10-16-2009

Maine State Cultural Building Augusta, Maine : Exterior Building Facade Investigation

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Maine State Cultural Building
Augusta, Maine

Exterior Building Façade Investigation

Project No. 09594

October 16, 2009
Final

Table of Contents

Exterior Building Façade Investigation

Scope of Study

The goal of the study is to develop an analysis by which the issues connected with exterior envelope and its façade can be identified and corrective actions quantified and addressed. The scope of our forensic analysis is to research the existing construction documents and details and provide a forensic demolition of the limestone panels at specific locations to determine the condition of the system. By removing the panels of the exterior wall, we determined the condition of the support structure and the anchorage of the panels. With the visual inspection of the installation, we were able to determine whether the panels and its back-up system were constructed as detailed.

The Maine State Cultural Building facilities personnel viewed and reported sections of the limestone panel facades breaking and falling off from various building faces. Upon the site observation of this condition, a number of factors were plausible causes. The demolition of the panels provided us with the answers necessary to determine the causes of this issue.

In the following sections, we will provide relevant information connected to our forensic observations of the limestone panels on each façade of the building. We will also provide recommendations and potential costs associated with the corrections; also identified is the recommended correction of the loading dock and its overhang.

We have evaluated the exterior glazed doors and will provide a cost to replace the doors with a more energy-efficient and durable system.

Maine State Cultural Building Exterior View

Exterior Building Façade Investigation

Location 1 – East Wall Panel Removal (EP)

The location image EP1 on the east wall of the building is the location on the façade where the bottom section of panel broke off as reported by the Museum personnel. Image EP2 shows the bottom section of the panel that was reported to have broken off and the removed panel. This location was chosen for a panel removal for a series of reasons. The fact that the panel was already broken and the review of the construction documents showed flashing details at the bottom of the cavity that were suspect, it offered a location to inspect multiple rows of structural angle supports and hangers. The selection of the bottom row of the façade panels was due in large part to the physical size and weight of the panels and the ability to remove and replace them. A crane would have been required to remove the larger panels in the rows above. With our first walk-through, we noticed that there was extensive water damage to the cement plaster soffits on three sides of the building and this generated questions as to the water entry points.

EP1 - East wall - broken panel and removed panel

EP2 -Removed panel and broken section

Exterior Building Façade Investigation

Location 2 – South Wall Panel Removal

As outlined, we selected a location on three sides of the building for visual inspection to determine the condition of the wall system on three exposures. The lower panel on the south wall was removed for the reasons stated in EP1. In the panel removal process, the section at the adjoining panel broke off pointing to the current unstable condition of the lower section of the panels. The details of construction and condition of the wall will be outline further in this report.

EP3 - South wall panel removal location

Exterior Building Façade Investigation

Location 3 – West Wall Panel Removal

As outlined, we selected a location on three sides of the building for visual inspection to determine the condition of the wall system on three exposures. The lower panel was removed at the west wall for the reasons stated in EP1. The removal of the panel at this location was made easier by a full-height control joint in the façade panel system on the left side. Another reason for panel removal here was the large amount of staining on the face of the panels and the joints and a desire to discover if water penetration into the cavity is one of the contributors to the panel face effect. The details of construction and condition of the wall will be outlined further in this report.

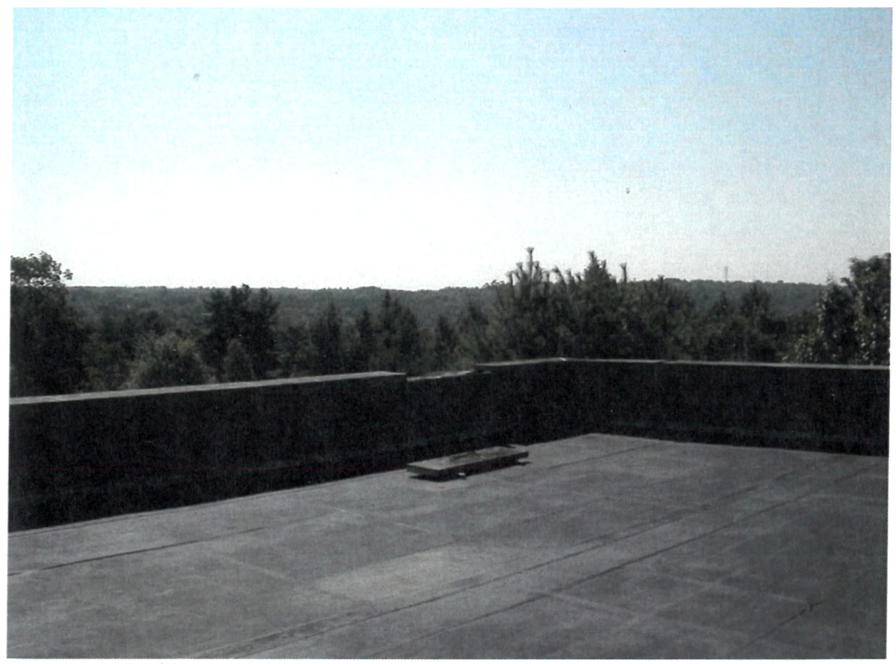
EP4- West wall panel location

Exterior Building Façade Investigation

Location 4 – Parapet Cap – Southeast Corner

To determine the condition of the existing wall system, a location on three sides of the building was selected. The parapet cap on the southeast corner was removed to observe the anchorage details of the components and installation of the flashings and the condition of the cavity. The limestone cap was removed as part of the existing conditions field observation. Per Harriman's request, a second limestone cap on the north side of the building was removed by Consigli Construction. See Parapets southeast corner inside (tab 4) for more information.

EP5 - Southeast corner parapet cap removed



Exterior Building Façade Investigation

EP6 - Southeast corner parapet cap removed

EP7 - North side parapet cap removed

Exterior Building Façade Investigation

Existing Construction Document Details – "As Built" (AB) Drawings

In review of the existing construction documents the following list of questions was assembled. From this list, a plan of panel removal and their locations was put into action.

1. Per a 2001 report, a question was raised about the CMU loadings on the perimeter steel supporting the panels. Is the potential of a structural deflection due to heavy backup load the cause of the panel breaking along the bottom edge?
2. Bolting of the structural angle support back to the perimeter beams – Is slippage of this connection occurring?
3. The flashing detail – The ability of water penetration to weep out of the cavity is questionable based upon the flashing detail. Also in conjunction with this flashing detail, is moisture trapped in the cavity? Is the angle support system galvanized as noted or has the potential of the support angle rusting a cause for failure of these support angles?
4. As we walked the perimeter of the museum on our first site observation visit, we noticed that the face panels were heavily stained. The cement plaster soffit system at the base of the wall indicated water stain locations on all soffits where the soffit and panel joined. The system also showed signs of water penetration without a means to relieve water properly.
5. Limestone Cap flashing detail – is this flashing potentially trapping water?
6. Limestone angle anchors to backup – How did these get installed based upon their configuration? Are they installed as detailed? They are noted as galvanized – are they rusting? Spacing and method of installation not consistent with this type and age period of building. Can these anchors handle current loadings?
7. Are the limestone panels slotted out and supported as detailed?
8. The third row of support – concrete encased steel channel – Is it anchored to steel columns? Are there any signs of deflection in the system that affects the overall panel system joints and cause a compressive load effect causing the bottom edges to crack?

Exterior Building Façade Investigation

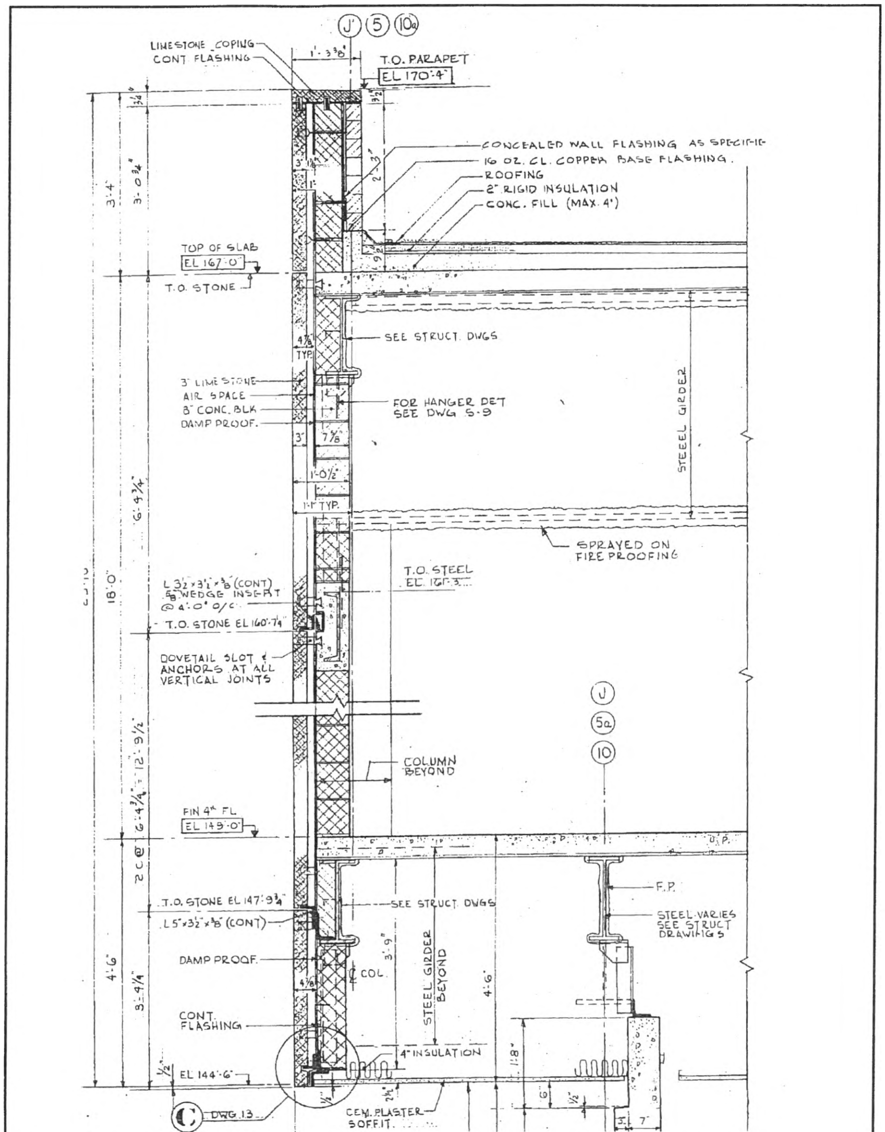
Image AB1 shows the entire wall section and its associated construction details. Our review of this as-built construction detail shows the base of this wall section and how it was assembled relative to the structural frame of the building. The issue of the limestone panel breaking off the building façade became a primary area of concern.

The limestone panels are supported on an angle support lintel which is anchored back to the main structural frame. The entire height of the façade is supported with three rows of horizontal angle support around all sides of the building. The bottom row of panels is vertically supported from a hung angle system connected to the floor structure. The second row is anchored back to the same floor beam structure. The third row of panels is supported from a concrete-encased steel channel which is hung from the roof perimeter beam support.

The 3" limestone panel is supported laterally to CMU backup system with an adjustable anchor on the vertical edges of the panels. There is an air cavity behind the limestone panels with damp proofing applied to the CMU backup face. The parapet construction shows a brick and CMU backup system sitting directly on the roof slab with the limestone panels supported on the support angle below and a limestone panel sitting and spanning the entire system.

Exterior Building Façade Investigation

AB1 -Wall section - east, south and west sides

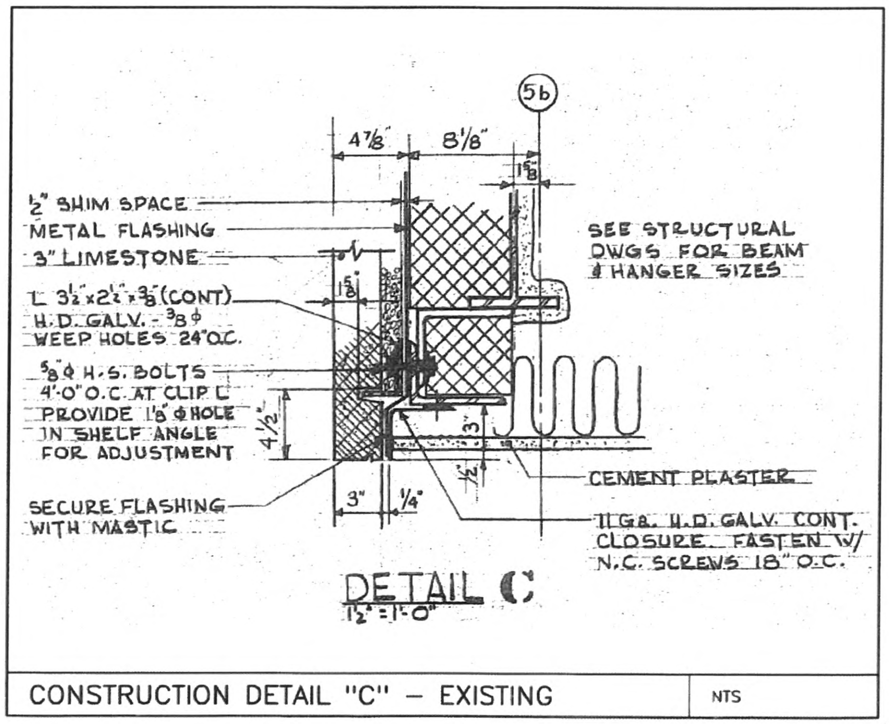


WALL SECTION – EAST, SOUTH AND WEST SIDES NTS

Exterior Building Façade Investigation

Image AB2 shows a large scale detail of the bottom wall detailing how the support angle is attached to the structure and the slot that was incorporated into the back of the limestone panel. It also shows the intent of how the wall cavity was to be drained and the connection of the suspended ceiling, flashing and panel system. Please note the location of the flashing at the ceiling and the intent to weep the cavity for moisture – we will delineate this further in the report.

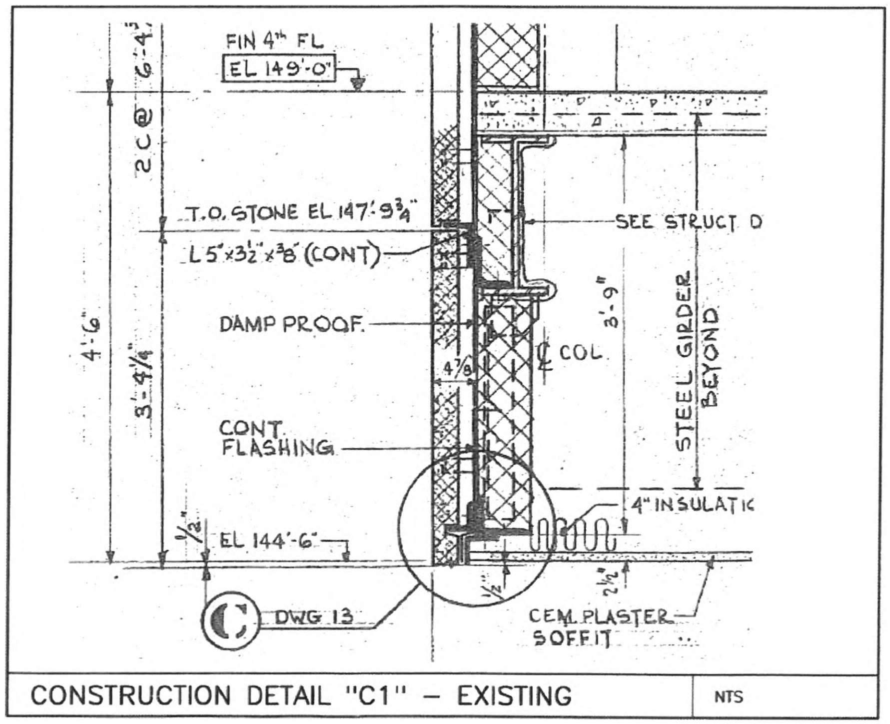
AB2 - Construction detail C - existing



Exterior Building Façade Investigation

Image AB3 details the bottom row of panels and the two rows of angle support and their connection to the structural frame of the building. This area was our focus for panel removal for the reasons outlined.

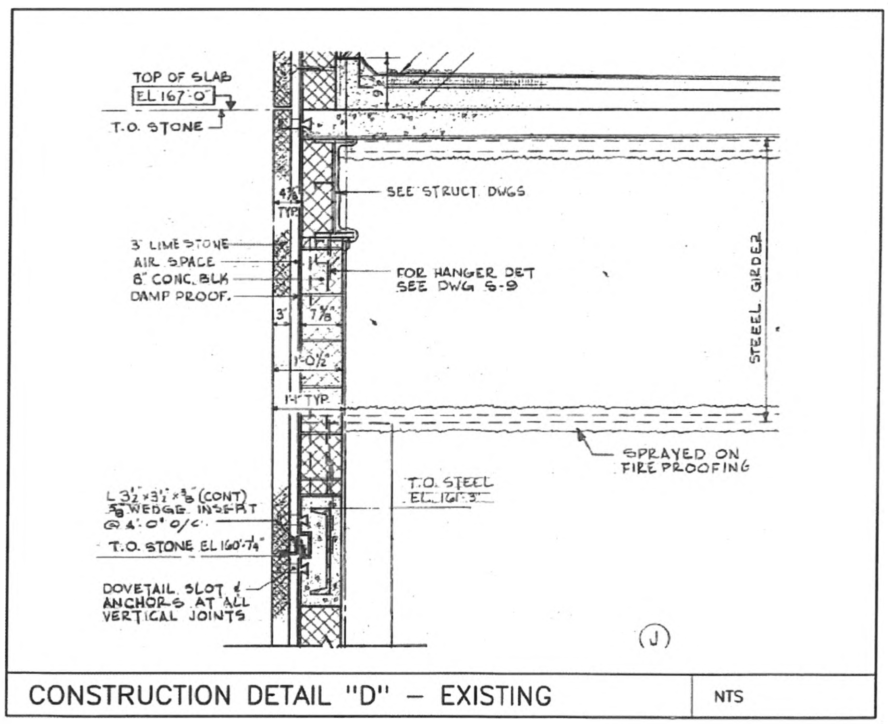
AB3 - Construction detail C1 - existing



Exterior Building Façade Investigation

Image AB4 details the top row panel support and its orientation and connection to the building frame. The concrete-encased steel channel is supported from the perimeter roof beam and encased with concrete so the CMU backup could be installed on both sides. The structural question with this support line is its connection to the structural column on each end. Based upon the difficulties of a forensic removal, this question could not be evaluated. The structural analyses contained within this report will outline the potential issues with this support location. Also of concern with this detail is the use of a wedge anchor and bolted connection to this angle with an elongated bolt hole. The potential of slippage due to loads could be the potential contributor to a compressive load failure. The observation notes from the locations with similar angle support connection will be noted in more detail.

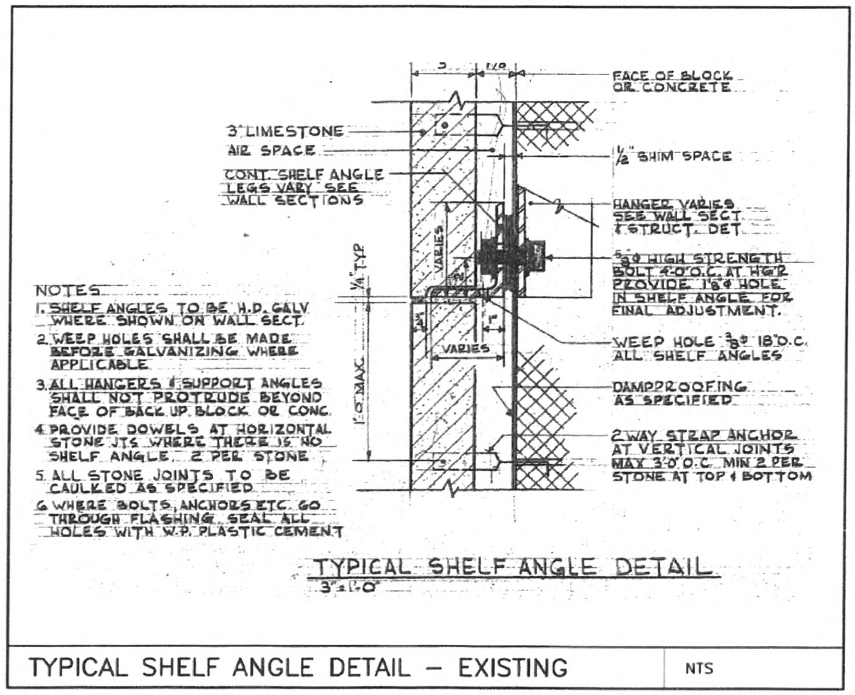
AB4 - Construction detail D - existing



Exterior Building Façade Investigation

On Image AB5, the items of importance for visual inspection were the strap anchors, the bolted connection and the weep hole that was drilled through the bottom leg of the support angle for moisture to drain through. All the support angles per this detail are noted as galvanized to resist rust due to exposure to moisture. Note #5 (below) states that all the joints shall be caulked.

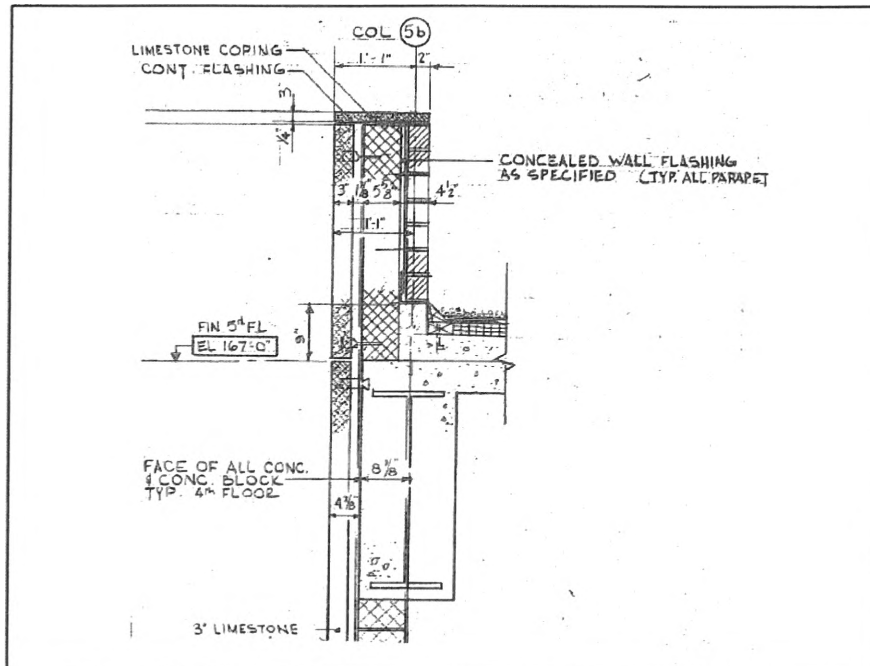
AB5 - Typical shelf angle detail - existing



Exterior Building Façade Investigation

Image AB6 shows how the limestone panels were integrated with the back up system above the roof. Of note here is how the top row of limestone panels are anchored to the block backup system without any noted structural lateral support for this extension above the roof.

AB6 - Parapet wall above the roof detail - existing



PARAPET WALL ABOVE ROOF DETAIL – EXISTING | NTS

Exterior Building Façade Investigation

Location 1 – East Side Observations (Panel Removal Location 1- PR1)

Image PR1-1 and PR1-2- This is the location on the building where the panel broke and fell to the ground as reported by Maine State Cultural Building facilities personnel. The picture shows that the panel broke along the line of the upper edge of the slot. The support angle and its condition is very evident. The support angle is galvanized as documented and the bolted connections are as detailed. The extra red support clip was initially installed as the connection to the horizontal support – the clip was not correctly sized and consequently was left in place and a new clip was installed to connect to the horizontal panel support angle. The cavity space is very dry and showed no signs of excessive moisture build-up due to inadequate weeping of the cavity. The fabric flashing to the right is inserted into a CMU joint and laid down over the angle hanger support system for the bottom panel support angle as protection from moisture in the cavity.

PR1-1 - Panel removal east side

Exterior Building Façade Investigation

PR1-2 - East side clip left in place, anchor and dampproofing.

Exterior Building Façade Investigation

PR1-3 and PR1-4 - The pictures below show that the support angles are bolted as detailed and galvanized as documented. The bolted connections do not show signs of slippage even with the multiple shims that were installed behind the leg of the angle. The shims were installed for alignment reasons and we did verify the elongated erection hole on the clip angle as noted by typical detail "C". It also clearly shows that the damp proofing on the CMU backup does not extend up behind the angle.

PR1-3 - East side connection

PR1-4 - East side horizontal support angle - bolted connection

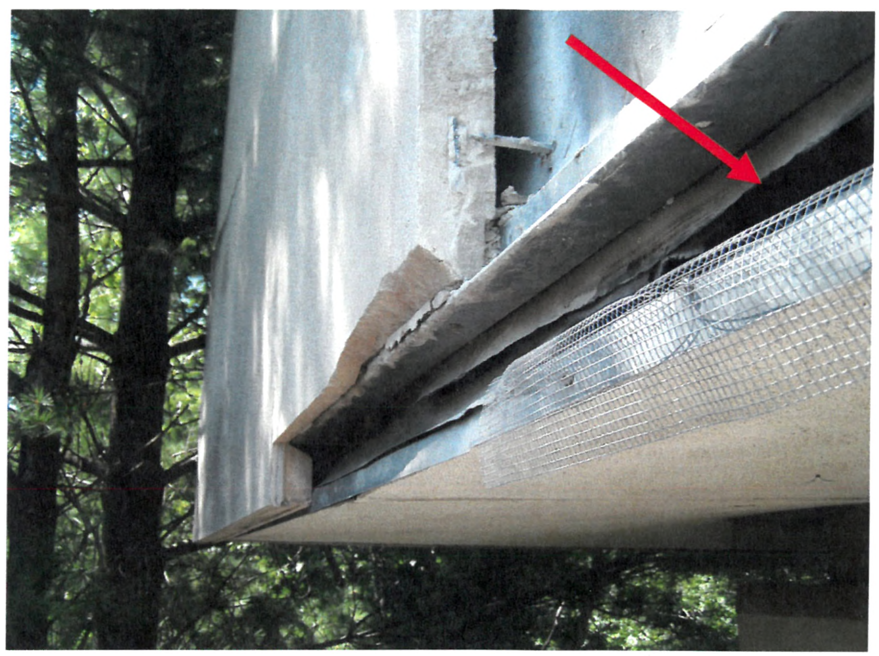
Exterior Building Façade Investigation

PR1-5 - The picture indicates that the panel anchors are not as specified on the contract documents. The contract documents indicated use of a flat strap anchor and this picture shows an adjustable anchor with threaded couplings for horizontal adjustment during erection. The back part of the anchor is a threaded coupling thru the flashing into the block cell and at some locations into a grout joint. This indicates that with this type of anchor, the panels were installed in a progressive manner using the bottom left corner of the façade and progressing both horizontally and vertically across the face of the building. The panel slot was set into a grout base on the angle and the biscuit end of the anchor was set into the slot on the edge of the panel and a full grout bed, then the next panel is set in place to form the panel layout. The detail called for pea stone fill to act as a grout catch during the panel installation but the distance from the back of the support angle and the flashing installation method will not allow the pea stone to remain in place.

PR1-5 - East side angle panel support

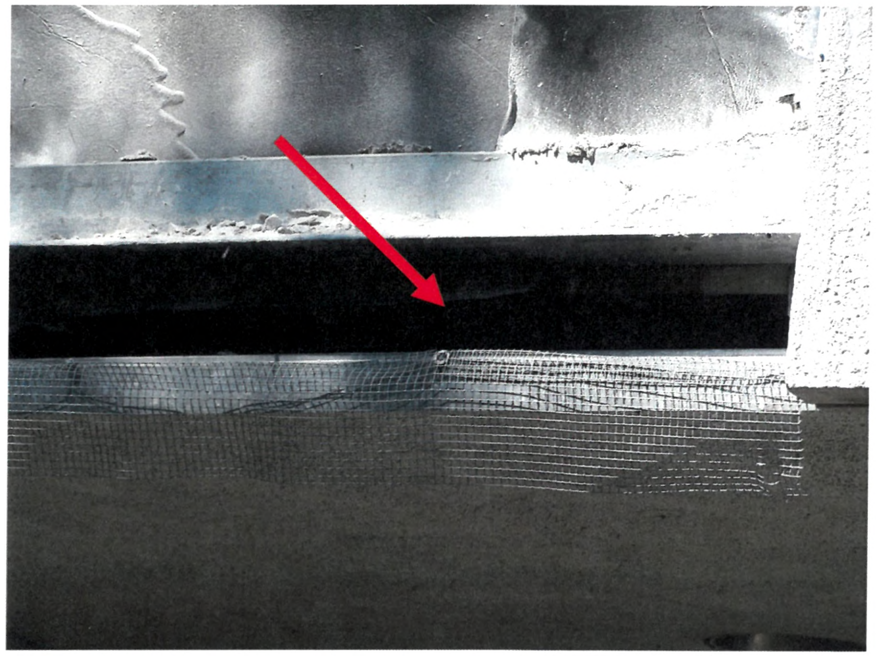
PR1-6 and PR1-7 - The failure of the panel occurred at the top of the slot above the angle – this is the weak point of the panel in regard to its thickness. It would be safe to assume that the slot was too deep for a limestone panel. This continuous slot out of the back of the panel is not recommended by Indiana Limestone Company, Inc. It also shows how the fabric flashing system was extended down behind the angle and laid over the galvanized metal edge at the end of the cement plaster soffit. This system was installed as detailed but is also the weak point for water weep from the cavity as evident in the image PR1-7. The figure also shows how the flashing was not extended down long enough to get out over the metal edge and allow water to migrate out of the cavity. The hole created by this poor installation is a major source of air infiltration into the building (see arrows on PR1-6 and PR1-7). The cold outside air penetrates over the soffit directly into the building's ceiling cavity.

PR1-6 - East side angle panel support



Exterior Building Façade Investigation

PR1-7 - Hole in flashing



Exterior Building Façade Investigation

Location 2 – South Wall Panel Removal (Panel Removal Location 2- PR2)

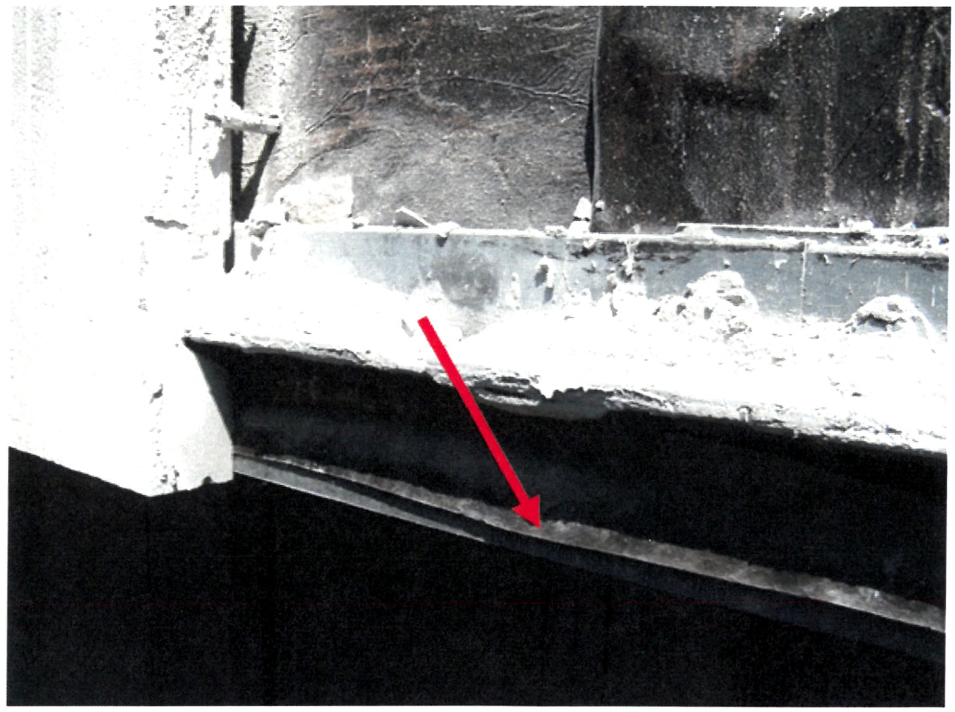
PR2-1 - The panel removal location on the south side revealed that the cavity and the backup system were in the same condition as location 1. No abnormal deterioration of the panel support system was evident and the panel and its associated flashings and backup system were constructed as detailed. The picture shows where the adjoining panel broke along the line of the upper edge of the notch during the removal process. The cavity space is very dry and shows no signs of excessive moisture build-up due to inadequate weeping of the cavity. The fabric flashing in the center was installed over the structural hanger system to protect it from moisture.

PR2-1 - South wall removal location

Exterior Building Façade Investigation

PR2-2 and PR2-3 - These images show that the limestone panel slot is supported on the angle and the system was set in a continuous bed of grout. The panel anchor is in a location consistent with the others observed. The anchor is the same type used in the other locations which is a different type then what was specified in the existing documents. Also note that the fabric flashing is tucked down behind the support angle and extended out of the metal edge at the soffit.

PR2-2 - Flashing extended down over metal soffit edge



PR2-3 - Flashing extended over metal edge

Exterior Building Façade Investigation

PR2-4 and PR2-5 - At this location, we discovered the fabric flashing did not extend down over the metal soffit edge leaving a large hole for water and air infiltration to the interior. This hole is the direct cause of all the water stains on the cement plaster soffit. The cement plaster soffit shows large stain areas near the outer edge of the soffit over many locations on all three building faces.

The soffits have large cracks and it is safe to assume that where these stains and cracks occur, water enters the hole in the flashing through the backside of the soffit. The hole also includes air infiltration which impacts the operating efficiency of the mechanical system for both the heating and cooling system.

Another item of concern is based upon the panel to soffit system detail outside air is infiltrating directly over the soffit direct to the internal ceiling cavity at that floor.

PR2-4 - Flashing not installed at hanger support

Exterior Building Façade Investigation

PR2-5 - Flashing not installed at hanger support (arrow indicates hole)

Exterior Building Façade Investigation

PR2-6 - Image PR2-1 indicates a large piece of fabric flashing extended vertically down through the picture. In lifting this piece of fabric flashing *(image PR2-6 below)*, the angle hanger system for the lower panel support angle is revealed. The image indicates that the vertical hanger was not galvanized; the angles have been coated with a damp proofing product and the CMU backup has been installed behind the angles. The fabric flashing was not sealed to the angle hangers nor was it extended down far enough behind the support angle below creating the hole (as noted previously). These angles were not noted as galvanized in the existing construction documents. The vertical slot between the angles revealed some surface rust on the vertical leg of the angles which is considered detrimental to its support capacity. This slot is also a point of entry for air-infiltration into the ceiling cavity.

PR2-6 - Flashing over angle hanger

Exterior Building Façade Investigation

Location 3 – West Wall Panel Removal (Panel Removal Location 3- PR3)

PR3-1 - The panel removal location on the west side revealed that the cavity and the backup system were in the same condition as location 1. No abnormal deterioration of the panel support system was evident and the panel and its associated flashings and backup system were constructed as detailed. The cavity space is very dry and shows no signs of excessive moisture build-up due to inadequate weeping of the cavity. The fabric flashing in the center was installed over the structural hanger system to protect it from moisture. The continuous joint on the right side is an expansion joint full height from the bottom edge of the panel to the underside of the limestone cap above. The sealant in the joint is deteriorating and there were holes opening up due to the sealant cracking and falling out.

PR3-1 - West wall panel removed

Exterior Building Façade Investigation

PR3-2 and PR3-3 - The panels were set in a continuous bed of grout at the support angle, grouted together, and the panel anchor shown was cut off and bent back during the removal process. The details are consistent with other locations and constructed as detailed. The amount of grout that piled up behind the side panel during its installation is a concern and would make the weeping of the cavity through the intended detail very difficult to accomplish. Image PR3-2 shows the connections and galvanized angle support of the first and second row was installed as detailed. Image PR3-3 shows that there is a considerable amount of grout that is adhered to the angle above; the documented detail calls for this joint to be a caulked joint. In this same figure, note the striations in the panel face above it – these striations are a direct result of how the panel was cut out at the quarry.

PR3-2 - Panel north on support angle

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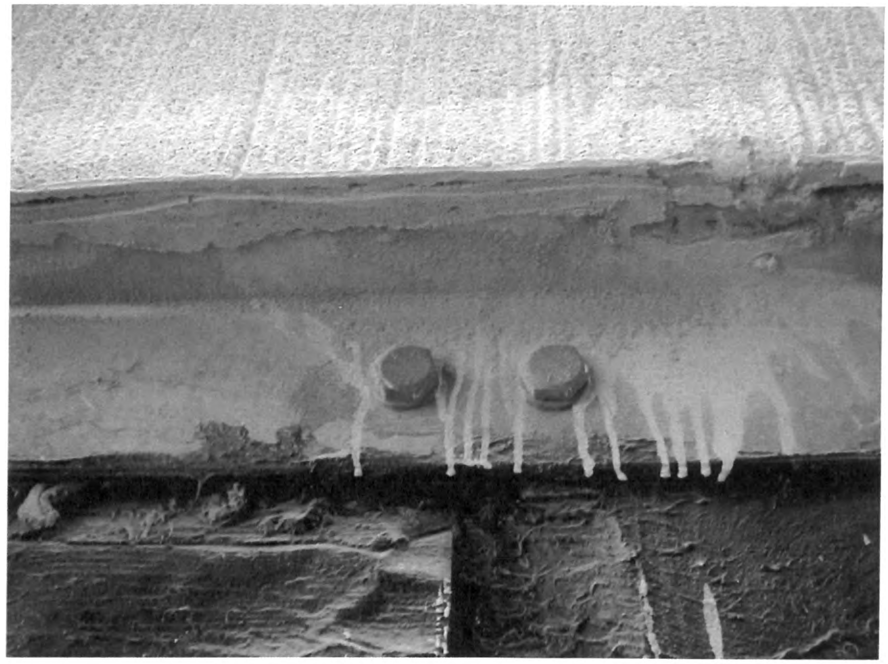
PR3-3 - Horizontal support angle connector and grout bed

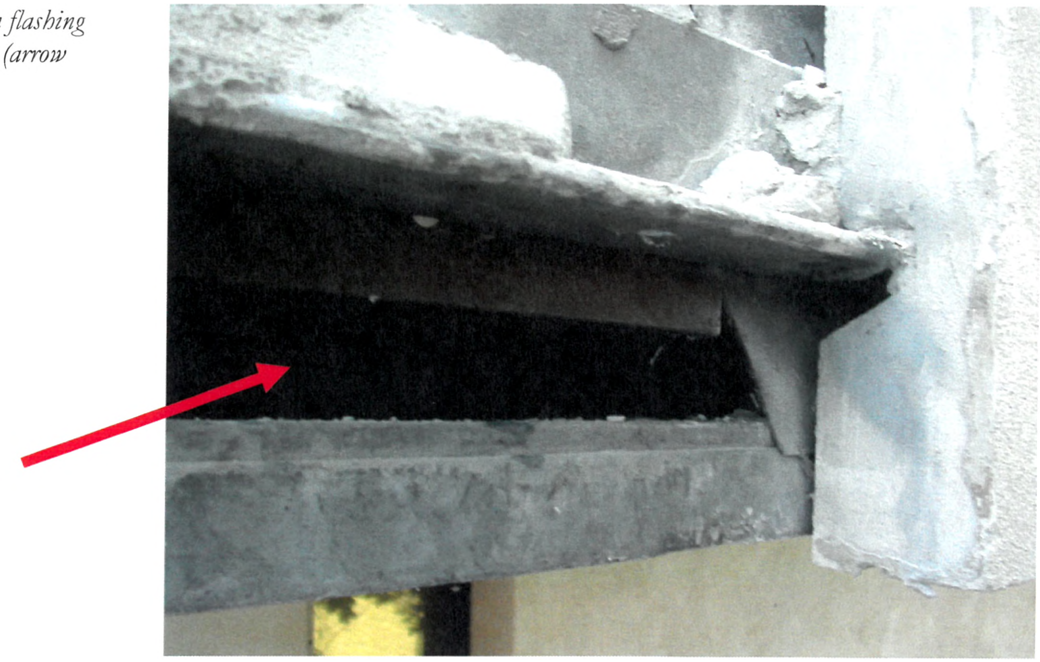


Exterior Building Façade Investigation

Image PR3-4 and PR3-5– At the structural hanger location, we lifted the fabric flashing and as observed in the other locations, the hangers were damp proofed and constructed as detailed. In lifting the side fabric flashing, it is noticeable that the CMU backup behind the fabric flashing was not damp proofed. In PR3-5 the hole in the flashing system below the angle support is clearly obvious and it is inline with the hanger system above. The connection of the hanger support system to the horizontal angle will not allow the fabric flashing to extend down over the metal soffit edge below and allow the weeping of the cavity as intended by the existing detail. The support hanger system on the existing drawings is denoted to be spaced at 4'-0" on center – it is safe to assume that this hole in the flashing occurs at this location around the entire lower perimeter of the building.

PR3-4 - Flashing at angle hanger

PR3-5 - Hole in flashing system at hanger (arrow indicates hole)



Exterior Building Façade Investigation

Location 4 – Parapets Southeast Corner, North Side
(Panel Removal Location 4- PR4)

Image PR4-1, PR4-2, PR4-3, PR4-4 – These parapet locations were defined by Harriman as a definite location for forensic removal after our initial walkthrough to view the building. The location of the limestone cap removal was easily identifiable through the existing documents. Once removed, our observation indicates that the parapet was constructed as detailed. The limestone cap spans the backup wall system and the façade panel and is held in place thru a series of metal pins inserted into the top of the panel and the CMU or brick backup. The cap joints were grouted together, raked out, a sealant applied and a lead strip joint cover inserted into the wet sealant joint. A layer of fabric coated copper flashing was installed over the backup system and limestone face panel. Image PR4-1 shows a worker from Consigli construction in the process of removing a cap on the north side, this a field time decision based on the discovery on the southeast corner. The construction at this location was similar to the southeast corner. One of the primary reasons for removal at this location is the amount of surface mold on the inside of the parapet face and the extensive amount of face stains on the limestone panel faces on the north facing exposure.

PR4-1 - South east corner cap removed

Exterior Building Façade Investigation

PR4-2 - Typical cap joint

PR4-3 - Cap removal north side

PR4-4 - Cap removal north side

Exterior Building Façade Investigation

Image PR4-5, PR4-6, PR4-7, PR4-8, PR4-9, PR4-10, PR4-11 – In removing the limestone caps, it was observed that the fabric-faced copper flashing has not deteriorated due to age. The insertion of flashing at this horizontal location in the backup wall has created a bond break between the cap and the brick backup as well as the limestone face panel – PR4-5. The joints on both sides have broken away from the flashing and have deteriorated to a point where portions have fallen out PR4-8. The flashing was not sealed at the laps and not sealed at the top of the brick or the face panel, thus allowing water entry points into the parapet system PR4-7. The limestone cap is held in place with a series of pins set into the panels and the brick backup and is held down by use of a grout pocket in the base of the limestone cap. In images PR4-9, PR4-10, PR4-11, these pins are shown in place, but are considerably shorter in height than what should have been used to properly hold down the cap. Since the grout joint under the cap has no bond left, and the pins are not secure into the cap, we can assume than the dead load of the cap itself is what is holding the system down.

PR4-5 - Flashing south east corner parapet cap

Exterior Building Façade Investigation

PR4-6 - End condition south east parapet cap

PR4-7 - End condition south east parapet cap

PR4-8 - Grout base at cap

PR4-9 - Pins and flashing at parapet cap

PR4-10 - Pin in brick backup

PR4-11 - Pin

Exterior Building Façade Investigation

PR4-12 and PR4-13 – In viewing down the cavity from above the panel anchors showed no visible signs of rusting to the 3/16" diameter threaded rod. There was some surface rust to the anchors at the top of the wall but looking down the cavity most the anchors appeared to be in good shape.

PR4-12 - Cavity looking down

PR4-13 - Cavity looking down - minimal rust on anchors

Exterior Building Façade Investigation

PR4-14 - The inside face of the parapet walls have vertical cracks on all sides of the building. The grout joints are totally broken at the cap and the backup system has vertical cracks from the caps to the base. This has presented a real structural concern to the top panel and parapet of the system. The existing construction documents show the top row of limestone panels anchored to the backup system in the same manner as noted previously. In the removal process of the limestone cap, we were able to physically move the backup system with little pressure. With no lateral structural element in the backup system shown on the construction documents, and the field forensics indicating the system was constructed as detailed, the vertical cracks in the backup system indicates movement in the system. We would consider this whole parapet system to be structurally unstable and should be addressed as soon as possible.

PR4-14 - Parapet backup cracked

Exterior Building Façade Investigation

General Façade Views – Limestone Panels (GF)

The following general views of the exterior limestone panels shows the condition of the limestone panels on all four exposures as they currently exist. The panels were supplied to the project from Indiana Limestone Company and are approximately 40 years old.

The following information is as documented from the contract specifications. The panels appeared to have been installed in accordance with these finish requirement of the specifications.

Buff Color – cream to light brown with some veining and grain movement in the face.

Rustic – large to coarse grain stone permitting an above average amount of inclusions and veining.

The finish visible is "shot-sawed". It is not caused by weathering - though it is conceivable that a very small amount of weathering has occurred over the 40 or so years the stone has been there. If photos were available of the building when it was new, you would see that the stone finish was cleaner but otherwise identical to what you see now.

As the stone is being sawed into slabs, chat (a coarse abrasive sand) and steel shot are introduced into the gang-sawing process. The result is what you see – a rough finish with grooves that are roughly parallel and of varying depths and widths. Some areas will have very few or very shallow grooves and look as though they've been sand-blasted. Other areas will have deeper grooves of varying number and "intensity". The grooves are formed by the shot and will vary from almost nothing to perhaps 3/8" deep and will be about the same width. The steel shot and the steel blades used in the sawing process will usually leave random rust stain spots.

There were some efforts made in the industry to replicate the finish or something close by other methods. It is possible, but highly unlikely, that what you see is one of those attempts at a faux shot-sawed finish. The rough finish holds dirt and other contaminants. It also provides an environment that often promotes algae growth.

Expected life of the stone:

If the water entry problems can be corrected and the bottom row of panel connection at the support angle and the resulting breakage from the continuous steel check, another 100 years at least and probably a lot more.

Exterior Building Façade Investigation

Water entry from the exterior:

The photos indicate there is or has been lots of water in the stone. The water content of the panels are most likely from a number of sources:

1. The vertical mortar joints. These joints don't carry vertical load and will shrink a small amount forming hair-line cracks. Also they tend to be less compacted. Water can pass through them pretty quickly. If there are no – or not enough - vertical control (movement) joints, then some of these joints will be cracked more than others as Mother Nature herself creates the joints.
2. Joints in the caps. Caps, copings and parapet walls are exposed to more rapid and extreme changes in temperature and tend to experience more thermally-induced movement. Mortar joints in these areas are more prone to premature failure and leakage than in other areas of the cladding. We believe that joints between caps should be sealant to better permit the small amounts of movement without failure.
3. Flashing at the caps. The detail was designed so as not to expose flashing edge. But to be effective, flashing under caps and copings should project out both front and back. Otherwise, there is a risk water will wick under the coping and back into the wall. Based upon the condition of the grout joint under the cap from both sides, this should be considered as a major water entry point into the cavity. Also, the flashing should be rigid enough or stepped or sloped so that it won't form a trough at the cavity and direct leakage water into the cavity via the joints.
4. Cracks or gaps at cant strips or at termination bars or reglets are all common sources of water entry.

The Broken Stone:

Continuous angles and continuous "steel checks" in the back of the stone are obvious lines of weakness. The configuration is also a stress concentrator. Good practice is to have continuous angles located at the bottom bed of the stone. If the support angles are to be located above the bottom bed (perhaps so they can't be seen) they should be short lengths that engage short pockets in the stone. Usually these pockets are located at the two vertical edges of the panel.
The fact that the continuous check was apparently filled completely with mortar made a bad situation even worse. It served to "lock" the leg of the angle into place in the stone. Any movement whatsoever – and there is movement however small - caused stress in the stone at this line of weakness. Since it lasted many years in this bad condition without failure, I would be safe to assume that water leakage has occurred to exacerbate the problem and finally over-stress the stone at this location.

From review of the specifications and discussions with Indiana Limestone Company the panels as specified with the grade of surface finish would come with the largest open grain finish and inclusions. The graining and color appearance are matched up by the company and shipped for field installation. From our research and discussion with Indiana Limestone Company and JKI Engineering as to why certain panels are not experiencing the same wearing

appearance on the same exposure is due in part to how the stone was cut out of the quarry and the directional grain on the panel face.

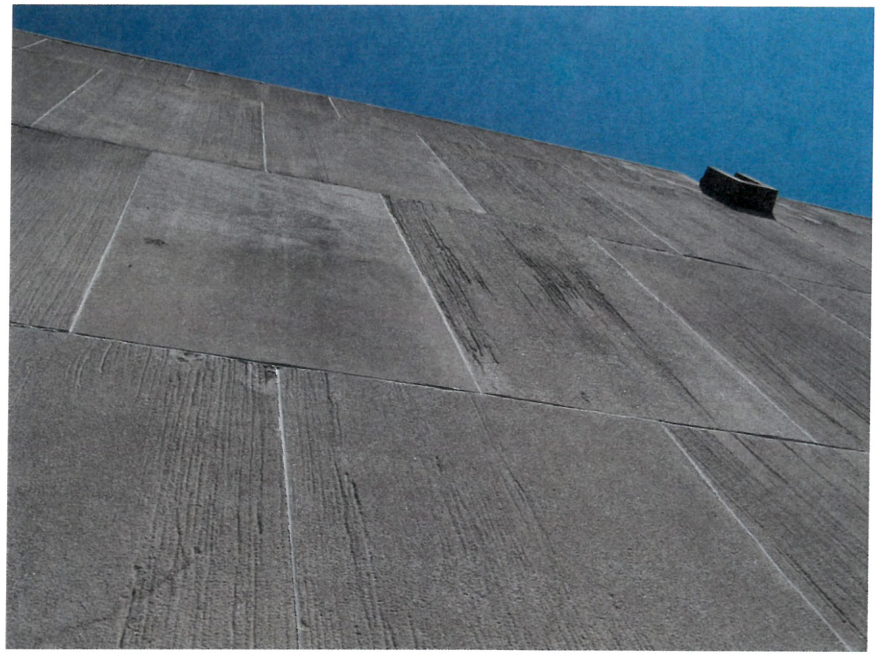
The heavy staining on the west and north faces – the north being the exposure facing the front entrance – is a combination of number of issues as noted above with moisture being driven into the panel from both sides – external face and the air cavity from the backside. The air cavity is designed to weep out moisture that may penetrate the cladding to the backside of the panel. Another component to moisture in the air cavity is the air-conditioning system and the change in humidity levels between the exterior and interior spaces causing a dew point to be formed somewhere between the inside face of the CMU backup and the inside of the limestone panel. Limestone is not a dense natural product it tends to be open grain stone that allows moisture to move through it readily easily. The fact that the backup system does not have an air/vapour barrier system nor insulation on the face is allowing the air-conditioning system to drive a condensation point through the backup into the limestone panel. Without an air/vapour barrier or insulation system in the exterior envelope is a direct cause to the inability to control the humidity levels within the museum during the cooling season. The lack of this system is the major weak point in the envelope with regard to energy efficiencies during both the heating and cooling seasons.

The most common causes of external stains on masonry products is water shedding down the walls across mortar joints causing an alkaline solution to be deposited on the face of the panel. As the panel face becomes saturated with moisture and evaporates it will increase the alkalinity and thus cause more staining. The air-conditioning in the warm summer months provides condensation in the mortar joints. This condensation draws the salts and alkaline out of the mortar washing down the face of the panel causing an interaction with the limestone panel and heavy brown staining. The north and west faces are not exposed to the drying effects of the sun, thus moisture sets in the mortar joints and panel faces for longer lengths of time causing to stains to appear.

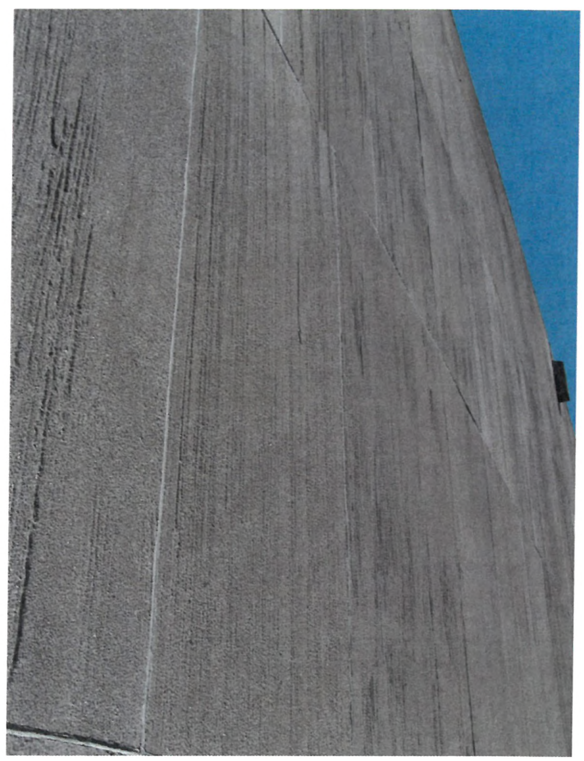
Images GF8 and GF9 show the parapet above the roofs of the Library and Archives. These images show that the exposed masonry backup at the parapet is not experiencing the same problems as the upper parapet over the main museum space. The detail here is different from the upper parapet in that there is no air cavity between the granite or precast concrete face panels and the brick backup but the connection detail to the system with flashing is the same as above.

Exterior Building Façade Investigation

GF1 - South wall

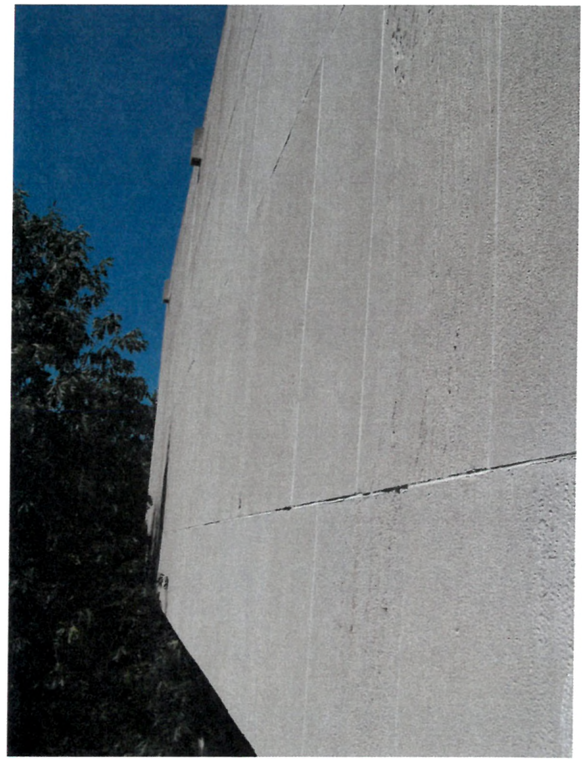


GF2 - South wall



Exterior Building Façade Investigation

GF3 - South wall



GF4 - North west corner

Exterior Building Façade Investigation

GF5 - North west corner

GF6 - North wall

GF7 - North east corner

GF8 - Roof parapet at Library

GF9 - Roof parapet at archives

Loading Dock Door (LD)

The damage to the limestone panels – LD1 – is a result of tractor trailer deliveries to the receiving door. The damage was caused from the impact from trailer box hitting and breaking off the bottom section of the limestone panel. Image LD2 shows the construction document detail of a similar connection at the limestone panel with a slot and seat on a support angle. The structural support frame at this opening does not appear to have been compromised by these incidents. The cement plaster soffit has been damaged beyond repair due to the impact of the trailer box. Our observation has shown that the soffit is bulging out and we can safely assume that the hung metal support system for the ceiling has been bent out of plumb and cannot be corrected to its original position without complete removal and replacement.

From the existing detail, a vertical height clearance was defined in the documents at approx 13'-4". The clearance requirements for today's tractor trailer units would be at a minimum of 13'-6".

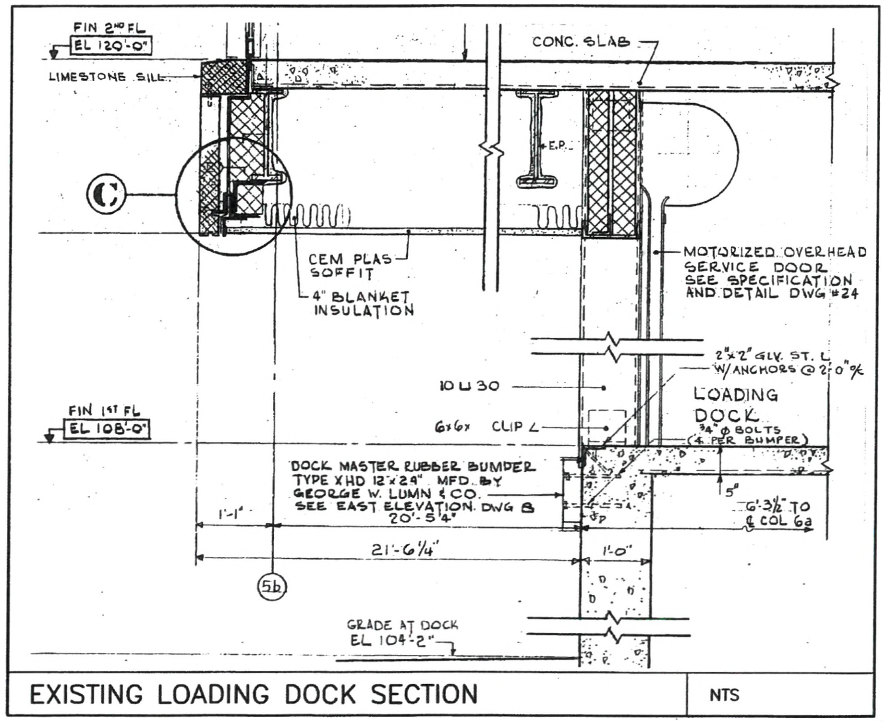
In discussion with the Facilities Director, the Maine State Cultural Building receives numerous deliveries from trucks of this size of artifacts and museum displays. The current door opening does not have an impact pad or any weatherization seals around the opening.

The recommendation for this loading dock area is to construct a new loading dock with the face of the extension out beyond the plane of the current limestone panel. With this new extension and opening, we would recommend a new dock pad and weather shield canopy to protect the loading and unloading operations during inclement weather. These new pads can also seal the loading dock area from the exterior temperatures during heating season.

Another important recommendation for this opening would be the installation of a dock leveler/dock lift to accommodate various truck heights and boxed displays during loading and unloading operations. The loading dock entry drive, drainage and grading should also be redesigned to allow for easier truck access.

Exterior Building Façade Investigation

LD1 - Existing Loading Dock Section Detail

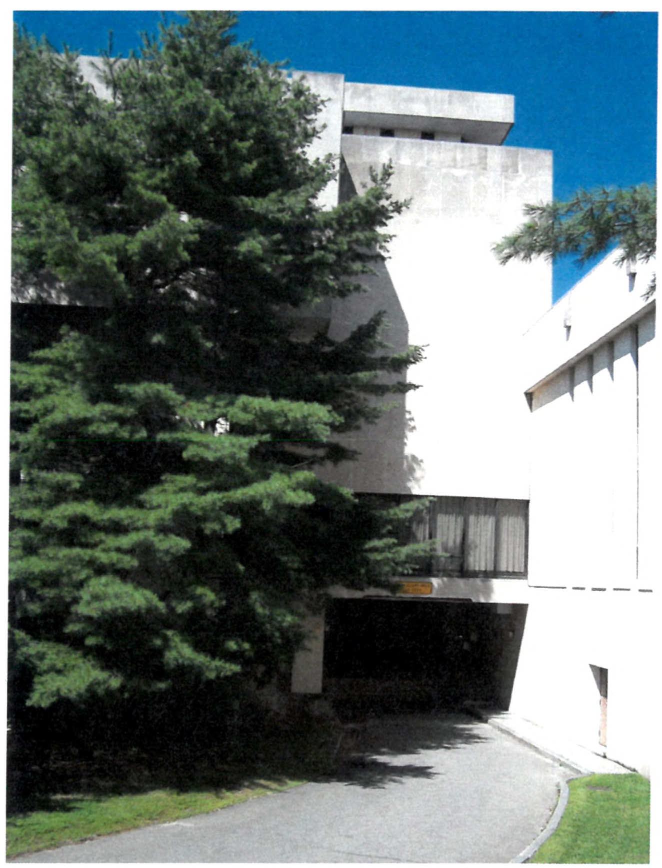


LD2 - Damage at panels over loading dock door

Exterior Building Façade Investigation

LD3 - Loading dock door entry

LD4 - Loading dock door entry



Exterior Building Façade Investigation

Exterior Doors (ED)

The existing exterior steel framed/aluminum doors are single-pane glazed openings in a steel framed curtainwall system. The opening on the south side has four single doors and the west side has three single doors. There is an additional single door opening on the south and west side of the building. All of these door openings are currently being utilized as emergency means of egress. At all these door openings the current hardware is beyond the point of repair and needs replacement. The weather stripping is also in poor condition. These openings have water and air leakage problems around the openings.

There are currently six existing door openings in the south side curtain wall system of which two openings are blocked; we question whether the blocked doors would be acceptable to code enforcement officials. The original design intent is that all these openings should be utilized for emergency egress.

The replacement system for these openings will have to be paired unequal leafs as stated by the life safety code for an existing condition and fully active and accessible at all time.

On the west side entrance there are three openings with all of these being used as an emergency exit. The entire existing steel frame and aluminum door system at this location will have to be removed in order for a new code-compliant emergency exit door system to be installed.

The single door openings on the west and south sides are also being utilized as emergency exit doors. These existing door openings are sized so that a new replacement door and frame will meet the minimum door width parameters as required by the current Life Safety Code.

Exterior Building Façade Investigation

ED1 - South wall emergency egress doors

Exterior Building Façade Investigation

ED2 - West wall emergency egress doors

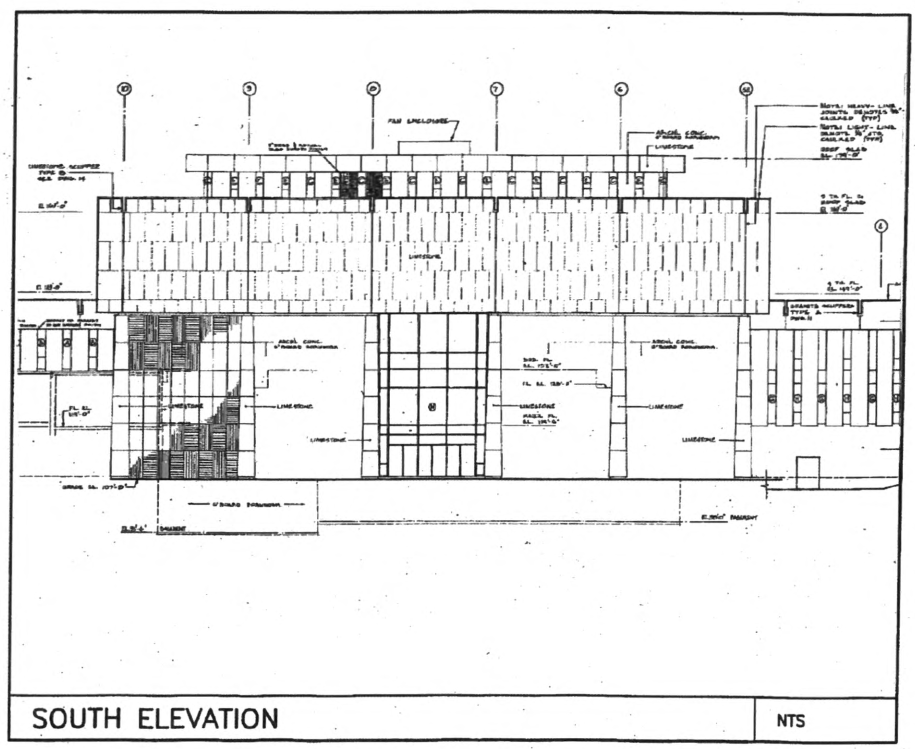
Exterior Building Façade Investigation

Structural Analysis

General

The area of the Maine State Cultural Building that has been considered in this investigation is the five-story Museum Wing of the building, and specifically, the limestone-clad perimeter curtain-wall of the fourth and fifth floors (Image S1). The existing structural framing and components that constitute the support system for the limestone panel veneer were reviewed by numerical analysis and field investigation, to determine whether there was a basis for failure of the stone panels founded on issues of structural inadequacy of the principal curtain-wall support system. Copies of most of the original construction drawings for the building were available, and were used as the primary reference for the structure of the building and its systems. A field investigation of the existing curtain-wall and its supporting structure was made. Individual limestone panels were removed from three locations along the base course of the curtain-wall, one each on the South, West and East elevations, and a section of the fifth floor roof parapet was opened at the south scupper of the east elevation.

Image S1 South elevation



Exterior Building Façade Investigation

Because the building is an existing structure, and it is not being added-to, modified, or being subjected to a change of occupancy, the current building code does not require that existing elements be in compliance with the current standards for new buildings. The code currently referenced by the State of Maine for new building construction is the 2003 edition of the International Building Code (IBC 2003). As an existing building, the design criteria used in the analyses to evaluate the existing structure were based on typical building code recommendations of model codes presumed to be in effect or available to the designers at the time of the building design. The dates found on the construction drawings indicate that the building structure was designed in 1968. Therefore, the National Building Code (1967) and, for comparison, the BOCA Basic Building Code (1970) were selected as the reference codes.

For the purposes of the numerical analyses, it was necessary to determine what original design load criteria were used by the Engineers of Record for the building structure. Typically, this data would be listed on the construction drawings; in the case of this project, much of the necessary information was unlisted on the available copies of the construction documents, and so was unknown at the time of this investigation.

For the structural analyses, the following material assumptions were used: Structural steel materials were ASTM A242 steel (Fy = 50 ksi) for structural shapes and ASTM A36 (Fy = 36 ksi) for angles and channels; field connections were bolted using ASTM A325 high-strength bolts in friction-type connections, or welded; concrete used for floors was 3000 psi compressive-strength light-weight concrete, and masonry block used for exterior veneer back-up walls was light-weight concrete masonry units.

These assumptions are based on the General Notes found on the available copies of the original construction drawings and the original project "Contract Documents and Specifications for General Construction".

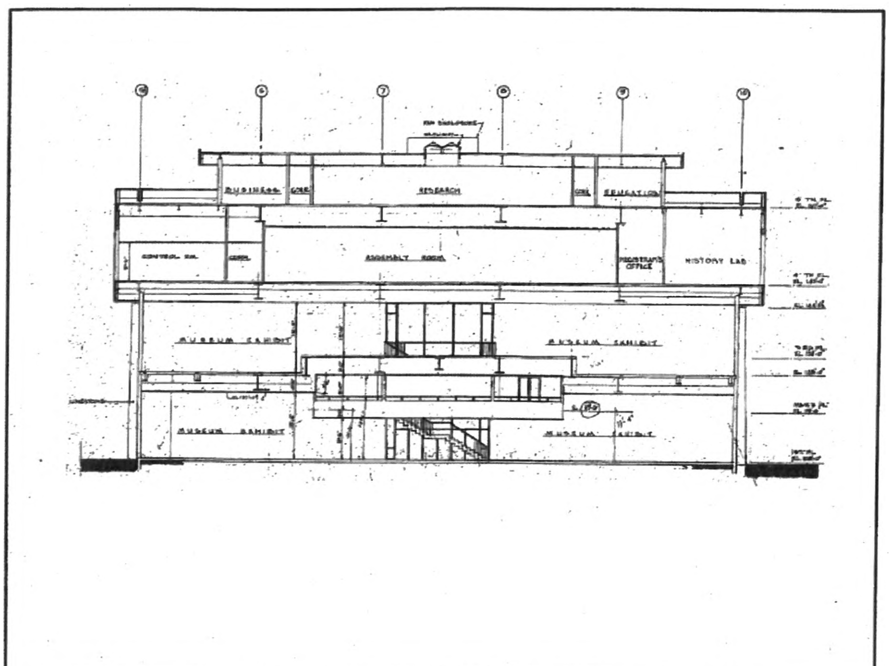
For the numerical analyses, imposed live load values used in consideration of the existing framing were conservatively taken to be the largest values for a given, anticipated occupancy, and no reduction of the live load values was utilized in the analyses.

Exterior Building Façade Investigation

The Building Arrangement

The overall construction of the museum wing of the building is a structural steel frame with light-weight concrete floor slabs cast over steel form decking. The fourth and fifth floors extend out beyond the boundaries of the third floor plan to provide an overhanging façade of limestone that cantilevers 4 ½ feet on the south, east and west elevations. To carry the floors and the exterior walls, steel girders cantilevering over the third floor level columns are the major support system for the total structure above. The occupiable space of the fifth floor is reduced in plan area to form a penthouse-like office space surrounded on all four sides by a parapeted roof area (Image S2).

Image S2 Cultural Building Cross Section



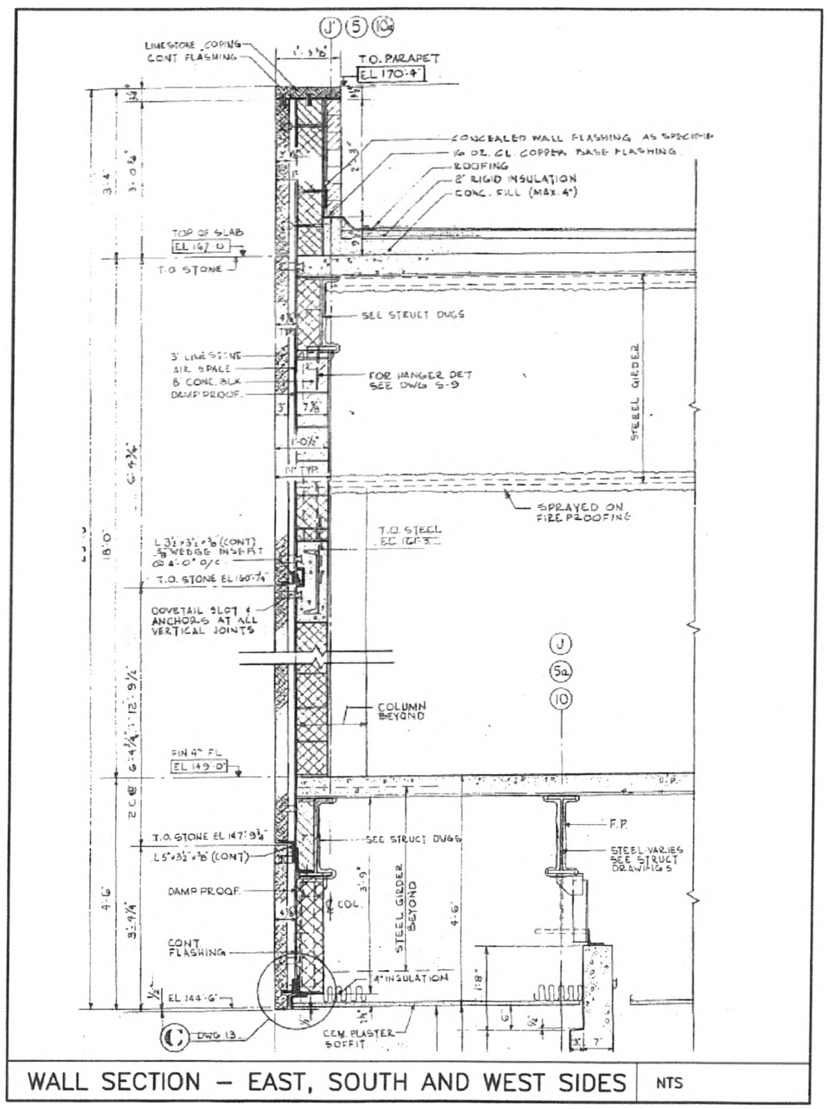
CULTURAL BUILDING – CROSS SECTION NTS

Three-inch thick limestone curtain wall panels are the exterior finish of the upper levels. The limestone curtain wall runs from the concrete soffit of the fourth floor overhang, up over 26 feet, past the level of the fifth floor/roof, and forms the parapet of the fifth floor roof perimeter. The weight of the limestone panels is supported by continuous steel relief angles that run horizontally around the perimeter of the wing. The relief angles are, in turn, supported by the steel framing of the fourth and fifth floors. The limestone panels are braced against lateral wind or seismic loads by 8" (nominal) thick light-weight concrete masonry back-up walls, to which they are connected, across an airspace of about 2" width by 3/16" diameter rod ties at a regular spacing.

Exterior Building Façade Investigation

Architecturally, the limestone panels of the curtain-wall are arranged in five horizontal courses of two depths: the bottom and top-most courses are short, measuring approximately 3'-4" tall at the base course and 3'-0" tall at the top parapet course. The three intermediate courses are each about 12'-8" tall. The widths of the panels vary, depending upon location, with the typical panel width being about 3'-0", while certain wide panels are more than double that width, at about 6'-8"or so. The base course and two lower intermediate courses are supported by relief angles at the steel framing of the fourth floor. Because the floor-to-floor height of the fourth to fifth floors is 18 feet, the upper intermediate course and the parapet course are supported by a relief angle line that is carried by a concrete-encased steel channel that is, in part, suspended from the fifth floor spandrel beams at about 6'-4" below the level of the fifth floor roof slab (Image S3).

Image S3 Wall Section-East, South and West Sides



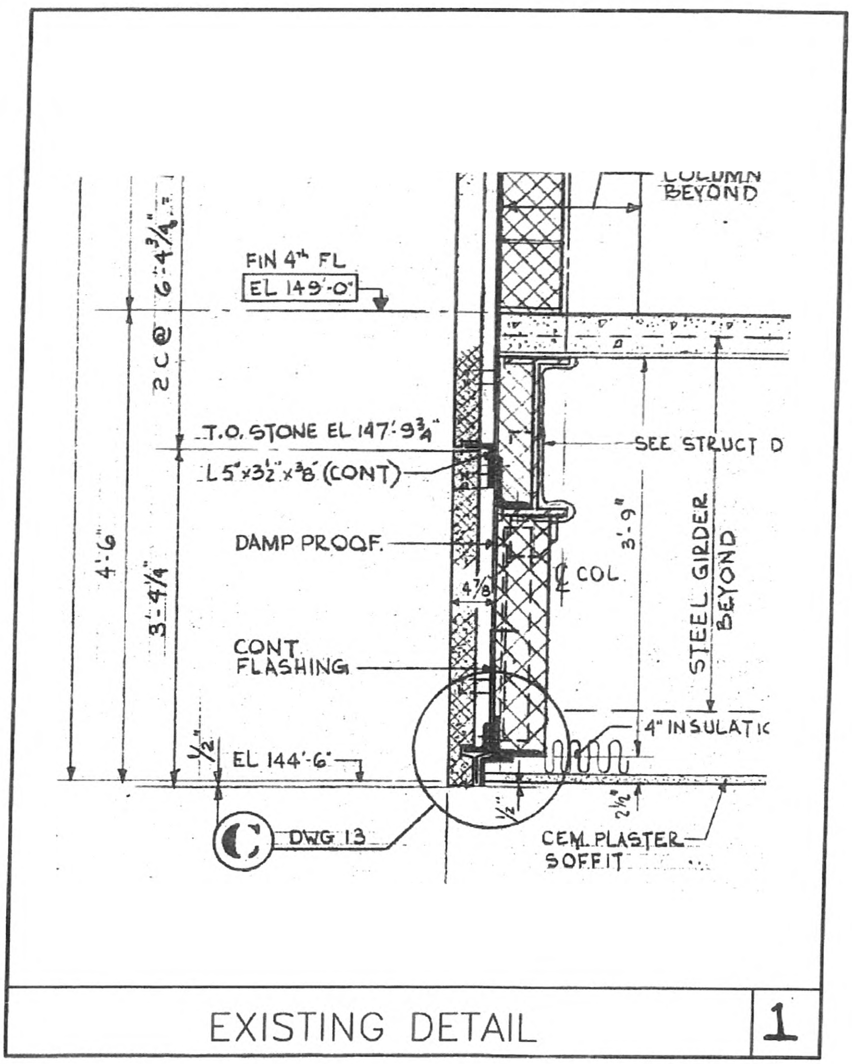
Exterior Building Façade Investigation

The Analysis

Relief angles:

Relief angles situated along the fourth floor level were the principal object of the field investigations. There are two lines of relief angle at this level, one which carries the weight of the 3'-4" bottom course of limestone, located at the bottom of the curtain-wall, and a second located above the base course, which carries the second and third courses of limestone. It is at the lower, first relief angle that some stone panels failed, spalling-off 4-to-6 inch by 3-foot long sections of individual stone panels. The second relief angle, however, carries the greatest weight of stone of all the levels, because it supports two stacked, tall panels of the intermediate courses of the curtain-wall, a nearly 13 ft. height of stone (Image S4).

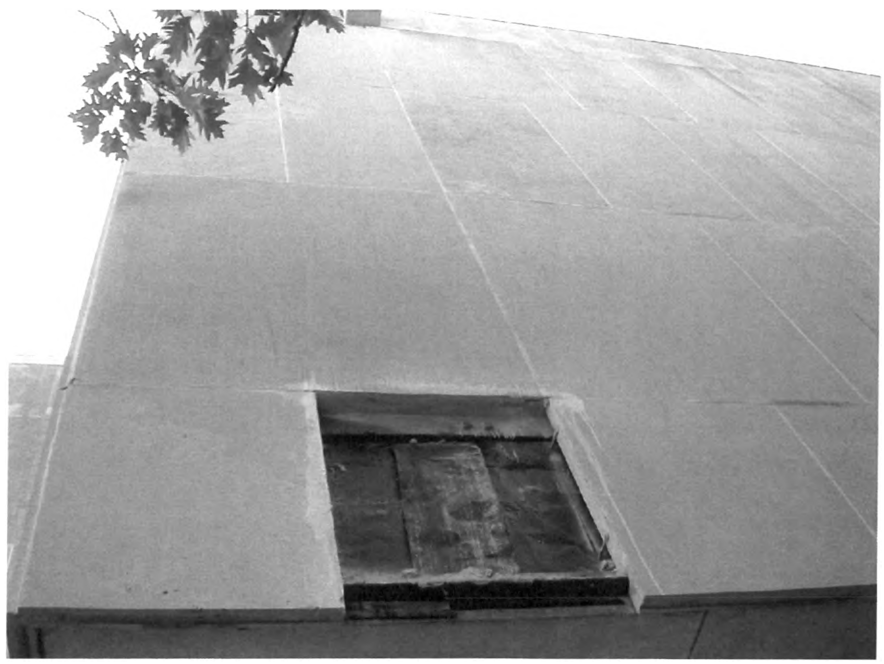
Image S4 Existing Detail 1



Exterior Building Façade Investigation

One of the most common causes of failure in stone curtain wall systems is deterioration of steel support angles by corrosion of the steel resulting from water intrusion behind the curtain wall panel veneer. In order to evaluate the extent to which this circumstance might be the cause of the stone failures, single-panel sections of the stone cladding were removed from the building at three locations of the façade, one each on the south, west and east elevations of the limestone curtain wall. These panels were located at the base course of the stone, at about the fourth floor level, where the worst effects would be expected to be found. Upon removal of the panels, it was discovered that the existing construction was built in accordance with the details of the construction drawings (Image S5). An inspection of the shelf angles and their hangers found that not only was there little evidence of damage to the steel of the relief angles, the condition of the shelf angles themselves was excellent, almost like new. The hangers showed some surface oxidation, but there was no significant deterioration noted.

(Image S5)



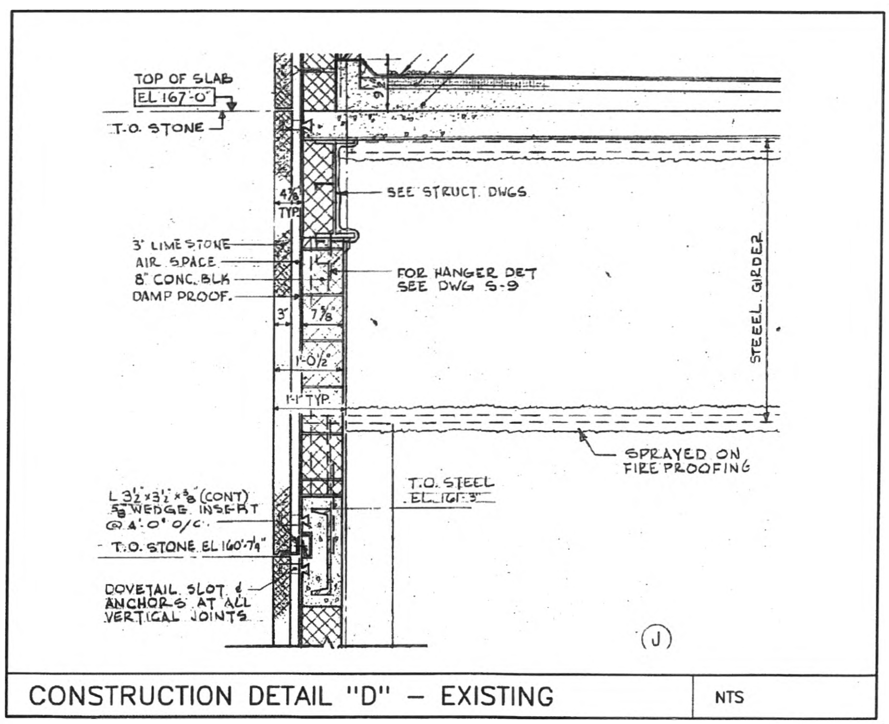
Often, damage to curtain wall panels can be attributed to movement of support framing due to slippage of bolted connections, such as those used to connect the relief angles to steel hangers or to the steel of the fourth floor. The lower angle is supported from the spandrel beams of the fourth floor framing by steel angle hangers spaced at 4'-0" on center. Close inspection of the angle connections was made to try to determine if there was any slippage notable in the bolted joints. No evidence of slippage was found.

Exterior Building Façade Investigation

Flexure of the relief angles along the run of the shelf was investigated. The typical relief angle of the fourth floor level is supported either by steel hangers or by steel clip angles, at a maximum spacing of 4'-0", according to the construction drawings. Based on this 4-foot span, a numerical analysis indicated that the existing steel angles were adequate in strength to support the limestone panels, and that expected deflection along the span of the relief angles was insufficient to cause damage to the stone.

The weight of the upper portion of the stone curtain wall veneer is, broadly speaking, supported by framing of the fifth floor and an intermediate-level beam which is composed of a steel channel-section member encased in concrete (Image S6). Relief angles supporting the stacked upper and the parapet courses of the limestone curtain wall are connected to the concrete-encased channel by the use of bolts, at 4-feet on-center, into wedge-type anchor inserts embedded into the concrete which encases the steel channel. An analysis of the upper level relief angle itself indicated that it is adequate for both strength and deflection criteria. There is a possibility of slippage to occur in the wedge-anchors, which would result in differential settlement and stress redistribution to the stone panels and their support, but there was no evidence apparent at the time of the field investigations to indicate that such slippage had occurred.

Image S6 Construction Detail D



Exterior Building Façade Investigation

Fourth Floor Framing:

The framing of the fourth floor perimeter consists typically of 18-inch deep steel I-section beams spanning 27 ½ feet along the exterior wall. These beams carry a strip of the fourth floor, about 2 ½ feet wide, as well as the weight of the limestone curtain-wall system and its masonry back-up wall. For the analysis of these beams, a live load for the tributary floor area was conservatively taken to be 150 pounds per square foot (psf). This load magnitude was chosen because one of the listed potential occupancies for the fourth floor spaces is "Collection Storage". Typical load values for "Light Storage" would be 125 psf, and for "Museums", 100 psf, but "Library Stack Room" design live loads are recommended to be 150 psf, minimum. Since collections of the facility could consist of library archives, 150 psf was selected as the minimum design value for this analysis. The construction drawings indicate that a "future" storage mezzanine was to be provided for, but apparently it was never constructed. The framing scheme for this mezzanine was not indicated on the documents, so it was not considered for this analysis.

The numerical analysis of the fourth floor spandrel beams indicates that the steel beams, used to support the perimeter wall and curtain wall, are of adequate load-carrying capacity. Checking for deflection of the 18-inch deep beams, under full floor loading, the calculations result in values that are greater than what would be considered appropriate for ideal performance of the curtain wall system. However, observations made during the field investigations seem to contradict this, as there was no evidence found to indicate that the actual deflections experienced have been of sufficient magnitude to cause displacement or damage to the stone panels of the curtain-wall or the masonry of the back-up walls.

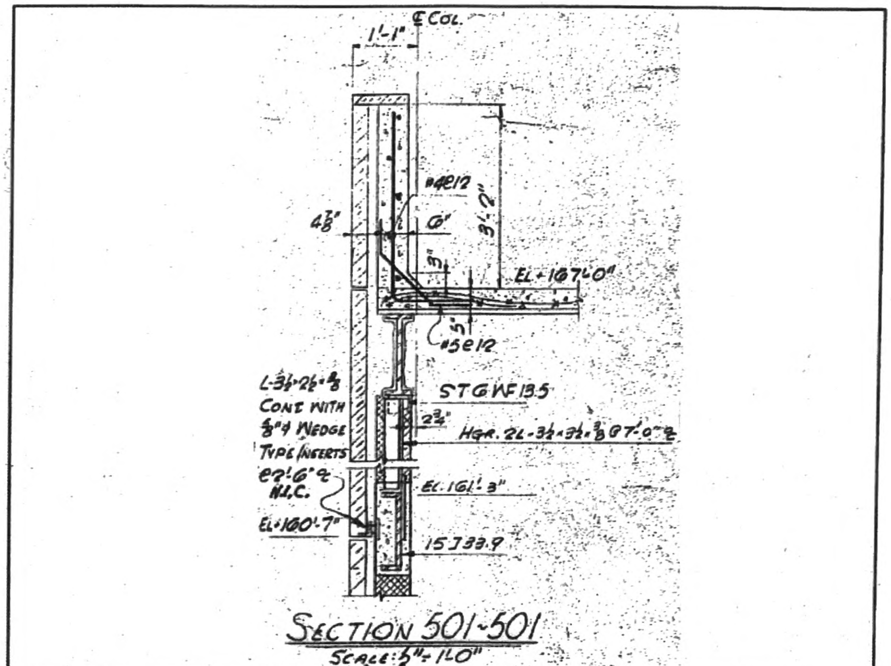
The use of high-strength steel for the building structure inadvertently contributes to the shortfall indicated by the numerical analysis for deflection: while the carrying capacity of the beams is increased for a given shape, allowing the use of smaller, lighter sections, the modulus of elasticity of the material does not increase, so the stiffness stays the same for equivalent shapes. Thus, the greater available load capacity of the smaller beams results in larger potential deflections, for a given steel shape. The actual deflections experienced would be the result of the actual loads imposed on the framing, and these values may never have reached the weight necessary to match that of the design capacity, with the corresponding level of deflection.

Exterior Building Façade Investigation

Fifth Floor/Roof Framing:

The framing of the fifth floor perimeter consists typically of 16-inch deep steel I-section beams spanning 27 ½ feet along the exterior wall. About 5 feet below the roof beams runs an intermediate level of 15-inch deep channel-section beams that are encased in concrete (Image S7). The roof beams and the intermediate channels are connected to each other by steel angle hangers at 7 feet on center. These beams carry a portion of the roof, as well as the weight of the limestone curtain-wall system and its masonry back-up wall.

Image S7 Construction Document Structural Parapet



CONSTRUCTION DOCUMENT STRUCTURAL PARAPET | NTS

For the analysis of these beams, the live load for the tributary roof area used for the original design is unknown. The reference building code does not specify a particular snow load value to be used for design. An attempt to back-calculate a live load resulted in a possible snow live load value of 40 psf. This value is less than the snow load values required for design by current codes for the Augusta area, and does not include the additional weight of snow resulting from drifting. The magnitude of the calculated load is, however, consistent with that of a snow load that might be recommended by some other, earlier building codes. Therefore, for the analysis of the framing of the fifth floor roof framing, a simple distributed snow load of 40 psf was used, to approximate the original design criteria for the structure. Because of the manner in which the framing of the fifth floor level was arranged, loads from the fifth floor interior floor spaces do not factor strongly in determining the capacity of the perimeter beams, and so these loads were not included.

Exterior Building Façade Investigation

The numerical analysis of the fifth floor spandrel beams indicates that the 16 inch wide-flange (WF) steel beams used to support the perimeter wall and curtain-wall are not, by themselves, of adequate load-carrying capacity, when the beams are considered to carry the full weight of the roof and upper walls. Checking for deflection of the 16-inch deep beams, under full floor loading, the calculations result in values that are greater than what would be considered appropriate for ideal performance of the curtain-wall system. An analysis of the 15-inch steel channels at the intermediate level between the floors, which support the upper curtain-wall relief angles and are connected to the steel spandrel beams of the fifth floor framing by steel hangers, shows that, if the channels are considered self-supporting, they are of sufficient capacity to carry the full weight of the upper curtain-wall and its masonry back-up. This suggests that, because the two levels of beams are connected, it may have been the intent of the designers that the WF16 and 15-inch channels work additively to support the upper portion of the curtain-wall and its back-up, as well as the roof loads. In order for this to be possible, it is necessary for the channel beams to be positively connected to the support columns at each end. The construction drawings available at the time of this review do not indicate whether this is, or is not, in fact, the case, but it would be likely that the channels were connected to the columns, if only to facilitate erection of the framing in the field. Because the channel beams are encased in concrete, the columns themselves are coated with a cementitious fireproofing and since the connection areas are not readily exposed to inspection due to the presence of interior and exterior finishes, it was not possible to verify that the channels are directly or sufficiently connected to the columns.

Observations made during the field investigations suggest that adequate stiffness is present in the existing construction. There was no evidence found to indicate that the actual deflections experienced have been of sufficient magnitude to cause displacement or damage to the stone panels of the curtain-wall or the masonry of the back-up walls.

Exterior Building Façade Investigation

Curtain-Wall Veneer Anchors:

Lateral support of the limestone panels, to brace them against dislocation by wind or seismic loads, is provided typically by the use of tee-plate anchors from the stone panels to the concrete masonry back-up walls or to other structural framing. These anchors consist of a 2" diameter x 2/23" thick steel disk welded concentrically to the end of a 3/16" diameter steel rod. The rod is threaded, and used in two (or more) lengths joined by sleeve nuts, to provide length adjustability. The disk of the anchor is set into a dado slot cut into the edges of the stone panels, allowing the rod of the anchor to run through the panel joint to the back-up wall or structure, where it is anchored, either by embedment into the masonry or by other mechanical fastening (Image S8).

Image S8

Exterior Building Façade Investigation

Failure of panel anchors can allow excessive movement of the stone panels, which can result in overstress in the stone and failure of the stone panel. Also, the movement of the panels causes cracks to form in the mortar joints between panels, resulting in failure of the building envelope, increased intrusion of water or moisture, and possibly leading to corrosion and failure of other structural or architectural components of the building.

According to the construction drawings, the spacing of the anchors was to be at 3'-0" on center maximum spacing, with a minimum of two anchors at the top and bottom of each panel. The architectural drawings indicate the use of strap-type anchors, which differs from the tee-anchors actually used. The condition of anchors observed during the field investigations was typically good. The anchors used were of galvanized steel. Some of the anchors had minor oxidation and surface rust on the threaded parts, but not to a structurally significant extent. Current industry recommendations are that stainless steel anchors should be used for this application.

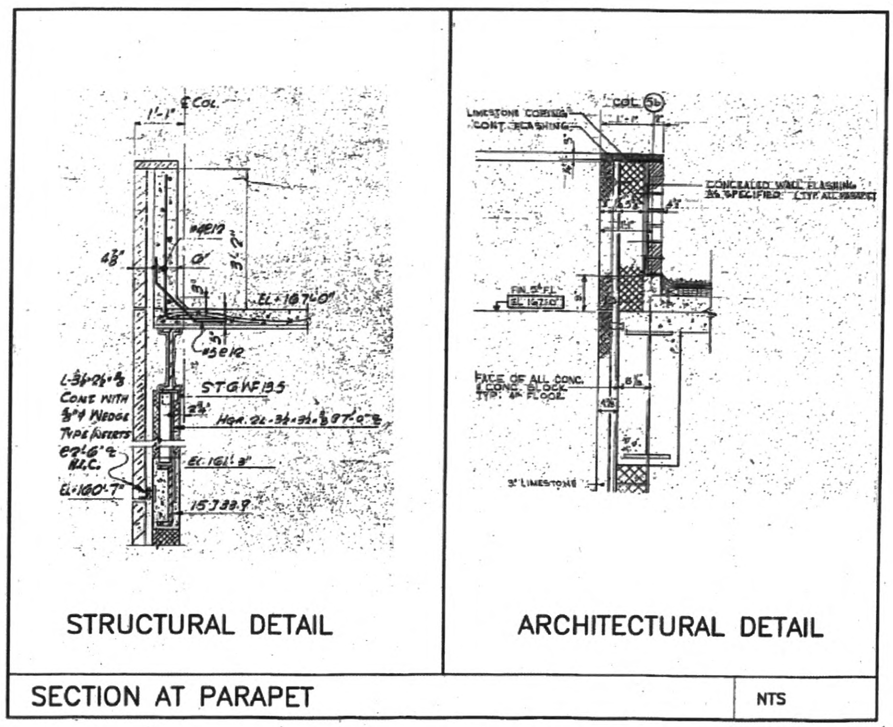
A numerical analysis of the ties actually used indicates that, for the case of the typical limestone panel, the capacity of the 3/16" tee anchors is sufficient to properly support the limestone panels against wind or seismic load, if the minimum anchor spacing is provided, i.e. two anchors each at top and bottom of each panel. The analysis also suggests that additional anchors are necessary to support the wider, 6-foot panels. The presence of additional anchors for these wider panels could not be verified. The analysis assumes that the connection of each anchor to the back-up structure is substantial enough to develop the full capacity of the individual anchor.

Exterior Building Façade Investigation

Parapets:

A section of the fifth floor roof parapet was opened on the east elevation, near the southern-most roof scupper, to inspect the existing condition of the construction for parapets and their scupper detail. The design structural drawings indicate that the structural parapet wall, which extends vertically about 3 ½ feet above the level of the roof slab and acts as back-up support for the limestone curtain-wall, was to be constructed using a cantilevered cast-in-place concrete wall. In contrast to this, the architectural drawings indicate that the back-up wall for the parapets was to be of concrete masonry. When the stone coping of the parapet section was removed, it was discovered that the structural wall was not constructed in accordance with the structural details, but rather was constructed according to the details of the architectural sections i.e., constructed of concrete unit masonry (Image S9).

Image S9 Section at Parapet



During the field observations, it was found that the masonry back-up wall could be displaced laterally by a man pushing at the top of the wall. This suggests that the masonry of the back-up wall may not have been positively dowelled to the concrete roof slab, despite such direction shown on the structural drawings, and that, therefore, the stability of the roof parapets is based simply on the mass of the masonry there. The result is that the entire, existing parapet masonry wall is not a reliable structure for lateral support of the limestone slabs of the parapet curtain-wall, and that there is a potential for failure of the parapets under large, dynamic wind loads or the occurrence of a seismic event. Essentially, the

Exterior Building Façade Investigation

limestone curtain-wall, which is intended to be "non-structural", and supported by the masonry parapet wall, now works in combination with the weak masonry wall to support itself. This is an unacceptable condition from a safety standpoint, and also compromises the expected service longevity of the construction.

Limestone Panels:

The limestone panels that are the curtain-wall are cut into 3" thick slabs. The typical panel dimension for the first, bottom course is approximately 3 ft. wide x 3'-4" high. The typical intermediate course panels are 3 ft. wide and 6'-4" high. Some of the intermediate course panels are double-width, near control joints and the building corners, to suit the architectural panel spacing scheme. The backs of the bottom course panels have a continuous, horizontal dado-cut recess at about 4" above the bottom of the individual slabs, to form a continuous bearing seat that receives the out-standing leg of the lower relief angle, and to provide for architectural concealment of that support and a boundary for the finished soffit of the building cantilever. The depth of the cut is about 1 ¾" into the back of the panel, resulting in a remaining thickness of stone of only 1 ¼" out of the total 3" thickness. Failure of several of the first course limestone panels has occurred because horizontal cracks formed along the dado recess, causing the bottom 4" of the panel extension, below the relief angle seat, to separate and fall off (Images S10 & S11). Due to a history of this type of failure, Indiana Limestone Institute of America, Inc. (ILI) does not recommend the use of this design detail for limestone curtain-wall panels.

Image S10

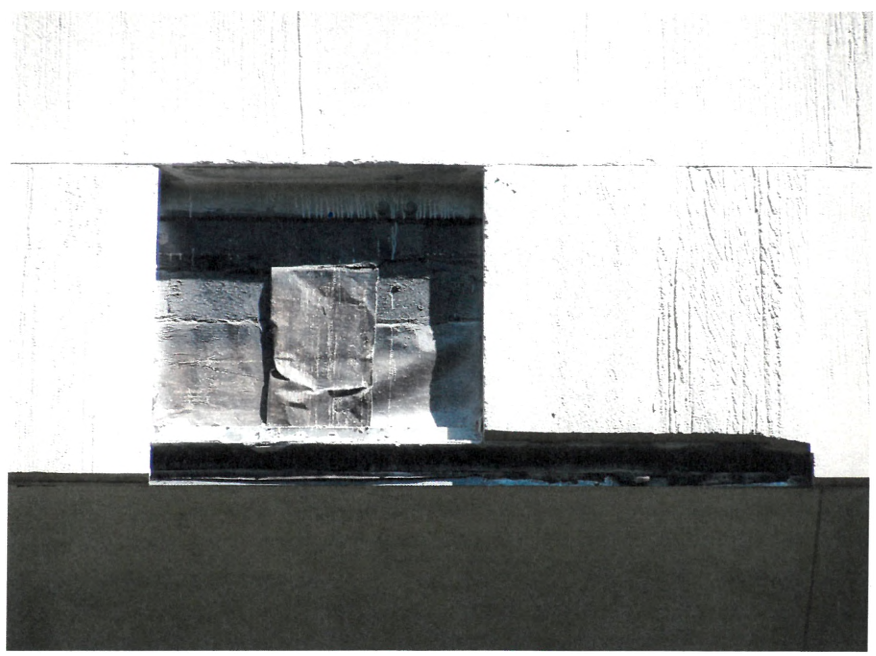


Image S11

A review of the limestone panels was made, with reference to "Recommended Standards and Practices for Design and Construction" from the "Indiana Limestone Handbook" by the Indiana Limestone Institute of America.

The thickness of the limestone panels is consistent with the ILI recommended minimum panel thickness for wind loads at the time of the original design. More recent building codes may call for higher wind load magnitudes, depending upon location of the panel on the building, which might suggest that a thicker panel would be more suitable, but the field observations indicate that, for the typical conditions, the 3" thick panels have been adequate, to-date.

Typical panel sizes are consistent with the recommendations of ILI, although the size of the double-width panels is larger than the recommended size for 3" thick panels. The thinness of the wider panels in proportion to the total panel size would have been problematic for shipping and handling during construction, but now that the panels have been installed, if there are sufficient numbers and appropriate location of curtain-wall anchors, this relative thickness is less important.

Exterior Building Façade Investigation

Limestone panels of the third course stack onto the second course panels, which are carried by continuous relief angles at the level of the fourth floor, resulting in a stack height of 13 ½ feet of stone. The parapet course panels are stacked onto the fourth course of panels, supported by continuous relief angles at the level of the intermediate channel-section beam, resulting in a stack height of 9 ½ feet of stone. The first course of the curtain-wall panels is independently supported at the base of the curtain-wall by the lower relief angles of the fourth floor level, and has a stack height of 3 feet. For three-inch thick limestone used in curtain-wall applications, ILI recommends a maximum stack height of 20 to 25 feet. Therefore, the stack height of the curtain-wall panels is within the recommended parameters of ILI.

For typical panels of the curtain-wall, lateral anchor support patterns, as described by the structural drawings and as found in the sample locations during the field investigations, appear to be consistent with the recommendations of ILI. Material thicknesses for the supporting legs of the steel relief angles also are consistent with the ILI recommended minimums. However, the presence of additional anchors necessary at wide panels could not be verified.

Exterior Building Façade Investigation

Probable Cost Options

1. Replace exterior doors
   $30,000 to $40,000 construction cost

2. New parapet structural backup and remove and replace panels
   approximately 540 linear feet of parapet system
   $300,000 to $400,000 construction cost

3. Remove top limestone panels and back-up with temporary cap
   approx 540 linear feet of parapet system
   $150,000 to $200,000 construction cost

4. New loading dock and entrance drive
   $550,000 to $650,000 construction cost

5. Remove and replace the limestone panels – new building envelope
   approximate 14,000 square feet of limestone panels
   a. Metal faced skin with air barrier and insulation
      $2,000,000 to $2,250,000 construction cost
   b. Terra cotta rain-skin system with air barrier and insulation
      $1,400,000 to $1,650, 000 construction cost
   c. Petrac rain-skin panel with air barrier and insulation
      $1,250,000 to $1,350,000 construction cost

Note:
Replacement of the limestone panel skin with a new envelope will have an impact on the potential of upgrading the structural system. This upgrade may be required depending on the new material system selected. A structural building frame analysis will be required before a total construction cost can be applied to a new envelope system.

Exterior Building Façade Investigation

Conclusions / Study Recommendations

Since there is no calculable way to predict when the bottom edge of the façade panel will break off, Harriman recommends that the Museum take appropriate measures block off bottom edge of the east, south and west façade to avoid personal injury to people below.

- The bottom limestone panel with its continuous cutout behind is not considered good design practice and will continue to break-off over time without a reconstructive correction to the support angle detail and replacement of the bottom row of panels.
- The majority of the limestone field panels are in relatively good shape and appear as they were installed. The staining of the panels can be cleaned thru a shot blast or sandblast process but will reappear over time unless the water entry problems or the air-conditioning envelope issues are addressed. The panels do have a considerable lifespan available to them.
- The current parapet system at the top of the museum – backup and panel needs to be corrected as soon as possible. The lateral structural support issue is of a major concern and if allowed to remain as is may lead to a major failure with in a short time.
- The forensic removal of the panels and parapet cap has revealed that the exterior envelope system of the museum, library and archives was constructed as detailed thru the construction documents.
- The structural frame and the metal panel support system is in good shape and the water entry issues has not deteriorated the frame and its support due to rusting.
- Deflection of the steel frame due to the weight of the backup and the panel system is not a concern relative to the limestone breaking.
- The building envelope cap details and its lack of a proper flashing and weep system is allowing water entry into the panels to remain and not dry out.
- The original contract specifications called for caulked joints around all panel joints. The existing condition has grouted joints and no sealant. Panels are basically locked together without any allowance for movement. Moisture in the cavity is being drawn out thru the joint and is a contributor to the staining on the face.
- The inability of the air-conditioning system to maintain proper humidity levels in the museum is directly related to the lack of an air-vapor and insulation system in the exterior envelope. The lack of this system in building envelope is the cause for all the energy loss and efficiencies and humidity control problems that the Museum is currently experiencing.

Exterior Building Façade Investigation

- Loading Dock area should be corrected to offer a more controlled loading and unloading process for the museum and its artifacts.
- Exterior doors should be replaced for energy and security improvements.
- Recommend extensive tree trimming and removal on three sides of the building. Moisture and grime from trees are contributing to the staining on the panels.
- The lead connector strip at the parapet should be addressed as hazardous abatement when the parapet is reconstructed.

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