from single-family homes to grand monuments. Until the advent of structural steel and reinforced concrete frames in the late nineteenth to early twentieth centuries, masonry construction was load-bearing with walls that varied in thickness from 8 in. (20 cm) to several feet (meters), depending on building height. These walls were typically designed based on empirical height-to-thickness ratios and used their mass as the primary waterproofing element.

The introduction of steel and reinforced concrete frames created a radical change in U.S. construction practices. The period between the late nineteenth century and the middle twentieth century is a transitional age in construction. Although load bearing wall construction was still used for low-rise buildings in the U.S. during this time, it was losing its popularity in tight urban areas due to its inability to create tall, slender structures that could maximize small areas of land.

The buildings constructed during the late nineteenth to early twentieth century are "transitional" because they are the precursor to modern curtain walls, which do not rely on their exterior facade for support of the building's gravity loads, but still use thick wall masonry for their waterproofing and fireproofing. There are many examples of buildings in the United States where steel framing elements were used in combination with massive masonry walls.

Common features of both load-bearing and transitional masonry facades include ornate cornices, parapets, water tables, and other articulated elements constructed of brick, terra cotta, or stone (Figure 1). Depending on their projection beyond the vertical face of the building, cantilevered elements such as cornices and decorative band courses may be reinforced with steel brackets, armatures, and hangers. In addition, parapets often contain metallic cramp anchors and pins tying facade materials to each other and to the back-up masonry.

Figure 1

Problem Areas and Consequences

Water is the primary cause of most damage to historic masonry facades. We commonly find water-related damage in the following areas:

- Locations of high exposure to wind-driven rain, such as parapets or other vertical element projecting above roof lines;
- Projecting architectural elements, such as cornices and water tables, especially those under damaged flashing and built-in gutters;
- Hung masonry soffits and decorative lintels at window heads;
- Areas below fenestration openings and roof line intersections;
- Cast-stone and face-bedded sandstone; and
- Areas of differential movement between structural framing and facades in transitional masonry buildings.

The consequences of water-related damage at these areas include:

- Bulging and lateral movement of masonry, resulting in partially or fully unsupported conditions and/or loss of material (Figures 2 and 3);
- Cracking, spalling, and loss of architectural elements due to freeze-thaw cycling and corrosion of embedded anchors and steel framing (Figures 4 and 5); and
- Increased water penetration resulting in accelerated damage and loss of material.

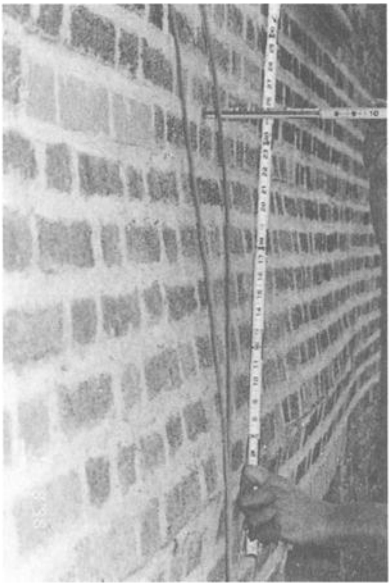

Figure 2

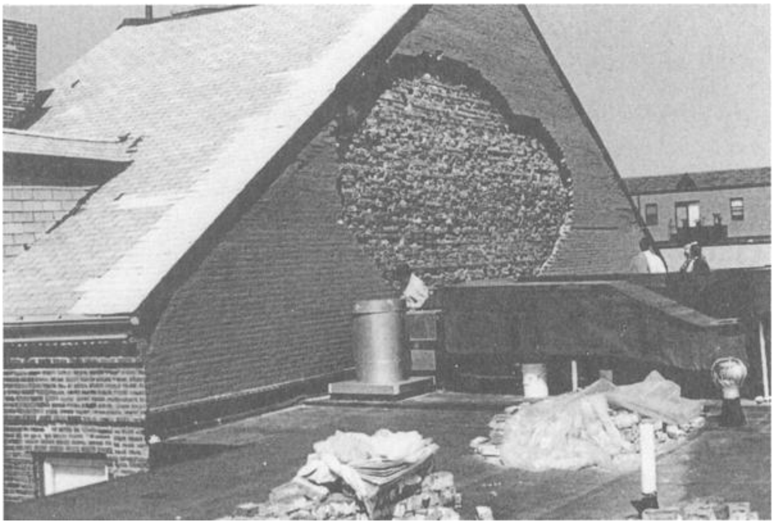

Figure 3

Figure 4

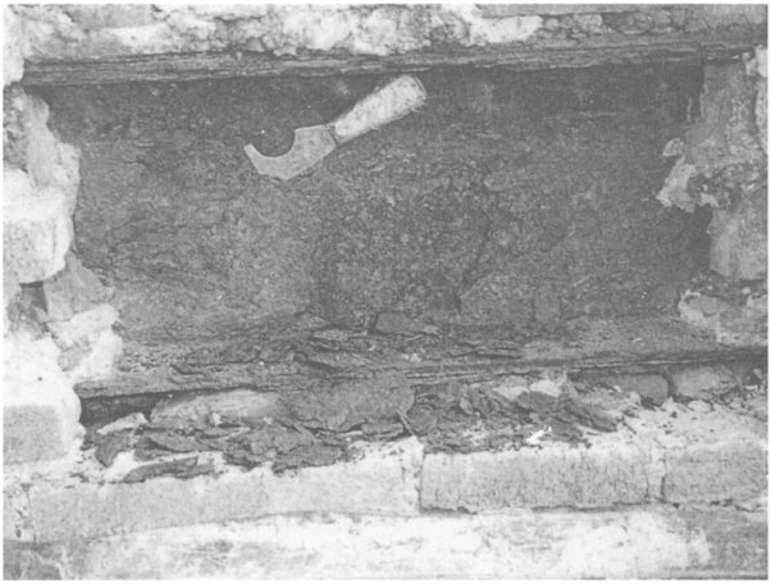

Figure 5

Facade Ordinances: What Should Be Required?

Due to the age and architectural characteristics of historic masonry buildings, owners of these buildings should require periodic inspections as a way of protecting their assets and avoiding injury. But as a building owner, what should be required of a facade inspection?

Facade ordinances generally mandate that licensed professional architects and engineers conduct inspections every four to five years (Detroit, MI, allows experienced workmen). If building owners do not comply with the ordinance, most ordinances impose a monetary fine. There is a wide variation between ordinances regarding the scope of the inspection. We suggest that building owners generally follow these basic rules.

- Eliminate Height-Based Requirements: Most facade ordinances allow binocular surveys of buildings less than 60 to 80 ft (19 to 25 m) in height and a "hands-on" visual inspection of those buildings greater in height. Due to the articulation of many historic masonry facades and the nature of distress often encountered, hazardous conditions often cannot be seen from the ground even with optical enhancement. Figures 6 and 7 illustrate a spall that was removed from the third floor of a building in a congested urban area. We surveyed all building elevations from the ground with binoculars before we began our hands-on inspection of all facade surfaces. The spall, which came off with little hand pressure, was not picked up in our initial binocular survey. Figures 8 and 9 illustrate a similar situation we encountered on a different building in a comparable urban setting. This example is a cast iron column capital only 15 ft (4.5 m) above the sidewalk, which could not be seen from street level.

Figure 6

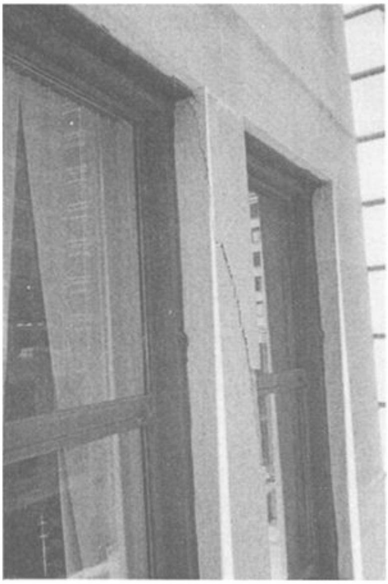

Figure 7

Figure 8

Figure 9

- Inspect all Facade Surfaces: The Chicago, IL, ordinance is the only one that requires that the hands-on inspection to be on all facade surfaces. On the other extreme, Detroit, MI, only requires the inspection of cornices or similar projections that are subject to deterioration or corrosion. The other three ordinances fall somewhere in between. Like the Chicago ordinance, we recommend that building owners have a hands-on survey conducted on all facade surfaces. This need is illustrated by a recent event in a large city in the northeastern United States. At 11:00 a.m. on a weekday, a large limestone modillion weighing approximately 100 lbs (Figure 10) fell 16 stories onto the sidewalk adjacent to the main entrance of the building. Fortunately, nobody was injured. The building owner had the facade inspected by a professional engineer two years prior. The ordinance in this city only requires one inspection drop on all elevations of the building. The report submitted to the city complied with the ordinance. The inspection drops were not made in the area of the fallen modillion.

Figure 10

- Probe Openings only when Required by the Qualified Professional: The Chicago, IL, ordinance requires three probe openings in each facade during every inspection. While this may be prudent in some circumstances, blindly requiring and specifying the number of probe openings may not be in the best interest of public safety due to the number of hazardous conditions encountered on a given building type or may inflict excessive damage to other buildings that are otherwise in sound condition. For example, load-bearing masonry buildings do not contain the same level of embedded steel as transitional masonry buildings so the number of probe openings in load-bearing construction may be limited and concentrated in only those areas where embedded steel is likely encountered, such as window lintels and large projecting cornices and band courses. Unless mortar is heavily deteriorated or bulges exist, probe openings in the field of load-bearing walls often reveal little useful information. In addition, load-bearing buildings built of brick generally do not encounter differential movement problems between exterior wythe and back-up. Probe openings to observe the condition of headers in this type of construction is usually not warranted, but may be necessary with transitional masonry buildings similarly built of brick.

  The Chicago, IL, ordinance also does not provide guidance regarding the size of the probe openings required. The size of probe openings will vary and is governed by the condition under examination (e.g., a steel spandrel beam-to-column connection). An opening in this location may require the removal of up to 4 ft^2 (0.37 m^2) of material. On the other hand, large limestone panels that are attached to the back-up with metal cramp anchors may only require a small opening at anchor locations. In this situation, non-destructive testing techniques may be the best diagnostic tool to determine the level of corrosion potential and rates coupled with limited probe openings to calibrate equipment and verify results.

The necessity of the number, location, and size of probe openings should be left to the judgment of the qualified professional and be based on the findings of the full facade hands-on inspection. Facade ordinances should be modified to reflect these changes. If building owners do not want probe openings to be made, cities can adopt language that gives the qualified professional the ability to withhold the facade inspection report until this work is complete.

Determining Hazardous Conditions

Facade inspections can produce evidence of hazardous conditions. Facade ordinances and licensing ethics require professional engineers and architects to promptly report all hazardous conditions to the building owner and local building officials. But what is a hazardous condition?

If not specifically stated in an ordinance or by a state licensing board, determining a hazardous condition is based on the judgment of an average professional engineer or architect identifying an outward sign of distress and determining it a hazard to the general public.

Often, outward signs of loose material, such as large spalls, cracked architectural features, and bowed masonry are identified by average professional engineers and architects as hazardous conditions. However, professional judgment based on experience and knowledge of historic masonry construction is also required to properly diagnose the significance of a hazardous condition. Understanding the significance of a hazardous condition is vital for protecting public safety and the interests of building owners by not placing them in a situation of expending funds to repair a situation unnecessarily.

Building owners should hire only those qualified engineers and architects with experience and special knowledge of the stability and deterioration mechanisms of the facade being inspected. In a competitive market, fees are often a factor in hiring a professional. However, they should never be the sole determinate.

Temporary Stabilization Techniques

Temporary stabilization of hazardous conditions is often performed prior to and, perhaps, in lieu of more durable repairs. Temporary stabilization should be considered as an interim measure to mitigate risks until more durable repairs can be designed or funds are secured to pay for the work. Once a hazardous condition is determined, it is the responsibility of the building owner to have it stabilized. Commencement of stabilization work should be immediate and not contingent on acquiring a building permit. Building owners should, however, inform the building department and architectural review boards, if appropriate, that the stabilization work is moving forward.

Temporary stabilization may include:

- Installing protective street-level staging;

- Removing unstable elements (Figure 11);

Figure 11

- Repairs to stop water intrusion; and
- Retaining elements in place, generally with pins, straps, or netting (Figures 12 and 13).

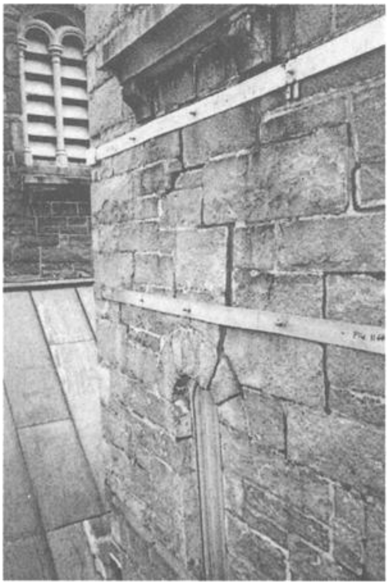

Figure 12

Figure 13

Installing Protective Staging

Installing protective staging is generally the first step in any stabilization project. Protective staging is often limited to pedestrian protection, i.e., covering just sidewalk areas. Sidewalk staging with rated planking should cover the entire sidewalk area as a minimum. Plywood eyebrows should extend above the top of the staging to help contain falling materials that may to bounce into streets. Most cities require that any sidewalk protection be lit during nighttime hours.

Depending on the nature of the distress, protective staging can be designed to protect against the loss of large wall sections. Protective staging, when used for this purpose, must be designed with the proper lateral tie backs and have enough capacity to resist the impact loads.

Sidewalk protection is considered an alternative in lieu of retaining elements in place (see below). Depending on the height of the hazardous condition and the articulation of the building, the likelihood of loose debris falling onto the sidewalk protection may be remote. Owners who have loose debris high on their building should insist that in-place retention methods be employed to prevent pieces from falling to staging or the street below.

Selective Removal

Broken or deteriorated elements that are hazardous should be removed from the building and replaced or reattached (see below). Large pieces can be removed and stored for reinstallation if historic renovation is required. During inspections, patches, cracks, and suspected spalls should be sounded with masonry tools. If areas sound hollow, they should be probed and removed as required. An easy determination of whether a spall should be removed is to "wiggle" the spall. If the spall wiggles under hand pressure, it should be removed.

In some cases, removing a hazardous condition may undermine a previously stable area. Holes that remain after pieces are removed may also require waterproofing to limit water intrusion and potential further deterioration. Careful consideration of these issues by a competent professional must be made before any material is removed from the building.

Stop Water Intrusion

The cause of most masonry problems is water penetration. Stopping water penetration can be an effective method of limiting deterioration. Roofs, gutters, drain or gutter leaders, cornices, and windows are common sources of substantial water penetration.

Retain Elements in Place

While removing loose pieces may be a relatively simple and quick method of reducing the risks of falling objects, it generally means more extensive restoration is needed in the future, particularly if decorative elements are removed. An alternative is to retain pieces in place. Retaining elements in place is often done using a combination of epoxy-set or mechanical anchors, external metal straps, hardware cloth, or polypropylene netting. These methods can temporarily secure historic fabric and maintain a semblance of the facade's appearance until final repairs are implemented. However, temporary pins and nets must be monitored on a continuing basis and augmented, if required, until permanent repairs are completed.

Owner Expectations

Facade inspections are no guarantee that all hazardous conditions have been identified on a building, and temporary stabilization is not a substitute for durable repairs. Professional engineers and architects should communicate clearly that building owners are responsible for the safe maintenance of their facades. Facade inspections and stabilization techniques reduce, but do not eliminate, the risk associated with historic facades in public areas.

Conclusions

Historic masonry buildings are unique structures that require special classification and guidelines. Maintaining these important examples of architectural history and protecting public safety requires planning and continuous effort on behalf of building owners. Periodically inspecting historic masonry buildings is a key factor in maintaining these buildings. Often, these inspections identify hazardous conditions that require immediate remediation. Temporary stabilization techniques can be effective in mitigating hazardous conditions, reducing the rate of further damage resulting from deteriorated conditions, and in "buying time" to make well-reasoned decisions for long-term repairs. The key in finding a prudent and equitable solution to hazardous conditions rests in the judgment of a competent and knowledgeable, qualified professional.

David A. VanOcker, P.E.[1]

Designer-Led Design/Build – Alternative Project Delivery Method for Façade Evaluation and Repair Projects – Case Study on an 11 Story Apartment Building

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Reference: VanOcker, D. A., **"Designer-Led Design/Build – Alternative Project Delivery Method for Façade Evaluation and Repair Projects – Case Study on an 11 Story Apartment Building,"** *Building Façade Maintenance, Repair, and Inspection, ASTM STP 1444,* J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: Traditional Design/Bid/Build project delivery methods are inherently cumbersome for complex façade repair programs. The repair program on an 11 – story apartment building is documented, demonstrating the aspects and benefits of utilizing a Design/Build contracting approach, with the Design-Professional leading the repair program. The project begins with an assessment that concludes with the recommendation to replace the façade. Early evaluation efforts document the need for immediate safety/ stabilization measures, then progress through the successful delivery of a complex façade repair program under a project delivery method that is growing in acceptance.

Keywords: design/build, design/bid/build, designer-led, specialty repair contractor, stabilization measures, risk exposure

Introduction

Traditional design/bid/build (D/B/B) project delivery is common in the construction industry, no less so in the repair and restoration of building envelopes. D/B/B can however, be inherently cumbersome on complex façade repair programs. Success on such projects depends in large part on the expertise of a team of professionals – the restoration engineer and constructor. Working together contractually in a design/build (D/B) relationship from the start, this team jointly diagnoses envelope conditions, fully assesses the feasible repair approaches (alternatives, logistical factors), then establishes realistic schedule(s) and budget criteria. The experience of the author's firm is that

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design/build (D/B) is a superior approach for serving the needs of a building owner with respect to building envelope repair and restoration.

D/B project delivery is a more adaptable approach on complex façade repair projects, providing a facility owner with the highest level of assurance of a quality repair, while at the same time leading to a shorter overall project schedule, reduced risk of encountering surprises after work starts, and ultimately, producing a less costly project. An engineer-led design/build team provides further assurances for the success of such a repair program. Our experience has been that with design/build (D/B), we are better able to establish the most realistic project goals in the earliest practical time frame, then assist an owner in understanding their options, setting realistic project expectations, and delivering a successful project.

Significant benefits of Designer-Led D/B delivery are demonstrated on a façade repair project:

- Shorter overall project schedule
- Design professional making ultimate decisions (Engineer-Led)
- Fully engineered, validated solutions, before the owner's final commitment
- Guarantees to owner – commitment to Guaranteed Maximum Price (GMP)
- Single source responsibility
- Lower risk for the D/B team through collaboration

Project Background

The subject building is an 11 story concrete-framed, flat plate structure, with a composite brick/concrete masonry unit (cmu) façade. Built in the mid 1960s, the building experienced leakage and distress within two years of completion of construction. A 1996 assessment by a facade consultant resulted in the recommendation to replace the entire façade.

The building measures 60 ft. (18.3 m) wide by 320 ft. (97.5 m) long, with projected bays occurring at four locations along the long sides. A photograph (Figure 1) of the southeast corner elevation delineates the typical configuration of the façade features.

Figure 1 – *Elevation of Southeast Corner*

Project Phases

The project was performed in multiple phases under several separate contracts. Reference will be made to the following discrete phases:

- Condition Survey & Evaluation Phase,
- Preliminary Design Phase – Part 1 Design/Build,
- Part 2 Design/Build – Pilot Repair Phase, and
- Part 2 Design/Build – Full Scale Repair Program.

The author's firm was initially retained to provide a second opinion of the above-mentioned assessment. After an initial evaluation and meeting with the owner, the author's firm was retained to perform a formal, in-depth condition survey and evaluation. The contractual arrangement for the survey and evaluation follows a typical initial phase owner – engineer relationship, where the owner retains a design professional, who in turn retains a specialty repair contractor (SRC), to provide labor and access for the field surveys. Figure 2 depicts the relationship for this initial investigative phase.

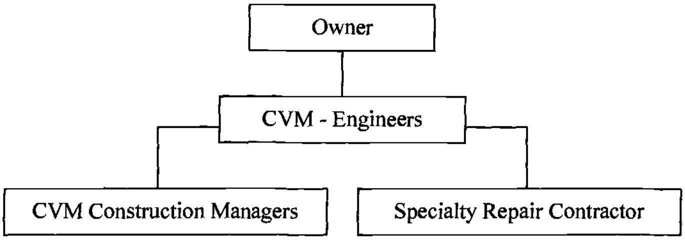

Figure 2 - *Contractual Arrangement for Condition Survey and Evaluation Phase*

CVM is really two separate firms: CVM Engineers (CVM-Eng) and CVM Construction Managers (CVM-CM). CVM-Eng performs traditional professional services – engineering, architecture, material science, etc., while CVM-CM performs preconstruction services, such as budgeting, scheduling, and constructability review, much like a traditional CM firm.

While this initial phase contractual relationship may not be considered as true Design/Build, certain project specific conditions made it so. As is often the case, stabilization measures were anticipated as part of the first phase services. In this particular building we could see loose, delaminated sections of fractured brick along many of the relief angle lines, a life-safety condition that we wanted to address while we were performing our surveys. Unstable façade conditions can most expeditiously be rendered "safe" through the timely implementation of temporary repairs while the surveys are being conducted. It is best when the engineer-lead directs the stabilization efforts, so they can assure the owner (and often Building Code officials) that the conditions have been rendered stable. A definite concern however, is the risk this may represent to the engineer's professional practice liability insurance coverage, as most such policies restrict the engineer's control of the contractor.

The Condition Survey and Evaluation Phase usually involves swing scaffold and/or high-reach equipment assisted surveys, exploratory probing, material sampling and testing. While it is just as common for the owner to retain the SRC as it is for the engineer to retain them directly, in our case we always look for more than just a means of getting on the façade (and the labor to support this routine). We look for experienced Project Managers (PM) from SRC's side, to assist in the evaluation. The engineers lead the evaluation effort and are not timid about leading stabilization efforts, but once the evaluation has proceeded to a certain point, the engineer – SRC team quickly begins to focus on just exactly what an ultimate repair program is going to entail. The SRC thus works with us early on as we begin to formulate the repair/restoration program. Our experience is that the technicians on the swing stages do not provide this service, but the experienced project managers within the SRC's organization do so.

The difficulty and/or complexity of rigging a façade is critical to factor into the time involved and overall costs of the evaluation phase. The engineer needs to decide which areas are most important to access while considering the SRC's perspective of safely getting there. Collaboration is necessary for the type and extent of probing/testing anticipated, the degree of stabilization that is expected to occur. When distressed parapets are a concern, the use of parapet hooks is obviously unwise. Pedestrian safety and ground control also enter into the picture.

Special Project Conditions Going Into the Evaluation

➢ Emergency stabilization: visual inspection from the ground level revealed spalling brick along relief angle lines (Figure 3). We knew that once we got on the facade, it made the most sense to do some level of stabilization (removal, temporary patching), thus this was factored into our contract.

➤ Hidden conditions: while we could see from grade the need to stabilize spalling masonry, we expected to encounter other areas requiring stabilization repair that were not discernable from grade level. An example is bulging regions of masonry resulting from rust jacking, especially at the roof level (Figure 4).

➤ Locations of swing scaffold drops: we wanted to cover the range of conditions on each façade and look at similar wall configurations over different exposures, but this was tempered by the desire to access visibly distressed areas to perform stabilization measures.

➤ Game plan: it is common to start out with a set plan on how to proceed around the building (number of swing drops, number of moves), but often the plan must be modified after a few drops, once we understand more fully the conditions we are encountering and their meaning, to correspondingly redirect the survey focus.

Figure 3 – *Spalling brick along relief angle line.*

Figure 4 – *Rust-jacking induced bulging at window corner.*

The above conditions are always cumbersome to deal with when they have not been anticipated and when the contractor was retained by the owner under a separate agreement, possibly with expectations of an arbitrarily low fixed cost limit. Consideration must be given to the suitability of the contractor to performing appropriate stabilization measures under the engineer's direction, in order for the engineer to be willing to sign off on them. Similarly, the contractor must be willing to adapt their schedule in moving around the façade to suit the engineer's needs.

Suffice to say that contractors are generally uncomfortable with doing all of the above. They are willing to supply labor and equipment, but not the PM level insight, the flexibility to adapt rapidly to altering the course/sequence of moving equipment, the experienced technicians needed for stabilization efforts, nor are they willing to stand by "their" evaluation of unsafe conditions and the degree to which such have been rendered stable. Many are unaccustomed to working early-on in a project without specific direction from the owner's engineer. A contractual design/build relationship assures that both the engineer and SRC work together to meet the obligations of the contract.

Condition Evaluation

Once the survey was complete, CVM began categorizing the areas of distress and performing engineering analysis to confirm the likely causes of such conditions. We worked jointly with the SRC to establish and quantify repair categories, but only on a very broad basis, due to the early stage of program development. A brainstorming session was held with the SRC where we summarized the engineering conclusions – certain conditions could not be corrected; leakage could not be controlled except through intrusive sill flashing repairs. These were significant findings, with substantial cost impacts. More importantly though, we agreed jointly that the masonry façade could be salvaged.

The most significant conditions were established.

- Extensive brick spalling along relief angle lines could not be alleviated solely through the installation of "soft joints" below relief angle lines, due to excessive slab deflection.
- Slab deflection in bay regions could only be alleviated through removal of the masonry wall load, in order to eliminate cracking and resulting moisture entry paths.
- Deficient window sills and the lack of sill flashing was a primary moisture entry path.

These conditions were the primary factors in setting the direction of any potential repair program. Other conditions warranted repairs, but most of these could be addressed through more traditional repair strategies. We spent the time during our brainstorming session with the SRC evaluating how we would potentially reclad the bay regions, how we could replace windows – all in an occupied building. Before we presented our conclusions to the owner, we wanted a high degree of confidence in our ability to actually perform such repair and reconstruction operations. In light of the previous recommendation to replace the façade, we did not want to offer possibly infeasible solutions and repeat the message of façade "problems". We went further in our evaluation by beginning to generate schematic level budgets and schedules for the solutions that we were going to recommend to the owner.

Evaluation Report

We presented the owner initially with a brief summary of our findings, supplemented with some graphic depictions of the conditions one could only see up-close. The emphasis was not on the problems, but the solutions that we felt were achievable. We met subsequently to discuss/review the recommendations in more detail, which proved to be a vital step in confirming the owner's understanding of conditions and establishment of their desire to consider investing not only in the repair of the façade, but in incorporating improvements that would enhance the value of their investment.

An important consideration at this point is that we did not want to offer any finite estimate of costs. We were however, prepared to talk in generalities of expected cost ranges, which we did through face to face meetings, rather than through reports. We took the time to understand the owner's long-term objectives with the property, which enabled us to respond more directly to their requests, defining a level of repairs and improvements that coincided with their investment objectives.

Budgeting, Scheduling

After completing our contractual obligations for the evaluation, the owner was interested in carrying on serious discussions of repair options and improvements. We drafted a traditional agreement for professional services for preliminary design, continuing with preconstruction services, following the D/B/B approach. At the same time we presented a contractual alternative to enter into a Part 1 design/build agreement. This alternative approach identified the following deliverables:

- Preliminary design program delineated in written and drawing documents;
- Preliminary project schedule;
- Preliminary cost estimate; and
- Formal Part 2 Design/Build proposal for implementation of repairs.

The above approach – Part 1 / Part 2 D/B – is the basis of several agency agreements commonly in use: Associated General Contractors of America (AGC)[1], Design-Build Institute of America (DBIA)[2-3], American Institute of Architects (AIA)[4].

A major feature of the Part 1/Part 2 D/B agreement for the owner's benefit, as compared to the traditional D/B/B, is in the deliverables achieved in a comparable timeframe (refer to Figure 5). In the time that it would take us to prepare a set of biddable Contract Documents under a D/B/B arrangement, we could complete a Part 1 D/B contract. Rather than having a set of bid documents and an engineer's estimate of the project cost and schedule, the owner receives GMP level documentation, with the engineering aspects integrated sufficiently with construction logistics, schedules, and other facets, to offer a price commitment. Further, the project would be defined enough to proceed into construction under the D/B scenario.

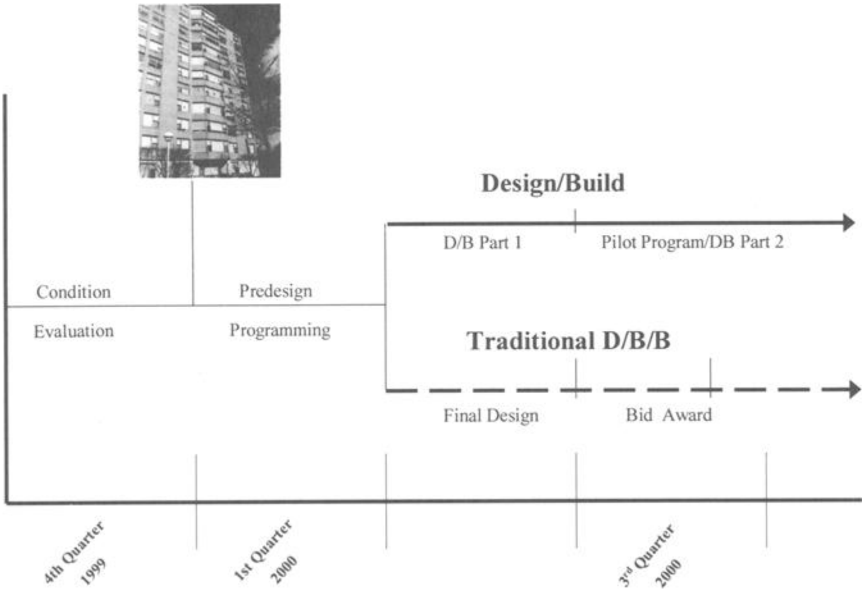

Figure 5 - *Preliminary Project Timeline*

The owner was undecided on an ultimate delivery (construction contracting) method at this point, but wanted to proceed with exploring options and further defining the program. We decided to proceed with program development, but under no contractual agreement to be compensated for any of these services. Our risk was thus our fees, until such time that we could secure a firm D/B agreement. The risk was shared among the parties – both CVM firms, the SRC, and two new players – a window contractor and interiors contractor, both of which we had previous relationships with, although only one under a D/B scenario.

We essentially proceeded down the path of a Part 1 D/B arrangement, but with a narrower objective of performing a pilot program – selecting a representative portion of the façade for performance of complete repairs, with the intent of fully validating as many aspects of the project as possible. We gained better insight and definition on a program that met the owner's objectives and began identifying priority items on the project schedule, the most important being long lead times for brick and windows. We focused on fully defining aspects of the pilot area, a two-bay wide region of the façade. Our feeling was that we could predict the costs and schedule to perform a pilot program very accurately, but we were still uncomfortable with guaranteeing program costs and schedules on a full-scale program that would essentially involve recladding portions of the façade, while the building was fully occupied. Part of our discussions with the owner at this point involved establishing/confirming his financial ability to perform such a project.

Several meetings were held through the first quarter of 2000, resulting in a formal repair program definition that incorporated improvements in the facades:

1. Rebuild brick masonry along significant portions of the relief angle lines;
2. Rebuild isolated areas of unstable masonry – resulting from severe rust-jacking;
3. Point 100% of the brick masonry; clean masonry; replace all sealants;
4. Replace windows 100%
5. Remove the brick outer wythe in bay regions and reclad the bays, incorporating new windows; and
6. Install windows in the blank endwalls.

The first three items were strictly repairs; items 4 and 5 combined repairs with improvements; item 6 was entirely an improvement. The establishment of a defined program enabled us to focus on a detailed formal D/B proposal, except we incorporated the Part 1 aspects into the Part 2 format, essentially having performed the entire Part 1 at risk.

Owner Options

It is imperative for an owner to know their options going forward at each stage in any project, no more so than with a single-source relationship. We felt we had taken the risk through the design/program development phase of the project to date, but expected certain rewards at the end – namely a D/B contract to at least perform the pilot program. We felt we could recoup our design costs through this facet of work. Not wanting to focus on this short-term goal exclusively, however, we kept a focus on the longer term objective of a full-scale repair program. The options to the owner, nevertheless, were kept open.

We offered the owner a choice at this point: execute a formal Part 2, D/B agreement for the pilot phase, or execute a D/B/B agreement for completion of design services through contract documents (on either the pilot or full-scale). If the owner chose the D/B option, an optional deliverable at the end of the pilot phase was a set of contract documents, enabling the owner to continue with the project with or without CVM, and in one of several roles. The most timely option for the owner was with the D/B scenario, in that for a relatively small portion of the projected program cost (~ 15%), they were assured of having a fully validated program, and a portion of their building facade repair completed, and would still have the option of proceeding on different paths of project delivery with no further financial commitment to the D/B team.

A major factor that perhaps influenced the owner's decision at this point was, given the time of year (late May), the prospect of delaying mobilizing for a projected fall pilot phase which had several crucial deadlines for ordering long-lead items. It was essential to reach an agreement to keep the project moving forward without weather related delays that would potentially push the program into the following spring.

An agreement was finally reached that aligned the D/B team as depicted in Figure 6.

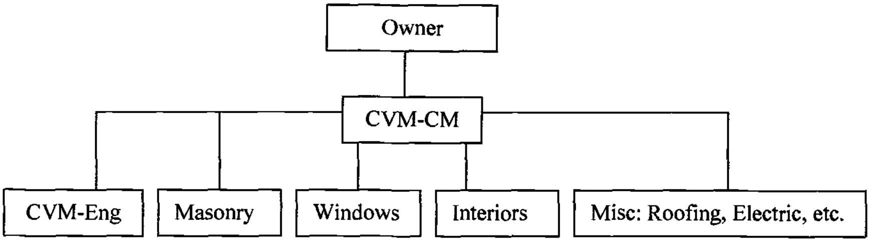

Figure 6 - *Contractual Arrangement for Pilot Phase*

Special aspects of the contract included the commitment for procurement of 100% of the windows, despite the fact that the D/B agreement only covered the pilot area of the façade – for masonry, interiors and other repairs. A substantial discount in the window price was achieved by the owner through this arrangement. The owner also agreed to purchase sufficient brick for the complete façade repair to assure consistency of material. We retained the owner's present roofing repair service contractor, even though there was no current warranty on the roof; the construction activities on the roof and the need to build bay region extensions factored into the decision to retain this subcontractor.

The pilot phase D/B contract was structured with detailed material and labor allowances for key repair regions and work activities. The intent was to track activities closely and make adjustments in final billings to coincide with actual material and labor utilized. More significantly though, was the establishment of full-scale repair program costs structured in a similar fashion. The goal throughout the pilot phase was thus to refine activities and costs to assure the highest level of accuracy when it was time to commit to the full-scale contract. Not only were labor and material costs projected and tracked carefully, but durations and turn-around of interior-related work was similarly tracked and refined, a key factor that impacted tenants.

In as much as we felt confident that our planning efforts had reduced the risk of encountering major unanticipated obstacles during the pilot phase, there were the inevitable surprises.

- Zoning issues became apparent during permitting, due to the introduction of endwall windows – the north face of the building was located at the property line setback limit, thus the window extensions encroached on the setback;
- Selective probing and removals along bay regions to take field measurements for pilot windows failed to detect significant variation in slab edge locations over the height of the building; only after the windows were ordered and we had begun formal demolition did we realize that the slabs on one side of the bay had been crudely cut back beyond the face of the relief angle, while on the other side, upwards of 2"of shims had been added behind relief angles (Figure 7).

- For brick, a final count had to be established, not only for the pilot but for the full-scale repair program – then placed in time for arrival in the late fall; a lot of guess work was needed as several program aspects were still in flux as the deadline for placing the order approached.
- Fire ratings – the decision to incorporate the use of full-height windows in certain floors of bays was aesthetically desirable to the owner, but we ultimately were unable to meet fire-code separation criteria; the window units had already been ordered by the time that we realized we had exhausted all options for making this feature compliant.

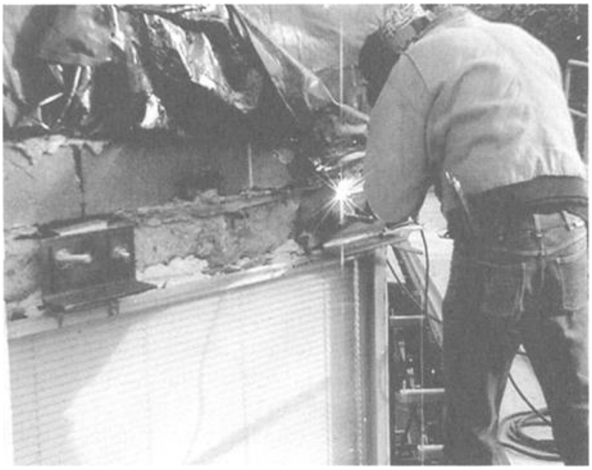

Figure 7 – *Variation in Slab Edge Locations*

The above issues were all resolved during the pilot phase. A major benefit of our D/B team arrangement allowed the avoidance of the traditionally lengthy RFI (Request For Information) routine that often introduces delays and suspicions/allegations on the engineer's and constructor's parts. The professional staff of CVM led the effort to address and resolve all such issues as we advanced through the pilot phase. While the efforts of the constructors focused on implementing the repair work, the staff of CVM acted as owner liaison on a constant basis, enabling the owner's property and maintenance managers and staff to continue to operate their facility in as close to a normal manner as possible.

Figure 8 – *Bay Demolition Proceeded Top to Bottom*

Figure 9 – *Reconstruction Proceeded Bottom to Top*

Working Toward a Full-Scale D/B Agreement

The actual performance of repair work during the pilot seemed to be the easy part. We obviously had to focus on successfully performing the work, but we needed to be creative in convincing the owner that their best interest(s) lay in retaining us to continue through the full-scale repair. We continued to define the project work effort through projections of man-power and work durations, broken down according to repair areas. From the masonry repair perspective, we utilized our own database of cost measures to confirm and validate the desired cost bases sought by our SRC in establishing final repair cost figures. We were somewhat at their mercy as far as confirming activity durations, but they knew there was always the possibility that the owner would insist that we bid out portions of the work to other subs, just to get a check on the final cost figures. From the standpoint of interior work, because the owner had extensive experience with interior fitout work on their other properties, they used their own cost database to reconcile our figures for this category. Detailed spreadsheets were compiled, breaking down all work aspects; these formed the basis for a schedule of values. We took advantage of efficiencies learned through the pilot to shorten durations for given full-scale activities. These translated into direct reductions in repair category costs from earlier projections.

During the pilot we constructed all necessary mock-ups, both interior and exterior. This enabled us to get signoffs on critical aspects of the project, but it also gave the owner the opportunity to fully assess the interior improvements. The owner utilized this aspect to create new model units for initiating a renewed marketing program for maximizing building occupancy.

The ultimate D/B contract that was developed and presented to the owner was a hybrid cost-plus and guaranteed maximum price (GMP) format. We had agreed on engineering and management fees through previous negotiations; the pilot satisfied all parties that the committed man-power of CVM was in line with the negotiated percentages. The owner had been kept advised of repair aspect cost breakdowns throughout the project, thus a lengthy review of the supporting schedule of values breakdown was straightforward. It all came down to the owner's comfort level with the design/build team that had been assembled and our performance through the pilot phase.

As mentioned previously, the owner still had options going forward, although they were perhaps somewhat narrowed at this point:

- Proceed with the proposed D/B agreement for the full scope;
- Retain CVM as Engineer and CM, but direct them to bid all aspects of trade work, maintaining an at-risk exposure;
- Retain CVM as separate Engineering and/or CM entities, acting as owner agent.

The benefit to the owner at the conclusion of the pilot, with respect to a traditional D/B/B contract arrangement, was receipt of a GMP contract proposal based on a fully validated repair program, compared to a set of complete bid documents and fixed price bids. It was certainly conceivable that during the pilot timeframe we could have received competitive bids for the repairs, thus at least validating the initially expected costs; however, through this process we would not have had the benefit of validating a program to the extent that assurances against unexpected cost increases could be offered. The design/build proposal based on the pilot reduced the probability of change orders during the subsequent full scale program.

The pilot program was successful combined the technical expertise of the design professional, the constructability expertise of the specialty subcontractors, and the managerial, logistical and cost estimating expertise of the construction managers, collaborating as a team. The team was led by the design professional. The owner had one single contract, avoiding the internal management effort necessary to oversee separate contracts between the design professional and constructor. An open book policy for cost accounting provided the owner with adequate knowledge of expenditures, tracked against a GMP contract limit.

Full-Scale Design/Build Program

The transition from the pilot program into the full-scale repair program coincided with the winter weather shut-down. Contract negotiations were completed in less than two months, avoiding demobilization costs and enabling a fast re-start before the end of winter.

Work progressed rapidly, but a major improvement aspect remained to be fully defined – installing new windows in the blank endwalls. Through collaboration with the SRC, we had developed a scheme that minimized the number of separate mast platform drops, with resultant time and cost savings, while minimizing initial wall removal and limiting the vulnerability of the building to leakage during this period. The initial pass removed only enough masonry to install and cast the cantilevered slabs edges (Figures 10 & 11). Once the slabs had all been cast, we worked from bottom to top, removing the balance of the masonry, installing temporary protection, following with the new curtainwall installation on a floor by floor basis. Turn-around time for interference inside residential units was limited to three days, as with the bay window replacements.

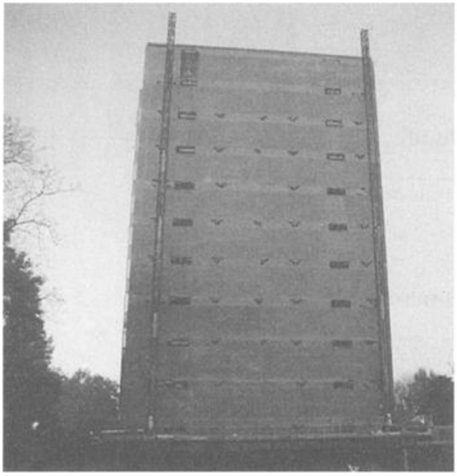

Figure 10 – *Endwall window sequencing.*

Figure 11 – *Cantilevered slab extension at new endwall windows.*

Completion of the full-scale repair project provided overall savings to the owner by implementing a façade repair program exceeding $5.0 mil., within 18 months after the decision to commence a pilot program under a Part 1 Design/Build agreement. An equivalent Design/Bid/Build program (Figure 12) would have added approximately 6 months to the schedule, and potentially thousands of dollars in change orders to the ultimate project. Throughout the project, the design/build team worked jointly to resolve conflicts, develop more efficient methods, and tackle unanticipated field conditions with the goal of avoiding change orders, due largely to the designer-led design/build relationship.

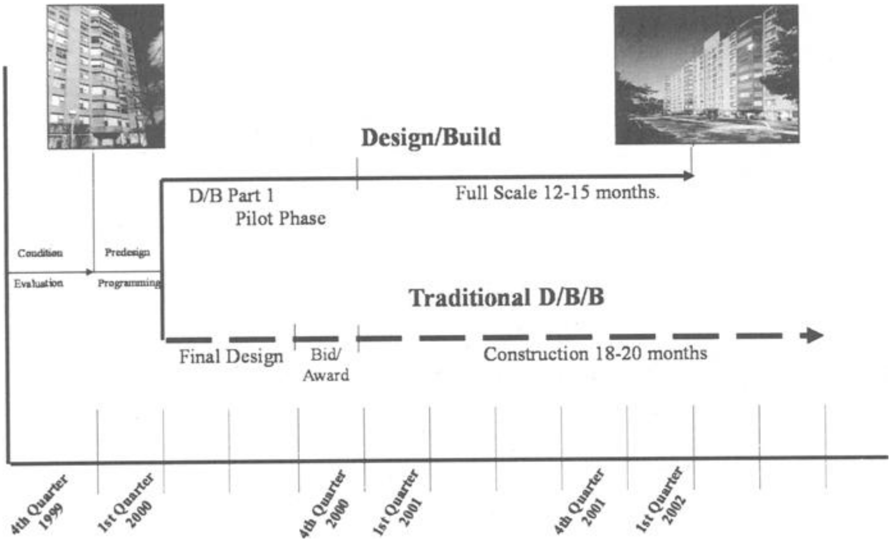

Figure 12- *Overall Comparative Timeline*

Teamwork

Reflecting on the early design development portion of the project and the budgeting, scheduling aspects necessary to fully define the program, as engineers we were not adequately equipped to accurately address all of the Owner's requirements during this phase. As a team of engineers and constructors, however, we were capable of providing much more comprehensive solutions. As engineers we rely on our records of cost data on previous repair projects, but pitfalls occur with this method of cost estimating. As an example, significant costs are attributable to rigging and protection in façade repair programs, but we could only assess these comprehensively through collaboration with our SRC.

Cost differentials for alternative rigging systems were evaluated in relationship to productivity and labor cost differentials with each system. The ultimate rigging systems selected for this project involved fixed mast rigs for major repair aspects, sequenced around the building in a cost-effective manner. The substantial cost of having as many as 4 rigs on the building at a single time, was limited through the performance of certain masonry repair work via follow-up swing stage rigging.

The schedule logistics of sequentially demolishing the brick masonry on the bays, while maintaining water-tightness, then removing/replacing windows and performing interior finishes, required intense planning between several trades. After the Owner's thumbs-down on our initial suggestion that this maneuver would take 8 – 10 working days of tenant interruption, our D/B team brainstorming resulted in a scheme that could accomplish this work with a maximum of three days of interior interference. Construction logistics thus essentially drove many of the final design decisions.

Designer-Led Design/Build

The ability of the Design Professional to lead a design/build contract is a function of proven leadership and an extensive background in engineering and construction management. Traditional engineering service agreements restrict the design professional's leadership ability during the construction process, in large part to avoid compromising professional liability insurance coverage. CVM's construction management philosophy is based on integrating project management decisions with engineering expertise, to provide the most comprehensive services possible, assuring owners of the continued engineering expertise throughout their complex repair/restoration programs.

It is unusual to find a design professional willing to take on the risk associated with design/build, especially when leading the D/B effort. There is however, a no more appropriate leader for such a team, especially when considering the needs of building owners faced with the auspices of a facade inspection program mandate and the demand of building officials to have design professionals vouch for the safety and integrity of a given facade upon inspection and stabilization.

Risk exposure is minimized by working with only proven contractor "professionals" – the specialty repair contractors with which a close working relationship has been established. This remains true from the early evaluation phase of a project, when life-safety measures are often a concern, through implementation of final repairs, when far more significant costs are a factor. The downside for an owner in proceeding with a D/B delivery can be very limited, especially proceeding down this path in steps. Having the professional services expertise (engineering, architectural, material testing, etc.) and construction management expertise in-house, to not only diagnose problems, but to implement the most complex envelope repair/restoration programs, provides the necessary confidence to offer owners an optimal arrangement of services conducive to their needs.

Summary

Design/Build project delivery is increasing in acceptance in the building construction and repair industry. While it is certainly not appropriate for every project, the benefits of incorporating such a delivery method with an experienced D/B team, led by the design professional, can be appreciable. Façade repair programs are fraught with the high potential of cost and schedule overruns, due to the inherent nature of widely varying hidden distressed/deteriorated conditions. The aging building infrastructure in our country produces an endless supply of repair and restoration needs. Alternative project delivery approaches such as D/B are growing in acceptance, to reduce the difficulties encountered in performing façade repair programs.

When we were first asked to consult on this project, to offer a second opinion on another professional's condemnation of the façade, the owner did not have a program. The owner did however, have a property and a long-term investment strategy. The technical challenges and complexities associated with the final program would have been extremely difficult to implement under a D/B/B arrangement. It is doubtful that we ever could have achieved such a refined program, integrating the repair and improvement aspects, without our D/B team arrangement. The mutual trust of the entire D/B team and the owner, resulted in the delivery of a complex repair and improvement program that will renew and extend the useful life and value of this property (Figure 13) for years to come.

Figure 13 – *Completed Façade*

References

[1] AGC Document 415, "Standard Form of Design-Build Agreement and General Conditions Between Owner and Contractor," The Associated General Contractors of America, Alexandria, VA, 1993.

[2] DBIA Document 525, "Standard Form of Agreement Between Owner and Design-Builder," Design-Build Institute of America, Washington, DC, First Edition 1998.

[3] DBIA Document 535, "Standard Form of General Conditions of Contract Between Owner and Design-Builder," Design-Build Institute of America, Washington, DC, First Edition 1998.

[4] AIA Document A191, "Standard Form of Agreement Between Owner and Design/Builder," The American Institute of Architects, Washington, DC, 1996.

Section V: Miscellaneous

Andrew P. Madden and Michael A. Petermann[1]

Preparation for and Collection of Facade Deficiencies at Large Complexes

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Reference: Madden, A. P. and Petermann, M. A., "**Preparation for and Collection of Facade Deficiencies at Large Complexes,**" *Building Facade Maintenance, Repair, and Inspection ASTM STP 1444,* J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA, 2004.

Abstract: Performing surveys at large buildings or a complex of buildings can be a daunting task. A large amount of data can accumulate quickly. Without an effective way to collect, store, sort, and analyze the data, the task can quickly grow unmanageable. Taking notes on paper and transcribing them in the office, onto a drawing or into a database, is not a problem for a small survey. However, in larger surveys, these tasks quickly become very time consuming and difficult to maintain. This paper examines electronic methods of recording data using handheld computers running a flat file database combined with a hierarchical database from a personal computer application to increase the efficiency of the data collection, recording, and analysis.

Keywords: palm pilot, database, microsoft access, query, survey, facade, analysis, reports

Introduction

Projects that involve surveying large buildings or a complex of buildings require unique solutions for recording survey information and sorting and analyzing that data after it has been recorded.

For a recent project involving the survey of nine buildings at a government complex in Albany, New York, an electronic system of recording data using handheld computers and laptops was used. The complex is comprised of nine buildings of varying sizes and layouts, from long lowrise buildings to a 44-story tower. All of the buildings were clad with marble panels. The total number of marble panels to be surveyed on the nine buildings was approximately 125 000.

Once it was decided to store all the recorded information in a database for easier sorting and analysis, it was a natural conclusion to use handheld computers with database software to record the survey data. This system enabled the data to be entered in to the database during the field survey without it having to be transcribed. This permitted the investigators to collect more data than traditional methods in the same amount of time and reduced the possibility for errors to be introduced in to the data.

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The choice for the database to store the data was Microsoft Access. This program was chosen for several reasons, including availability, ease of use, robust programming, and query writing features. For the handheld platform, the Palm Pilot IIIxe and Handspring Visors were chosen for their reliability and low cost. Once a handheld computer with the Palm OS (Palm Operating System) was selected for use, HanDBase 2.5 was selected as the database platform.

HanDBase is a scalable database program for the Palm OS. It is customizable and easy for the end user to manipulate in the field. It is lacking in desktop operation and the ability to perform advanced queries. However, since Microsoft Access was used for this function, HanDBase's lack of functionality in this area was not a problem.

Unfortunately HanDBase 2.5 could not convert between its native database system and Microsoft Access. A conversion program had to be written for this process. Access' robust programming features through Microsoft's Visual Basic made this task easy.

It was also decided that every panel that was to be surveyed would be numbered prior to surveying. This provided a unique way to identify every panel. The task of numbering this many panels in an AutoCAD drawing could be daunting, but a utility to help automate these tasks was written in-house, which significantly sped up the process.

Preparation for Field Work

Preparation for field work consisted of producing field booklets, customizing the HanDBase database for use with our project, producing the Microsoft Access database that would store all of the final survey information, and creating the utility program to import the information from the HanDBase database to Microsoft Access.

Field booklets were produced showing all of the elevations of each of the nine buildings. In total, nine books were produced—one for each building. The nine bound booklets were printed in tabloid format (11 in. x 17 in.) for ease of use while in the field. In selecting the size of the drawings for recording field observations, a trade off was made between portability and legibility. To allow the field booklets to remain a manageable size and legible, it was decided to split up each elevation over many pages. Each booklet was scaled so that the individual panel identification numbers were legible (Figure 1). Minimizing the size of the identification number permitted the field survey team to sketch cracks, spalls, etc. onto the various panels. In order to show all the panels of the nine buildings, 1 000 AutoCAD tabloid sheets were required.

Development of the database consisted of three phases. Phase one involved deciding what information was to be recorded and how it was going to be recorded. Phase two involved designing the database tables to hold the desired information. Phase three consisted of making adjustments to the database.

It was decided that three types of information were going to be recorded about the panels. The first type of information to be recorded included information specific to each panel. This information consisted of such items as building, elevation, panel number, and panel type. This information was stored in the first table, with each record being a separate panel (Figure 2). The second type of information concerned the specific

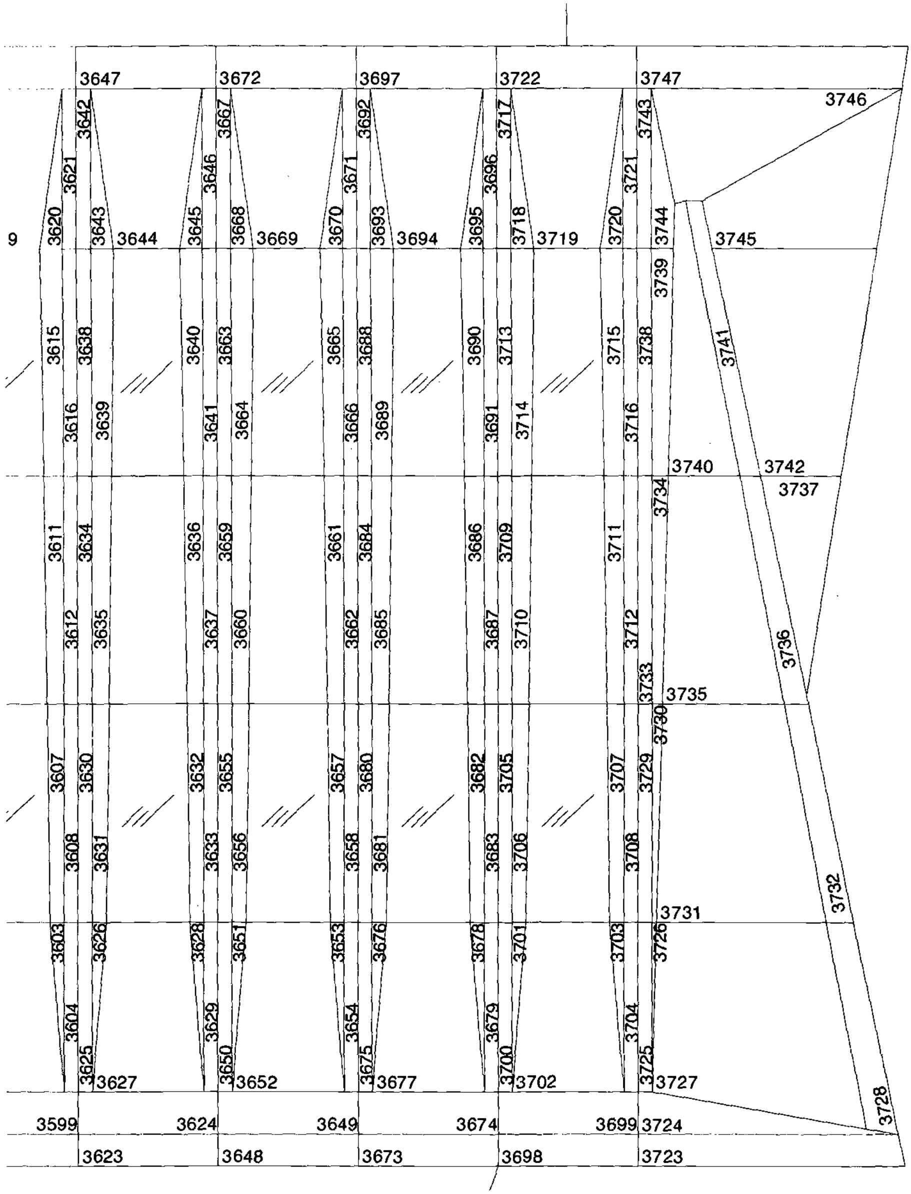
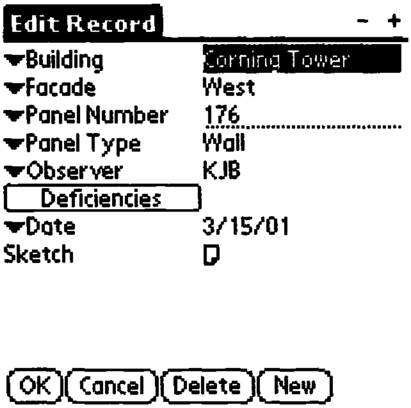


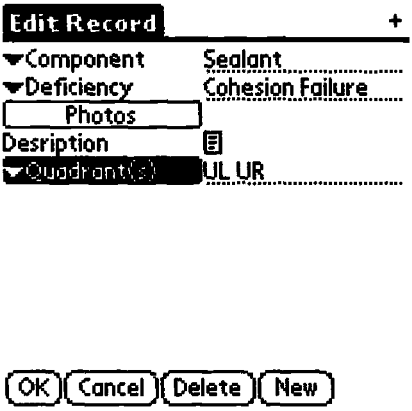
Figure 1 - *Partial Elevation: panels were automatically numbered using a custom written utility in AutoCAD*

deficiencies found on each panel such as deficiency type and a description. This information was stored in a separate table, linked to the first table by the panel number. This allowed an infinite number of deficiencies to be recorded for each panel (Figure 3). The third type of information to be recorded was information about photographs taken of the deficiencies. This information included the film roll identification number, frame number, and photographer. This information was stored in a third table, also allowing an infinite number of photographs to be recorded in the database for each deficiency.



▾Building	Corning Tower
▾Facade	West
▾Panel Number	176
▾Panel Type	Wall
▾Observer	KJB
▾Date	3/15/01
Sketch	

Figure 2 - *Typical data entry screen for panel information*



▾Component	Sealant
▾Deficiency	Cohesion Failure
Desription	
▾Quadrant(s)	UL UR

Figure 3 - *Typical data entry screen for deficiency information*

After the initial design of the database, it was tested in the office and improvements were suggested for ease of use. These suggestions included the addition of pop-up menus that contained predetermined information for ease of entry (Figure 4). The types of information stored in pop-up menus included information that the user would have to enter repetitively, such as the observer's initials, the building, the elevation, and information that should be entered uniformly, such as the deficiency type field. These types were predetermined to allow the deficiencies to be separated into categories such as spalls, cracks, and patches, etc. By placing these categories into pop-up menus, it assisted the users in entering the category type uniformly, allowing for more efficient queries during the analysis.

Finally, a small application was written to allow conversion of the HanDBase database into a Microsoft Access database. The conversion process had to read the output from HanDBase and convert it into Access as well as keep track of all the linked records that were being input from up to six users each day.

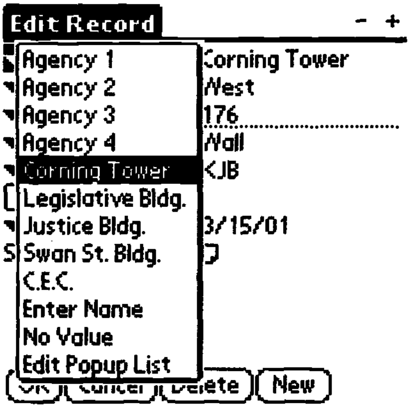


Figure 4 - *Typical example of pop up menus*

Field Work

Fieldwork began with a trial run to allow the observers to become familiar with the system and to make sure that all of the components functioned properly. This was also the time for any final debugging of the system.

After the trial run, a walkthrough with the field survey team was conducted to calibrate observations. Representative photographs of various deficiencies on each building facade were recorded digitally. The digital photographs were then loaded into a laptop and projected onto a screen for the field survey team to review and discuss in terms of categorizing the deficiency. This "calibration meeting" enabled the field survey team to agree upon how to categorize various deficiencies; cracks, cracks at anchors, spalls less than 1 sq.ft. (9 dm^2), spalls greater than 1 sq.ft. (9 dm^2), etc. In addition, the categories were modified to include additional deficiencies that were not originally included in the pop-up menu. The simplicity of the database permitted edits in the field so adding or subtracting categories was not a problem.

As the observers entered data into the system, they also recorded the deficiencies on the AutoCAD drawings contained in the field booklets. A different symbol was used for each different category of deficiency and the approximate location of the deficiency was recorded on the drawings.

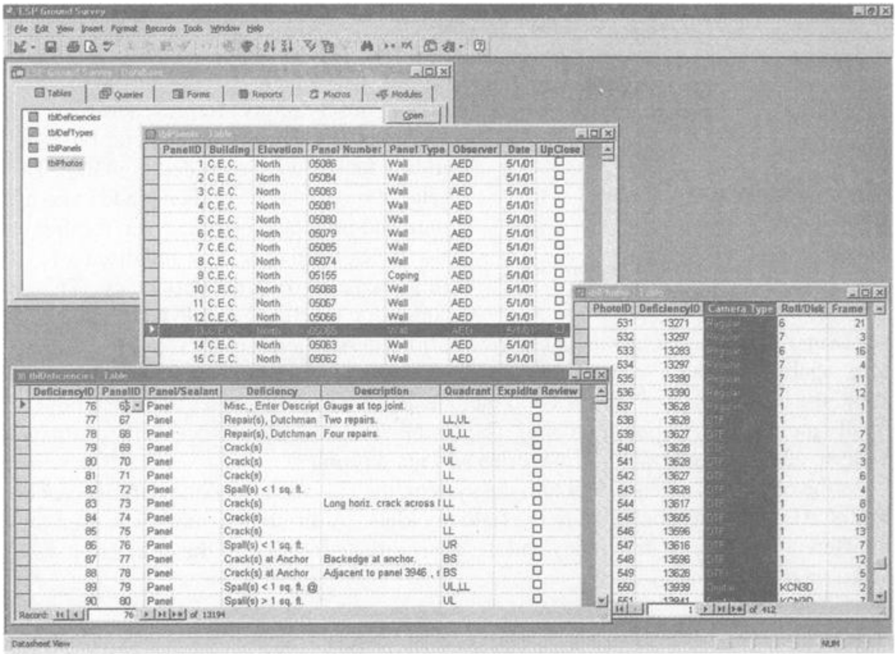
Review

Each day the data collected was synchronized with a laptop and then electronically mailed back to the office for review and conversion into the Microsoft Access database. At this point the data was checked for errors or missing data. Any errors that were found were relayed back to the project team in the field so they could send corrected data back to the office. Once the fieldwork was finished, the database was checked against the data recorded in the field booklets. Any panels found to have errors or to be missing were re-entered. One type of error was duplicate entries for a select panel. To protect against duplicate entries, each line of data entered received a unique number in the database. When duplicate entries were made, their unique numbers were compared and subsequently flagged during queries.

During the survey, some of the electronic data was lost for various reasons that included malfunctions of the handheld computers (loss of battery power erased the data), operator error (synchronization with a laptop erased previous data), and damage (the units were sometimes dropped to the ground). Lost data was re-entered from the information recorded in the field booklets.

Analysis

Once the data was converted and entered into the Microsoft Access database, it was now easily accessible for retrieval and analysis (Figure 5). Microsoft Access includes many tools for easily designing queries for analysis of data. One of the most useful tools is the Crosstab Query Wizard. This tool helps format queries to calculate data and display it in tabular form. The tabular form expressed depends upon the data desired, such as the number of panels with various deficiencies expressed per building per elevation.



PanelID	Building	Elevation	Panel Number	Panel Type	Observer	Date	UpClose
1	C.E.C.	North	05086	Wall	AED	5/1/01	☐
2	C.E.C.	North	05084	Wall	AED	5/1/01	☐
3	C.E.C.	North	05083	Wall	AED	5/1/01	☐
4	C.E.C.	North	05081	Wall	AED	5/1/01	☐
5	C.E.C.	North	05080	Wall	AED	5/1/01	☐
6	C.E.C.	North	05079	Wall	AED	5/1/01	☐
7	C.E.C.	North	05085	Wall	AED	5/1/01	☐
8	C.E.C.	North	05074	Wall	AED	5/1/01	☐
9	C.E.C.	North	05155	Coping	AED	5/1/01	☐
10	C.E.C.	North	05068	Wall	AED	5/1/01	☐
11	C.E.C.	North	05067	Wall	AED	5/1/01	☐
12	C.E.C.	North	05066	Wall	AED	5/1/01	☐
13	C.E.C.	North	05065	Wall	AED	5/1/01	☐
14	C.E.C.	North	05063	Wall	AED	5/1/01	☐
15	C.E.C.	North	05062	Wall	AED	5/1/01	☐

tblDeficiencies : Table

DeficiencyID	PanelID	Panel/Sealant	Deficiency	Description	Quadrant	Expidite Review
76	65	Panel	Misc., Enter Descript	Gauge at top joint.		☐
77	67	Panel	Repair(s), Dutchman	Two repairs.	LL,UL	☐
78	68	Panel	Repair(s), Dutchman	Four repairs.	UL,LL	☐
79	69	Panel	Crack(s)		UL	☐
80	70	Panel	Crack(s)		UL	☐
81	71	Panel	Crack(s)		LL	☐
82	72	Panel	Spall(s) < 1 sq. ft.		LL	☐
83	73	Panel	Crack(s)	Long horiz. crack across I	LL	☐
84	74	Panel	Crack(s)		LL	☐
85	75	Panel	Crack(s)		LL	☐
86	76	Panel	Spall(s) < 1 sq. ft.		UR	☐
87	77	Panel	Crack(s) at Anchor	Backedge at anchor.	BS	☐
88	78	Panel	Crack(s) at Anchor	Adjacent to panel 3946 ,	BS	☐
89	79	Panel	Spall(s) < 1 sq. ft. @		UL,LL	☐
90	80	Panel	Spall(s) > 1 sq. ft.		UL	☐

Record: 76 of 13194

PhotoID	DeficiencyID	Camera Type	Roll/Disk	Frame
531	13271	Regular	6	21
532	13297	Regular	7	3
533	13283	Regular	6	16
534	13297	Regular	7	4
535	13390	Regular	7	11
536	13390	Regular	7	12
537	13628	Regular	1	1
538	13628	DTF	1	1
539	13627	DTF	1	7
540	13628	DTF	1	2
541	13628	DTF	1	3
542	13627	DTF	1	6
543	13628	DTF	1	4
544	13617	DTF	1	8
545	13605	DTF	1	10
546	13596	DTF	1	13
547	13616	DTF	1	7
548	13596	DTF	1	12
549	13628	DTF	1	5
550	13939	Digital	KCN3D	2

Record: 1 of 412

Figure 5 - *Example of Microsoft Access database tables.*

For reporting purposes, several queries were written depending upon the information desired. Each query is essentially a question about the data collected. Some of the questions posed were as follows:

- How many deficiencies were recorded and on how many panels?

- How many deficiencies occurred on each facade?
- How many deficiencies occurred on the top half of the building?
- How many deficiencies occurred at the upper left quadrant of a panel?
- How many deficiencies were observed from a scaffold as compared to distant observations through binoculars?

The answers to these queries were used to assist the investigators in establishing various data points such as an order of magnitude for each type of failure, the potential of weather-related failure depending upon facade or height exposure, the potential of a construction defects causing repetitive failures at distinct locations, and a multiplication factor for facade that were not observed 100% close-up from a scaffold. Depending upon the individual queries and buildings, the data was then presented in tabular form suitable for inclusion in a report to the client (Table 1).

Table 1 – *Sample table showing deficiencies observed and quantities*

Visual Deficiency	Amount	Number of Panels Affected	Percentage of Panels Affected[1]
Spall(s) less than 1 sq.ft. (9 dm^2)	1007	920	2.76%
Crack(s)	466	445	1.33%
Spall(s) less than 1 sq.ft. at an anchor	280	251	less than 1%
Cracks at Anchors	214	196	less than 1%
Erosion, Sugaring	209	206	less than 1%
Repair(s), Mortared Patch	133	127	less than 1%
Lean Out Top	100	97	less than 1%
Spall Repair	69	68	less than 1%
Lean Out Bottom	16	16	less than 1%
Repairs, Dutchman	27	26	less than 1%
Spall(s) greater than 1 sq.ft. at an anchor	5	3	less than 1%
Spall(s) greater than 1 sq.ft.	1	1	less than 1%

[1] Percentages are based upon 33,000 panels

Conclusion

This system enabled the investigators to keep track of and analyze a large amount of data with relatively little effort. The use of handheld systems reduced the time it took to collect and transcribe the enormous amount of data that was collected. The database program also allowed quicker sorting and analysis of the data, and enhanced the investigators' ability to ask various questions of the data when it came time to report survey findings to the client. In fact, the client also posed queries of the database.

Now that the database exists for this particular complex, the client will endeavor to maintain the database and build upon it during future periodic inspections. The combination of the database and AutoCAD drawings provides a coordinate system for each panel in the complex. Future observations of any panel can be compared to the database to establish if the deficiency is new, already exists, or has worsened. In addition, future testing data of select panels can also be added to the database for future reference and tracking.

New developments in both hardware and software should make the process of using handheld devices in the field even easier. The latest version of HanDBase supports direct exporting to Microsoft Access. On platforms other than Palm OS machines, such as Windows CE, there are many options. Windows CE machines running Pocket PC 2002 allow direct conversion of Microsoft Access databases. These databases can be run with commercial front ends or a custom one can be written using Microsoft Visual Tools for Windows CE.

Amy Peevey Brom [1]

Guidelines for Inspection of Natural Stone Building Facades

REFERENCE: Brom, A. P., "***Guidelines for Inspection of Natural Stone Building Facades,***" *Building Facade Maintenance, Repair, and Inspection, ASTM STP ASTM STP 1444*, J. L. Erdly and T. A. Schwartz, Ed(s)., ASTM International, West Conshohocken, PA, 2004.

ABSTRACT: Existing building facade inspection ordinances are inconsistent and leave much of the investigation methodology to the discretion of the professional performing the facade inspection. Guidelines for facade inspections that establish the critical inspection elements and special considerations for particular facade materials would ensure that the professional performing the inspection will provide all the key components of the facade inspection while allowing that professional to evaluate the most suitable approach for each inspection component of a particular building.

For natural stone facade inspections, the key elements of the inspection process include: precautionary measures assessment; building facade history review; visual surveys; up-close inspections and field testing; inspection openings, probing, or interior surveys; and determining the need for laboratory analysis and calculations. Special considerations that need to be accounted for during the stone facade inspection are the classification of the stone; material properties and environmental effects on stone; and problems relating to the design, construction, maintenance and repair of exterior stone cladding.

KEYWORDS: facade inspection, field testing, inspection opening, precautionary measures, stone facade, visual survey

Introduction

All materials, whether natural or man-made, deteriorate over time with exposure to environmental elements. The facades of our nation's buildings are no exception. Facades not only provide a building's exterior aesthetics but the first line of the building's defense from the exterior environmental elements. Exterior walls of buildings must sustain the effects of wind, rain, ultraviolet radiation, extreme temperatures, and pollution over the life of the building. Under these environmental conditions a building's exterior deteriorates, and deteriorated facade materials can pose a potential threat to the safety of the public and adjacent structures.

Because of the deterioration of our nation's building facades and subsequent facade failures that have resulted in injury, loss of life, and property damage, some local building officials have instituted local facade ordinances that require the periodic inspection of facades. These local facade inspection ordinances are not consistent from city to city and contain different requirements for facade inspections. Additionally, because it is not the responsibility of the local building official to dictate the methodology of building-facade inspections, the professional employed to perform the inspections

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determines the inspection approach and is responsible for determining the unique issues pertinent to each facade material type. While a professional may meet the general requirements of the local building-facade inspection ordinance for performing inspections, he may not have expertise in the inspection of each type of facade material or system requiring inspection. Because of the inconsistency of the existing facade inspection ordinances and variety of issues to be considered for each type of facade material or system, there is a need for the development of inspection guidelines so that professionals can be provided with the key inspection elements and considerations that are relevant to different facade types.

The purpose of this paper is to provide professionals performing building facade inspections with guidelines for the inspection of natural stone facades. These guidelines are to serve as a supplement, not a replacement, to the requirements of local building facade inspection ordinances and regulating codes. The following provides the elements of facade inspection that are imperative for an adequate natural stone facade inspection as well as the considerations unique to natural stone facade systems.

Inspection Guidelines for Natural Stone Cladding

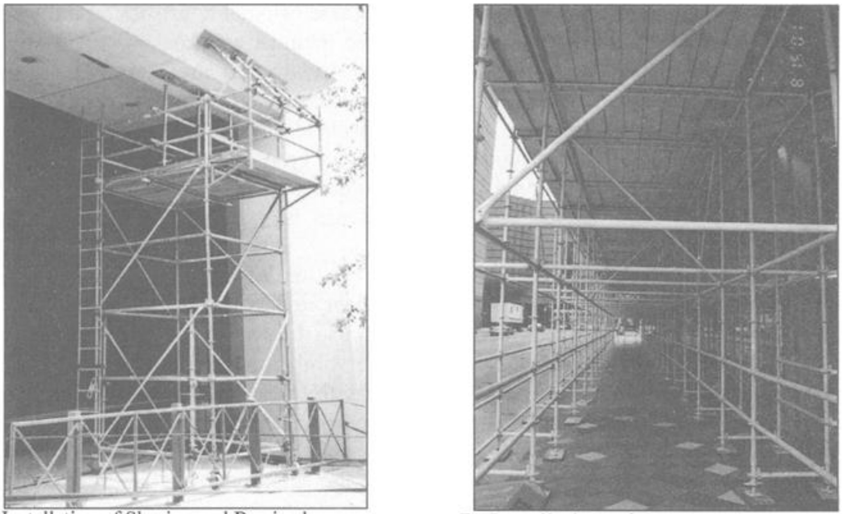
There are many elements to a facade inspection that should be addressed for all facade materials. However, it is not the purpose of this paper to provide the minimum inspection requirements for all facade inspections. Instead, the focus of this paper will be on establishing the key facade inspection elements and considerations for natural stone facades. The following facade inspection elements serve as guidelines and are critical to the adequate inspection of natural stone facades. Therefore, these elements should be considered in addition to the facade inspection requirements of the local governing authorities'.

Precautionary Measures Assessment

Because of the highly variable properties of natural stone and the slow evolution of stone cladding standardization [1], many stone facades may experience unstable conditions that can pose immediate threats to the public and adjacent structures as well as to the professionals performing the facade inspection. Consequently, a professional should conduct a preliminary assessment to determine if initiating a precautionary measures plan, prior to the beginning of the facade inspection, is required.

Precautionary measures are efforts to protect the public and adjacent property from the potential of falling facade elements and debris. Protection can be provided in a number of ways, including stone panel removal, stabilization of existing unstable portions of the facade, installation of screening at unstable exterior wall portions, or installation of overhead protection or barricades, as shown in Figure 1. The precautionary measures protection is temporary and is not intended to serve as a permanent means of facade repair or protection. Therefore, long-term repairs to those facade portions deemed to be unstable will be required prior to the removal of the precautionary measures protection.

The inspection process can induce additional loads on the facade. Because these loads may further exacerbate the existing deteriorated or unstable facade conditions, the precautionary measures assessment should be performed prior to the overall facade inspection. The assessment to determine the need to initiate precautionary measures should include a brief review of previous inspection reports and the building's service history. Additionally, a brief visual survey of the entire building facade with the use of high-powered optics should be performed to attempt to identify panel locations that may be unstable and may pose a falling hazard. Once any required precautionary measures are in place, the actual evaluation of the facade can commence.



A. Installation of Shoring and Barricades

B. Installation of Overhead Protection

C. Panel Strapping

Figure 1 - *Precautionary measures installed as a means of temporary protection from the potential falling of unstable stone cladding portions. Examples shown include (A) installation of shoring and barricades, (B) installation of overhead protection and (C) panel strapping stabilization.*

Building Facade History Review

If available, all documentation that provides the history of the facade should be reviewed as a part of the facade inspection. Architectural elevations and documentation of the building's exterior wall system can be utilized in the development of inspection documents. Additionally, reviewing the history of the building facade can be useful in evaluating the overall condition of the stone cladding system, the evaluation of potential trends, and identification of facade problems.

All of the available building facade documentation should be evaluated to determine the overall history of the facade. The review of available original construction drawings, specifications, and relevant shop drawings should be included as a part of the building documentation study. Review of the original construction documents will provide information regarding the use of proper detailing and the specification of proper facade materials. The original shop drawings for the stone cladding can provide details of the stone panel support and stone connections that may not be provided in the architectural and structural drawings. These original construction documents can provide information about the existing construction of the facade that can be verified during the facade observations.

In addition, the relevant codes and standards at the time of original construction can be a useful part of the original construction document review. Maintenance and repair records as well as previous facade reports should also be reviewed. Special attention should be paid to ongoing water infiltration, facade problems, or poor repair designs. Other building facade information can be obtained through interviews with building maintenance personnel, previous contractors performing the maintenance and repair work, and previous exterior wall consultants.

From the review of the building documentation, the actual as-built conditions of the facade should be compared to the original construction documents and, where applicable, the repair drawings. Furthermore, the information obtained from the documentation review can be utilized to determine locations of the stone facade that may require more detailed observations.

Visual Survey

In order to determine the general condition of the facade, a visual survey of all of the facade elements must be performed. This survey should be conducted from the ground level and accessible adjacent structures with the use of high-powered optics. Some types of natural stone distress, such as panel displacement or bowing as shown in Figure 2, are more evident under tangential daylight conditions, yet other types of distress are easily observed in daylight or direct sunlight. Therefore, each facade elevation should be

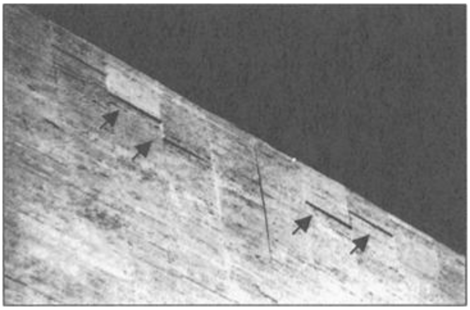
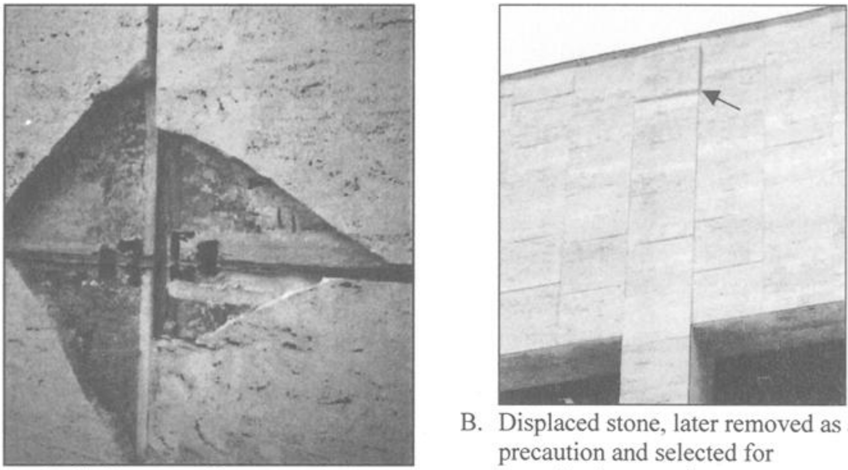


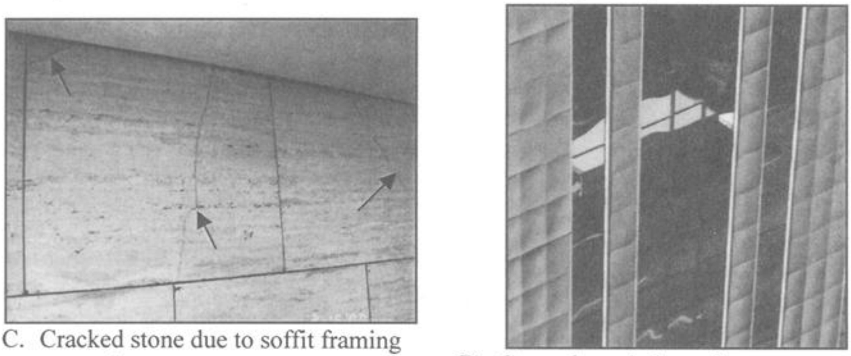
Figure 2 - *Observations at stone facade locations where displacement is visible under tangential daylight when observed from an angle to the building face.*

inspected in daylight, sunlight, and tangential daylight [2]. Additionally, each elevation should be observed from different angles and checked for panel distress and displacements or bowing from each vantage point. Consequently, a single elevation of exterior stone cladding may be inspected several times during the visual survey phase of the facade inspection.



A. Spalled stone at ferrous embeds

B. Displaced stone, later removed as a precaution and selected for investigative opening



C. Cracked stone due to soffit framing expansion

D. Cupped and bowed stone panels

Figure 3 - *Examples of typical stone distress including (A) spalled travertine stone panel corners at ferrous chairs within concrete panel, (B) displaced travertine stone panels as a result of corrosion of backup, (C) cracked travertine stone panel due to expansion of soffit steel framing, and (D) bowing and cupping of marble stone panels.*

During the visual survey, the exterior wall cladding is to be inspected for all types of stone facade distress, including evidence of water infiltration. Examples of stone distress are shown in Figure 3 and include deterioration or erosion of the stone surface, delamination or spalling of stone portions, cracking of panels, displacement of panels, and bowing or cupping of panels. Examples of water infiltration that may be observed from the exterior include: rust staining and efflorescence.

A means of recording the various types of distress observed within the stone cladding is to be utilized while conducting the visual survey. Methods of recording stone distress and as-built conditions, as shown in Figure 4, include field observations recorded on elevation drawings, data collection using hand-held computers in conjunction with graphics programs like AutoCAD [2], or photographic means like perspective-corrected photography. These records will be utilized for the documentation of existing conditions and analyzed to determine the existence of failure trends within the stone facade and potential locations for up-close observations.

Up-Close Inspections and Field Testing

While the overall condition of a facade can be generally determined from a visual survey with the use of high-powered optics, the visual survey should be supplemented with an up-close inspection and field testing. An up-close inspection is a "hands-on" investigation of the stone facade that is performed at an exterior wall portion accessed from suspended scaffolding or aerial lifts, by rappelling, or other means of exterior wall access. The up-close inspection provides close-range observations of the stone facade where additional stone distress or patterns may be observed that were not visible in the visual inspection due to distance, obstructions or other causes.

During the up-close inspections a more detailed "hands-on" inspection, which incorporates field testing of the stone cladding, is performed. Field testing methods typically performed during the up-close inspections include nondestructive testing, such as sounding or impact testing; metal detection surveys; and moisture detection surveys. These testing methods provide additional information about the soundness of the stone panel material, the soundness and locations of the stone panel connections, and areas of possible water infiltration.

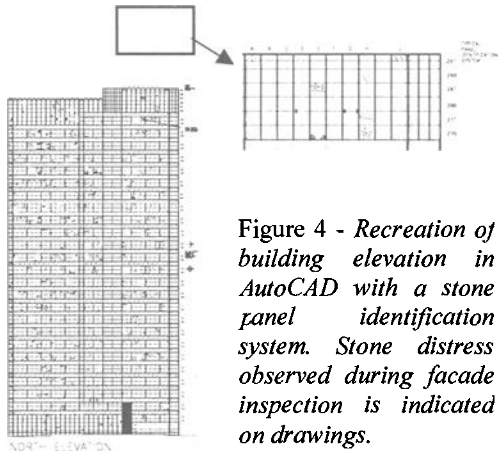


Figure 4 - *Recreation of building elevation in AutoCAD with a stone panel identification system. Stone distress observed during facade inspection is indicated on drawings.*

Inspection Openings, Probing, or Interior Observations

Observations of the exterior wall natural stone cladding alone do not provide sufficient information to assess its general conditions. The backup conditions of the stone cladding must also be observed to ensure that the conditions of the stone panel connections and the backup material(s) are acceptable. As Figure 5 illustrates, spalls may be present on the interior side of the stone panels or the backup material may be severely deteriorated. These conditions are not visible during the visual survey or up-close survey; therefore, inspection openings, probing, or interior observations are necessary to confirm the condition of the cladding backup and connections.

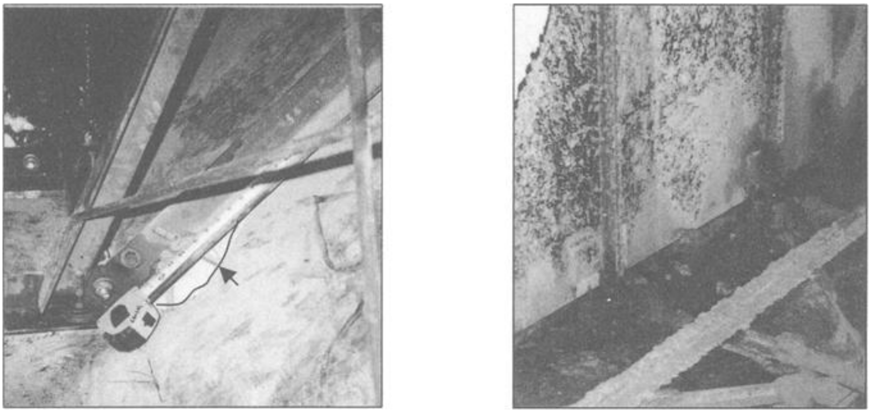
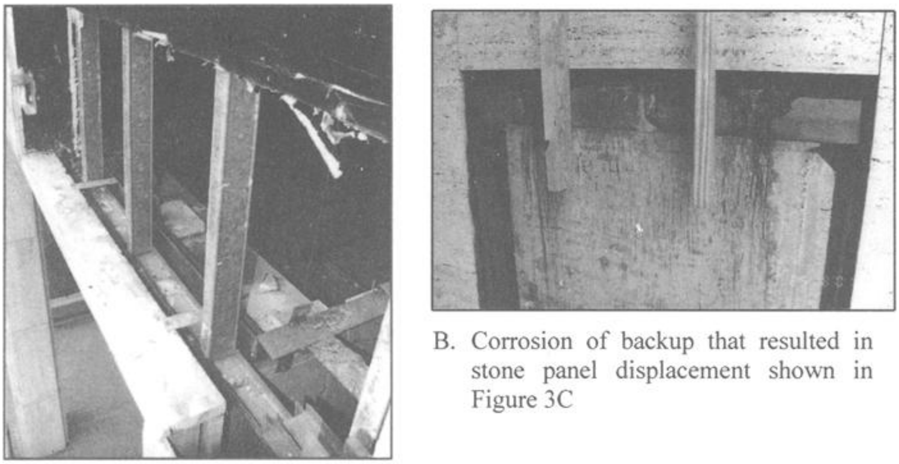


Figure 5 - *Spall on interior side of stone panel at panel connection and corrosion of backup observed during survey of the interior side of the stone panels.*



B. Corrosion of backup that resulted in stone panel displacement shown in Figure 3C

A. Structural steel and stud wall backup

Figure 6 - *Inspection openings for observations of backup and connection conditions.*

Inspection openings, as seen in Figure 6, are locations where a portion of the exterior cladding is removed revealing the backup system. Probe openings are small holes through the cladding into which a visual aid device, such as a borescope, is inserted so that the backup conditions can be observed without the removal of a portion of the cladding, as shown in Figure 7. Probing is sometimes used in conjunction with, or in lieu of, inspection openings. Probing can be used to supplement the findings of inspection openings or determine potential inspection opening locations. Additionally, probe openings may be used when the excavation of an exterior wall portion is not feasible due to access or may seem unnecessary. However, probe openings provide limited visibility and inspection openings may be required to determine the actual backup conditions. Some panel systems, such as truss-supported thin-stone cladding systems as shown in Figure 8, can be visually inspected from the interior side of the cladding system. Visual interior inspection of these systems can be performed and, consequently, inspection openings or probing at these accessible locations may not be required.

The locations of exterior wall inspection openings, probe openings, or interior surveys are generally determined from the results of the visual survey, up-close inspection, and field testing. During the inspection opening, probing, or interior survey observations, the as-built conditions should be recorded and studied to determine if they conform to the available construction documents. Additionally, the types and conditions of the stone panel connections and back-up system should be observed and documented.

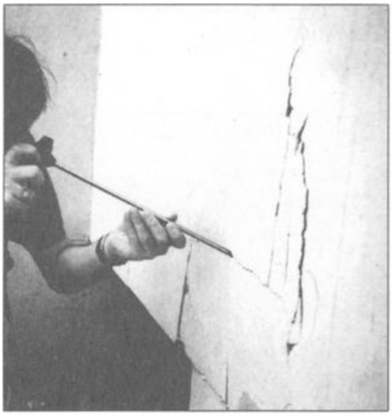


Figure 7 - *Use of a borescope at a distressed limestone facade panel*

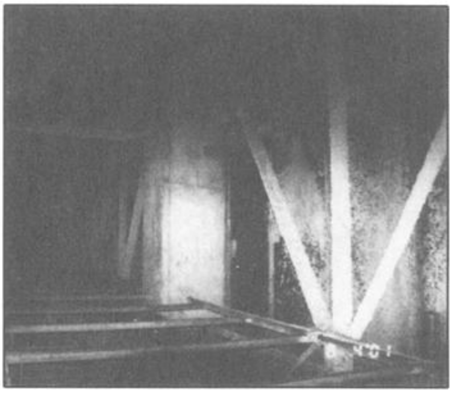


Figure 8 - *Interior observations of truss-supported thin-cladding system*

Laboratory Analysis

During the up-close inspections, inspection opening, or interior survey observations, samples of the stone cladding system can be collected for laboratory analysis. Laboratory analysis of the stone panels will provide information about the effects of the environment in addition to the characteristics and composition of the facade materials. The physical properties of the stone cladding materials can be determined from standard laboratory testing for compressive and flexural strength, modulus of rupture, and permeability. Material analysis, through methods such as petrographic microscopy, X-ray diffractometry, chemical analysis, or infrared spectroscopy of the materials, can provide the material composition, [and possibly the] cause(s) of distress and anticipated durability [3]. Furthermore, thermal and moisture cyclical testing and accelerated weathering testing can provide more accurate information about expected long-term durability of the material.

Laboratory analysis of the natural stone or backup materials is not always necessary. However, an assessment of the need for laboratory testing should always be included as a part of the stone facade inspection. Also, laboratory analysis can provide useful information regarding the types of repairs, repair materials, and cleaning methods that should be used in the repair design of the stone cladding facade.

The types of distress observed within a natural stone facade can be utilized to determine the need for laboratory analysis. Bowing of stone panels, deteriorated mortar spot connections, stone panel crack distress, deteriorated stone surface, and the presence of delaminations within the stone panels are all types of distress that may be indicative of strength or material problems of the stone cladding system. Laboratory analysis can detect the presence of chlorides within the stone or backup, presence of expansive materials like gypsum within the facade system, and the loss of stone strength due to environmental exposure. Analysis may also provide other useful information to determine possible stone facade failure mechanisms and to assess the condition of the stone facade.

Calculations

From the physical laboratory testing of the stone panels, calculations can be performed to evaluate the existing support conditions and joint locations of the stone facade system. In some instances, the stone panels were originally inadequately designed or constructed to resist the wind and self-weight loads imposed on them. In other cases, the exposure of the stone panels to the environmental elements has resulted in a loss of stone strength. To assess the existing support conditions of the stone panels, the strength results from the laboratory testing can be utilized, along with the corresponding factors of safety, to perform calculations evaluating the existing placement and type of stone panel supports. The laboratory results can also be utilized to evaluate the placement of the existing soft joints to permit structural movement of the backup system or thermal movement of the stone panels.

As with the laboratory testing, calculations evaluating the existing support conditions and joint placement of the stone facade are not always needed. However, the need for calculations should be evaluated as a part of the stone facade inspection. Types of distress that may indicate a stone panel strength or joint placement problem include spalling at connection locations, cracking of the stone panels, and spalling at panel-to-panel joints.

Stone Cladding Considerations

Because there are many different types of facades, there cannot be a single methodology for performing all facade inspections. Special considerations for the type of material or facade system being investigated must be accounted for during the inspection.

It is the responsibility of the professional performing the inspection to determine the inspection method that will address the unique considerations for the facade being examined. Just as the previous section provided guidelines for the minimum inspection elements, addressing the following issues should provide the special factors to be incorporated into a natural stone facade inspection.

Classification of Stone

There are many different types of natural stone facades. The most common types of stone historically used in the construction of exterior wall cladding are: granite, sandstone, limestone, travertine, slate, and marble. Typically, granite, travertine, slate, and marble are thin veneer stone cladding systems with stone thicknesses less than 5 cm thick, while sandstone and limestone cladding are generally 6 cm or greater in thickness. All stone is formed from geological processes that result in three stone classifications: igneous rocks, such as granite; sedimentary rocks, such as sandstone and limestone; and metamorphic rocks such as slate and marble.

The classification of the stone cladding being evaluated is important. Because a natural process creates stone, the properties of stone can be highly variable. This variability must be taken into account as a part of the exterior wall cladding design and, consequently, safety factors are incorporated into the design. These safety factors are based on the classification of the stone and sometimes on the variability of the laboratory determined stone strength [4]. Consequently, these safety factors should be utilized if it is determined that calculations are to be performed to evaluate the existing stone panel support as a part of the facade inspection.

Material Properties and Environmental Effects on Stone

As mentioned previously, because stone is a natural material, it has highly variable properties. Natural stone contains veins, voids, deposits, and other anomalies within it. In addition, the quarrying and finishing process can create micro-cracks within the stone. Stone anomalies and the presence of micro-cracks can affect the strength and durability of the stone. An abundance or concentration of anomalies, as shown in Figure 9, within a stone panel can create a weak plane or zone within the panel. The presence of anomalies can also make it difficult to inspect the stone cladding because it is sometimes hard to distinguish between an anomaly and distress. However, in many cases, cracks or distress form within these anomalies, as shown in Figure 10.

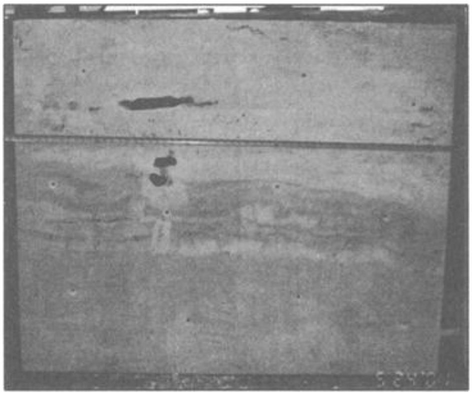


Figure 9 - *Excessive veining and voids within a travertine stone panel.*

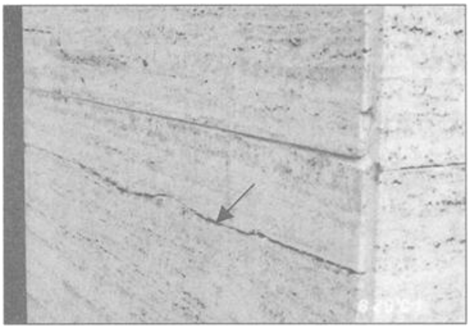


Figure 10 - *Crack distress observed within natural vein of stone.*

Stone anomalies, stone composition and structure affect the permeability of a stone. Stone permeability can vary from class to class or even within the same class of stone. The higher the permeability is, the more susceptible the stone will be to environmental factors such as water. Water is the most harmful of all environmental agents [3], and is responsible for a large portion of stone distress. Corrosion of supports or connections, efflorescence, biological growth, deterioration of joint or connection materials, and stone spalling distress related to freeze-thaw and acid rain are all types of distress that can result when natural stone facades come in contact with water. Also, the composition of some stones or other facade materials may contain chlorides or gypsum, which can be detrimental to the stone, stone supports, and stone cladding backup when water is introduced into the exterior wall system.

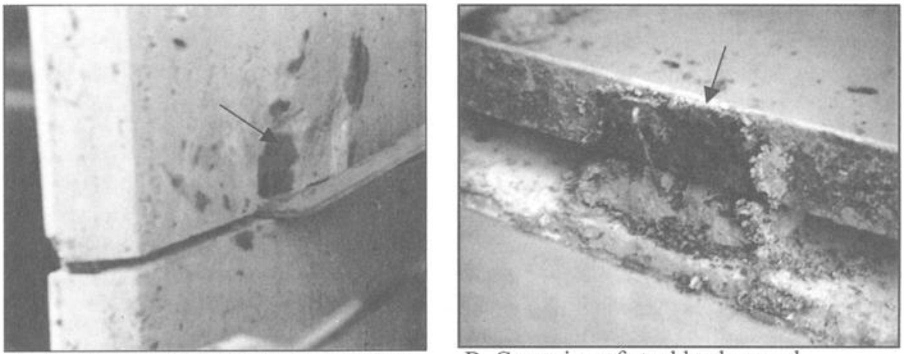
All of the material properties and environmental distress should be considered during the inspection of a natural stone facade. Laboratory testing can be performed to determine the material properties and environmental elements that may be causing stone distress. Finally, observation of the existing means of water evacuation for the exterior stone cladding system should be evaluated as a part of the facade inspection because water infiltration can be detrimental to a natural stone facade.

Design and Construction

Figure 11 - *Spalled limestone panel corners at ferrous washer*

Poor design, detailing, and construction practices during original construction of the natural stone facade may cause future distress. Inadequate stone connections, poor selection of facade materials, lack of adequate wall joints to allow for movement of the exterior wall system, and lack of adequate exterior wall drainage are some of the most prevalent original construction problems.

As previously stated, while the stone facade design and construction techniques have been updated over time, the practice and installation of stone facades did not follow as quickly [1]. There have been instances where the result has been inadequate stone cladding connections that are in non-conformance with the applicable standards. Stone panel connections must be positive mechanical connections that equally distribute the loads on the stone panels in order to provide adequate panel support. However, many of the inadequate stone panel connections that have been used do not satisfy these requirements and are not able to withstand the design wind loads and self-weight loads.



A. Spalled epoxy-filled pin connection B. Corrosion of steel backup and embrittled wire-tie connection

Figure 12 - *Failed epoxy and wire-tie stone panel connections.*

These deficiencies result in stone distress or failure. Consequently, the capacity of the stone connections should be verified through calculations as a part of the facade inspection. Additionally, the stone facade inspection should incorporate a detailed inspection for the following types of potentially problematic stone panel connections.

1. Ferrous or dissimilar metals used for stone connections, which corrode and deteriorate in the presence of water. This condition may lead to spalling of stone or failure of stone connection, as shown in Figure 11.
2. Non-durable connection materials that are not suitable for severe exterior exposure like interior-grade epoxy and non-copper or non-stainless steel wire-tie connections. These materials can deteriorate over time when exposed to exterior environmental conditions or may melt in the event of a fire. Figure 12 illustrates failed epoxy and wire-tie stone panel connection locations.
3. Gypsum based mortar connections or joints that expand in the presence of water, as previously referenced in Figure 12.
4. "Blind" stone panel connections, such as wire-ties embedded into mortar spots. There is no means of verifying adequate anchorage of the stone panels for these types of connections. Consequently unstable panel conditions can result as a

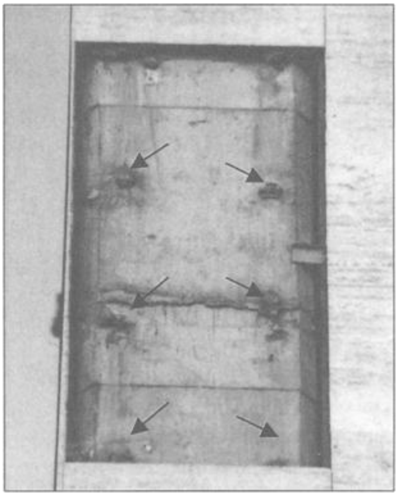


Figure 13 - *Location of panel collapse where blind mortar spot with wire-tie connections deteriorated and failed.*

part of initial improper installation. Figure 13 illustrates the use of a blind mortar spot with wire-tie connections where both the wire-tie and gypsum based mortar have deteriorated and failed resulting in the collapse of the stone panels.

Figure 14 - *Distress in stacked travertine panel at shim support*

Another form of inadequate stone connection is the poor placement of stone connections. The stone panel must resist the imposed wind loads and stone gravity loads between the connection locations so that the loads can be transferred back through the connections to the structure. Consequently, the placement of gravity connections for stacked panels and lateral connections for individual panels is critical. Failure of the stone will occur if the stone connections are placed so that the stone cannot adequately support the loads. Also, the lack of adequate connection or shim elements to transfer the stone loads can result in failure. Figure 14 illustrates distress within a stone panel at a shim support. The shim is too short to adequately transfer the gravity load from one stacked panel to the next one below. This inadequacy results in excessive, concentrated stresses within the stone resulting in a failure at the shim location. As a part of the exterior stone cladding inspection, calculations can be performed to determine if the placement of the existing stone connections is adequate.

In addition to inadequate connection details and materials, the lack of proper joint placement to accommodate exterior wall movements can result in failure of the natural stone facade. The exterior wall design must account for the thermal movements of the stone panels as well as the deflection, creep, and thermal movements of the structure. The restriction of stone panel or building movement through the lack of proper joint placement and joint width can result in the displacement, spalling, and cracking of stone panels. In addition, the presence of hard materials within these “soft” joints can create concentrated stresses in the stone facade. As a part of the facade inspection, calculations should be performed to verify that the stone facade has adequate joint placement.

Poor drainage of the natural stone facade can also contribute to future cladding distress. If the cladding does not drain infiltrating water from the wall system, the water can lead to the distress previously discussed. Not only should the existing waterproofing system be evaluated as a part of the facade inspection, but a survey of the building interior to determine the leak history or leak patterns is recommended. Interviews with building maintenance personnel can provide information regarding the leak history of the building. Additionally, field water testing can be performed to determine the amount or locations of water infiltration into the wall system.

Maintenance and Repair

A key factor in the longevity of a natural stone facade is maintenance. Consequently, the lack of maintenance is a major factor in the accelerated deterioration of a building facade. The most important maintenance item for a natural stone facade is the water drainage system. In order to prevent the infiltration of water and subsequent distress of the stone cladding system, the waterproofing of the stone facade must be maintained. Measures should be taken to avoid the effects of water related distress in natural stone cladding.

Figure 15 - *Ferrous through-stone repair anchor*

Repairs may be performed as a part of the building's exterior maintenance. However, sometimes these "repairs", if unsuitable, may create more problems for the natural stone cladding. Allowing the passage of water and vapor from the natural stone facade is important. However, some "repairs" include the use of vapor impermeable materials that can potentially lead to stone distress. For instance, the use of vapor-impermeable coatings or hard, vapor-impermeable mortars forces moisture and salts within the exterior wall system to become trapped within or pass through the stone panels [5]. The salt then crystallizes within or on the surface of the panels, resulting in distress. In some instances, ferrous repair materials like the supplemental through-stone anchors shown in Figure 15 may have been used. As the ferrous anchors corrode and expand, they create additional distress in the stone panels they were intended to repair.

The aesthetics of a building's exterior and the function of the facade as the building's defense from the exterior elements are preserved through a proper, ongoing facade maintenance program. Records of all of the maintenance and repairs performed on the facade should be maintained. These records can be utilized to evaluate the cladding system in the future. They provide a gauge for the relative deterioration rate of the natural stone cladding and a history of previous problems associated with the facade.

The maintenance and repair of a building's natural stone facade is typically the responsibility of the building owner or manager. Much of the facade maintenance and repair can be conducted as a part of the regular maintenance program for the building. A detailed guide that can be utilized by building owners and managers for the maintenance and repair of dimension stone facades is ASTM 1496 "Standard Guide for Assessment and Maintenance of Exterior Dimension Stone Masonry Walls and Facades". This standard provides elementary knowledge of dimension stone facades and guidelines for conducting ongoing facade inspection and maintenance. However, this standard and other existing references do not address all natural stone facade systems and all types of stone distress or conditions. The building owner or manager should consult a professional experienced in the inspection or repair of stone facades for other types of cladding systems and facade conditions that are not readily repaired as a part of the regular building maintenance.

Conclusion

This paper has provided general guidelines and special considerations for the inspection of natural stone facades. The intent is for the professional to supplement the local ordinance or regulations pertaining to facade inspection with this information so that the critical elements and unique considerations for the inspection of natural stone facades are included as a part of the facade inspection process. However, these guidelines and considerations are general and do not account for all conditions of or types of distress in natural stone facades. Additional special considerations or unique occurrences for a particular building's natural stone facade may need to be included as a part of the facade inspection.

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Douglas R. Stieve, Alicia E. Díaz de León, and Michael J. Drerup. [1]

Assessing the Apparent Watertight Integrity of Building Facades

Reference: Stieve, D. R., Díaz de León, A. E., and Drerup, M. J., **"Assessing the Apparent Watertight Integrity of Building Facades"** *Building Facade Maintenance, Repair, and Inspection, ASTM STP 1444,* J. L. Erdly and T. A. Schwartz, Eds., ASTM International, West Conshohocken, PA 2004.

Abstract: Many city mandated facade inspections require the inspection team to evaluate whether the building facade is vulnerable to water leakage. Due to the large number of facades that require inspection, the professional charged with this essential task must be trained to determine quickly if a facade is vulnerable to water infiltration. If there is evidence that the watertight integrity of the building facade is compromised, further investigation should be performed to determine the source(s) of water leakage.

This paper addresses methods that inspectors can utilize to quickly evaluate whether there may be a problem, and discusses techniques that can be applied to determine the source(s) and magnitude of water penetration

Keywords: Facade ordinance, facade inspections, water infiltration, water testing, leakage, corrosion, masonry, steel, curtain walls

Introduction

Many source(s) of water penetration through a building's facade are often overlooked. Most building owners realize that their roofs have an expected useful service life, but do not realize that facades and other components of the building also require periodic maintenance, repair or replacement.

Water penetration into a building's facade can lead to corrosion of embedded ferrous metals, degradation of facade materials, or water leakage into occupied portions of the building. Corrective measures to prevent water penetration are usually not implemented until an unsafe condition or water leakage is reported. Many municipal facade ordinances recognize that water infiltration through a building's facade should be addressed.

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There are more than 13 000 buildings in Boston, Chicago, and New York City that require periodic inspection at intervals ranging from one to five years. As more municipalities enact facade ordinances, this number is sure to grow. To keep pace, the industry will need additional qualified inspectors who can quickly and effectively perform these inspections.. Proper diagnosis of water entry and the implementation of maintenance and repairs can reduce the potential of hazardous conditions developing at a later date.

Understanding the Facade System

The first step to evaluate the apparent watertight integrity of the building facade is to gain a basic understanding of the facade system and to know how each of these systems reacts to water penetration. Are the walls "skin barriers," cavity or other type of drainage walls, or thick load-bearing walls?

Walls of commercial buildings constructed before the 1900s were usually load-bearing masonry walls. These walls gained their strength and load carrying capacity from the mass of the masonry, which often included both brick and stone. Floor and roof beams were usually pocketed into the wall structure, transferring internal building loads to the exterior walls. These walls did not have a drainage cavity. During dry weather, water would gradually evaporate out of the masonry. As the walls became wet, especially during rainy weather, the masonry absorbed water. This type of construction is particularly vulnerable to extended periods of rain, which may fully saturate the masonry and reach the building interior.

Signs of potential water penetration into load-bearing walls include, but are not limited to, water staining; efflorescence; surface scaling; open mortar joints; and spalled masonry units.

The advent of steel skeleton framing at the turn of the century dramatically changed the face of American architecture. Much taller buildings were possible due to the ability of the steel skeleton to carry the loads of the building. Many early steel frame buildings can be mistaken for their load-bearing masonry ancestors because the steel framing was commonly encased in thick masonry walls to provide fireproofing protection. The use of terra cotta as a facade material also flourished during this time period. Masonry constructed during this era was often supported from steel outriggers connected to the building's steel skeleton frame. However, there was a downside to the use of masonry encased steel framing. Water penetration into the exterior walls now had the potential to corrode underlying ferrous metal, which will expand, crack or spall the surrounding masonry (Figures 1 and 2). Water penetration can cause much more damage to these walls than older load-bearing walls.

Signs of potential water penetration into masonry walls with encased steel include the same signs as in load-bearing walls plus corrosion staining, and cracked or distressed masonry, which may or may not be a potentially hazardous condition.

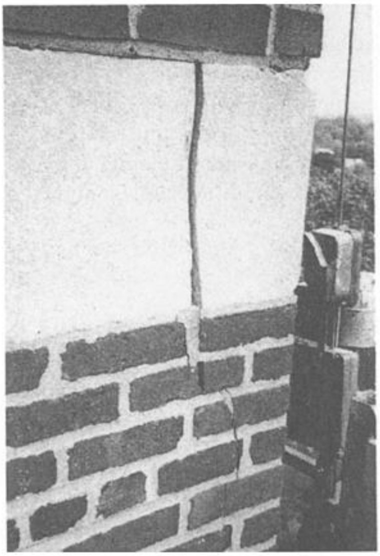

Figure 1 - Cracked and displaced masonry

Figure 2 - This corroding steel beam was found when the masonry in Figure 1 was removed.

The 1950s brought the advent of the drainage cavity. Drainage cavities are utilized with masonry veneers, stone, and glazing systems. The design of these walls recognizes that some water will penetrate through the outside face of the wall surface. Water is allowed to flow down through the wall or glazing system where it is diverted back to the exterior via a system of flashings and weep openings. The walls' overall ability to control water infiltration to the interior of the building is dependent on the flashing system and internal seals between vertical and horizontal members of glazing systems. The performance of drainage cavity walls is usually much more dependent on the configuration and integrity of the internal flashing system than on the watertightness of the exterior cladding. Because these elements are hidden, assessing the watertight integrity of cavity walls can be a significant challenge.

Signs of potential water penetration into these walls include the same signs as noted before. However, interior water leaks often occur below the flashings. Look for water stains on the interior surfaces of outside walls and ceilings one floor below the flashings (Figure 3). Pay attention to corners and other locations where the flashings make turns or steps (Figure 4).

The construction industry continues to develop lighter and thinner claddings that can be quickly erected, thus saving time and money. Many of these walls can be classified as skin barriers, meaning that they rely entirely on the integrity of the cladding to keep out water. Skin barrier systems are inherently vulnerable to water leakage because they lack redundancy and because the waterproofing components are fully exposed to climatic elements. Wall panels may be composed of aluminum, stainless steel, and composite panels with brick or tile veneer and a back-up board. Many exterior insulation and finish systems (EIFS) are also skin barrier systems, although new generations of EIFS are being constructed with drainage cavities.

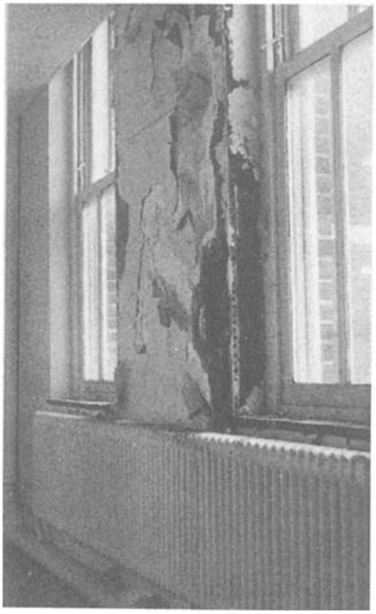

Figure 3 - Severe interior damage from water leakage

Figure 4 - Unsealed flashing at corner. Arrow indicates open end of metal flashing. Also note mortar blockage.

Most skin barrier walls rely on elastomeric sealants to close gaps between building panels and other adjacent facade elements. These sealants have varying useful service lives. Once water penetrates a skin barrier wall system, it can readily flow down through the wall, damaging wall components and causing interior water leakage (Figure 5). A facade inspector can often evaluate a skin barrier system for potential water leakage more quickly than other types of walls because the waterproofing components of the system are readily visible.

Signs of potential water penetration into skin barrier walls include the same signs as drainage cavity walls. However, pay particular attention to locations where exterior walls terminate directly over occupied portions of the building such as setbacks and rooftop bulkheads. These locations are especially vulnerable to water leakage. Look closely at components of the wall that degrade as they age such as sealants and gaskets (Figure 6). It is also important to look for cracks in barrier panels as potential sources of water leakage.

Figure 5 - Water leakage through this skin barrier stucco system has caused corrosion and other damage to interior wall components.

Figure 6 - Coating and sealant failure allows water to enter skin barrier systems.

Visual Observations

The inspector's goal is to assess the watertightness of the building facade. In pursuing this goal, the inspector is responsible for evaluating possible ways in which water can penetrate the building. While performing the facade inspection, the inspector should try to interview building maintenance personnel about reported water leakage and any repairs implemented to address water infiltration:

- Were there any water leaks in the past? Get a historical background.
- How long has water been leaking into the building? Remember that leaks usually develop for some time before they are observed or reported.
- Does water leakage seem to follow a pattern? Look closely at interior and exterior stains. In particular, note the location of water leakage signs relative to features such as window heads and sills, setback walls, and roof drains.
- Under what circumstances is water leaking into the building? Does leakage occur quickly after rain starts, or does it take a while for leakage to develop? If water leakage lags behind rainfall, water may be accumulating under a roof membrane, inside a wall system, or traveling a substantial distance from the infiltration point to the observed leak.
- Does the building leak every time it rains, only during rainstorms with high winds, or during any other specific weather conditions?

- Were facade repairs done to address previous water leakage? Find out when and where these repairs were made, and evaluate whether they were effective.

Visual signs of water leakage are found both inside and outside the building. Exterior symptoms should be correlated to interior leakage patterns. The following symptoms are indications of potential water leakage that warrant further investigation.

- Water stains - Water leakage typically creates a moderate discoloration of interior finishes, and staining of exterior components may be observed as well. Orange staining indicates probable corrosion of embedded metal, and dark discolorations may indicate mold growth.

- Efflorescence - Efflorescence is caused when soluble components, usually from masonry or cementitious materials, are dissolved in water flowing through the materials, then deposited on the face of the wall as the water evaporates. While evaluating efflorescence, look for white residues on the surface of the wall. Efflorescence is usually found on the exterior wall surface, where the material is exposed to the weather.

- Open joints - This is perhaps the most typical cause of water leakage and the easiest to identify. Open joints allow water to leak; even a small breach can allow a significant amount of water to penetrate into the building. Examples of open joints include the following.

    - Breaches in the sealant: Look for cracked sealant on the edges of the joint and in the middle of the sealant strip.
    - Weathered or open mortar joints - Porous mortar joints may contribute significantly to water leakage. Look for soft areas, cracks on the edges of the joint, loose mortar segments and areas of missing mortar.
    - Defective window glazing - look around the window glazing for loose or missing glazing elements. Loose glass panes are not only a sign of water infiltration, but a safety concern as well.

- Coatings – Coatings are often applied to combat water leakage. Unfortunately, they seldom solve the problem and often trap additional water in the wall by sealing exit paths. Coatings may be most visible at roof bulkhead walls and other hidden locations. These areas are readily accessible to building maintenance personnel (who usually are the first people to address water leakage), and they are often located directly above occupied space that may be experiencing leakage.

- Sealed weepholes, lintels, and other openings intended to release water – Free-flowing weep openings are critical to the performance of a wall's flashing system.

- Previous repairs – Look for repaired wall areas, new metal flashings, replacement windows, even a new roof. The driving force behind much facade repair and maintenance work is water leakage. Water leakage affects not only buildings; more importantly, it affects the people who use them. For facade ordinance inspections, whose performance and timing are mandated by law, any water leakage observed probably started long before the inspection, and in most cases the building owners or occupants have tried to do something about it. Unfortunately, water leakage is usually not as straightforward as it seems, and well meaning maintenance personnel often employ inappropriate corrective measures that make the problem worse, create new problems, or both.

- Organic growth - Porous materials that are chronically moist are highly vulnerable to organic growth. Water trapped on the surface of the material can potentially cause this condition. On the exterior surface, look for stains with a green or brown hue.

Reporting

Evidence of water leakage and observed vulnerabilities to water penetration should be addressed in the facade ordinance report. If allowed to continue, uncontrolled water penetration will usually lead to degradation of components of the exterior walls. The professional performing the inspection should use her/his judgment in recommending repairs to mitigate water infiltration. Often a phased plan to perform the maintenance repairs before the next reporting cycle is appropriate. However, the time period should be shortened if there are active leaks.

Recommendations

Once symptoms of water leakage have been identified, a more detailed investigation should be performed to evaluate the cause(s) of these symptoms. Water leakage investigations typically include a combination of observations, water spray testing and probe openings. Water spray testing and probe openings are typically beyond the scope of a facade ordinance report, but are often required to address properly the deficiencies identified by the inspection. It is important for the building owner to be aware of the need for additional investigation because the cost for this type of work often exceeds the cost of the facade ordinance inspection. However, the cost of a well-planned investigation is almost always more than balanced by reduced long-term costs.

For these reasons, it is important for architects and engineers performing facade ordinance inspections to have a working knowledge of common investigation and repair techniques; this will facilitate a more thorough report and better communication to the client regarding the nature and extent of required work.

This section presents guidelines for planning and performing a water leakage investigation, and discusses appropriate repairs for commonly encountered deficiencies.

Planning the Investigation

Pinpointing water leakage is not always straightforward, and an investigation based on the architect or engineer's experience should be designed to test likely paths of water infiltration. An important point to remember is that water seldom travels up; it is usually reasonable to look for infiltration points at and above the location of the observed water damage or reported leak. However, it's not unusual for water to travel significant distances horizontally from the infiltration point to an observable interior leak.

Controlled Water Spray Testing

The purpose of water spray testing is to reproduce observed and reported leakage under controlled conditions, so that water leakage paths can be isolated and evaluated in detail. Standard procedures for various types of water testing have been adopted by ASTM. These tests were developed to evaluate compliance with performance standards for new construction, but many are increasingly being put to work as diagnostic tools for existing buildings. Some but certainly not all of the possible tests are listed below.

ASTM – Standard Test Method for Field Determination of Water Penetration of Installed Exterior Windows, Doors, and Doors by Uniform or Cyclic Static Air Pressure Difference (E1105). – As its name implies, this test was developed for installed manufactured components of a facade, but it has also been adapted to a wide variety of applications. The test approximates a wind-driven rain over a small portion of a facade using a grid of calibrated spray nozzles to uniformly wet the wall surface.

AAMA 501.2 – Field Check of Metal Storefronts, Curtain Walls, and Sloped Glazing Systems for Water Leakage – In its essence, this test is similar to troubleshooting leaks with a garden hose; the only difference is that a special nozzle, specific water pressure, and specified time intervals are used.. This test is good for verifying readily apparent weaknesses, such as improperly sealed flashings. It should be used when quick results are expected.

Repair Recommendations

Based on the results of the preliminary visual assessment, and any supplemental investigation, repair recommendations must be prepared. Repairs to address water leakage identified during a facade ordinance inspection may range from replacement of weathered sealant to replacement of an entire deteriorated curtain wall system. Most buildings require a program between these extremes.

Conclusions

Many of the causes of facade deterioration are related to water penetration into the exterior walls. Often the signs that this is occurring are subtle. Professionals performing facade inspections must develop the skills to properly diagnose water penetration. Reporting professionals must be able to apply their judgment to the individual situation. They must not be overly conservative or too relaxed and be able to educate building owners so they are able to make sound decisions regarding the long-term performance of the building.

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