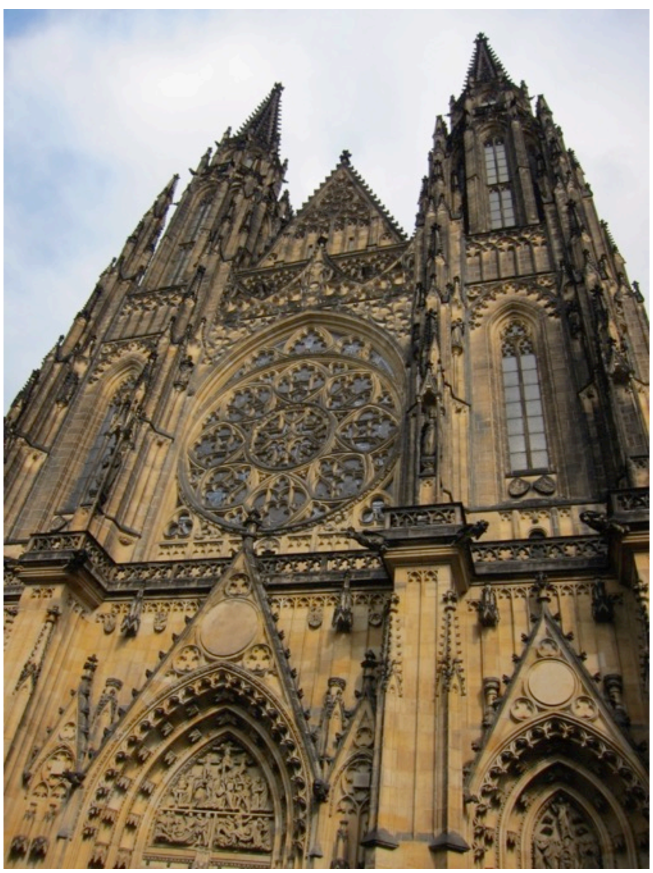
About the Author

I, Paul Lukes, consult in building exterior enclosure systems, from the roof to the sub-grade, and have been focused in this area for over 30 years; as a practicing architect for over 12 years prior to that, with a few years overlap; and with significant hands-on construction experience starting in my late teens and ongoing currently, or roughly 45 years of total construction industry experience, ranging from site laborer to architect and builder, eventually focusing on building enclosure systems, or building envelopes. During this time, I have consulted on roughly 800 projects, ranging from ultra-high-end residences to brand new, $ 200,000,000 medical centers.

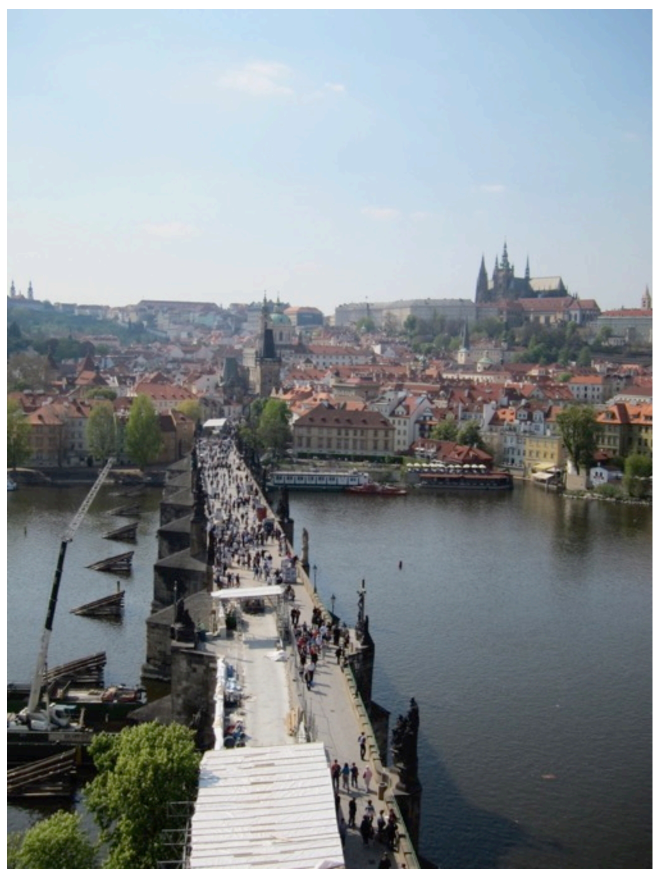
In fact, I believe that I had coined the now-ubiquitous term “building envelope” when re-naming my firm in the 1980’s to more closely align with my focus. While I had planned to call myself simply “The Envelope”, a client cautioned me that I may be confused with a stationery business, so I changed it to “Building Envelope” with his concurrence. This was my first time hearing the term, and I believe that is how it was born, and my limited search has found no earlier use. Unfortunately, due to a profusion of other firms now using “building envelope” in their names, and to avoid the resultant recurring confusion, I added my personal name to my business name to re-distinguish my firm, wishing my reputation be neither harnessed nor damaged by such confusion, so my firm is now “PAUL LUKES: Building Envelope Consulting Services LLC”, or PL:BECS for short. I have no affiliation with any other firm with “Building Envelope” in its name.

Though I consult in all aspects of building enclosure systems, I have particular fondness for and familiarity with masonry, dating back to my 4th birthday, when I asked for, and received, a brand new brick as a gift, then carried it proudly, unwrapping its wax-paper covering to show it off to my friends. Growing up the first 11 years of my life in Prague, a city of incomparable beauty as well as replete with masonry buildings of all types, hundreds dating back 600 years, some even a thousand, only cemented my spiritual bond with masonry, as well as affording me the opportunity to observe how various masonry elements weathered, not over 40-50 years, but centuries. Thus I believe my career began.

Please visit my web-site, plbecs.com, or request a firm brochure for additional information concerning PL:BECS.



St. Vitus Cathedral, 1300 hundreds, Prague



Charles Bridge, 1300 hundreds, Prague

Re-Cladding the Alaska State Capitol:
A Case Study in Rational Historic Preservation

Alaska Capitol Re-Clad in Progress.
New, seismically upgraded, re-clad portion is left of corner, original building is right of corner.

Primary Team Members:

Client:	State of Alaska, Legislative Affairs Agency
Architect:	Jensen Yorba Lott Inc.
Structural Engineer:	Swenson Say Faget Inc.
Building Envelope Consultant:	Paul Lukes: Building Envelope Consulting Services LLC.
Electrical Engineer:	Haight & Associates Inc.
Mechanical Engineer:	Murray & Associates P. C.
General Contractor, Portico:	Alaska Commercial Contractors
General Contractor, Building:	Dawson Construction Inc.

Introduction

This article concerns the still ongoing re-clad and seismic upgrade of the Alaska State Capitol, scheduled for completion at the end of the 2016 construction season, a project I have been involved with over the course of 11 years.

One of my primary motivations in preparing this article is to advocate for rational historic preservation of notable buildings, which I hope this project illustrates ever-so well, rather than insisting on rigid adherence to absolute replication of the original design without regard for possible technical errors in it, and insistence on re-using existing elements, regardless of how badly degraded these may be.

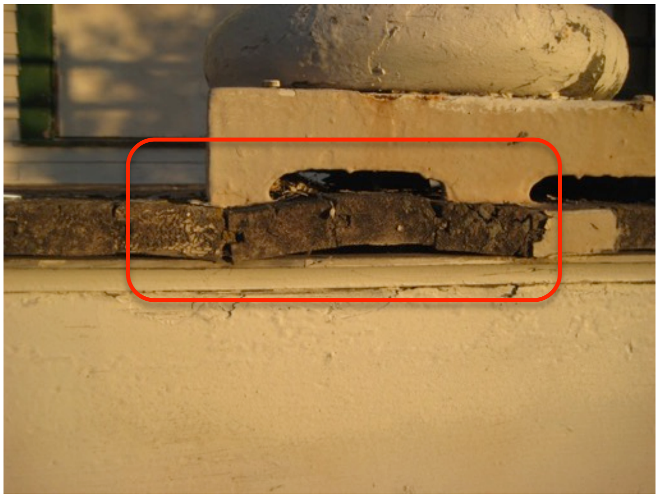
Along the way, I hope to briefly touch upon related technical subjects as illustrated by this project, such as:

1. what kills masonry,
2. how buildings get wet and how this affects, (or should affect) building envelope configuration,
3. accommodation of drainage from masonry claddings,
4. accommodation of venting from masonry claddings,
5. accommodation of dimensional variation in the cladding support system,
6. accommodation of inherent movement resulting from thermal fluctuations, curing, moisture variations, etc.

Let me begin by explaining what I mean by rational historic preservation. Over the course of my now-lengthy career, I have on many occasions worked on projects of historic significance, where Historic Preservation Boards governed any proposed work on the buildings, in particular work affecting their exteriors. In my experience, such boards have at times insisted on absolutely unwavering adherence to the original design, even when that original design, wonderful and beloved though it may deservedly be, makes serious technical errors which plague the resulting buildings and their occupants and owners. This very approach seems to be based on the idea that architects of 80 years ago were somehow infallible demi-gods, all knowing of all fields in building, that builders of 80 years ago similarly built to utter perfection. Of course, this is not so, as we all make the occasional error, and no architect can be expected to know in detail every aspect relating to construction, ranging from fire exiting requirements to accommodation of thermal movement in claddings.

In one such project involving several historic Seattle Carnegie Libraries experiencing widespread degradation of their aged wood windows, the preservation boards insisted that decayed portions of individual window frame members be replaced by cutting-out the rotten portions, then gluing-in new sections of wood to replace these. Due to inherent cross-grain movement in the wood, such patched-in pieces will rip any paint coatings applied to these frames, leading to moisture intrusion and fungal decay, re-starting the failure process immediately, and forcing ongoing costly maintenance, while always appearing somewhat degraded. Further, this approach was vastly more costly than re-fabricating and installing exactly matching wood windows, which would have afforded a new lease on life, to last many decades into the future, which would also have allowed retrofitting some concealed sub-sill flashings to resolve the leakage these libraries were suffering from the leaky original windows, which lacked such flashings. I do not believe this approach served to benefit these buildings, nor their owners, nor the general public.

On another historic project, the original design placed decks of tightly-fitted tongue & groove decking, (T & G), in weather-exposed locations, with the predictable result of the wood decking buckling up and lifting column bases bearing on this decking, again due to cross-grain expansion of the wood in this location's wet climate. This was merely a technical error, one I suspect the original architect, given the chance to see the results, would opt to change. Yet, the preservation board insisted that this error, one relatively easily solved with very limited visible change, be duplicated exactly, dooming the new decks to the same failure mode within months of replacement.



Buckling Decking Under Columns Resulting From Cross-Grain Expansion

It is my contention that buildings of historic significance are best preserved by using judgment to maintain or duplicate the building's original appearance, while taking the opportunity to enhance performance and correct technical errors plaguing these buildings. We already upgrade historic buildings structurally to enhance the survivability of their occupants and of the buildings themselves in earthquakes, and retrofit insulation to enhance energy-efficiency, for example. Correcting for technical errors, and enhancing a historic building's enclosure performance, particularly where such enhancements can be largely concealed and not visually distracting from the original design, similarly makes overwhelming sense in my opinion.

I believe that we should similarly be willing to make often concealed modifications to resolve leakage to the interior, to slow-down cladding degradation, to enhance energy-efficiency, etc., rather than blindly duplicating every aspect of the original, including its technical errors, and it is my hope that the recladding and seismic upgrade of the Alaska State Capitol, described in this article, will help illustrate this.

Phase 0: Project History

The Alaska Capitol building was designed as the Federal Territorial Building in 1929 and completed in 1931, just as the Great Depression was in its infancy. It consists of a concrete frame structure, with some riveted steel girders at the house chambers, and with multi-wythe masonry infill walls, which include brick, limestone, granite, marble, as well as colorful terra-cotta elements.

Fig. 0.2: Concrete Skeleton Under Construction, 1930

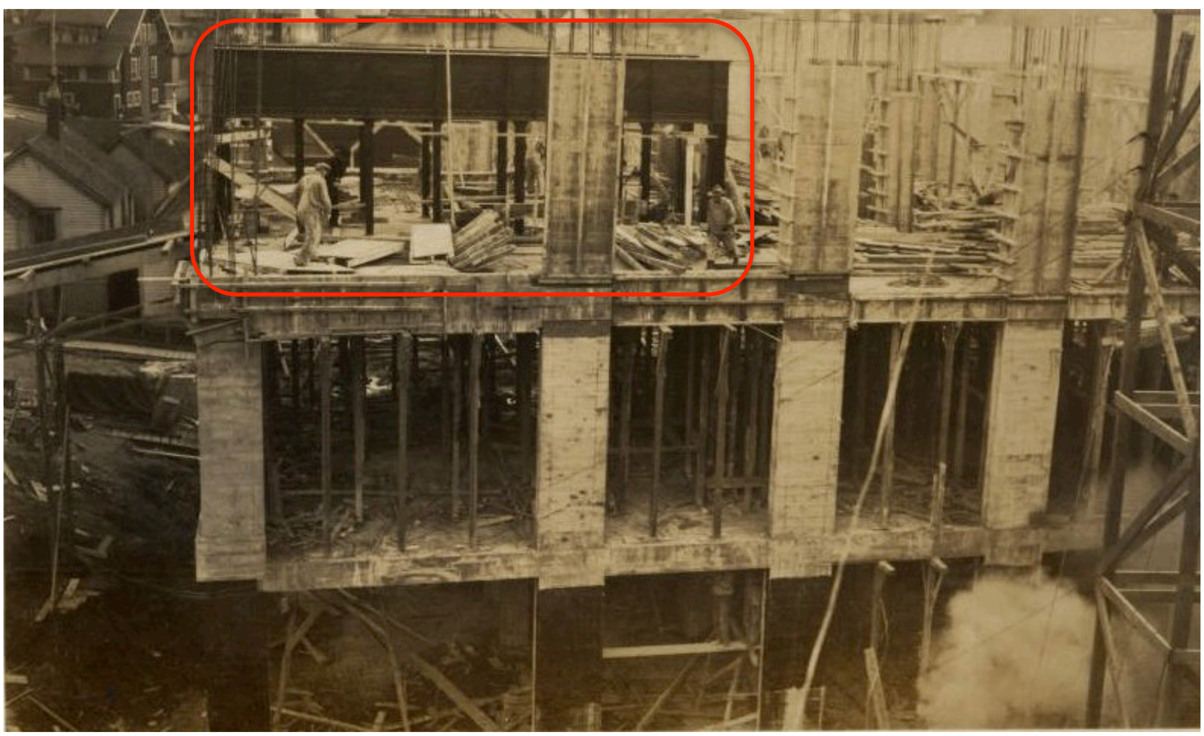


Fig. 0.3: Riveted Steel Girders in House Chambers

Fig. 0.4: Masonry Exterior Walls Under Construction, Using Wood Scaffolding, 1930

Fig. 0.5: Completed Building, 1931

Amazingly, the building's entire complex exterior masonry construction was largely described on 2 1/2 drawing sheets, each densely packed with information and artfully arranged. For perspective, 52 drawing sheets were required to define the current re-cladding.

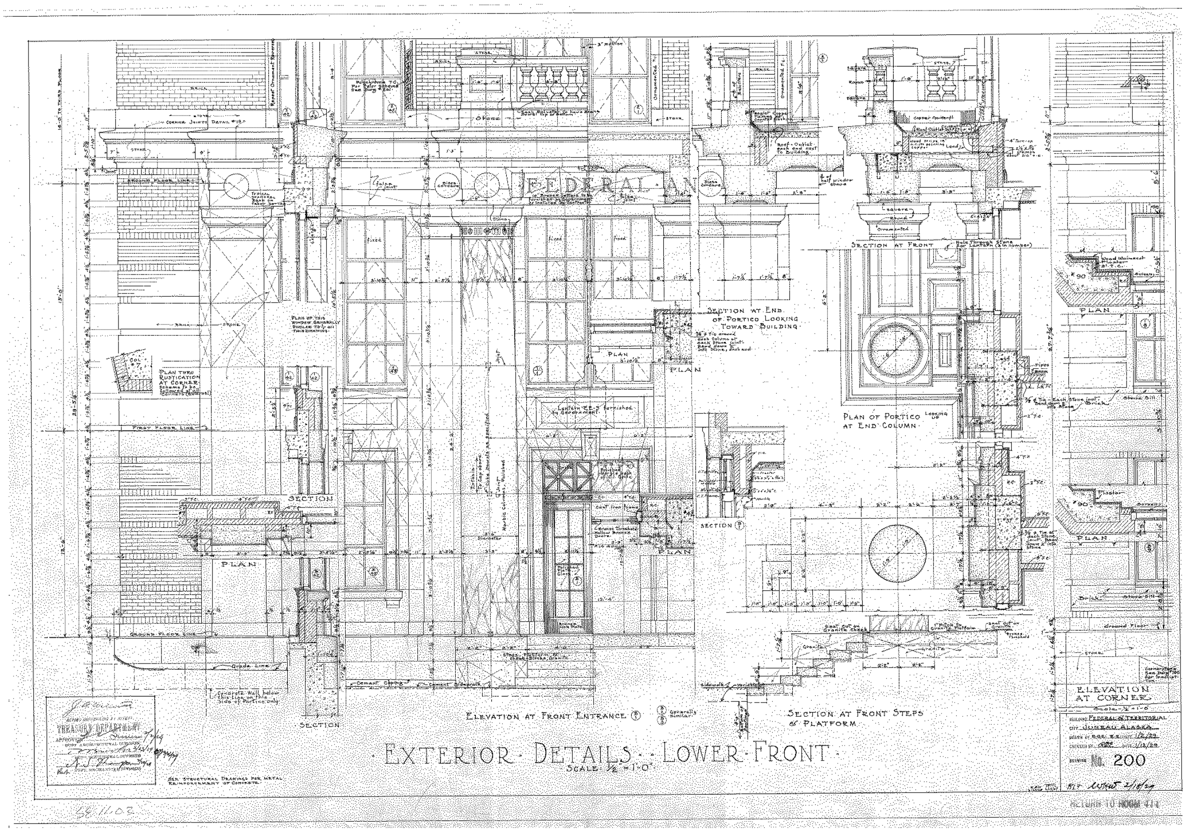


Fig. 0.6: Information-Packed Sheet 200 of the Original Construction Drawings, 1929

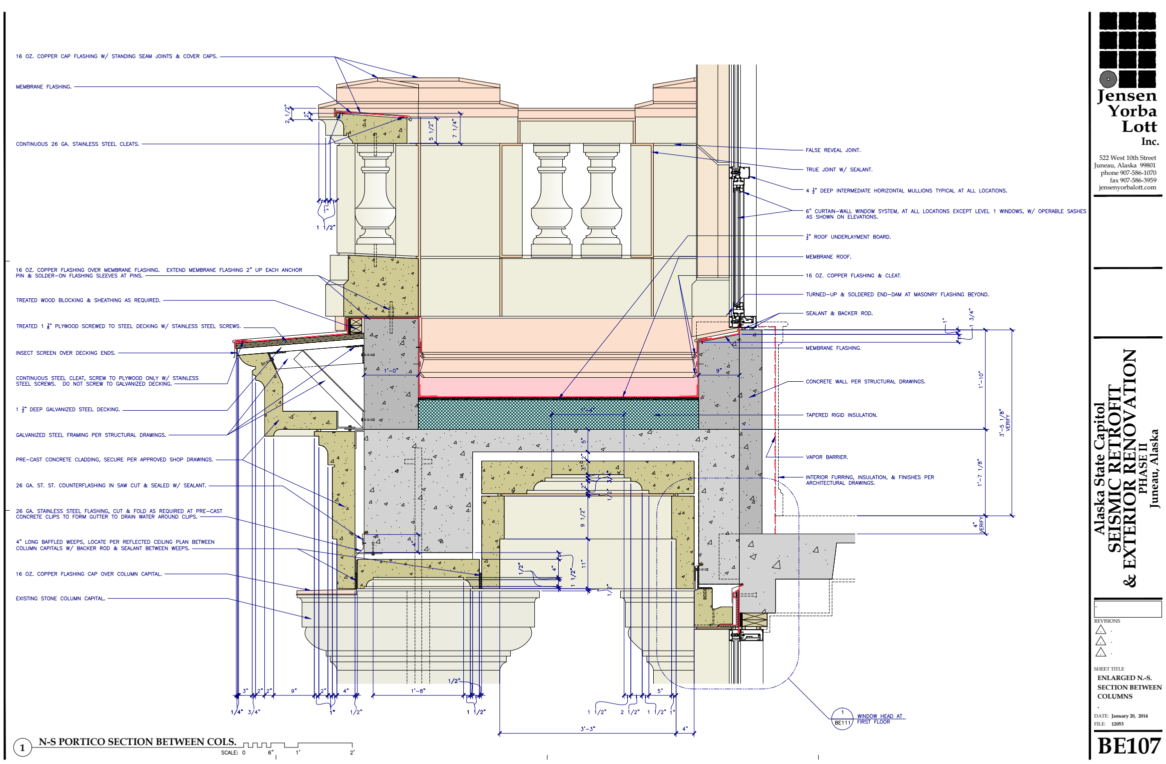


Fig. 0.7: Retrofit Design Drawing, Portico, 2015

Phase 1: Initial Investigation, 2006

My involvement with the building began in 2006, when I was asked to take a brief look at its exterior masonry and provide a verbal summary.

This examination revealed unexpectedly severe masonry degradation and cracking, relatively widespread leakage to the interior, spalling of the stone cladding resulting from anchor corrosion, among many other symptoms. The brick itself appeared very rough as if sandblasted, and contained various cracks, some extending over 10 feet. In short, the building's exterior masonry was in rather poor condition, particularly in view of the building's relatively young age.

Further, the stone entry portico appeared to suffer both severe water infiltration and associated damage as well as seemingly dangerous cracking of stone beams supporting the portico's roof.

These issues were brought forth in my summary, which recommended that the portico in particular be more closely evaluated due to its seemingly dangerous cracking and degradation.

Fig. 1.1: Degradation of Stone Base

Fig. 1.2: Degradation of Stone Base

Fig. 1.3: Brick Spalling

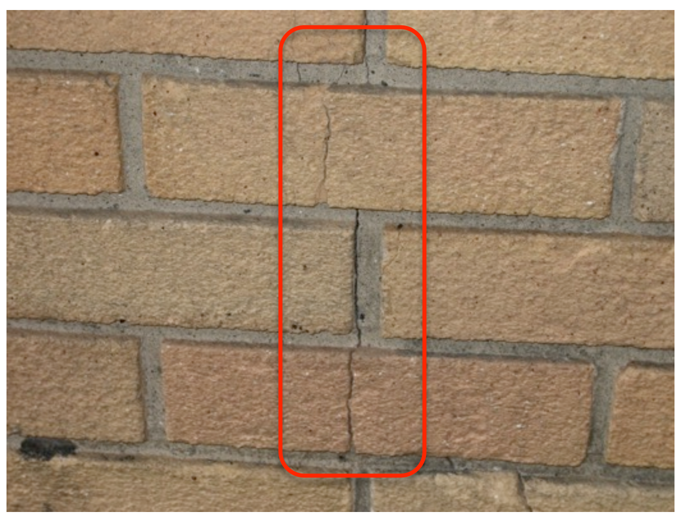


Fig. 1.4: Brick Erosion & Cracking

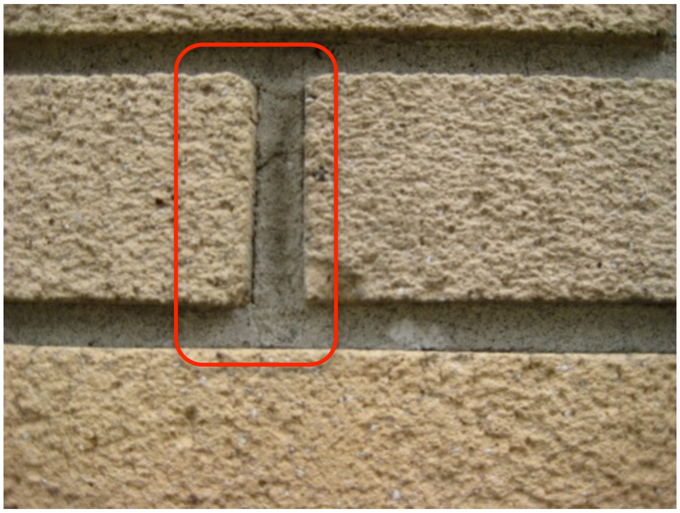


Fig. 1.5: Erosion, Mortar Delamination

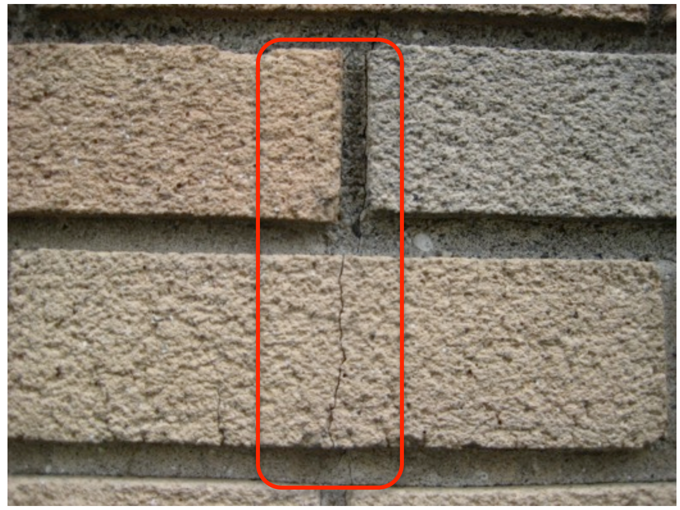


Fig. 1.6: Brick Erosion & Cracking

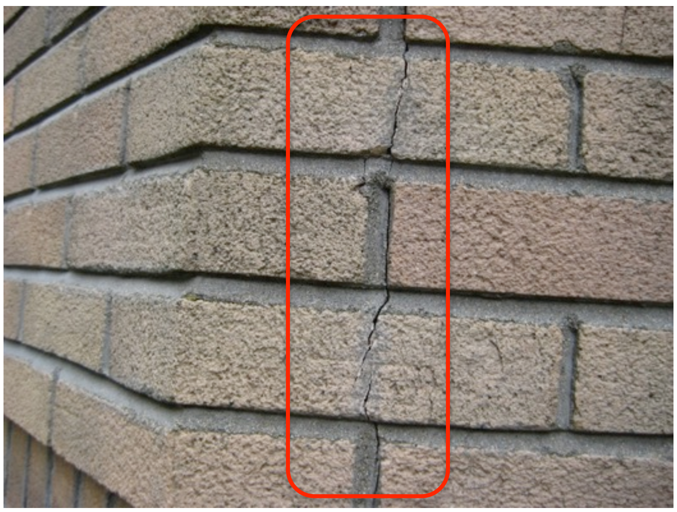


Fig. 1.7: Brick Surface Erosion & Cracking

Fig. 1.8: Brick Surface Erosion

Fig. 1.9: Spalling Stone Cornice Band

Fig. 1.10: Spalling Stone Cornice Band

Fig. 1.11: Water Infiltration to Interior

Fig. 1.12: Water Infiltration to Interior

Fig. 1.13: Water Infiltration Into Portico Clg.

Fig. 1.14: Water Infiltration Into Portico Clg.

Fig. 1.15: Cracking of Stone Portico Beams

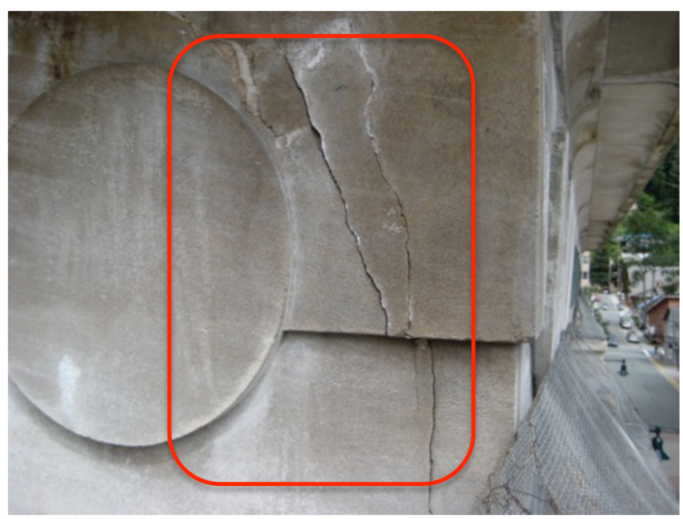


Fig. 1.16: Cracking of Stone Portico Beams

Phase 2: Portico Evaluation, 2010

My next opportunity to see the building came in 2010, when I was asked to take a much closer look at the portico and provide a report of Observations, Analysis, and Recommendations for this particular element.

This confirmed my earlier concerns about the portico, whose multi-ton stone structural beams and adjacent window headers were seriously cracked, seemingly vulnerable to collapse in a seismic event. Double-fist sized chunks of stone had already spalled off in the past, apparently from seismic events, onto the granite floor of the entry below.

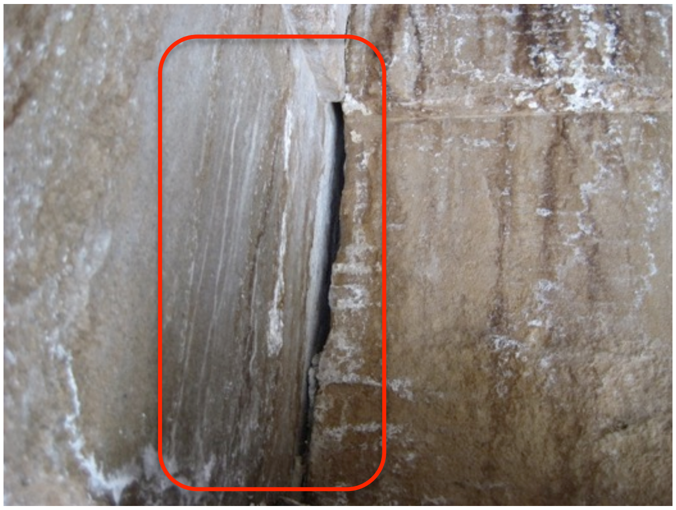


Fig. 2.1: Separation of Stone Beams

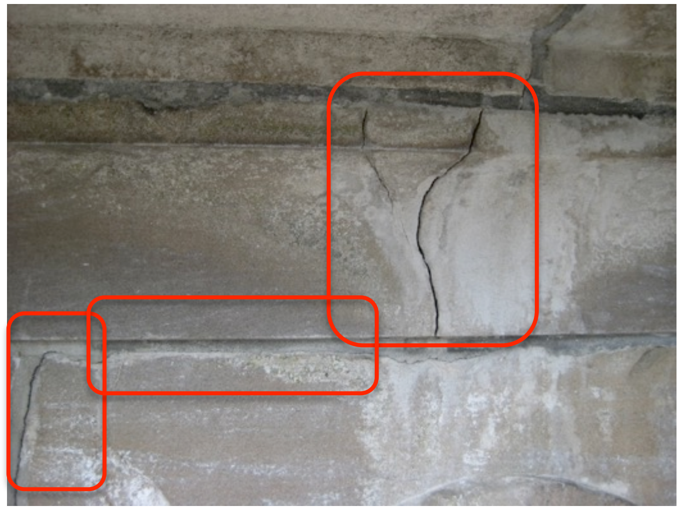


Fig. 2.2: Cracking of Stone Portico Beams

Fig. 2.3: Spalled-Off Chunk of Stone

Fig. 2.4: Cracking of Stone Portico Beams

Fig. 2.5: Cracked Stone Beam & Header

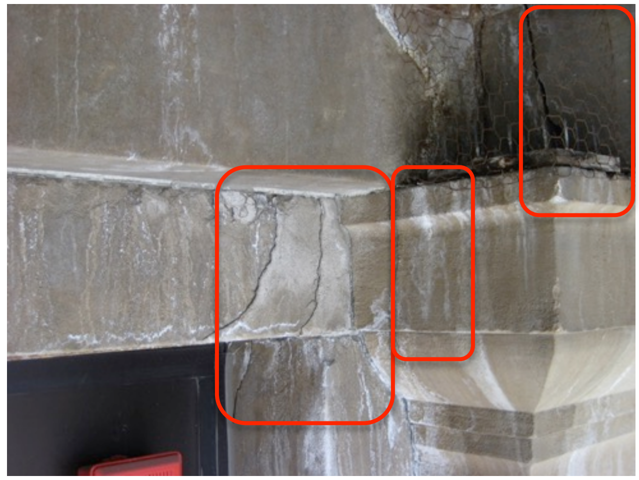


Fig. 2.6: Cracked Stone Beam & Header

Fig. 2.7: Cracked Stone Beam End

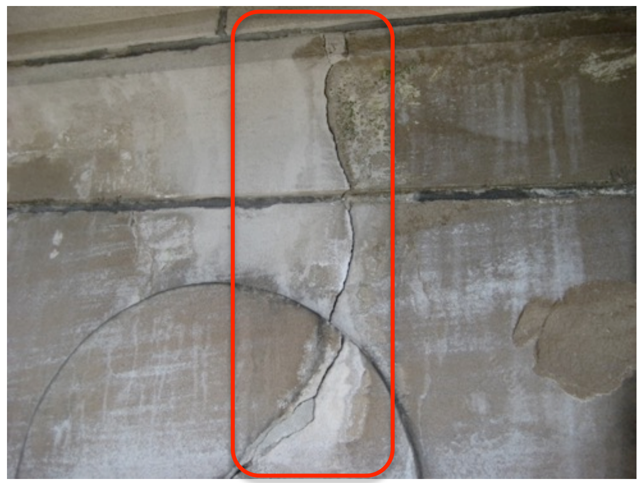


Fig. 2.8: Cracked Stone Beam End

My concern with the seemingly significant cracking of the stone portico beams was only exacerbated by the vertical cracking in the bottoms of the stone pilasters which actually provided the structural support for the cracked beams above. The constellation of these symptoms implied exactly the type of twisting motion this portico would be expected to experience in an earthquake. I saw no prior mention of any observed damage following earthquakes, though some of the damage was readily apparent.

Fig. 2.9: Cracked Middle of Struct. Pilstr.

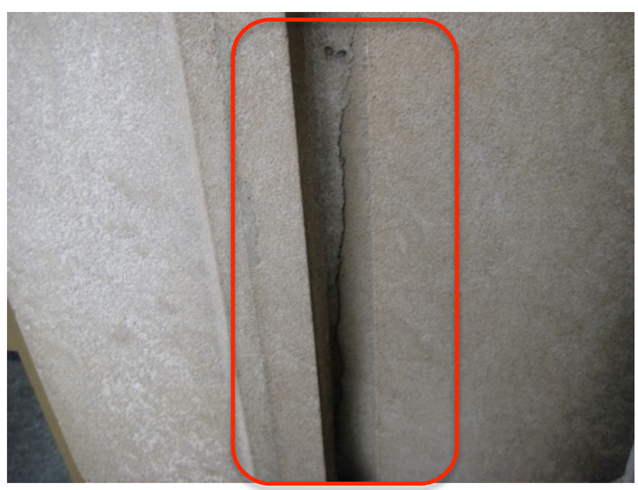


Fig. 2.10: Cracked Middle of Struct. Pilaster

Fig. 2.11: Cracked Base of Struct. Pilaster **Fig. 2.12: Cracked Base of Struct. Pilaster**

The four marble columns supporting the portico themselves displayed not only serious surface weathering and oxide staining, but also some possibly deep cracking, and while absorption testing indicated these columns to be generally well-sealed, extremely high absorption at even the tiniest of these cracks implied that these fissures may be deep and extensive. Similarly, the granite portico base beneath these columns and abutting stairs showed differential displacement of up to ¾" in places, revealing significant movement in the past. Per the original construction drawings, the three 8-foot tall, 3-foot wide marble sections, each weighing roughly 13,000 pounds, comprising each column were interconnected only with short "cube dowels" at their joints, making the columns little more than loosely-stacked stones, exacerbating seismic concerns.

Fig. 2.13: Portico Columns **Fig. 2.14: Oxide Staining on Portico Column**

Fig. 2.15: Staining, Cracking at Portico Col.

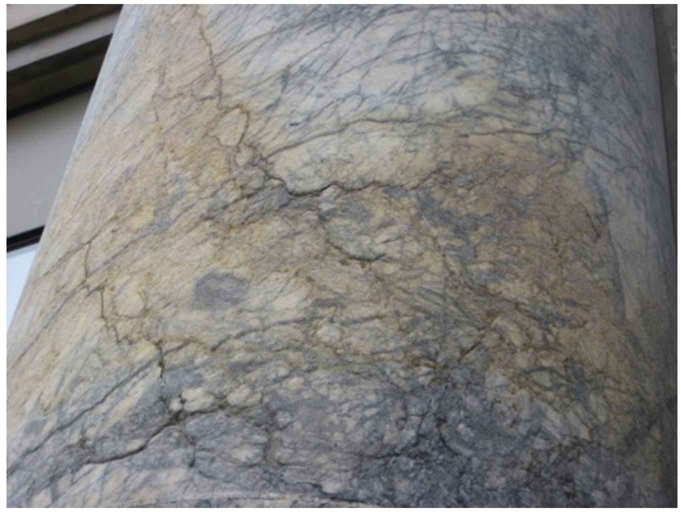


Fig. 2.16: Staining, Cracking at Portico Col.

Fig. 2.17: Cracking at Portico Column

Fig. 2.18: Cracking at Portico Column

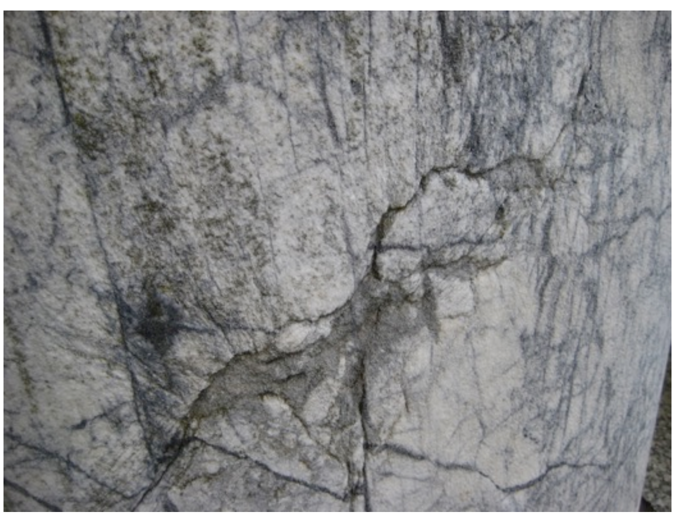


Fig. 2.19: Cracking at Portico Column

Fig. 2.20: Cracking at Portico Column

Fig. 2.21: Cracking at Portico Column

Fig. 2.22: Cracking at Portico Column

Fig. 2.23: RILEM Absorption Testing of Col.

Fig. 2.24: Downward Displacement Bel. Col.

Fig. 2.25: Downward Displacement Bel. Col.

Fig. 2.26: Downward Displacement at Stair

The portico's limestone railing elements also displayed cracking and displacement, and I was able to push some 200 pound stone caps from their positions, indicating that these had not been connected to the structure in any way other than mortar, which had cracked apart decades earlier.

Fig. 2.27: Portico Railing and Roof

Fig. 2.28: Portico Roof and Building Wall

Fig. 2.29: Cracking in Portico Railing

Fig. 2.30: Cracking at Portico Railing

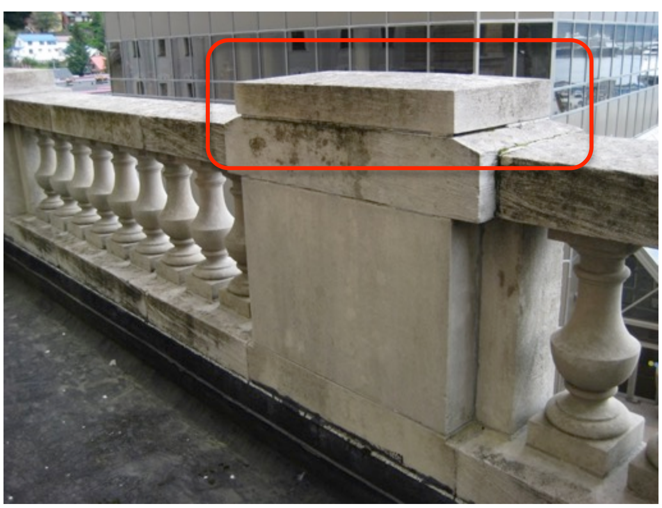


Fig. 2.31: Completely Loose Railing Cap

Fig. 2.32: Cracking at Portico Railing

Further, it was clear from the severely stained, in places eroded stone ceilings that water had been seeping down through the portico deck structure since its original construction, causing 80 years of water damage and probable corrosion of reinforcing steel embedded within the portico's concrete deck and beams.

Fig. 2.33: Severe Staining of Portico Ceiling

Fig. 2.34: Severe Staining of Portico Ceiling

Fig. 2.35: Severe Staining of Portico Ceiling

Fig. 2.36: Severe Staining of Portico Ceiling

Fig. 2.37: Staining & Erosion of Portico Clg.

Fig. 2.38: Staining & Erosion of Portico Clg.

Fig. 2.39: Staining & Erosion of Portico Clg.

Fig. 2.40: Staining & Erosion of Portico Clg.

The infiltration at the portico also migrated down within the building's exterior wall, manifesting as leakage and lime staining at the windows within this portion of the building's wall, which was completely sheltered from direct rain contact by the portico.

Fig. 2.41: Lime Stains @ Window Bel. Port.

Fig. 2.42: Lime Staining Below Portico Roof

The infiltration even appeared to continue down to the ground level, manifesting as oxide staining on the interior marble wall cladding and highly elevated moisture readings. The oxide staining further implied that the steel wire anchorage of the structural stone pilasters, exceedingly minimal to begin with, was probably corroding.

Fig. 2.43: Oxide Stains @ Interior Marble

Fig. 2.44: Oxide Stains @ Interior Marble

Though the focus of my second investigation was the portico, this element was so integrally intertwined with the building's exterior wall that analyzing the problems plaguing the portico unavoidably required analyzing the full height of the building's exterior wall above the portico, and as much of the entire building is built identically, my "portico-focused" investigation ended up analyzing much of the building's exterior by default.

For example, the severe leakage plaguing the portico ceiling did not originate with its roof, but rather resulted from the downward migration of moisture within the building's multi-wythe exterior masonry walls above, which, reflecting construction methods of its time, did not incorporate through-wall flashings or weeps to capture and drain water back out of inherently absorbent masonry, and attempted to rely on the masonry thickness and mass to limit infiltration to the interior. While this "mass masonry" approach may suffice for many exterior detailing conditions in much drier climates, it has little chance against Juneau's 220 days of precipitation annually.

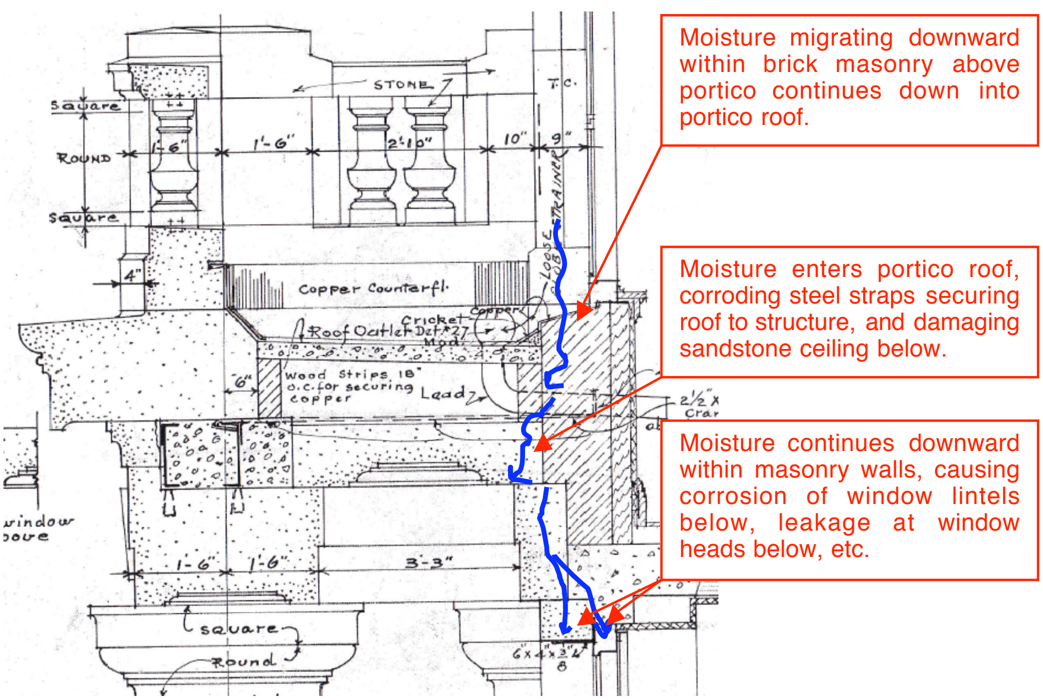


Fig. 2.45: Infiltration Pathway Into Portico From Masonry Walls Above

Masonry's inherent absorptivity, combined with this building's complete absence of through-wall flashings and Juneau's particularly wet climate affected all of the building's exterior walls, causing infiltration near many windows, corroding some steel lintels and anchors securing the terra-cotta elements, etc.

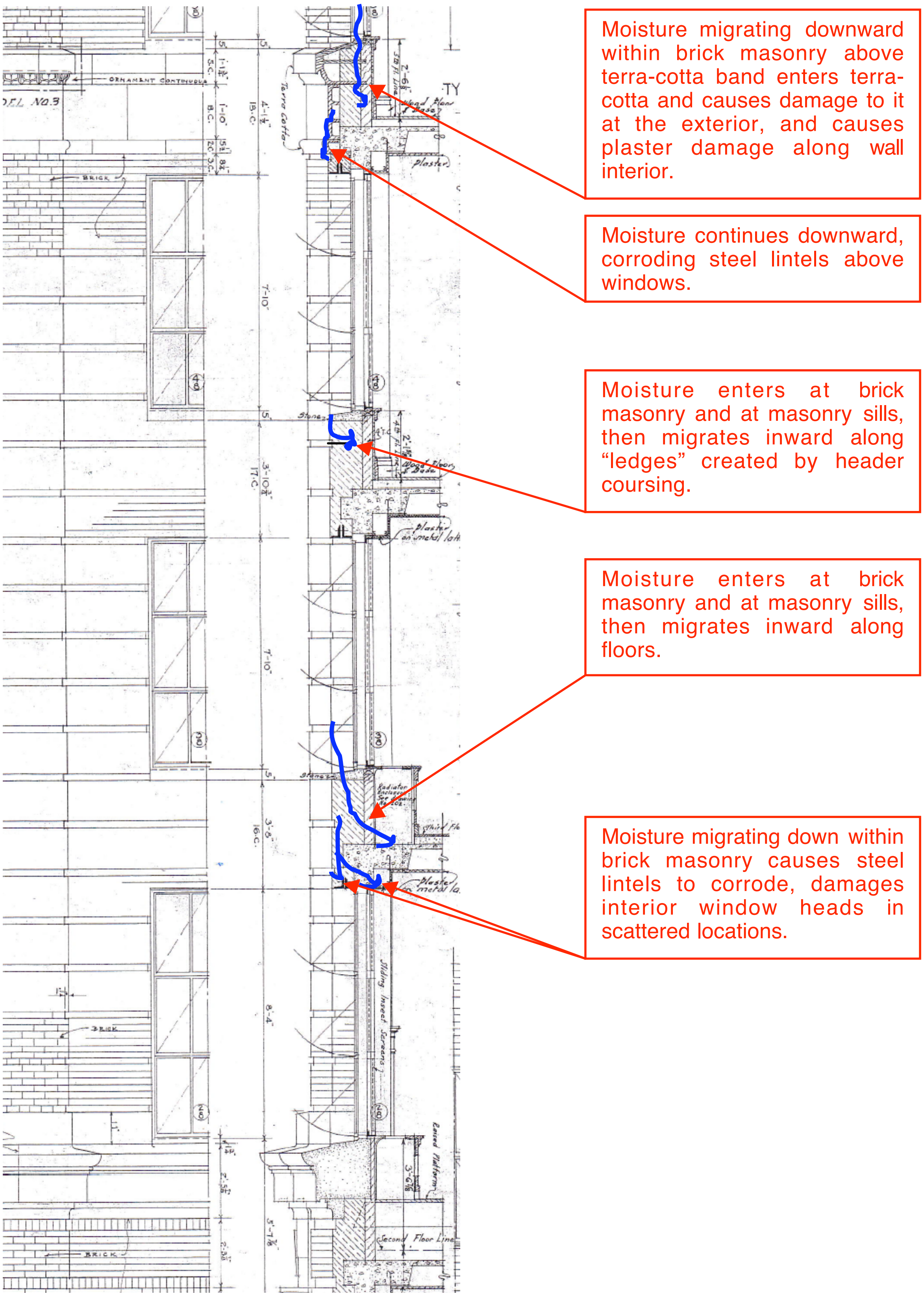


Fig. 2.46: Infiltration Pathways Via Exterior Masonry Walls

As part of my portico evaluation, I also had the opportunity to review a prior report by a structural engineering firm, which among other conclusions also surmised that the entire building's structural concrete frame was inadequate, and susceptible to collapse in earthquakes of plausible magnitudes, and though not a structural engineer, even my own limited structural expertise sufficed to discern that the building's concrete frame columns appeared too slender to provide adequate lateral resistance. Yet, the building's design seemed to accommodate a possible seismic upgrade consisting of adding new concrete walls to the interior faces of the exterior walls to create new concrete shear walls.

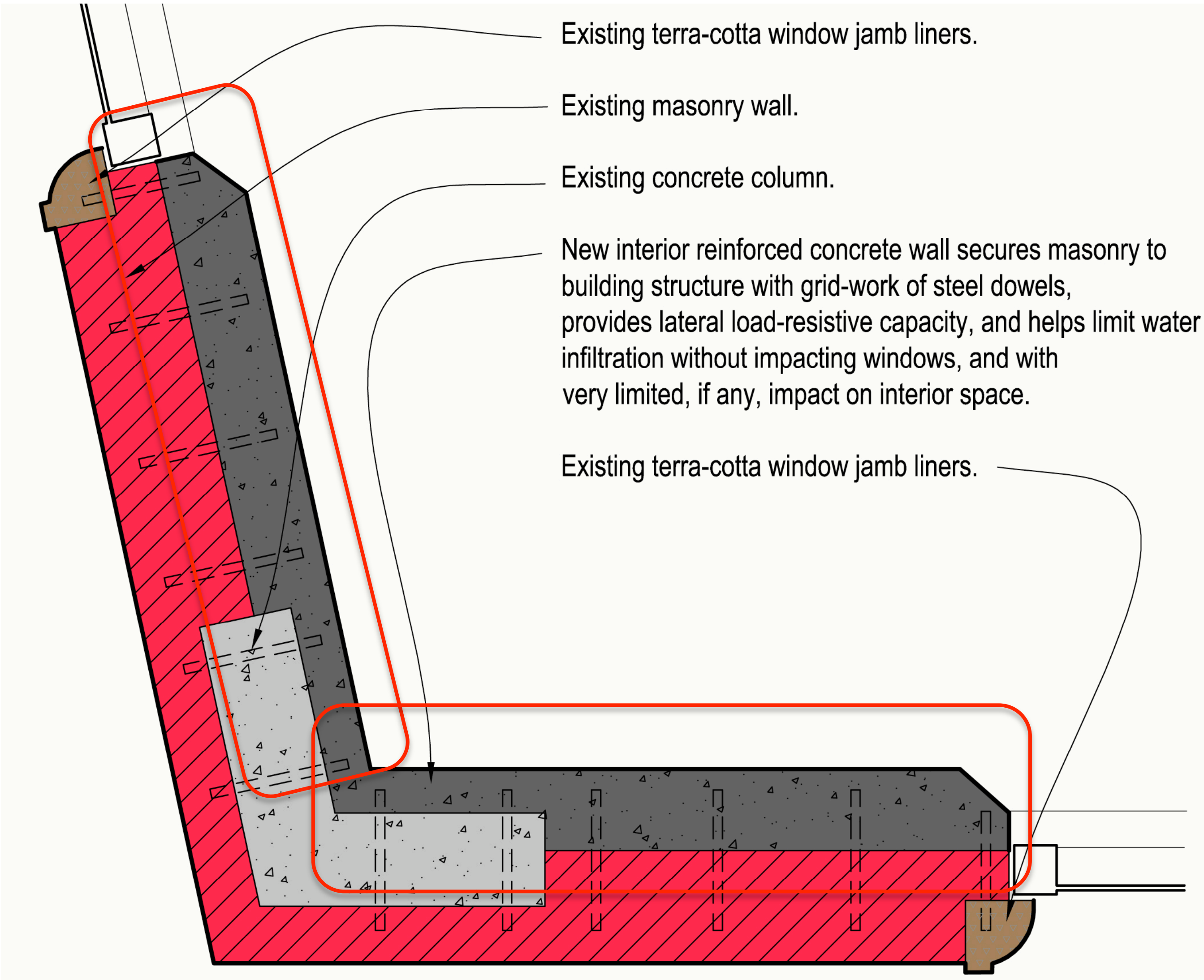


Fig. 2.47: Possible Addition of Concrete Shear Walls at Building's SW Corner

In short, my closer look revealed that the portico had suffered seismic damage in the past and appeared very vulnerable to potentially severe damage in any future earthquake, and that it had been plagued by severe infiltration for 80 years, compromising the integrity of its stone ceiling panels and possibly also of its concrete roof structure through corrosion of its reinforcing, embedded steel tie-straps and steel beams. Further, the entire building appeared vulnerable to severe damage during future earthquakes, and by virtue of the absence of through-wall flashings and drainage provisions within its exterior walls, infiltration to the interior and damage to interior finishes as well as to the masonry plagued various parts of its exterior walls.

My phase 2 report's corrective recommendations for the portico included two basic options.

In brief, the "Technically Preferable" approach included core-drilling and reinforcing the existing marble columns, then completely reconstructing the portico, with the new structure consisting of normally-reinforced concrete clad with a pre-cast concrete cladding to resemble the existing damaged limestone.

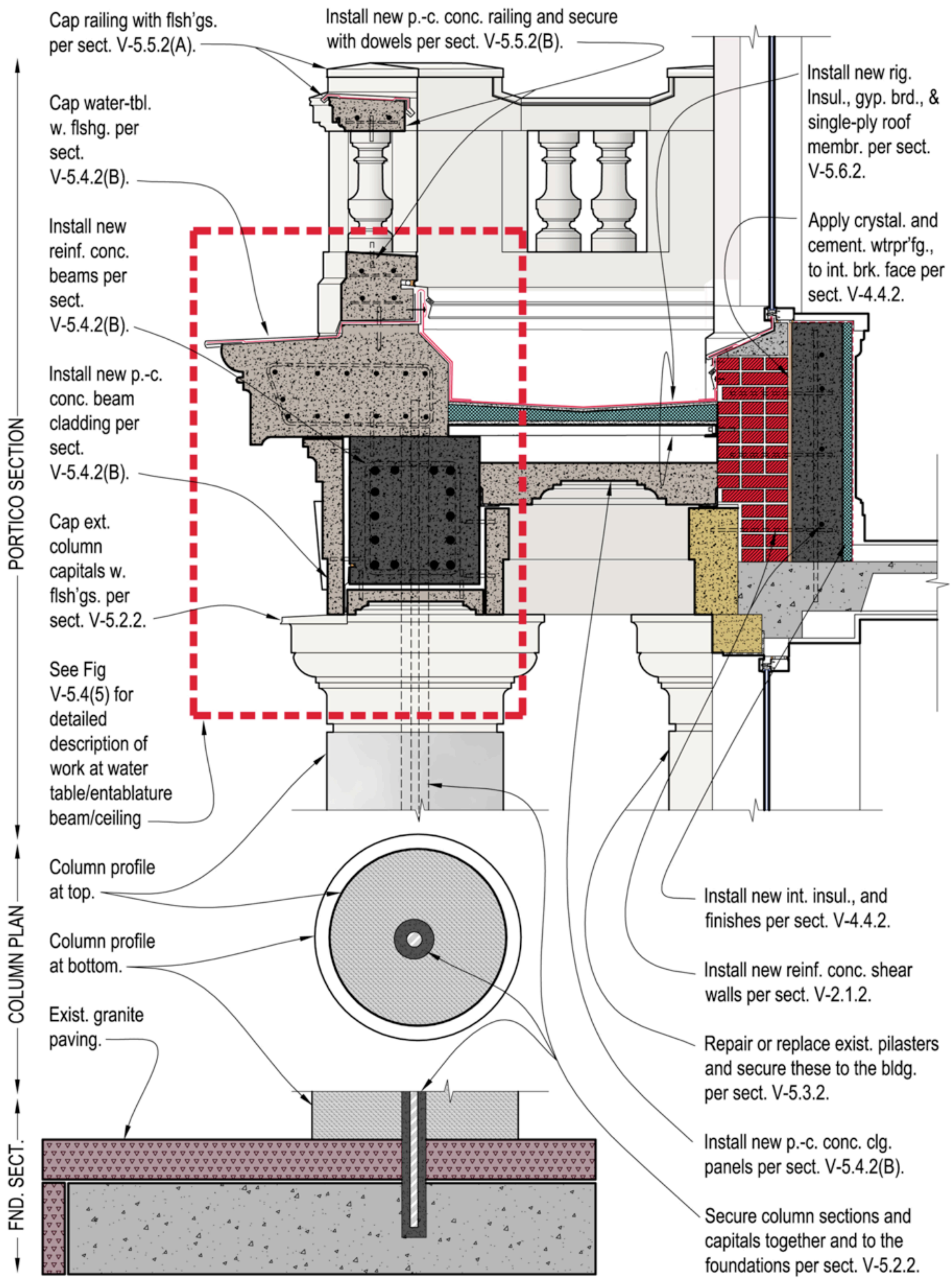


Fig. 2.48: Technically Preferable Approach, 12/31/10 Phase 2 Report

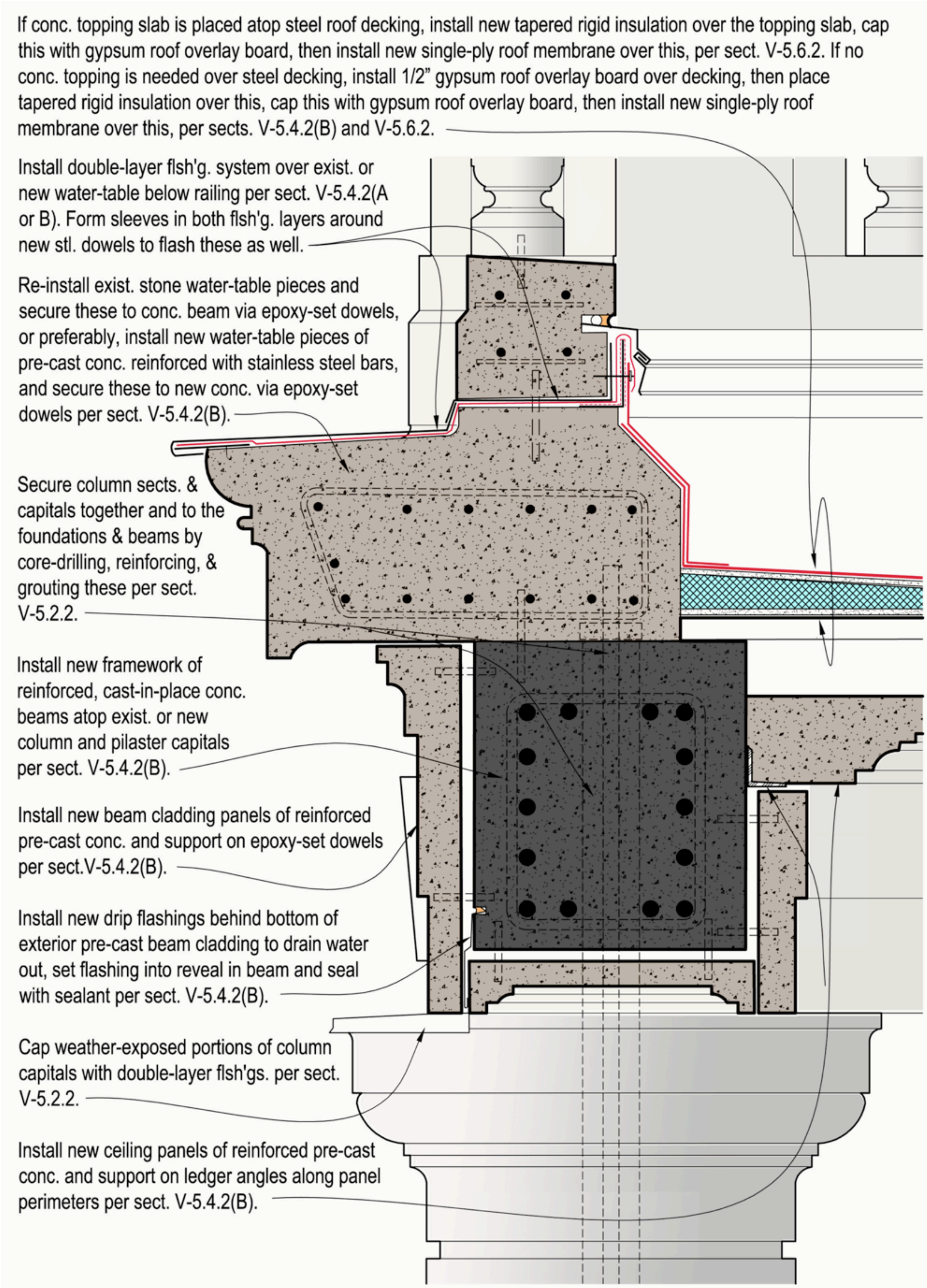


Fig. 2.49: Technically Preferable Approach @ Portico Edge, 12/31/10 Phase 2 Report

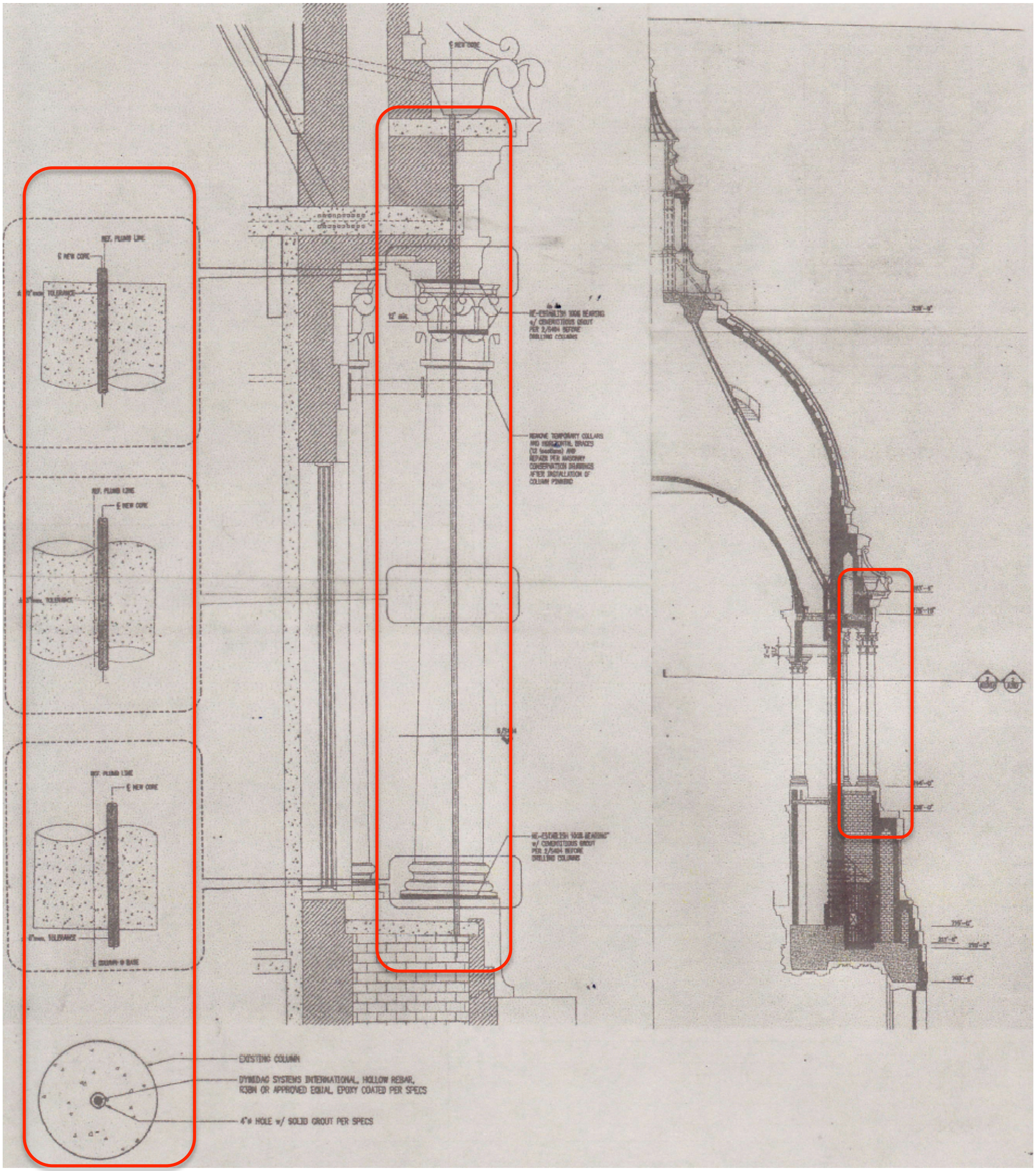


Fig. 2.50: Sim. Column Reinforcing at Washington State Capitol, 12/31/10 Phase 2 Report
Excerpted from structural drawings prepared by Swenson Say Faget structural engineers.

The alternate, technically much lesser approach also included reinforcing of the existing marble columns, but rather than reconstructing the portico roof structure, a maximal effort to maintain the existing construction would be made, consistent with resulting safety and water integrity. This required core-drilling laterally through the damaged stone beams to structurally re-integrate them and to tie the portico to the building. This approach also inherently required retrofitting of through-wall interceptor flashings in the building wall above the portico, which is complex and costly, yet can not even be fully guaranteed to suffice due to practical limitations.

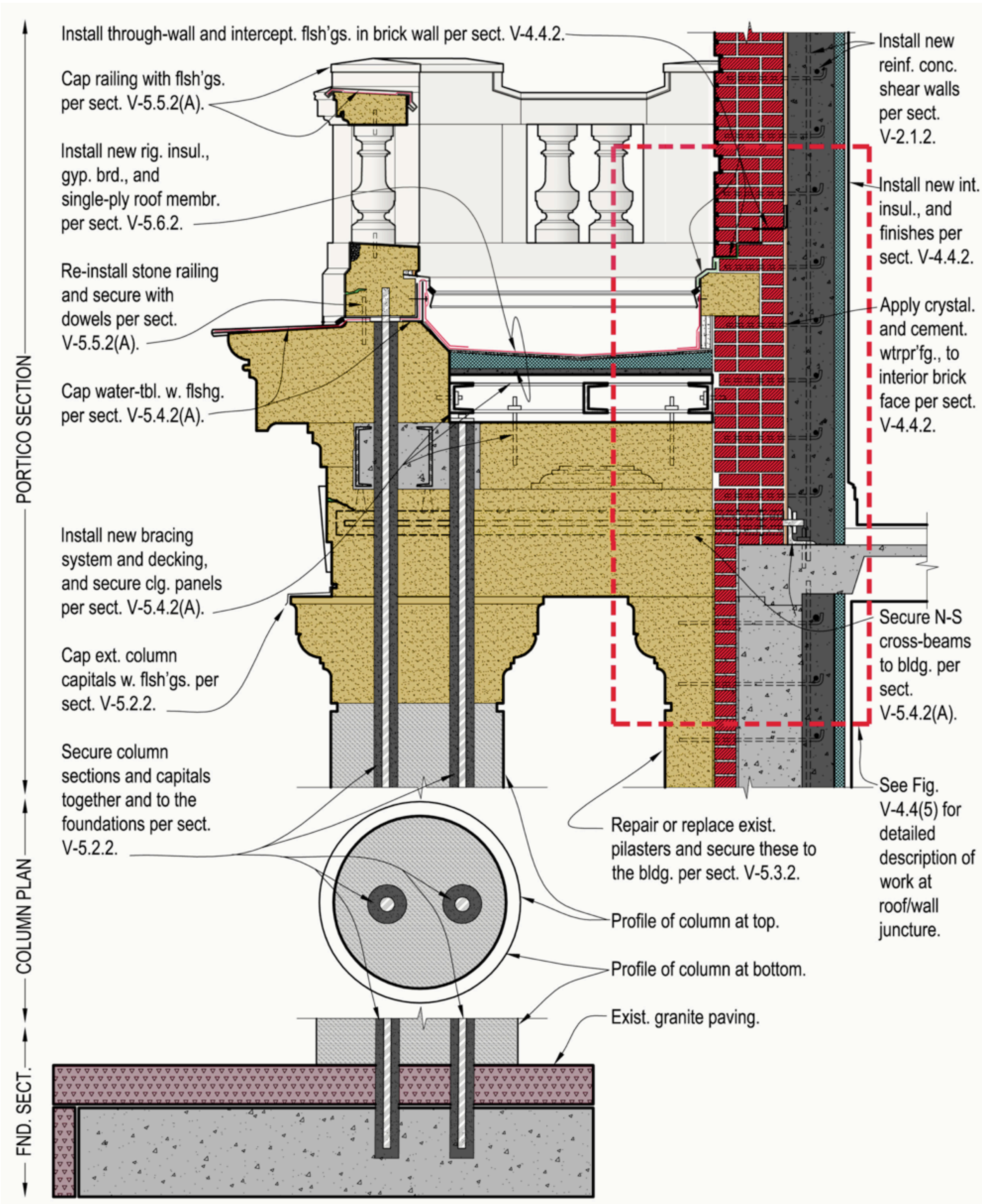


Fig. 2.51: Technically Lesser Approach @ N-S Cross-Beams, 12/31/10 Phase 2 Report

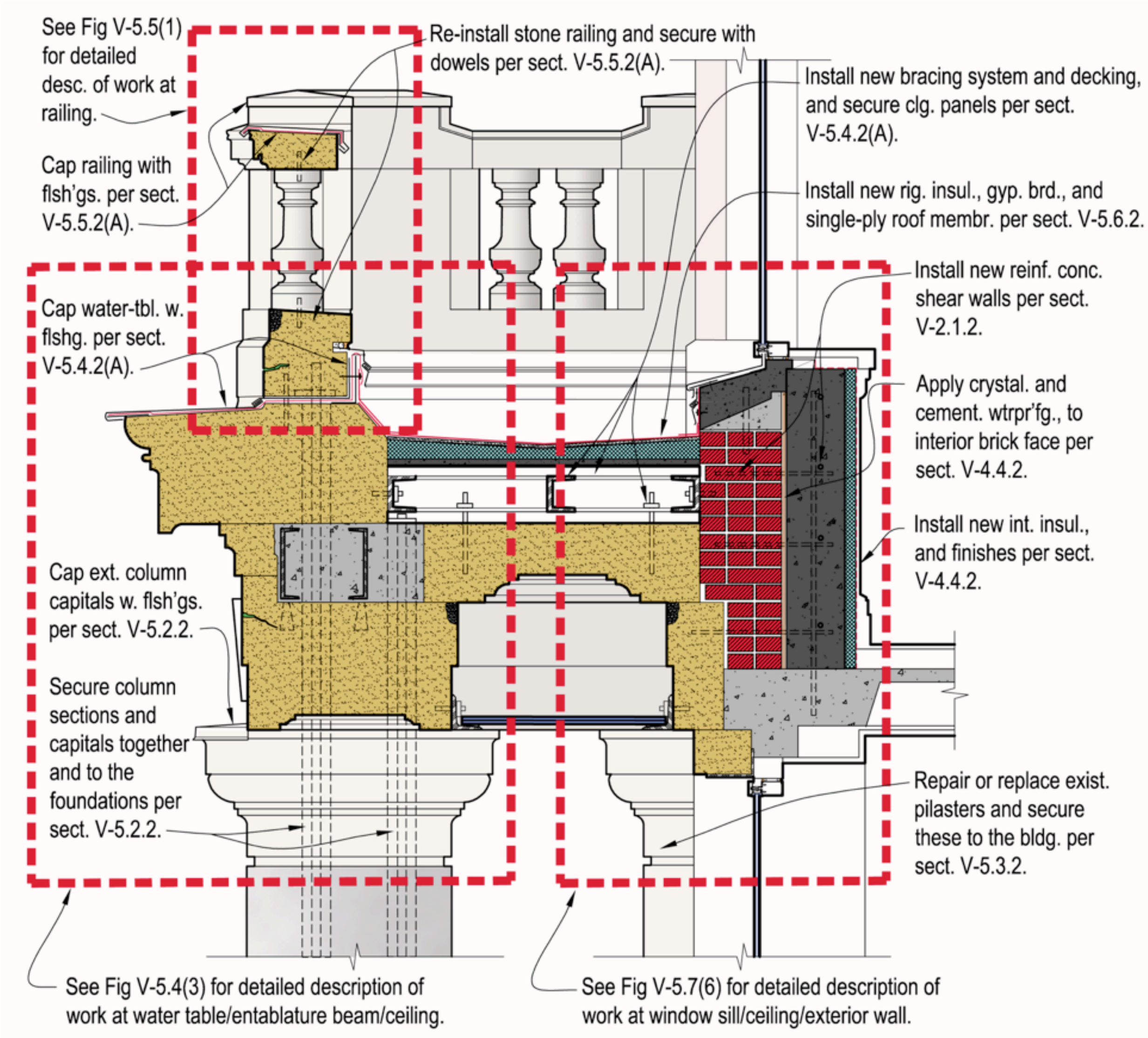


Fig. 2.52: Technically Lesser Approach Between N-S Cross-Beams, 12/31/10 Ph. 2 Report

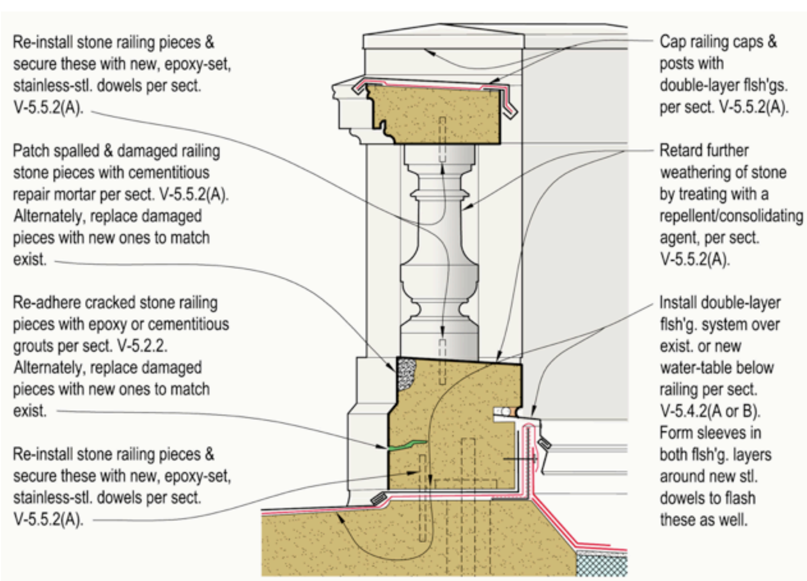


Fig. 2.53: Technically Lesser Approach @ Railings, 12/31/10 Ph. 2 Report

Phase 3: Holistic Evaluation of Corrective Options for Entire Building, 2012-13

In view of the already extant damage to its masonry, my phase 2 report cautioned that seemingly random shedding of fist-sized masonry chunks off its façades should be expected.

Just such an occurrence manifested about 2 years later, when, if my recollection of the story as related to me remains accurate, a senator's aide was just entering the building for his work-day, and a fist-sized chunk of masonry came crashing down 80 feet and shattered next to him. This crystallized the potential risks of inaction, leading to my 3rd visit to the building, when I was asked to assemble a team of experts to evaluate the building in its entirety and develop corrective options. The team included architect Wayne Jensen of the Juneau Architectural firm of Jensen Yorba Lott Inc., whose professional experience with this building preceded mine by decades, and who in turn brought on-board a cost-estimator as well as mechanical and electrical engineering firms. Greg Coons and Paul Faget of the Seattle-based structural engineering firm of Swenson Say Faget formed an integral part of the team by virtue of their prior assistance with my 2nd evaluation as well as due to their recent design of similar structural retrofitting of the Washington State Capitol.

All examined the building and its detailed design from their discipline's perspective over several days to begin developing appropriate corrective options addressing the building's multi-layered problems. This afforded the opportunity to examine portions of the building's exterior which I had not previously accessed, revealing more of the degradation symptoms expected of so-designed a masonry building in Juneau's climate, namely serious weathering of its masonry.

For example, starting at the building's top, the roof-level masonry band which replaced the original cornice was spalling extensively, in particular along a projecting narrow band, posing appreciable risk to pedestrians below.

Fig. 3.1: Spalling Roof-Level Band

Fig. 3.2: Spalling Roof-Level Band

Fig. 3.3: Spalling Roof-Level Band

Fig. 3.4: Spalling Roof-Level Band

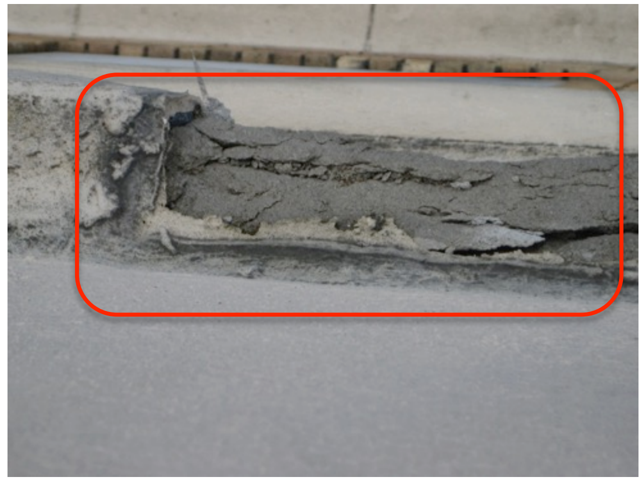


Fig. 3.5: Spalling Roof-Level Band
Note “ready-to-fall” piece.

Fig. 3.6: Spalling Roof-Level Band
Note “ready-to-fall” piece

Fig. 3.7: Spalling Roof-Level Band
Note sidewalks below.

Fig. 3.8: Spalling Roof-Level Band
Note sidewalks below.

The potential risk to pedestrians below was readily illustrated by the accumulation of stone chunks of varying sizes atop the portico roof, which was cleaned less frequently than the sidewalks. In fact, I happened to be atop the main roof, looking down onto the portico, when a fist-sized chunk fell and shattered on the portico roof 55 feet below.

Fig. 3.9: Spalled-Off Pieces on Portico

Fig. 3.10: Spalled-Off Pieces on Portico Roof

Similarly, the building's level 5 terra-cotta water-table was experiencing in-places severe freeze-spalling, and scattered locations of reddish oxide staining oozing from cracks in the terra-cotta implied that embedded steel anchors were corroding.

Fig. 3.11: Spalling Level 5 T.-C. Band **Fig. 3.12: Spalling Level 5 Terra-Cotta Band**

Fig. 3.13: Spalling Level 5 Terra-Cotta Band **Fig. 3.14: Spalling Level 5 Terra-Cotta Band**

Fig. 3.15: Spalling Level 5 Terra-Cotta Band **Fig. 3.16: Cracking Level 5 Terra-Cotta Band**

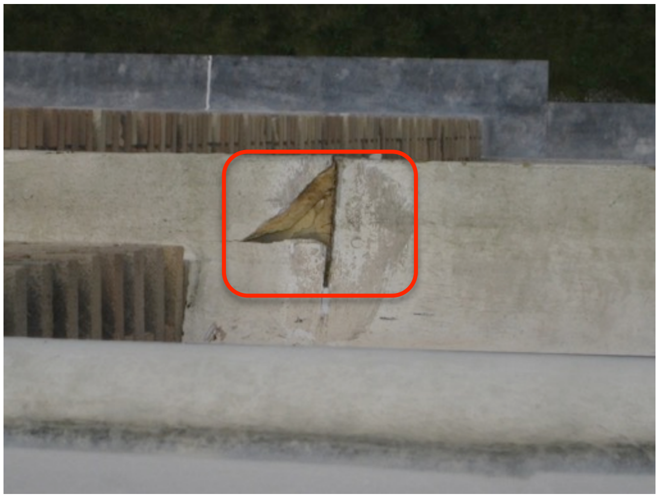


Fig. 3.17: Spalling Level 5 Terra-Cotta Band **Fig. 3.18: Spalling Level 5 Terra-Cotta Band**

Fig. 3.19: Oozing Oxide Stain @ Lev. 5 T.-C. **Fig. 3.20: Close-Up of Oxide Stain @ T.-C.**

The multi-colored, ornate terra-cotta window surrounds were also experiencing freeze-spalling of variable degrees, though generally of lesser severity than at the projecting bands, due to their more weather-sheltered locations. In places, significant lime deposits had discolored these surrounds also.

Fig. 3.21: Spalled T.-C. Window Surround **Fig. 3.22: Spalled T.-C. Window Surround**

Fig. 3.23: Spalled T.-C. Window Surround

Fig. 3.24: Spalled T.-C. Window Surround

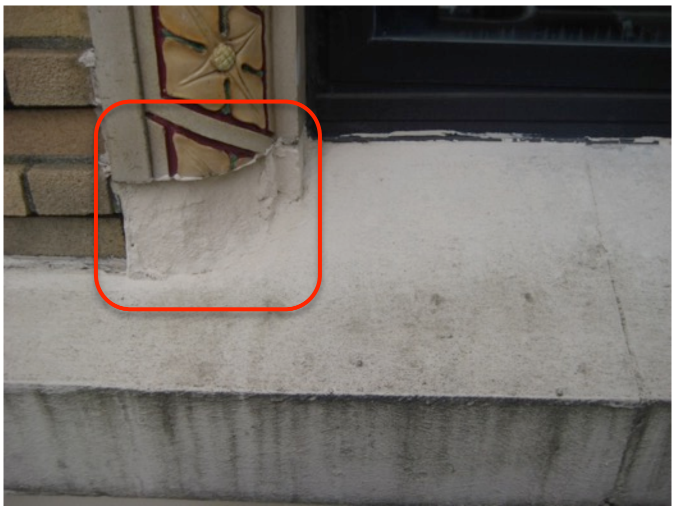


Fig. 3.25: Spalled T.-C. Window Surround

Fig. 3.26: Spalled T.-C. Window Surround

Fig. 3.27: Lime-Stained Window Surround

Fig. 3.28: Lime-Stained Window Surround

The building's terra-cotta spandrels, which separate windows vertically, displayed variable condition, ranging from still apparently decent to moderately degraded. However, reflecting these spandrels' lack of drainage provisions and lack of sill caps, many had damaged, spalling bottom edges, some showed cracking which could reflect early signs of spalling due to corrosion of embedded anchors, others had severely damaged cement-wash sills, etc.

Fig. 3.29: Spalling, Damaged T.-C. Bottom

Fig. 3.30: Spalling, Damaged T.-C. Bottom

Fig. 3.31: Crack in T.-C. Spandrel Panel

Fig. 3.32: Cracked T.-C. Spandrel Panel

Fig. 3.33: Failing Cement-Wash Sill

Fig. 3.34: Failing Cement-Wash Sill

Working down the building to the Level 2 Water-Table, composed of un-capped, weather-exposed limestone, its symptoms were as expected of its design and still relatively young age. Namely, as it is always ill-advised to expose any masonry upward to the open sky, the top surface of this water-table was starting to show variable degrees of delamination, ranging from incipient and detectable, but not yet visible, to fully spalled. Vertical cracks through these pieces were scattered all around, and edge damage and spalling affected various locations.

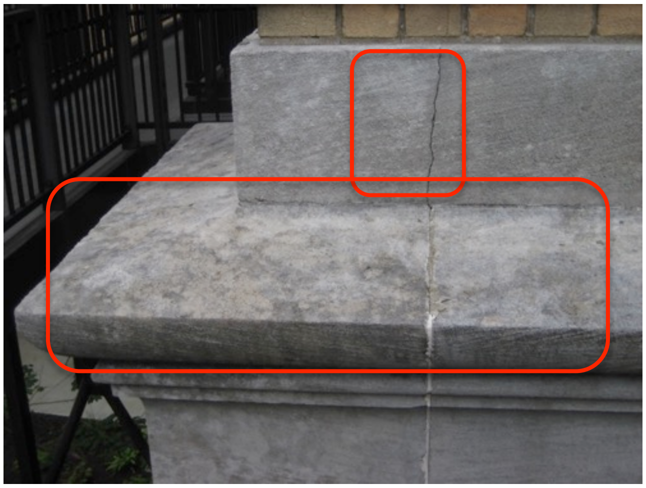


Fig. 3.35: Crack, Erosion at Water-Table

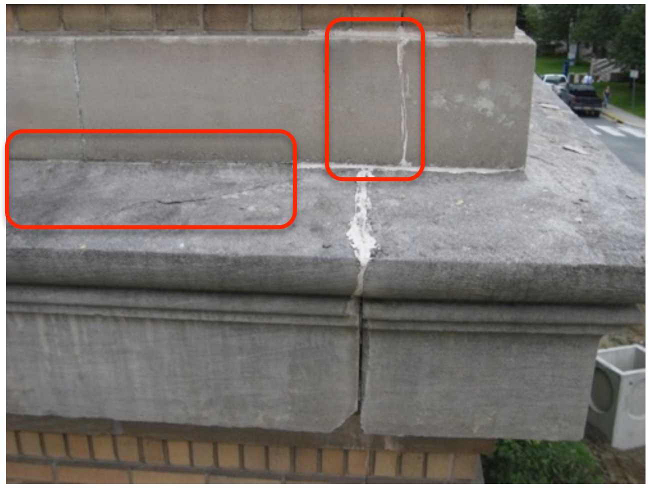


Fig. 3.36: Crack, Top Delamination

Fig. 3.37: Water-Table Edge Spall

Fig. 3.38: Water-Table Top Delamination

Fig. 3.39: Water-Table Top Delamination

Fig. 3.40: Close-Up of W.-T. Top Delamination

The limestone cladding extending from grade to the level 2 water-table on the building's south side also displayed an unexpectedly high degree of weathering and other symptoms for this cladding's relatively young age, reflecting in part its specific design and materials, Juneau's particularly masonry-challenging climate, as well as a history of some movement, manifesting for example by the fact that all of the ground-floor window sills were cracked through at one end. Serious differential surface erosion affected many areas, and some locations had spalled due to corrosion of embedded steel anchors.

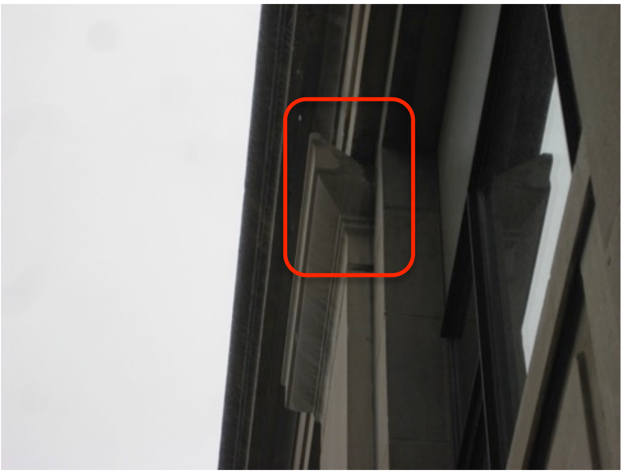


Fig. 3.41: Spalled Pilaster Capital

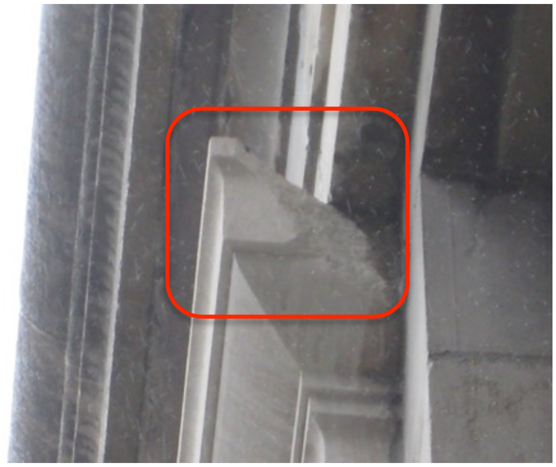


Fig. 3.42: Close-Up of Spalled Capital

Fig. 3.43: Cracked Stone Sill

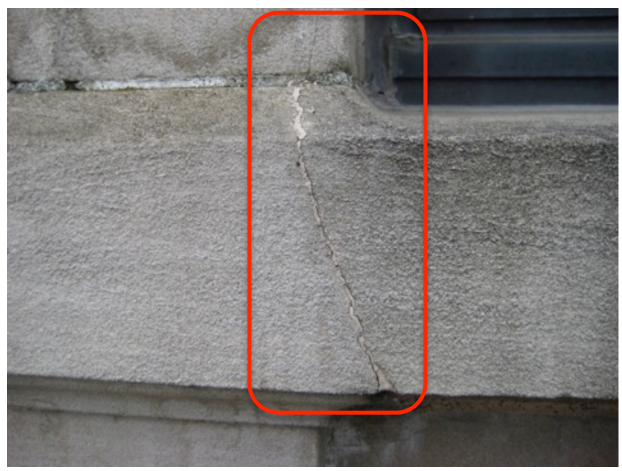


Fig. 3.44: Cracked Stone Sill

Fig. 3.45: Differential Surface Erosion

Fig. 3.46: Spall Due to Anchor Corrosion

The brickwork in general displayed a far more advanced age than one would expect of 80-year old brick. In fact, during my second visit, I was also working on another historic brick building in Seattle, 25 years older than this capitol, but whose brick would not show degradation comparable to the capitol's for a good 200 years. At first, the brick's surface was so rough that I was convinced that it must have been sandblasted, a very damaging yet not uncommon practice 5-6 decades past, as I had rarely seen brick as rough which had not been blasted. However, on my 3rd visit, I was finally able to closely access the relatively weather-sheltered west face of this building's east wing, whose brick was in obviously much better condition, many decades younger in appearance. I similarly observed that the portico's marble columns remained well honed on their weather-sheltered NW faces, yet were seriously eroded on all other exposures, so I believe that the observed damage reflects Juneau's particularly masonry-challenging climate, as addressed later in this article. This, combined with aspects of the building exterior's design, significantly accelerated the masonry's weathering.

A primary design factor exacerbating this included the complete absence of through-wall flashings under sills and anywhere within the masonry, thus greatly increasing infiltration into the brick cladding, contributing to interior leakage as well. The brickwork, which is articulated with admittedly visually pleasing effect by repeating recessed coursing as well as deeply raked mortar joints, creates many small horizontal ledges which absorb water, which can spall the brick when frozen, something which happens roughly 150 times annually in Juneau. Consequently, the brickwork was significantly spalled.

Fig. 3.47: Spalled, Eroded Brick

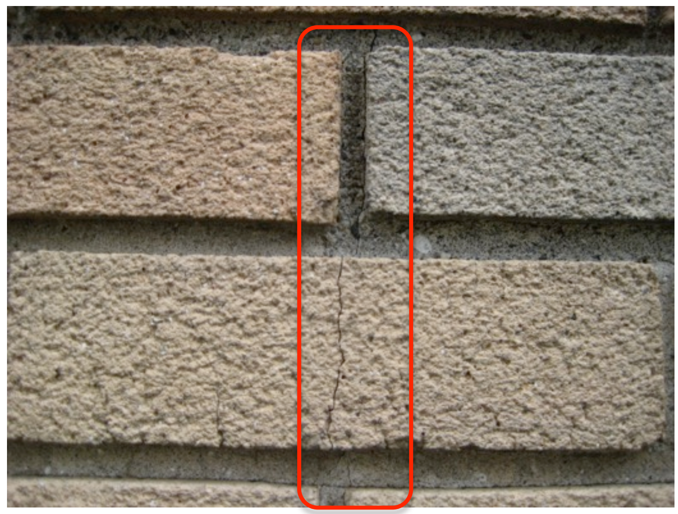


Fig. 3.48: Eroded, Cracked Brick

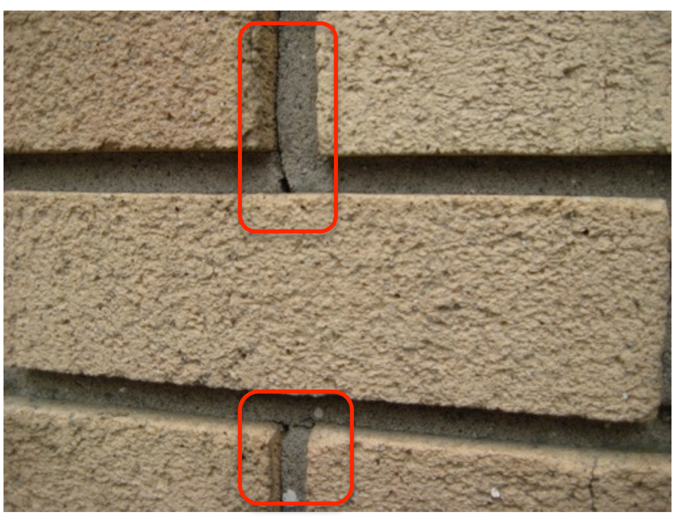


Fig. 3.49: Eroded Brick, Cracked Mortar

Fig. 3.50: Eroded Brick

Fig. 3.51: Spalled, Eroded Brick

Fig. 3.52: Eroded Brick

Fig. 3.53: Spalled, Eroded Brick

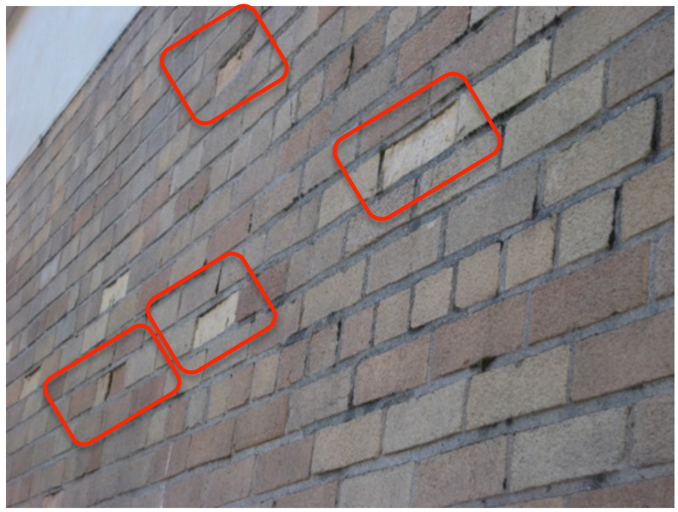


Fig. 3.54: Spalled Brick

Fig. 3.55: Spalled, Eroded Brick

Fig. 3.56: Spalled, Painted Brick
Note that brick was coated to limit leakage.

As expected, in the one location where the interior face of the brick walls could be examined, extensive interior face spalling and efflorescence were also observed, and though this was the only visible area where the interior brick face could be observed, similar degradation was likely occurring at many concealed inner brick faces.

Fig. 3.57: Spalling & Efflor. on Int. Face

Fig. 3.58: Spalling & Efflor. on Interior Face

The brickwork also contained numerous cracks, most relatively short but some exceeding 10 feet in length, some penetrating vertically through the brick units and others stair-stepping through the mortar joints. In one location, a corroding window head lintel had sagged down, causing two brick courses above the lintel to sag also.

Fig. 3.59: Vertical Brick Cracking

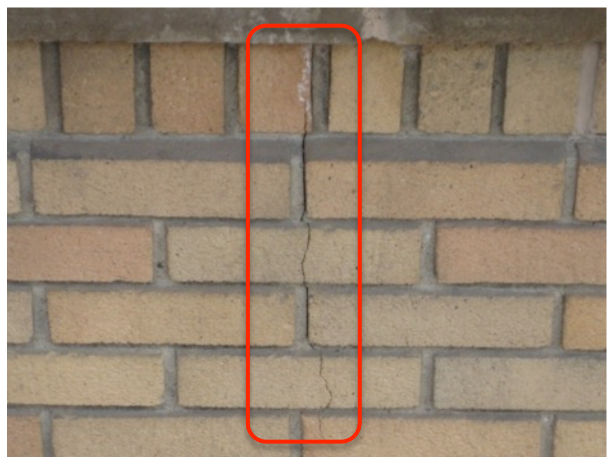


Fig. 3.60: Vertical Brick Cracking

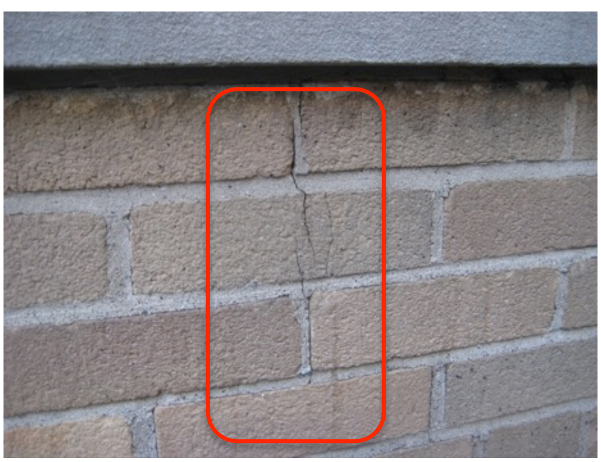


Fig. 3.61: Vertical Brick Cracking

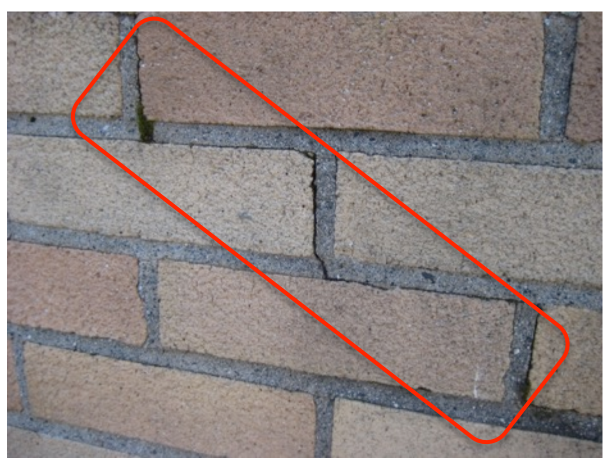
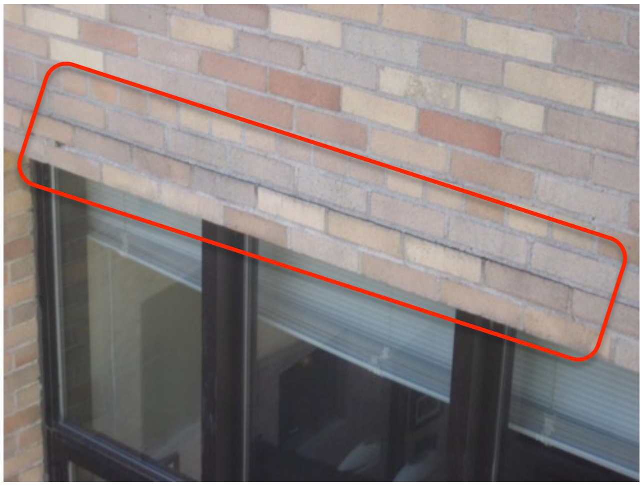


Fig. 3.62: Stair-Step Cracking



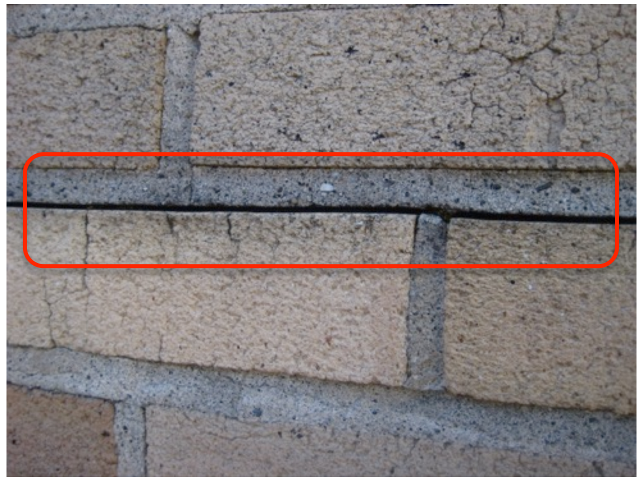


Fig. 3.63: Horiz. Crack @ Sagging Lintel **Fig. 3.64: Horiz. Crack @ Sagging Lintel**

Although lintel corrosion was surprisingly limited for a building of this age in so wet a climate, variable, though generally no worse than moderate corrosion was observed at scattered locations. This was exacerbated by the complete absence of through-wall flashings above these lintels, as well as absence of any drainage provisions, with the gaps above the lintels typically sealed.

Fig. 3.65: Lintel Corrosion **Fig. 3.66: Lintel Corrosion**

Fig. 3.67: Lintel Corrosion **Fig. 3.68: Lintel Corrosion**

Significant interior leakage manifested near many windows, of which there were at least three distinct types, including the original steel-sash units, which were left in some locations on the lee north side, and two types of extruded aluminum windows of much more recent vintage, plausibly dating back to the 1960's, which were used at all other locations. Both of these types, however, were ill-conceived by design and improperly installed, with all possible drainage pathways mistakenly sealed with sealant. Neither window type appeared to have any integral drainage system, although many holes in the frames obviously allowed water entry into the frames. Interior leakage symptoms associated with these windows included plaster damage, elevated moisture readings, etc. In one location with a seismically-deflected window frame, severe corrosion of the steel anchorage was apparent.

Fig. 3.69: Steel Jamb Corrosion

Fig. 3.70: Steel Jamb Corrosion

Fig. 3.71: Lime Stains on Mullion

Fig. 3.72: Lime Stains on Mullion

Fig. 3.73: High Moisture @ Jamb Bott.

Fig. 3.74: High Moisture Below Sill

Fig. 3.75: Plaster Damage Above Wind. Head

Fig. 3.76: Plaster Damage on Jamb

Fig. 3.77: Lime Stains on Window Frame

Fig. 3.78: Lime Stains on Mullion

Fig. 3.79: Lime Stains Exuding From Joints

Fig. 3.80: Lime Stains on Window Frame

The structure itself displayed some moderate cracking of seemingly seismic origin at the ground floor concrete slab in the building's west wing.

Fig. 3.81: Cracking of Conc. Floor Slab **Fig. 3.82: Cracking of Floor Slab**

Examination of the crawl-space under the building revealed many running brooklets, as well as serious, structurally significant corrosive spalling in many concrete floor joists. Similar spalling on footings and support columns confirmed that the building's concrete base had been sucking water from the very wet soils for all of its 80 years.

Fig. 3.83: Corrosive Spalling of Fl. Joist **Fig. 3.84: Corrosive Spalling of Fl. Joist**

Fig. 3.85: Corrosive Spalling of Fl. Joist **Fig. 3.86: Corrosive Spalling of Fl. Joist**

Fig. 3.87: Corrosive Spalling of Fl. Joist

Fig. 3.88: Corrosive Spalling of Fl. Joist

Fig. 3.89: Corrosive Spalling of Fl. Joist

Fig. 3.90: Corrosive Spalling of Pier

Fig. 3.91: Corrosive Spalling, Efflor. of Pier

Fig. 3.92: Efflorescence on Foundation Pier

In short, the Alaska State Capitol, perhaps the state's historically and architecturally most significant building, one more than worthy of preservation, was in quite poor condition with many of its exterior masonry elements very near, in many cases beyond their safe, usable lifespans; posed life safety risks to pedestrians below its walls due to the extant damage to its masonry; was experiencing interior leakage in many locations; and its very structure was vulnerable to complete seismic collapse, thus risking the state's effective decapitation just when the government's resources could be most critically needed, following a significant earthquake.

The "expected" pathway for addressing the host of issues plaguing this venerable building, the pathway which based on my prior experience would have been absolutely mandated by any overseeing historic preservation boards, would be to do all possible to preserve all existing masonry, while addressing the structural and other deficiencies.

Yet, it was clear that given the full constellation of problems plaguing this structure, this would involve a massively costly effort while still yielding at best marginal results, and extending this building's day of reckoning by at most 40 years. This approach would require removal of all hollow clay tile lining the inner faces of all exterior walls to allow new concrete shear walls to be shot-creted against the existing masonry to provide the structurally needed shear walls. It would require costly retrofitting of through-wall flashings to preclude infiltration into the portico ceiling and below many windows. It would similarly require that all existing masonry be anchored to the new concrete shear walls with tens of thousands of steel pins. The masonry would need to be patched with suitable repair mortars and treated with consolidating agents to help stabilize its degraded integrity, which even under the best circumstances would have bought 40 years before another round of very costly repairs would be needed. In this approach, the overall building would become heavier by replacing thin hollow clay tile with thick concrete shear walls, thus exacerbating seismic stresses and requiring addition of yet beefier foundations and shear walls. The exterior masonry would still continue to erode away and drop chunks onto sidewalks below, though hopefully with less frequency for some years. Further, this approach allowed no significant enhancement of the building's energy efficiency, leaving its exterior walls largely un-insulated, with total R-values ranging between R-3 and R-4, depending on location. Although insulation could in theory be added inward of the new shot-crete walls, this would not only reduce already tight interior space, but posed a risk of accelerating further degradation of the masonry, and was thus inadvisable.

In short, this approach seemed to make no sense, so I suggested that another approach be considered, namely complete reconstruction of the building's exterior to match as closely as possible the original design, while also taking advantage of the opportunity to technically enhance the cladding's performance, and to correct the technical flaws inherent in the existing design. Although this seemingly radical suggestion at first met with understandable hesitation, the potential advantages of this approach afforded compelling arguments. This approach would ironically simplify the work, as all exterior walls would be removed to allow easy access for installing new concrete shear-walls, which would then remain fully accessible to allow new masonry to be anchored to them. It would lighten the building, replacing in many locations 16 inches of masonry with 8"-12" of concrete and brick, thus reducing seismic risk yet further, and reducing the needed amount of new concrete shear walls. It would provide a new masonry cladding closely resembling the original, but with a plausible lifespan of 100-150 years even in Juneau's masonry-challenging climate. It would also allow major enhancement of energy-efficiency, increasing the exterior walls' insulating value from their original R-3 and R-4 to roughly R-20 in some locations and to over R-40 in many other areas. This approach would also allow easy correction of the original design's technical flaws, by installing suitable flashings atop all ledgers and lintels, below window sills, and at similar suitable locations to drain water back out of the cladding; to cap over ill-advised, skyward-facing masonry surfaces with historically compatible copper flashings, and similar enhancements with very limited visual impact. Although cost-estimating falls outside my focus, it seemed plausible that this technically much better approach may also be comparable in cost.

With either approach, I also strongly recommended that the original roof-level cornice be re-constructed of pre-cast concrete, as this would not only restore the building closer to its original appearance, but would appreciably help protect the new masonry from weathering, helping extend its life-span.

And this is a good departure point for a discussion of how this reconstructed cornice, projecting only 3 feet beyond the 85-foot tall building walls, would accomplish this. This begins with a discussion of "what kills masonry". In general, most, though not all, masonry materials are not harmed by water itself. Though I have obviously not done so, I am confident that one could place a good brick in a pail of water, and retrieve it a century later with little harm to the brick. However, water does harm masonry through two distinct mechanisms.

In the first, absorption of water, followed by freezing, induces the absorbed water to expand as it freezes within the masonry matrix, causing the outer faces to spall off. This freeze-spalling manifested on this building in most locations, reflecting Juneau's masonry-challenging climate. This is why the 1931 Alaska Capitol displayed so much more advanced degradation than a 1904 Seattle building, for while Seattle's 160 rainy days annually approach Juneau's 220-day rain frequency, its 23 yearly sub-freezing nights pale in comparison to Juneau's 150 days.

Fig. 3.93: Spalled Brick on 1931 AK. Capitol, Fig. 3.94: Intact Brick on 1904 Seattle Bldg.

A second water-related masonry-damaging mechanism involves movement of water in one direction through masonry. On the capitol, such movement has been taking place since the building's construction, with rain water migrating inward through the brick walls. In doing so, this migrating water extracts salts from the masonry and carries these in solution toward the inner brick faces, where the water evaporates to the interior, leaving the salts near the innermost masonry face. As this process continues over decades, the concentration of salts near the inner masonry face becomes ever-greater, and much like water freezing, the crystallization of these salts within the masonry matrix causes expansion, leading to spalling and pulverization at the inner masonry faces. Where the inner face on this building could be examined, this inner-face phenomenon was also observed.

Fig. 3.95: Spalling Interior Brick Face

Fig. 3.96: Spalling Interior Brick Face

The bottom line, however, is that it is very advisable to keep masonry as dry as possible, to limit the frequency, severity, and duration of wetting to the greatest possible degree, and to keep it from freezing when wet to the greatest possible degree. A properly designed cornice projecting even a few feet beyond the building face can greatly help keep the masonry below dry most of the time.

As my assertion that a 3-foot wide cornice can significantly help protect the full height of an 85-foot tall wall below it often meets with incredulity, let me repeat the original explanation for this as offered to the state of Alaska in advocating the reconstruction of the cornice.

Many hold the impression that since rain typically falls at an angle, a projecting cornice can only shelter the uppermost portions of the wall below it, as one might naturally project the falling angle to assume that rain will strike the building face below this line. Figure 3.97 below illustrates this common, though mistaken, assumption.

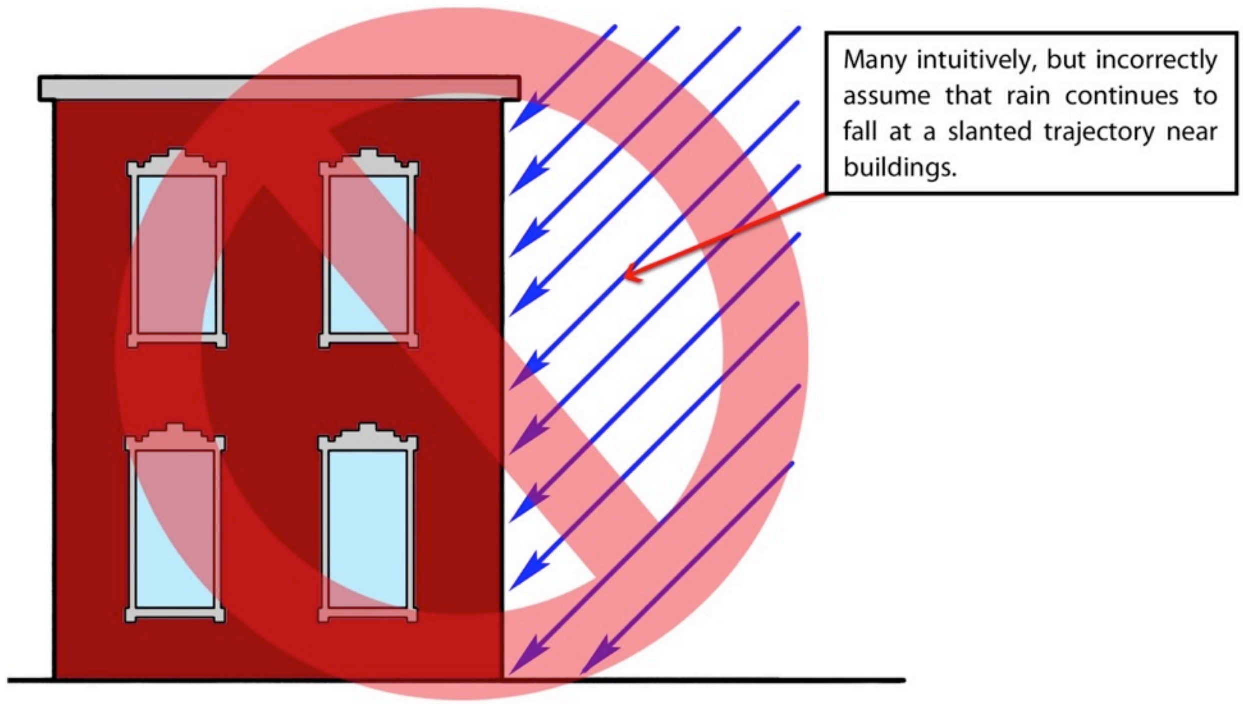


Fig. 3.97: Incorrectly Assumed Rain Trajectory Near Building Faces

In reality, the reason why rain typically falls at an angle is that much of the time, some minor wind pushes the droplets sideways, producing the sloped fall-line, which otherwise would be straight down. This lateral wind force needs to be continually applied, for if this wind is somehow removed, the droplets would fall along a curved, steepening path.

Since wind can not blow through a building, it is deflected around it. The air-flow near its top is deflected upward over its roof, and the air-flow below splits and travels around the corners. This removes the lateral force on the rain droplets, causing them to fall along steepening arcs, rather than wetting the building. Under most conditions, this effect will cause only the uppermost bands of building walls to become wet, even if not sheltered by a cornice or roof overhang. The outer vertical building corners also typically receive more rain exposure than mid-faces. Figure 3.98 on the following page illustrates this wind effect. As this claim has often met with disbelief, Figures 3.99-3.104 show actual buildings during rains or showing stain evidence of this phenomenon. All of these photos clearly show that most water reaching the wall surfaces drains down from the uppermost band, rather than resulting from direct rain strikes. This, in turn, should illustrate the benefit afforded by a projecting cornice, which can help deflect away from the building the vast majority of water which would otherwise drain down the walls to damage the masonry.

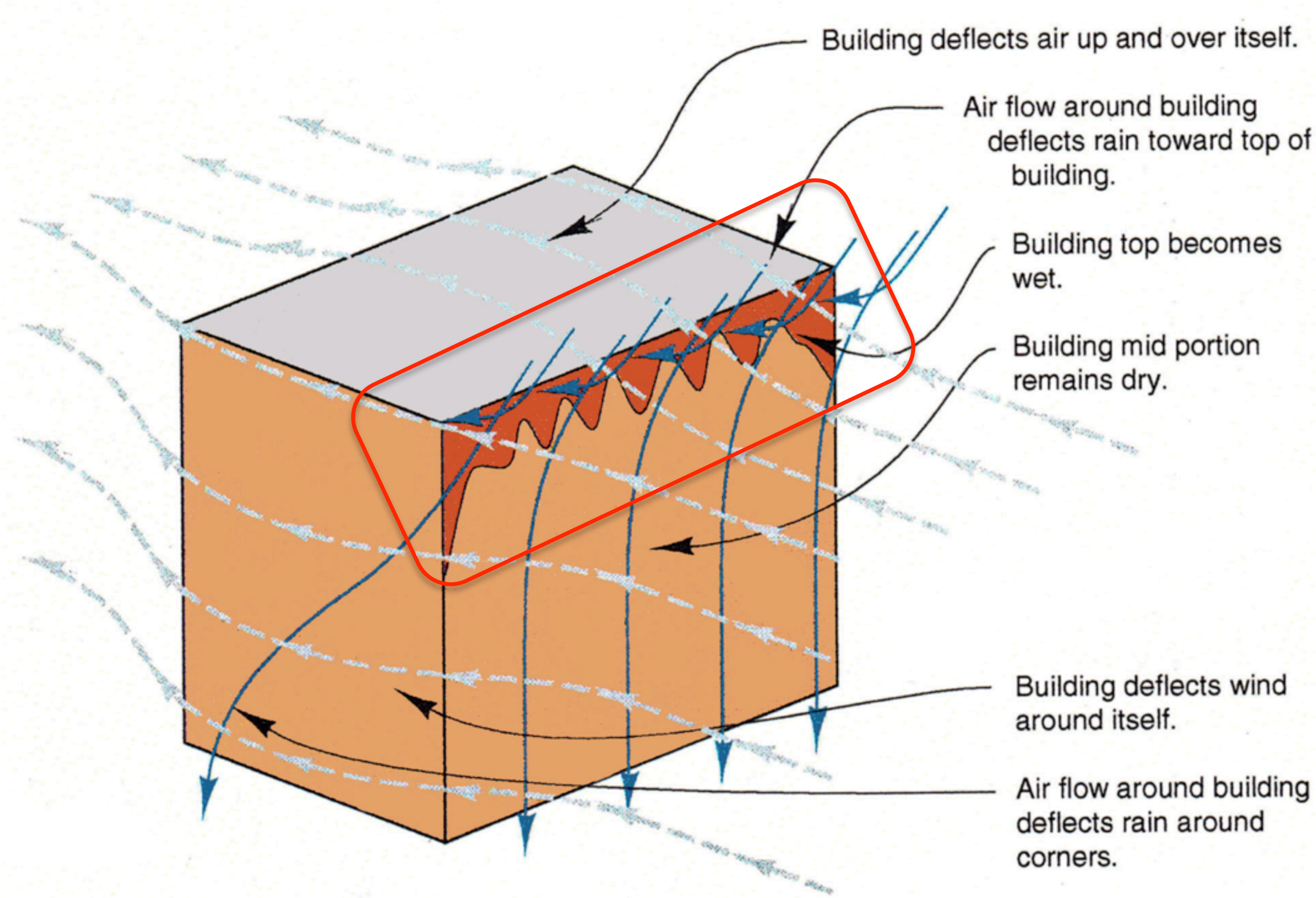


Fig. 3.98: Typical Wind-Flow and Rain Trajectory Near Buildings

Fig. 3.99 Wetting Pattern on Lee Side

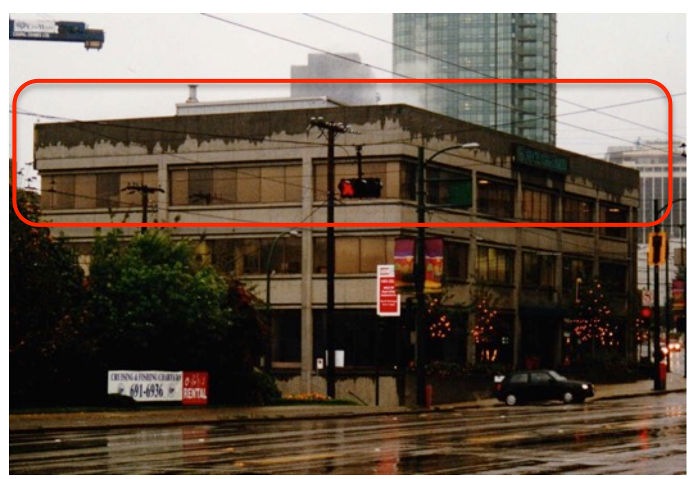


Fig. II-3.100: 3rd Rain Day Wetting Pattern

Fig. 3.101: 3rd Rain Day Wetting Pattern

Fig. 3.102: 3rd Rain Day Wetting

Fig. 3.103: Stain Pattern, Juneau

Fig. 3.104: Stain Pattern, Juneau

In short, even a minimal cornice or similar sheltering projection near a building's top can do much to keep its masonry dry, thus appreciably slowing down its degradation. Thus, reconstruction of the cornice would not only restore the building's original appearance, it would help protect the life-span of all exterior masonry.

This also leads to a brief discussion of why it was inadvisable to add significant interior insulation to the existing walls to enhance energy-efficiency. The reason again relates to water migration through these walls. As explained, water can severely harm masonry by freezing after it absorbs into the masonry, and by continually migrating inward, thus transporting salts to the masonry's interior surface. An un-insulated masonry wall, though energy-wasteful and requiring significant additional heating to maintain interior comfort, otherwise helps protect the integrity of the masonry by both keeping it warmer and above the freezing point more often and for longer durations, and by helping to dry the masonry, thus also reducing the damaging salt-transport through the wall. If much insulation is added to the interior face of a masonry wall, this cools the wall and exacerbates the duration and severity of water absorption, which poses the risk that the masonry will begin degrading yet more rapidly. For this reason, the Phase 3 report advised that only limited interior insulation be added to the existing walls, and only if the roof cornice is also reconstructed to help offset the loss of the drying effect by the escaping heat.

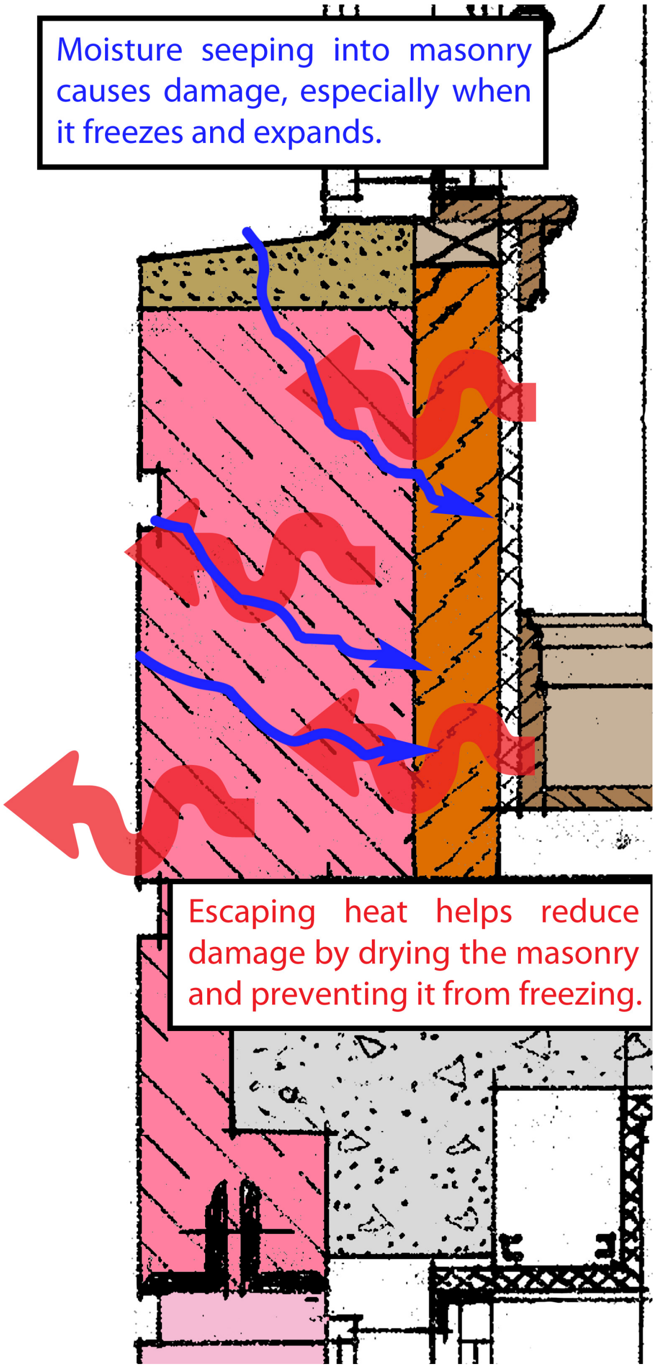


Fig. 3.105: Warming & Drying Effect of Existing Energy-Inefficient Masonry Walls

Given the existing building's serious degradation and seismic inadequacy, the phase 3 report evaluated three different corrective approaches.

The first basically consisted of installing new shot-crete shear walls at all interior faces of the exterior walls to provide seismic adequacy, while exerting all effort to retain the existing exterior masonry, and rebuilding the original, albeit enhanced, roof-level cornice.

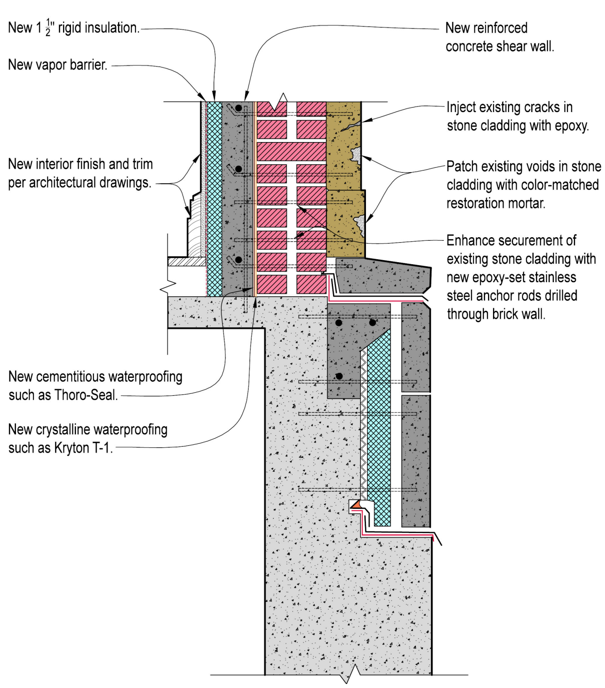
The second approach consisted of removing all exterior masonry to strip the exterior structure to its concrete skeleton, adding new shot-crete shear walls to provide seismic adequacy, then reconstructing the exterior masonry as a veneer to closely resemble the building's original appearance. This approach also included reconstruction of the original cornice, along with incorporating technical enhancements, such as integration of through-wall flashings, capping upward facing masonry elements, etc.

The third approach was similar to the second, and also included removal of all exterior masonry and reconstruction of a closely matching masonry veneer cladding, reconstruction of the roof-level cornice, and incorporation of technical enhancements. The primary difference was that in this approach, new concrete shear walls would be added only where needed, while other exterior walls would be reconstructed using steel stud framing.

The Phase 3 report described each of the corrective approaches in some detail, provided drawings depicting how each of the various exterior conditions would be addressed within each approach, outlined the relative advantages and draw-backs of each approach, and provided rough cost estimates for each.

This revealed that the Option 1: "Maximum Preservation" approach would cost roughly $ 18 million; Option 2: "New Masonry Veneer with Concrete Walls" approach would cost roughly $ 22 million, and Option 3: "New Masonry Veneer with Steel-Framed Walls" would cost roughly $ 23 million. As the Option 3 approach was both technically less optimal than the Option 2 approach as well as the most costly, the Phase 3 report strongly advised against it. While the Option 2 approach cost roughly 20% more than Option 1, it offered such compelling advantages in safety, energy-efficiency, projected lifespan, much lower risk of continued infiltration and degradation, among many others, that the Option 2 approach was recommended as the only truly viable option. Recognizing the powerful advantages of the Option 2: "Reconstruction" approach, the state of Alaska accepted this recommendation.

Figures 3.106-3.121 depict these three basic options at various locations on the building, generally starting at the ground level and working upward. To best illustrate the differences between the three options, all approaches for each condition are grouped together to allow side-by-side comparison. Please note in particular the obvious differences in total wall mass, thickness, and insulation levels.



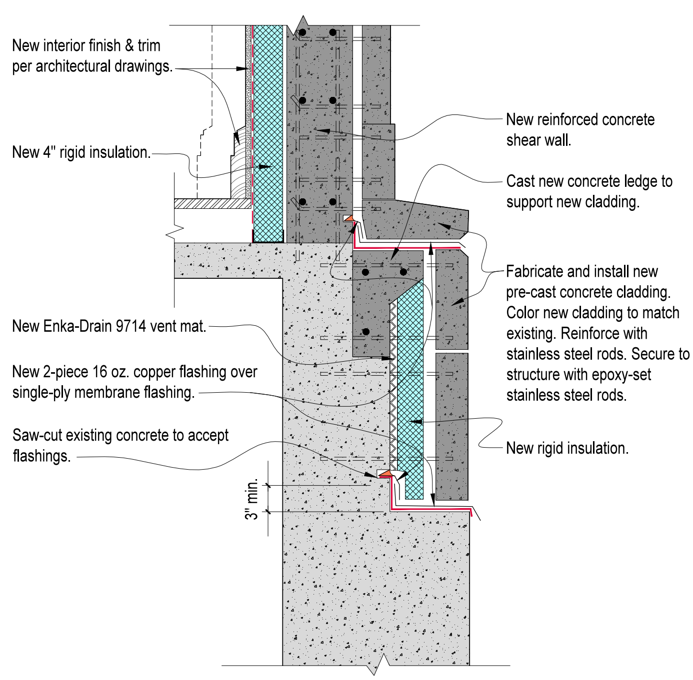


Fig. 3.106: Opt. 1 "Restoration" @ Grnd. Fig. 3.107: Opt. 2 & 3"Reconstruction" @ Grnd.

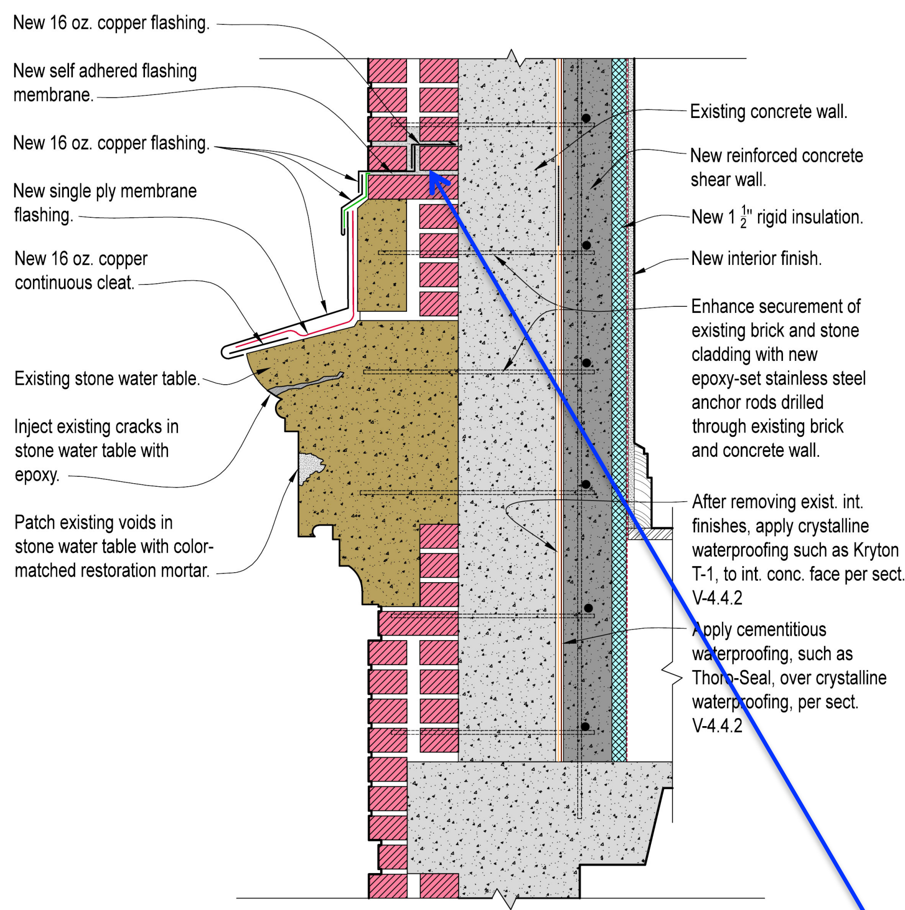


Fig. 3.108: Opt. 1 “Restoration” @ Level 2 Water-Table

Note differences in total wall mass & insulation levels.

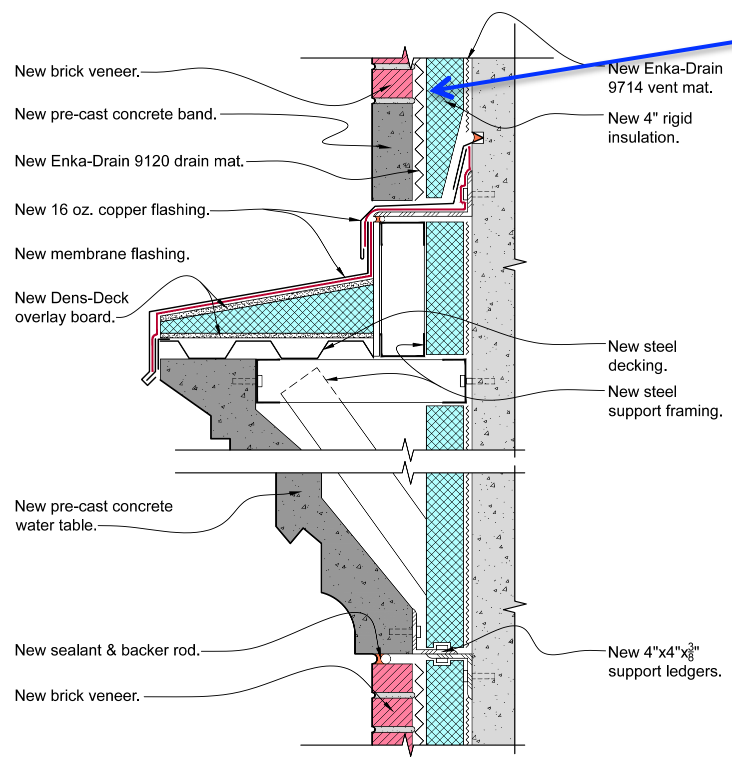
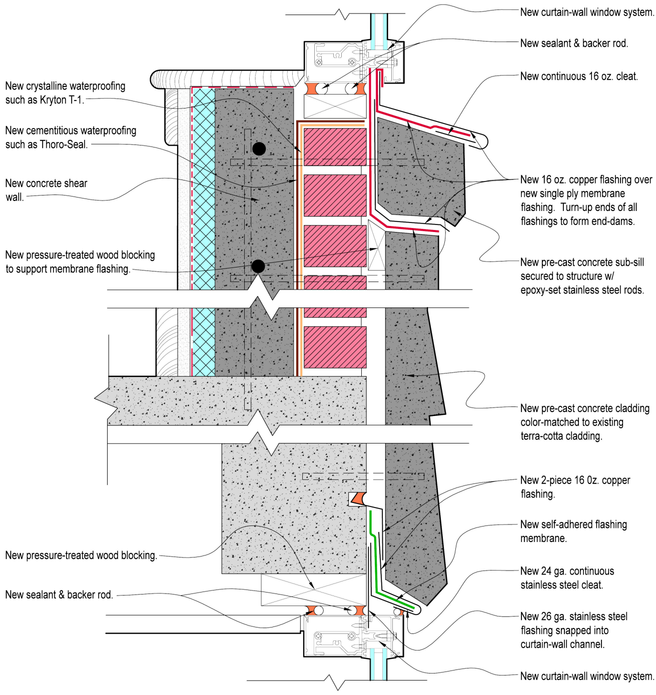


Fig. 3.109: Opt. 2 “Reconstruction” @ Level 2 Water-Table



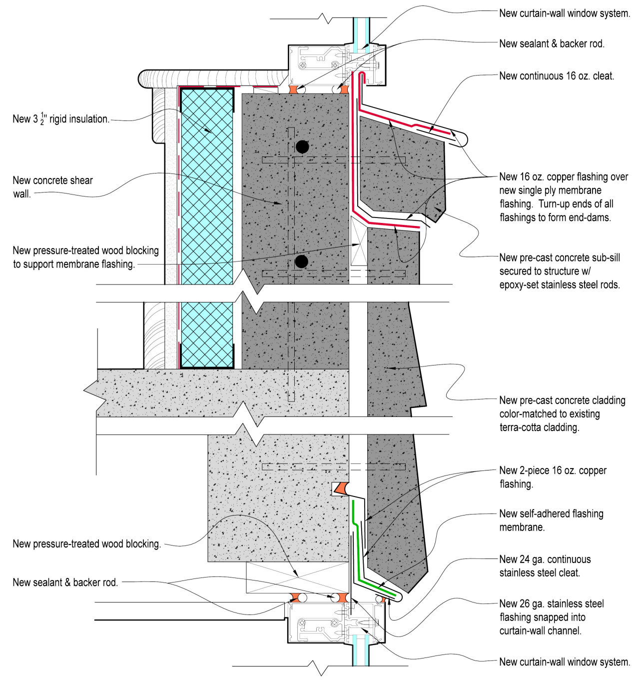


Fig. 3.110: Opt. 1 “Restoration” @ Spandrel   Fig. 3.111: Opt. 2 “Reconstruct.” @ Spndr.

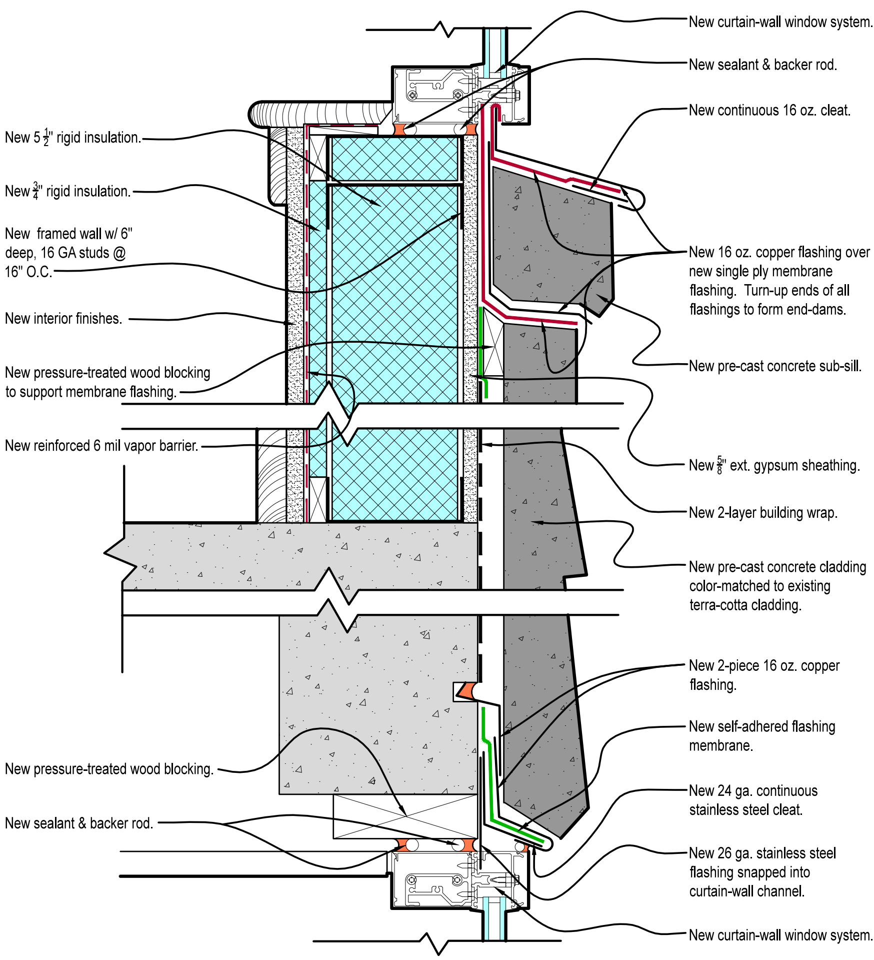


Fig. 3.112: Option 3 “Reconstruction Approach” at Window Spandrels

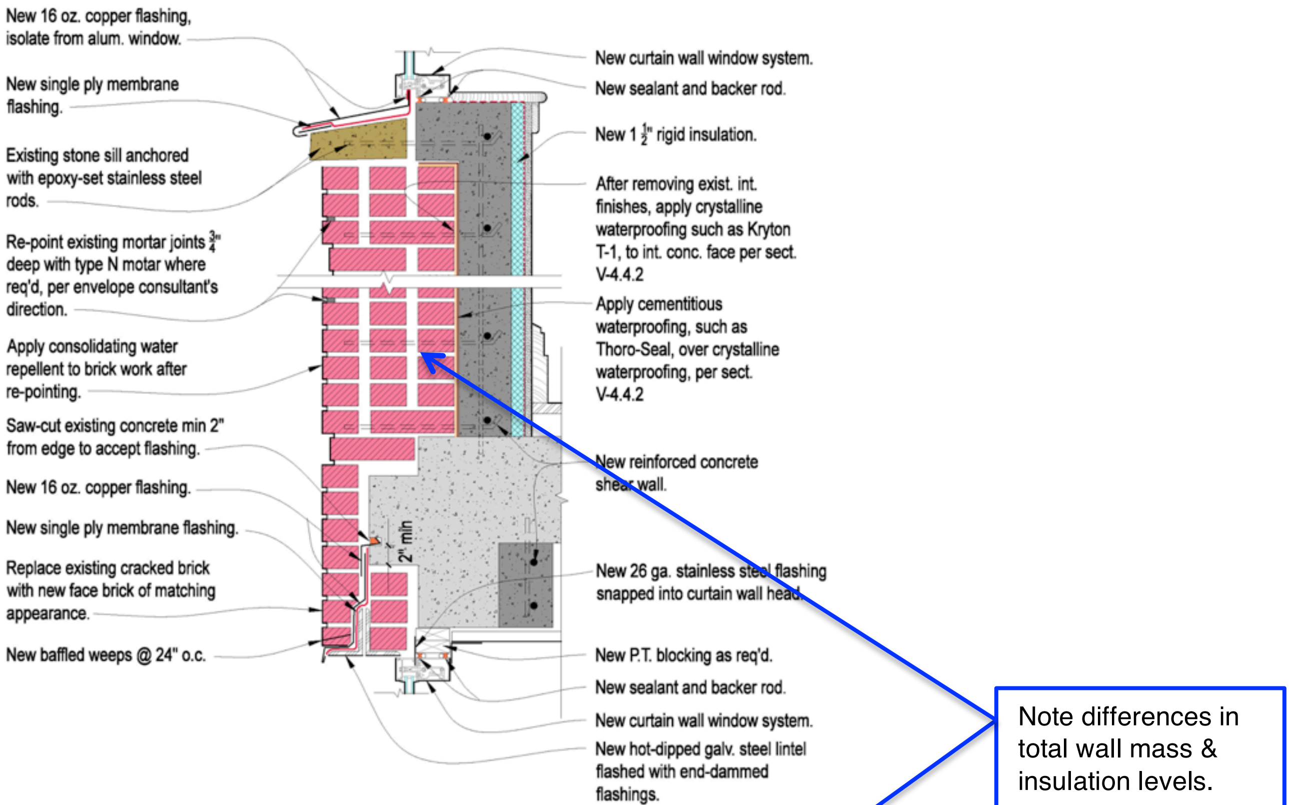


Fig. 3.113: Opt. 1 "Restoration" @ Brick Walls

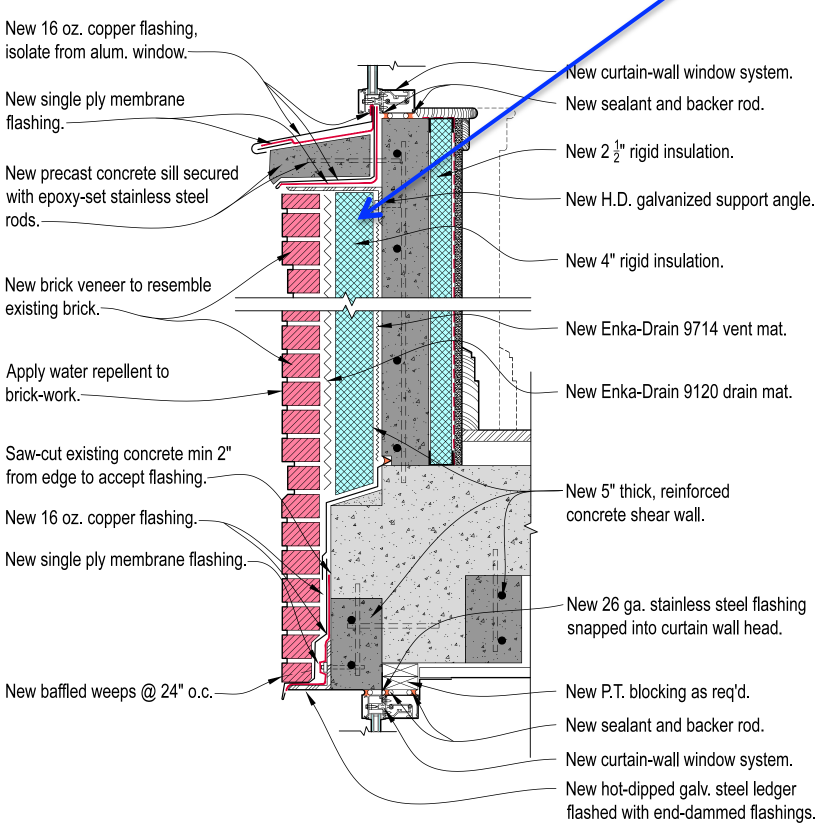


Fig. 3.114: Opt. 2 "Reconstruction" at Brick Walls

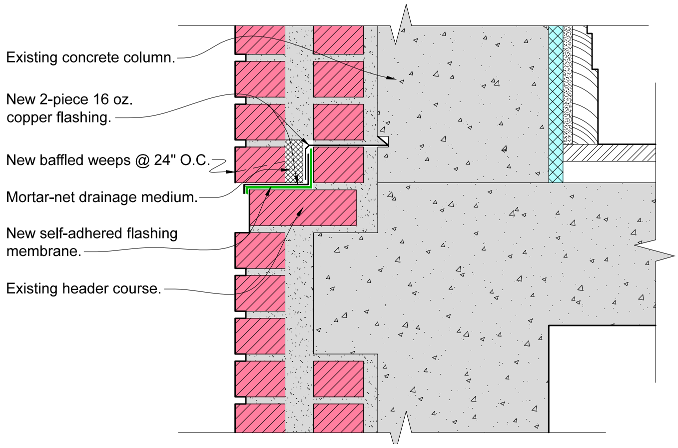


Fig. 3.115: Opt. 1 "Restoration Approach" at Retrofitted Floor-Line Through-Wall Flsh'gs.

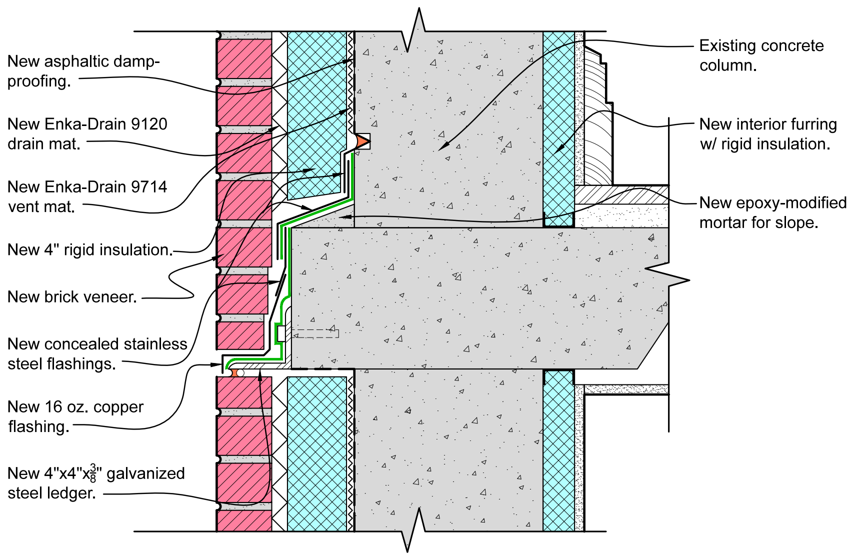


Fig. 3.116: Opt. 2 "Conc. Reconstruction Approach" at Floor-Line Through-Wall Flsh'gs.

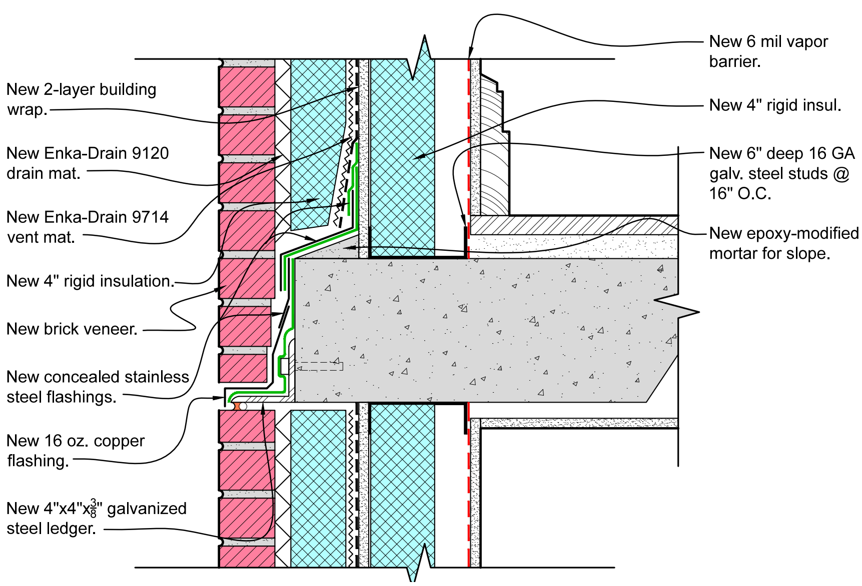


Fig. 3.117: Opt. 3 "Steel Reconstruction Approach" at Floor-Line Through-Wall Flashings

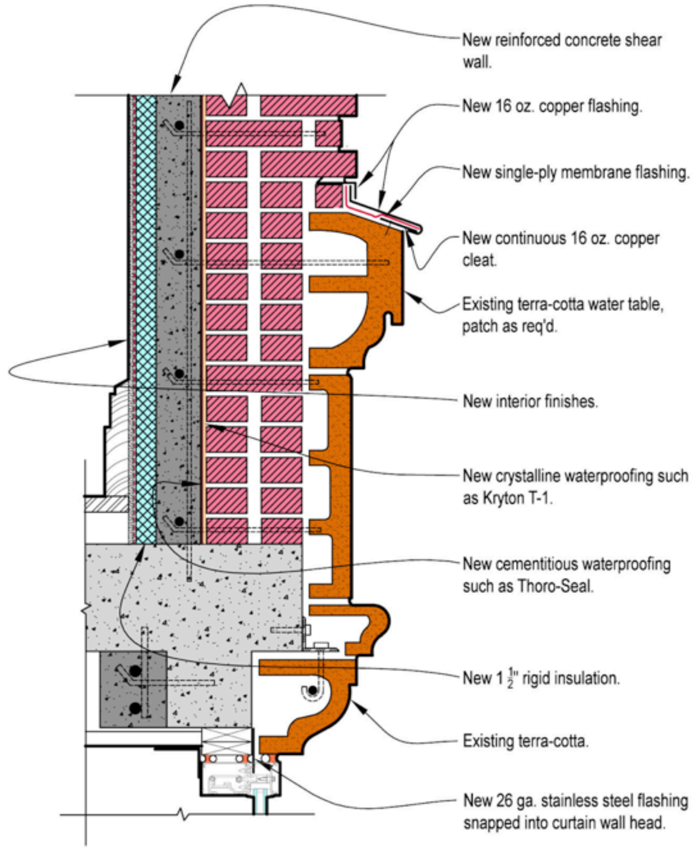


Fig. 3.118: Opt. 1 "Restoration Approach" at Level 5 Water-Table Band

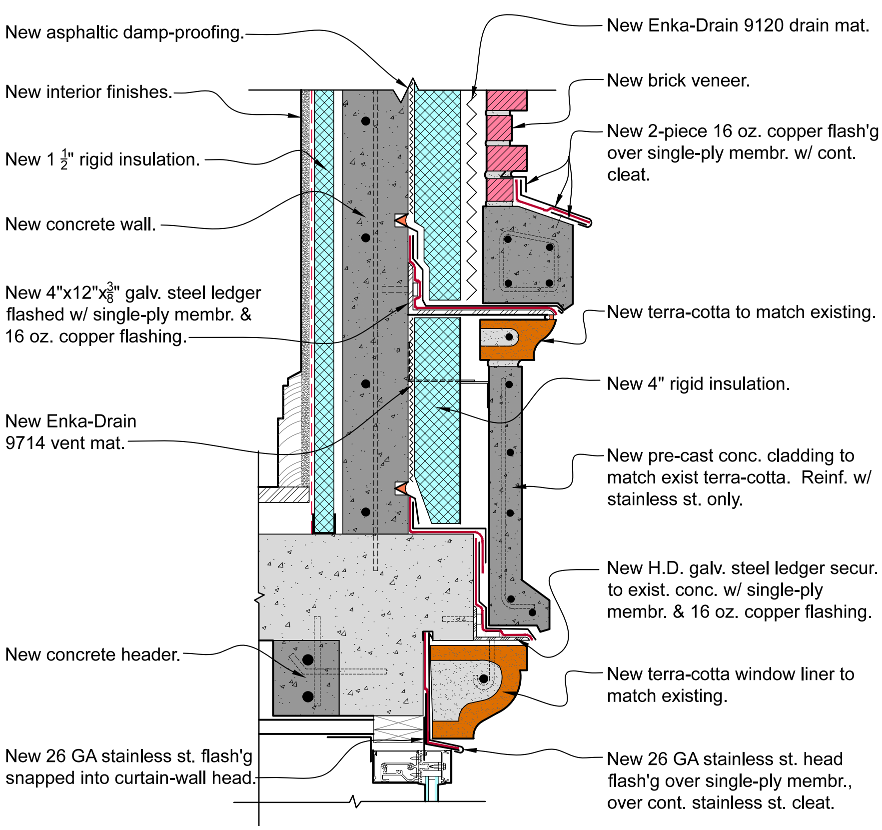


Fig. 3.119: Opt. 2 "Reconstruction Approach" at Level 5 Water-Table Band

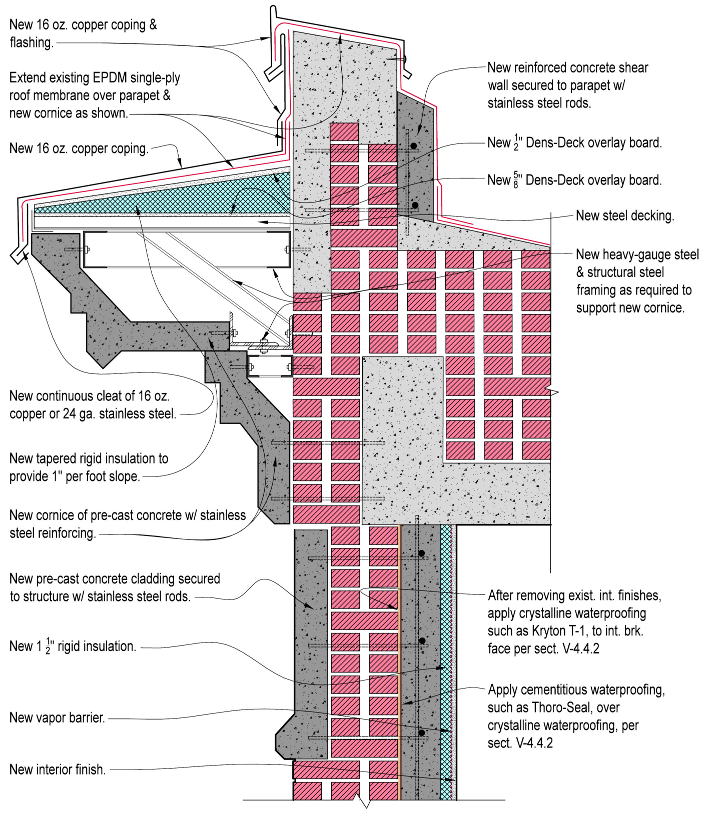


Fig. 3.120: Opt. 1 “Restoration Approach” at Reconstructed Roof-Level Cornice

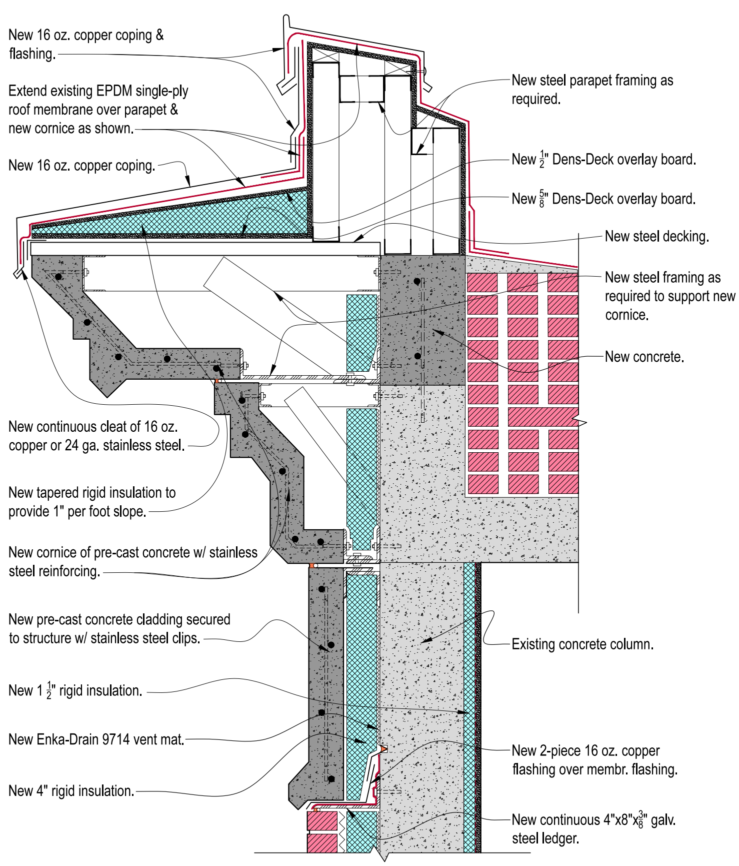


Fig. 3.121: Opt. 2 “Reconstruction Approach” at Reconstructed Roof-Level Cornice

All three options also included retrofitting a drainage system in the very wet crawlspace to drain the existing streams running through it, and to reduce the high levels of humidity to slow-down further corrosive spalling of the concrete floor joists.

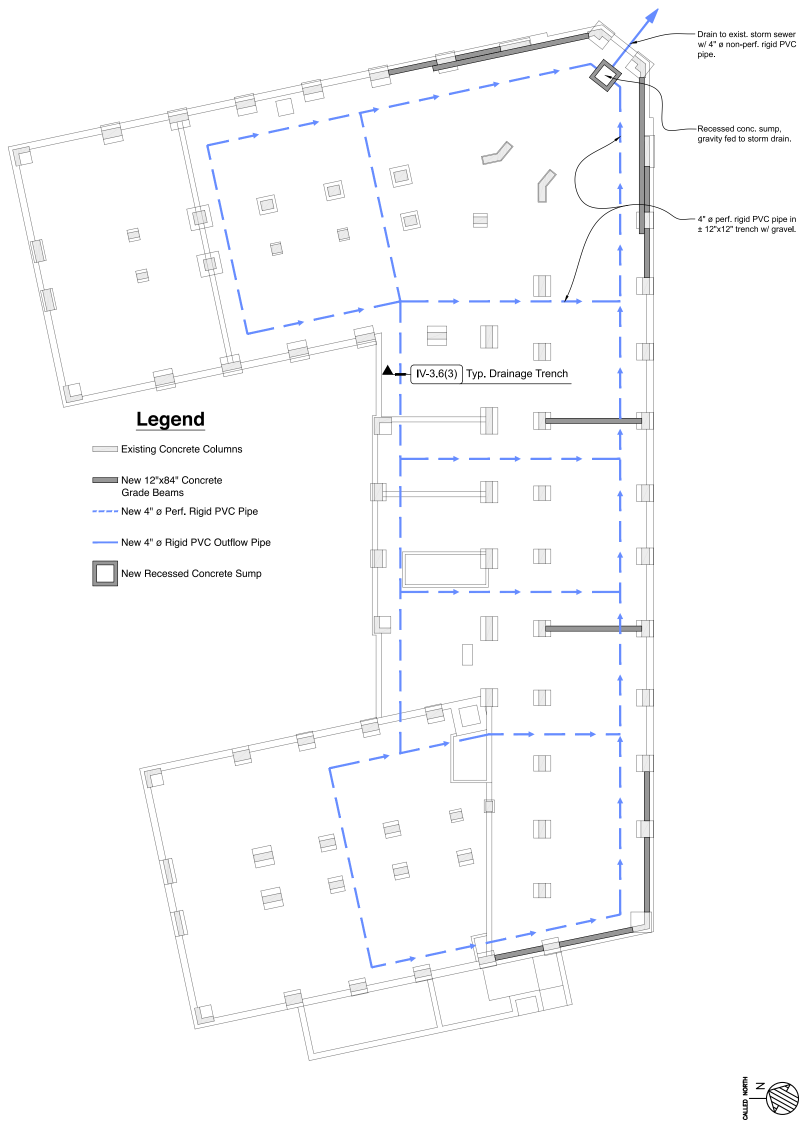


Fig. 3.122: Opt. 1-3 Crawlspace Drainage Layout

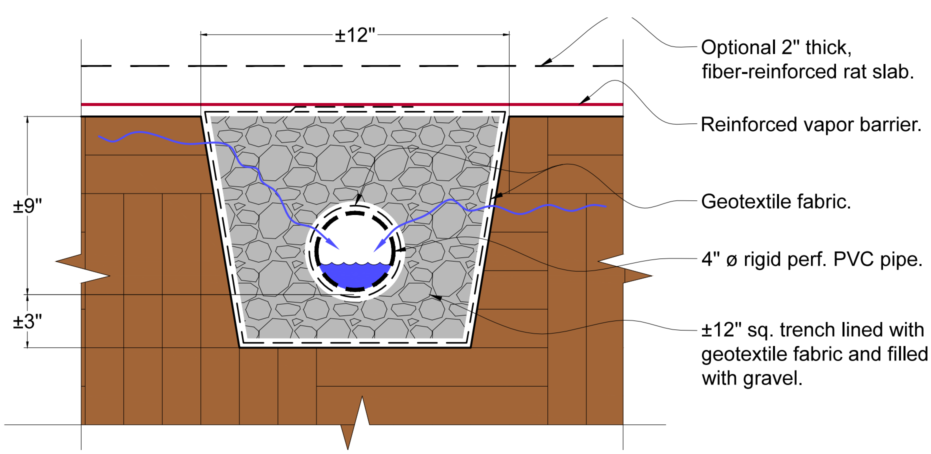


Fig. 3.123: Opt. 1-3 Crawlspace Drainage Line Section

Phase 4: Corrective Design, 2013-14

Given the Phase 3 team's expertise and deep familiarity with the Alaska Capitol's extensive problems, the same team was selected to carry forth with the corrective design, with the architect, Jensen Yorba Lott as the lead consulting firm, with all others sub-consulting to it.

Due to the potential life-safety risks posed by the seriously damaged portico, the Phase 4: Corrective Design was actually divided into two sub-phases, the first of which pertained to the reconstruction of the portico structure, while the second described work at the remainder of the building. This allowed the most critically needed corrective construction at the portico to proceed still in 2013, while the design for the following years' corrective work continued.

Corrective work at the portico would begin by removal of all portions of its structure, except for its four marble columns, which would then be core-drilled through their entire height to allow reinforcing strands to be grouted through these to tie the separate marble sections together and to the foundations. A new concrete-frame structure of beams, pilasters, and a roof slab would be cast atop these columns, with a temporary EPDM roof over this to protect the structure till the following year, when this skeleton would be clad with pre-cast concrete cladding to match the existing, severely damaged stone.

Beside the portico, the Phase 4 corrective design extended to the building's crawlspace, which would be excavated, with a new drainage system installed to contain and drain out the brooks running through the existing crawlspace, along with structural repairs to the corrosively-damaged concrete floor joists.

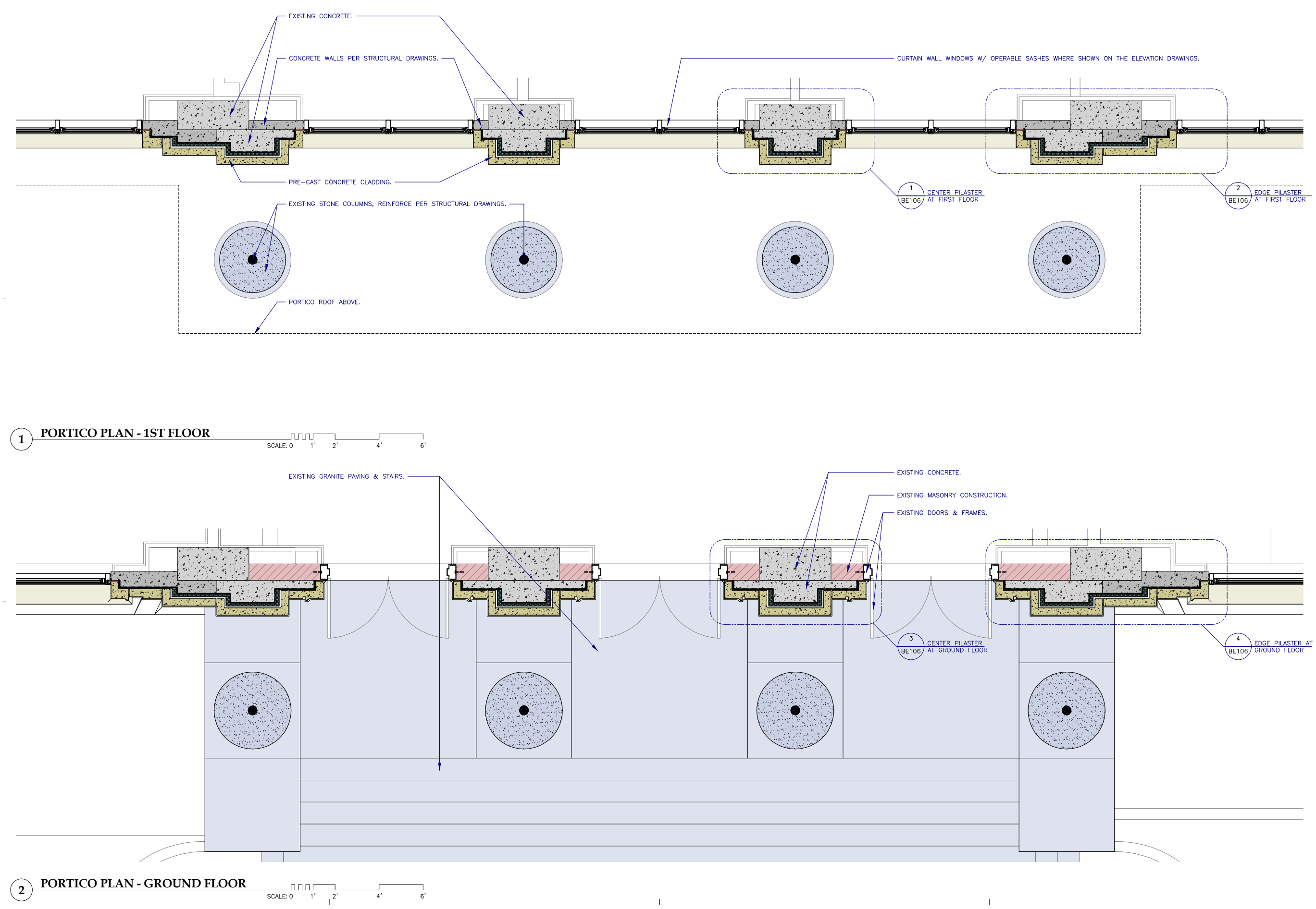


Fig. 4.1: Plan of Portico Corrective Work, Ground & 1st Floor

Fig. 4.2: E-W Section of Portico Corrective Work

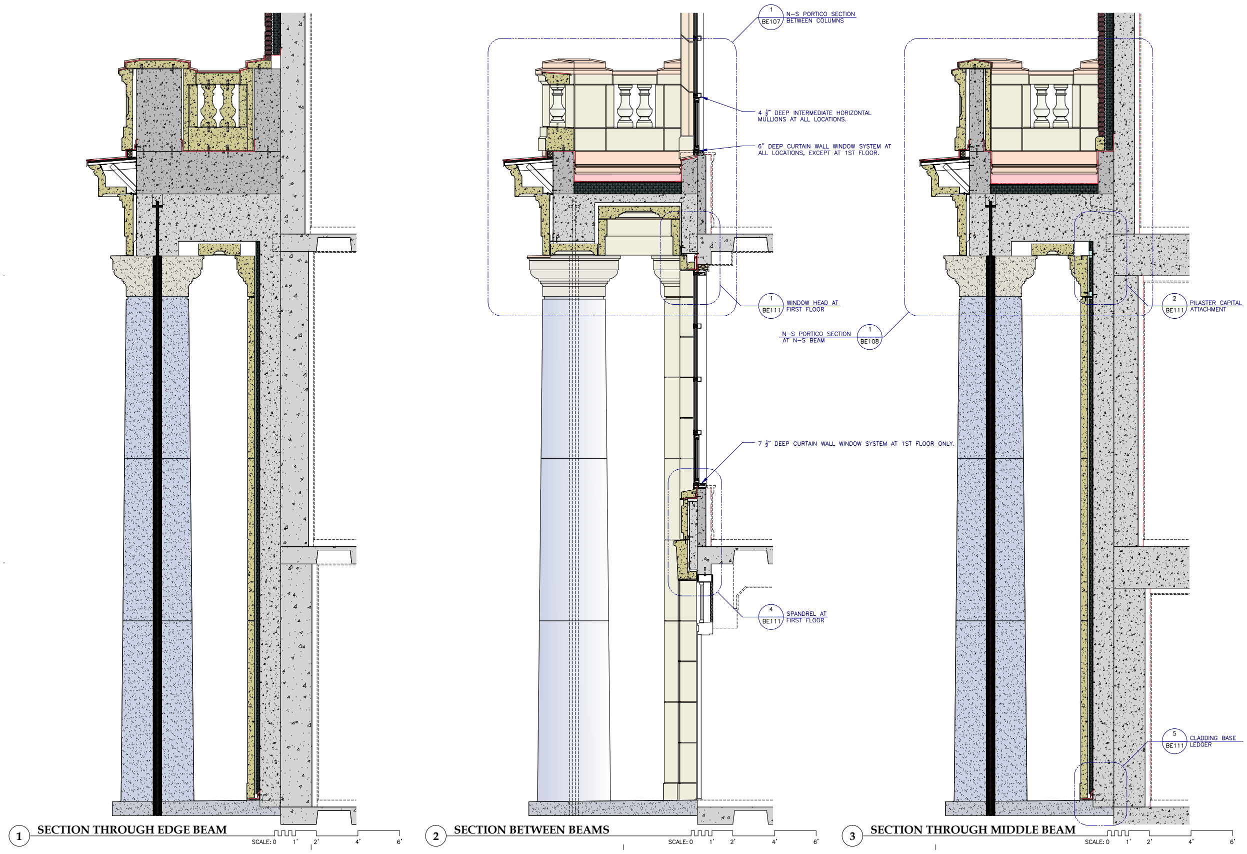


Fig. 4.3: N-S Section of Portico Corrective Work

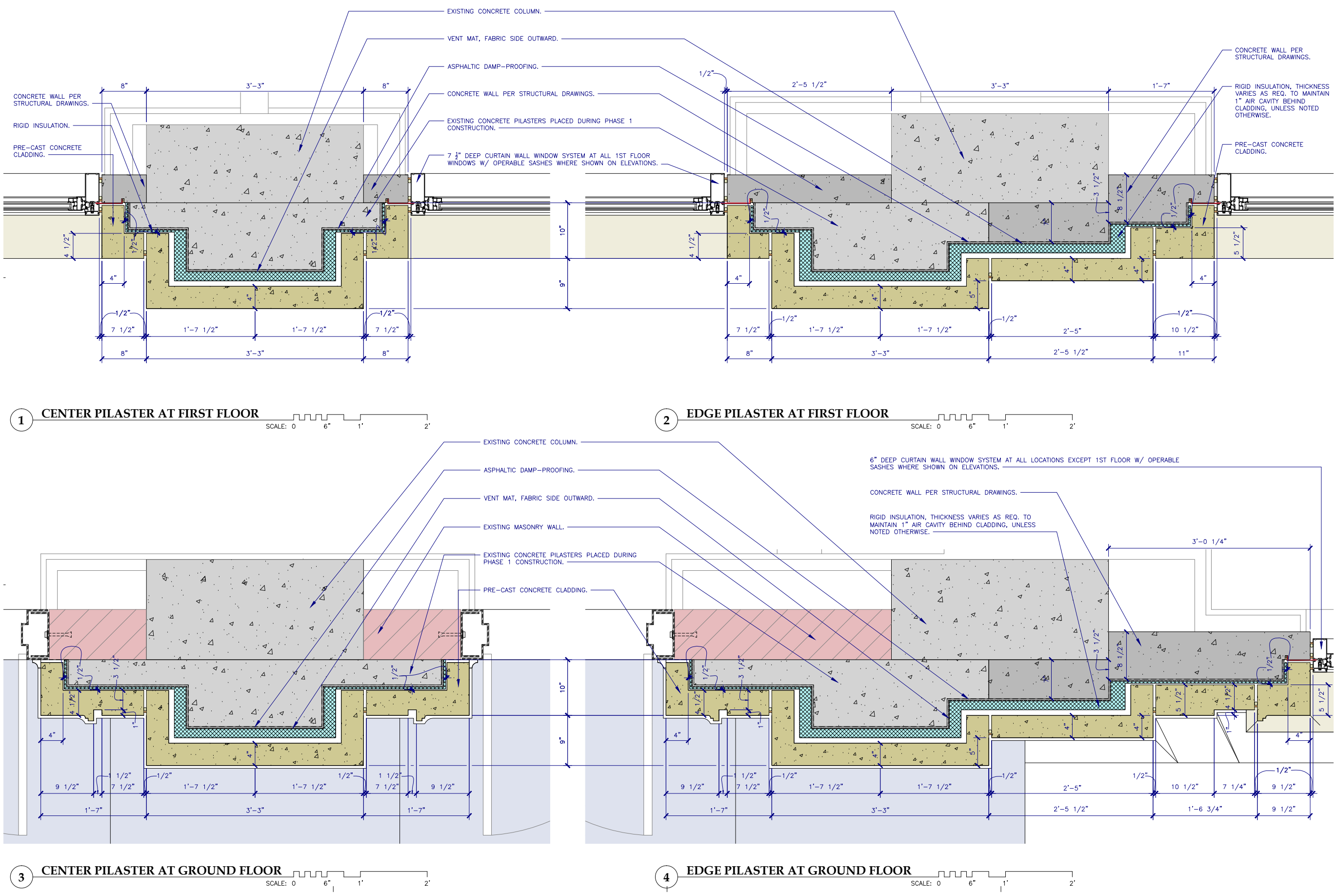


Fig. 4.4: Detailed Plan of Portico Corrective Work, Ground & 1st Floor

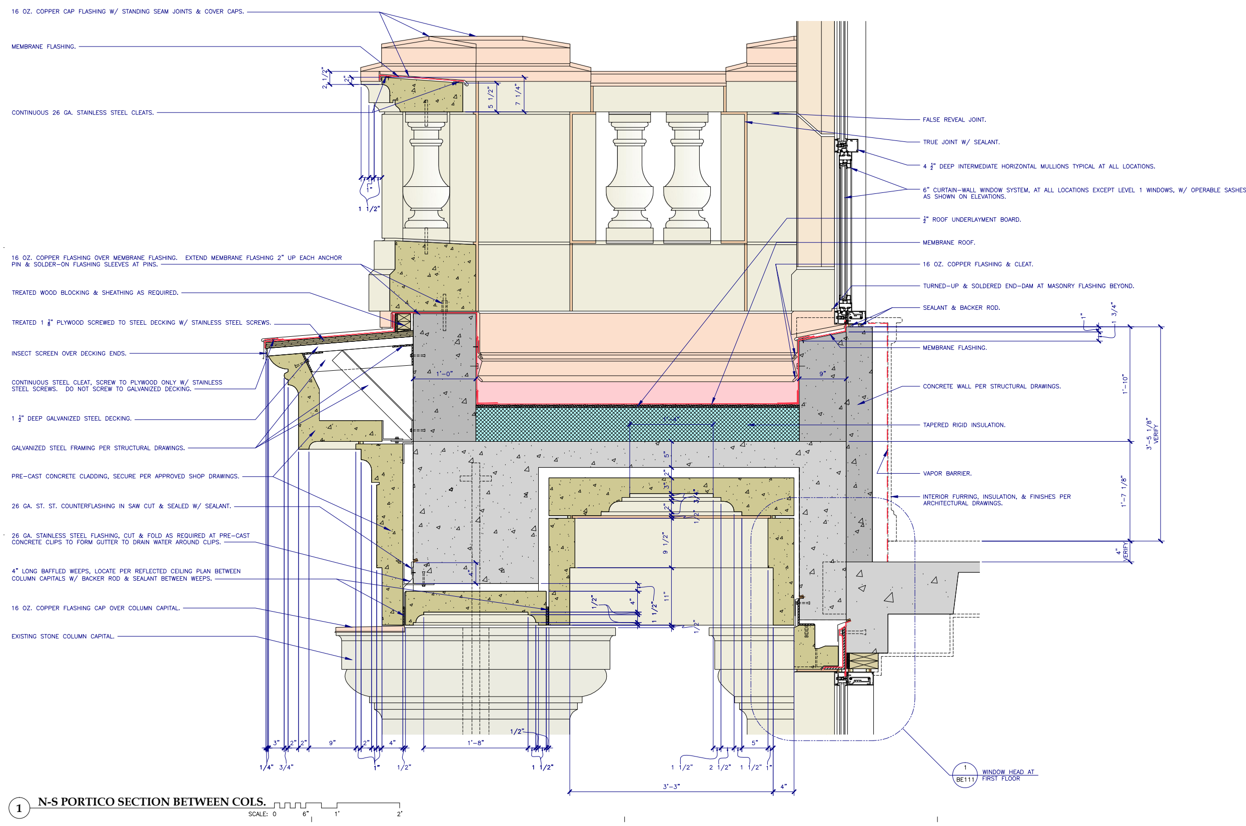


Fig. 4.5: N-S Section of Portico Corrective Work Between Columns

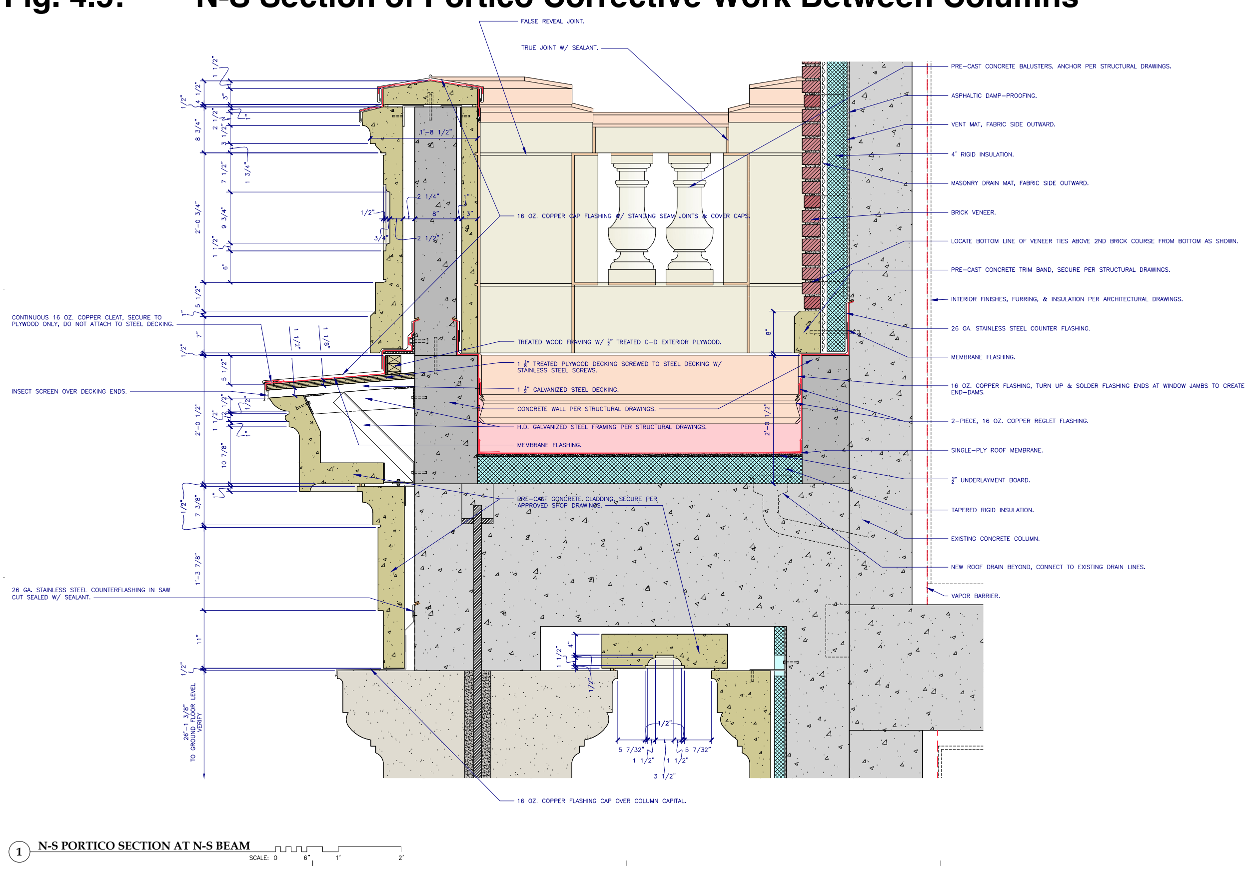


Fig. 4.6: N-S Section of Portico Corrective Work @ Columns

In brief, corrective work at the rest of the building, to be executed in 2014-16, consisted of complete removal of all exterior masonry to fully expose the building's concrete skeleton, installation of new shot-crete shear walls for seismic enhancement, and over-cladding the structure with a new masonry cladding to closely resemble the original building, while also incorporating many technical enhancements, including insulating the building with rigid insulation outside the concrete structure and adding interior insulation also for maximum energy-efficiency enhancement. As the existing brick face was 9" outside the concrete structure in many locations, this allowed addition of 4 ½" of rigid insulation. Elsewhere, where the masonry fell closer to the concrete skeleton, lesser amounts of insulation could be placed within the masonry cavities, where added interior insulation was of greater consequence. Depending on location, the new exterior walls had insulating values ranging from about R-20 to over R-40, compared to the R-3 to R-4 range possessed by the existing walls.

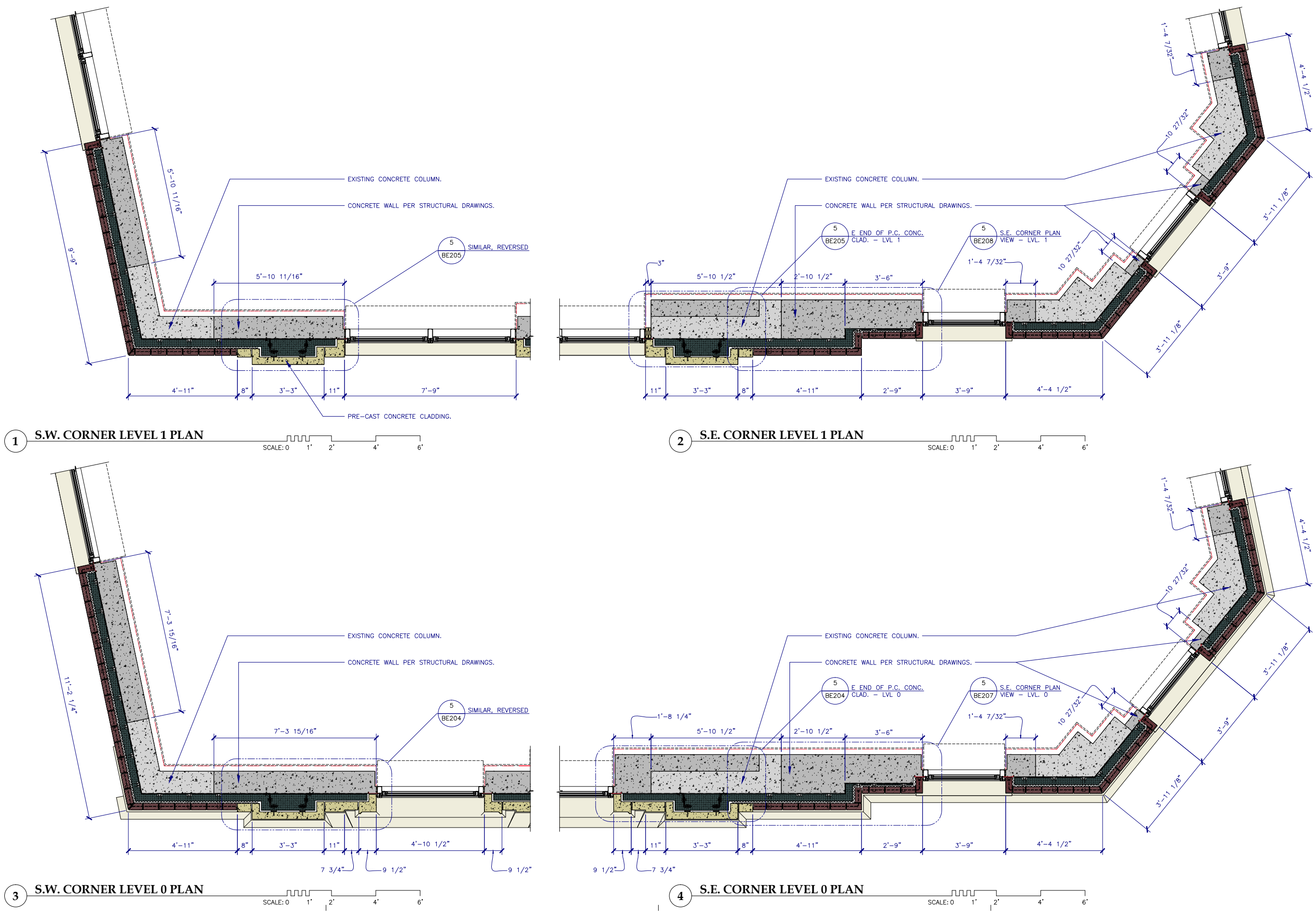


Fig. 4.7: Plan of Building Corrective Work, Ground & 1st Floor

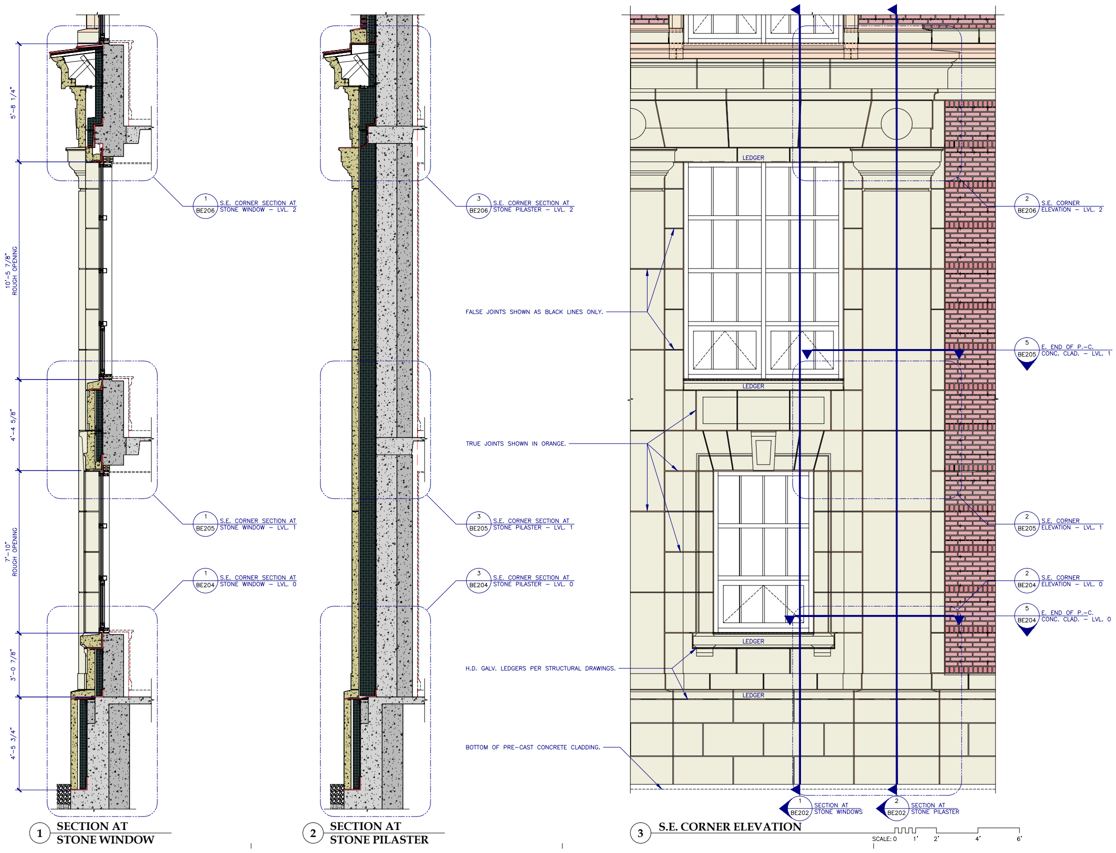


Fig. 4.8: N-S Section of Ext. Walls @ New P.-C. Conc. Cladding, Ground & 1st Floor

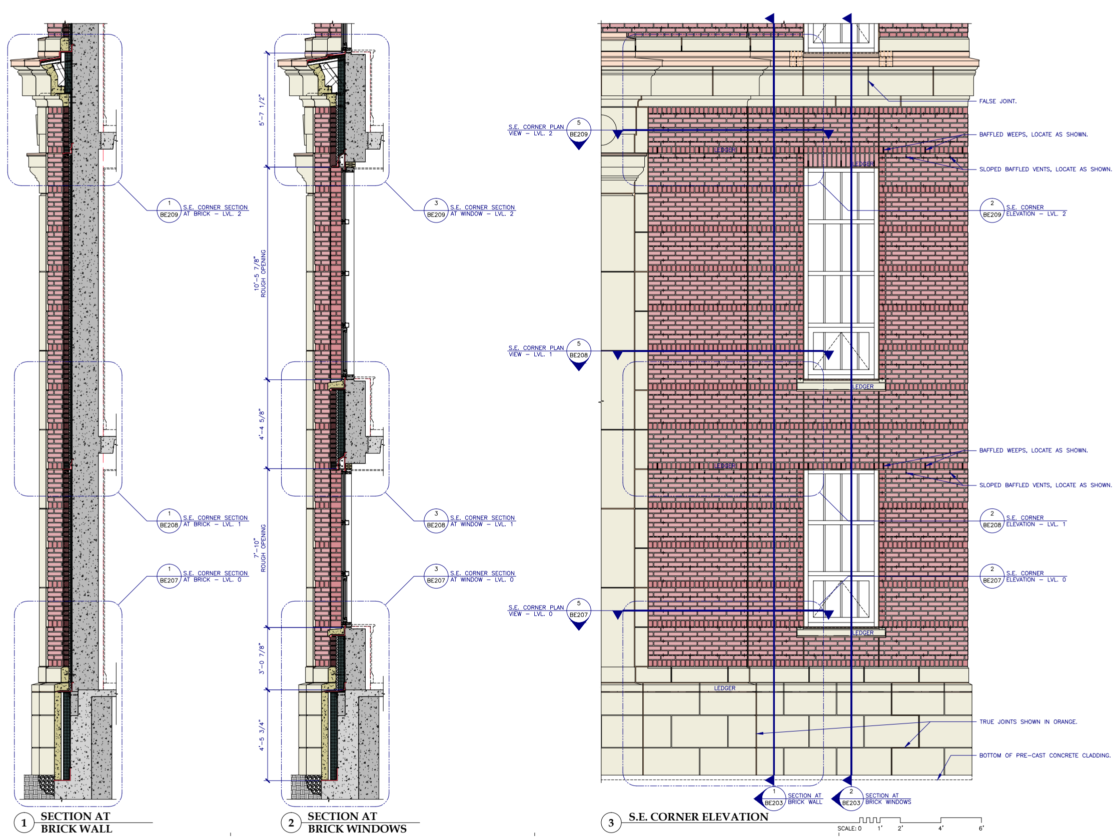


Fig. 4.9: N-S Section of Ext. Walls @ New Brick Veneer Cladding, Ground & 1st Floor

The building's degraded limestone level 2 water-table would be replaced with a new, much lighter pre-cast concrete water-table, capped with a membrane and copper flashings to preclude infiltration into, and degradation of the water-table and all wall elements below.

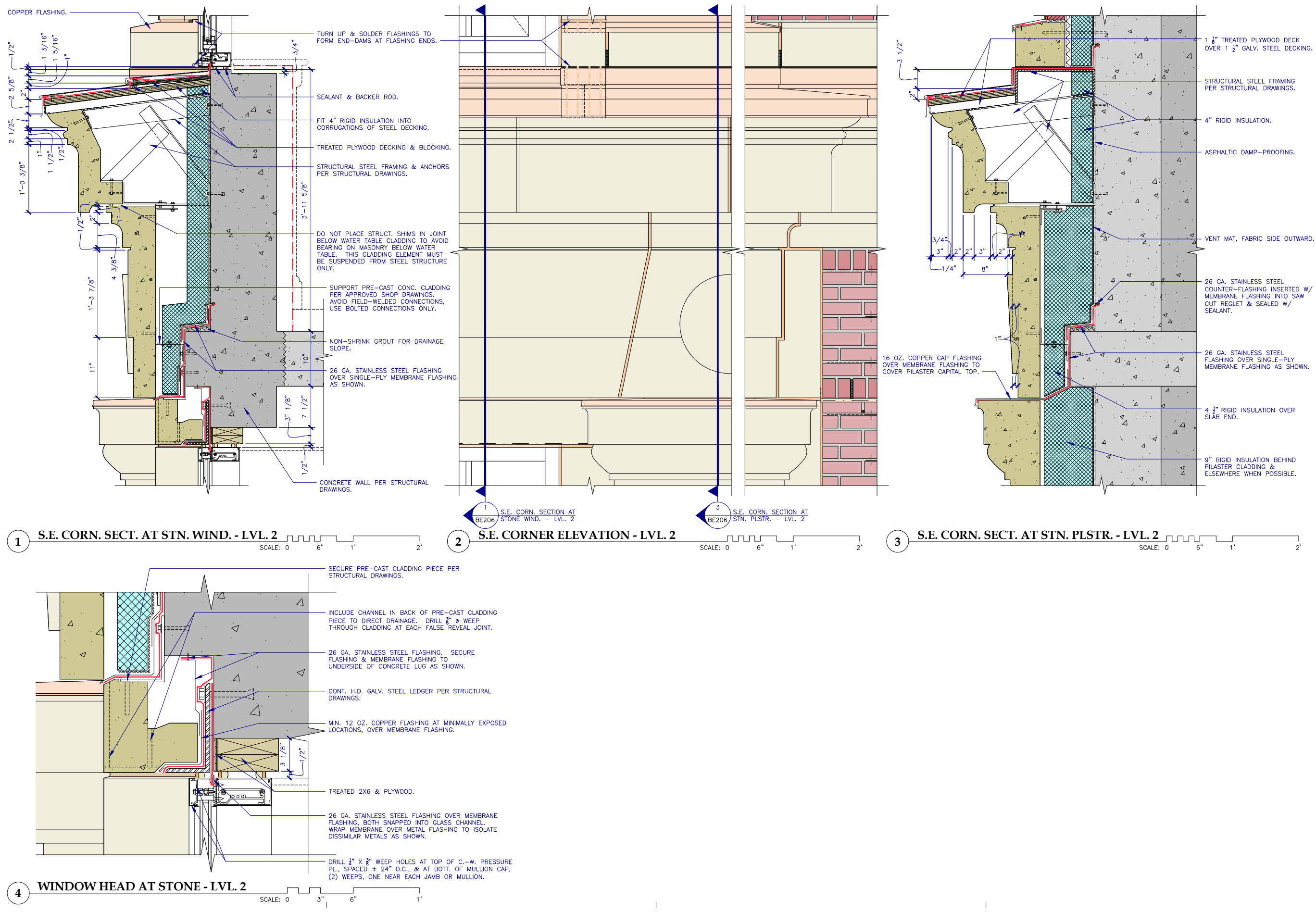


Fig. 4.10: Work at Level 2 Water-Table

Where the existing masonry was 9” outside the building’s concrete structure, as was the case at many of the “public” exterior walls, 4 ½” of exterior rigid insulation was added, sandwiched between a drainage mat directly inward of the brickwork, and a thinner vent mat placed against the damp-proofed exterior concrete walls. The significant gap between the structure and the brick at these locations also required that the ledgers supporting the brickwork be furred about 4” away from the concrete walls, and to accommodate variations in the existing concrete surface, an adjustable, double-angle furring system was designed for these ledgers. Locations where the brick was near the structure could use the conventional method of securing the ledgers directly to the structure.

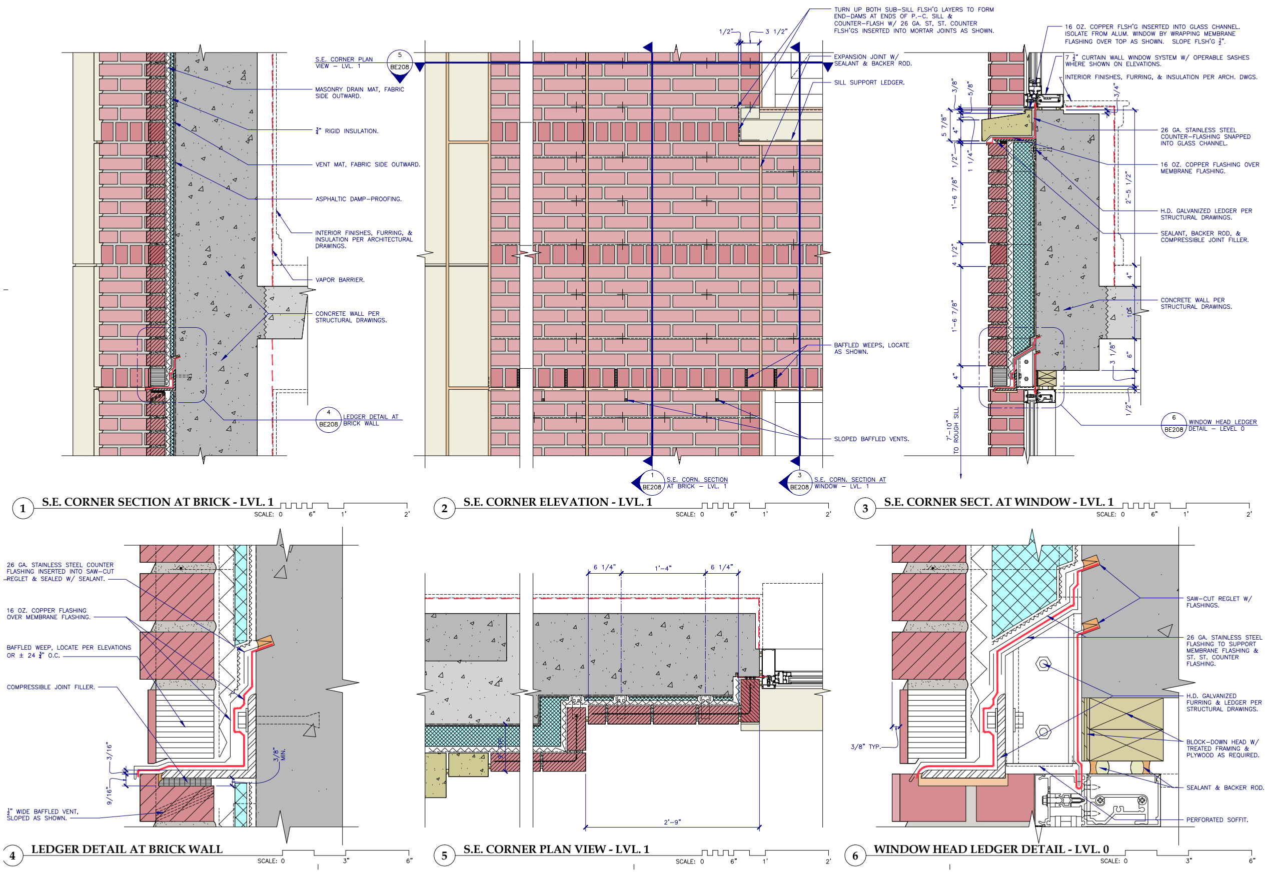


Fig. 4.11: Typical Brick Walls @ Furred & Un-Furred Ledgers

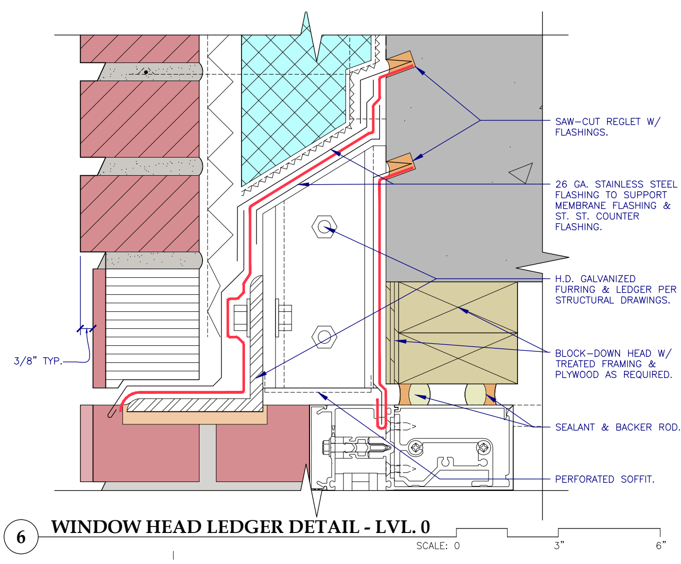


Fig. 4.12: Furred Ledger Above Window Head

The building's degraded terra-cotta spandrels, which separate windows between different floor levels, were replaced with new terra-cotta spandrels matching the existing ones in appearance, but incorporating copper sill caps to greatly reduce water-infiltration; drainage flashings at their bases to allow drainage from behind the spandrels; double-stage, drained sealant joints to resist water entry at the joints, and new rigid and batt insulation.

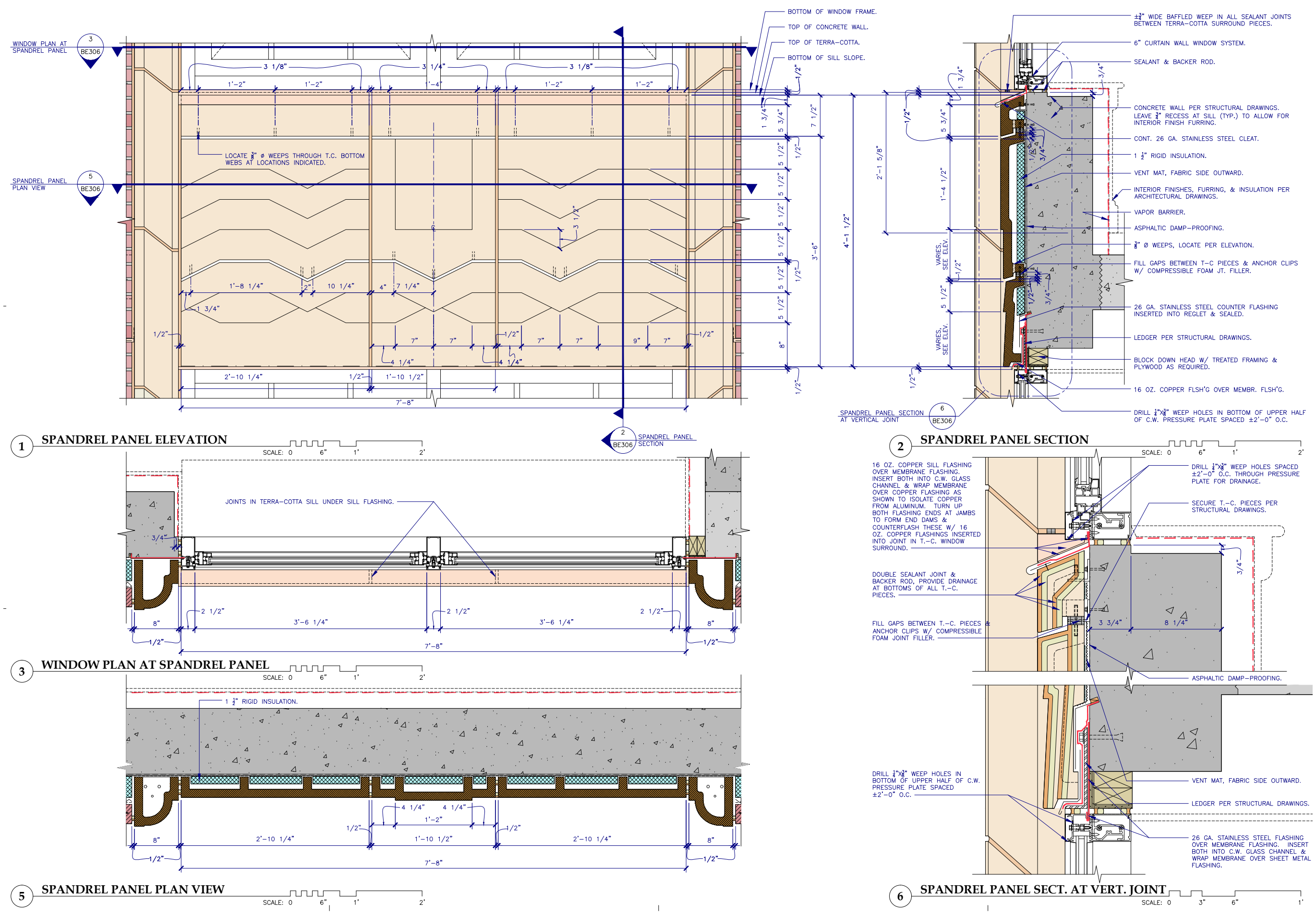


Fig. 4.13: Replacement of Terra-Cotta Window Spandrels

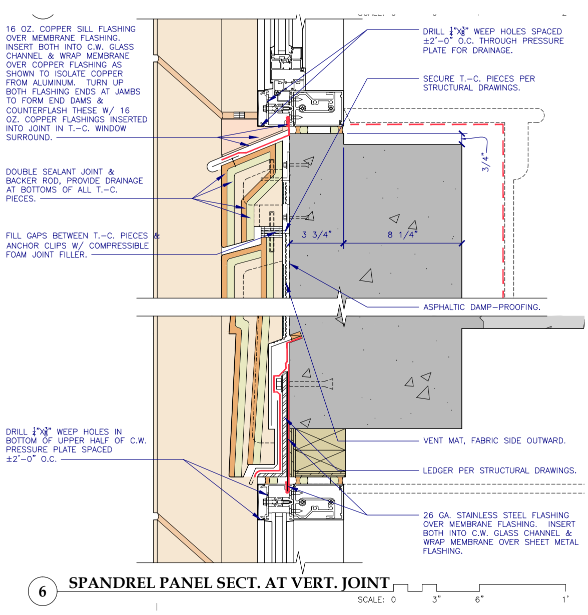


Fig. 4.14: Double-Stage, Drained Sealant Joints @ Terra-Cotta Spandrel Joints

The building's level 5 terra-cotta water-table band was replaced with a new terra-cotta water-table atop a pre-cast concrete band, essentially matching the existing water-table's appearance, but again incorporating copper sill caps to greatly reduce water-infiltration; drainage flashings at their bases to allow drainage from behind the band; double-stage, drained sealant joints to resist water entry at the joints; and exterior rigid and interior batt insulation to greatly enhance energy-efficiency.

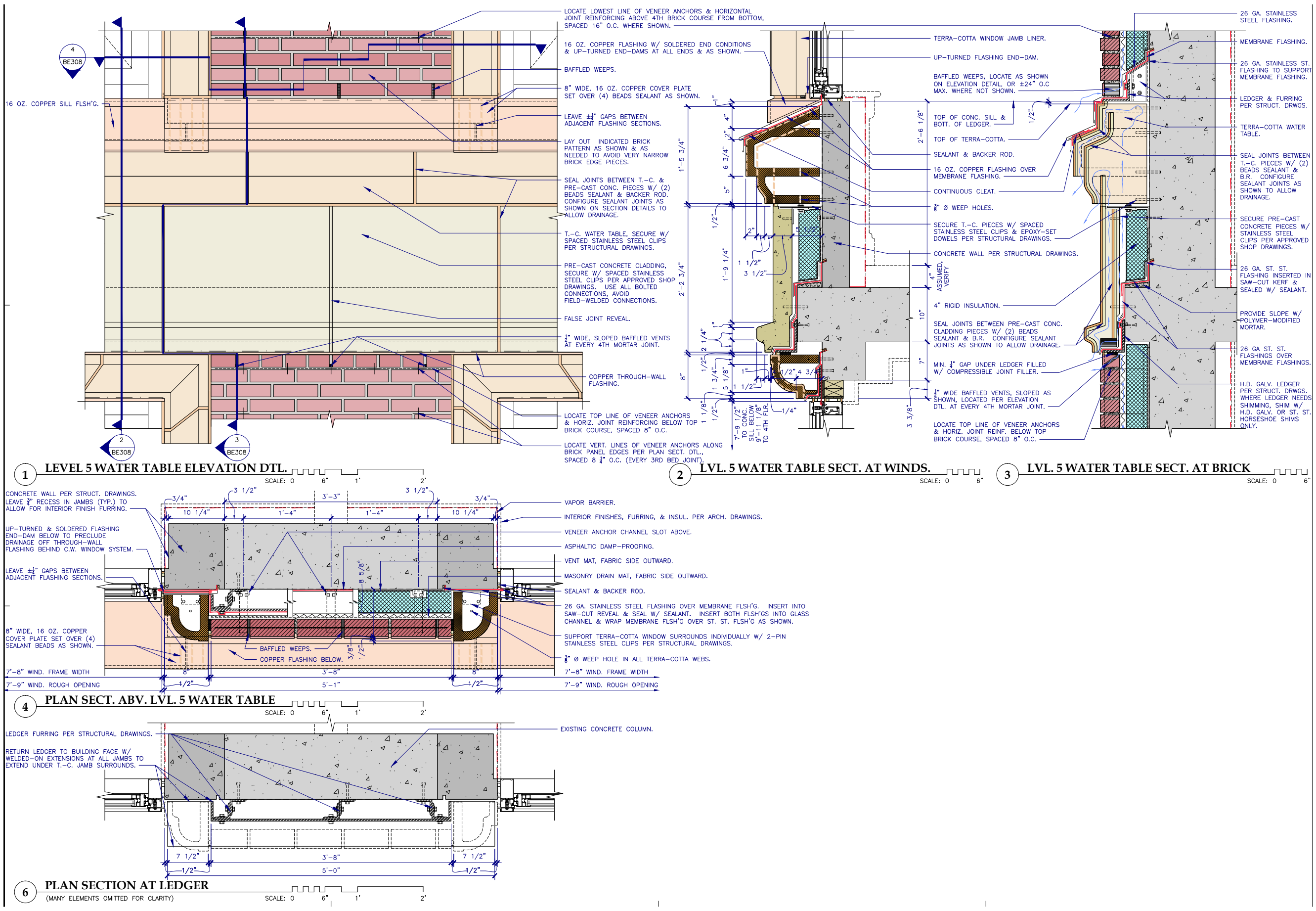


Fig. 4.15: Replacement of Level 5 Terra-Cotta Water-Table

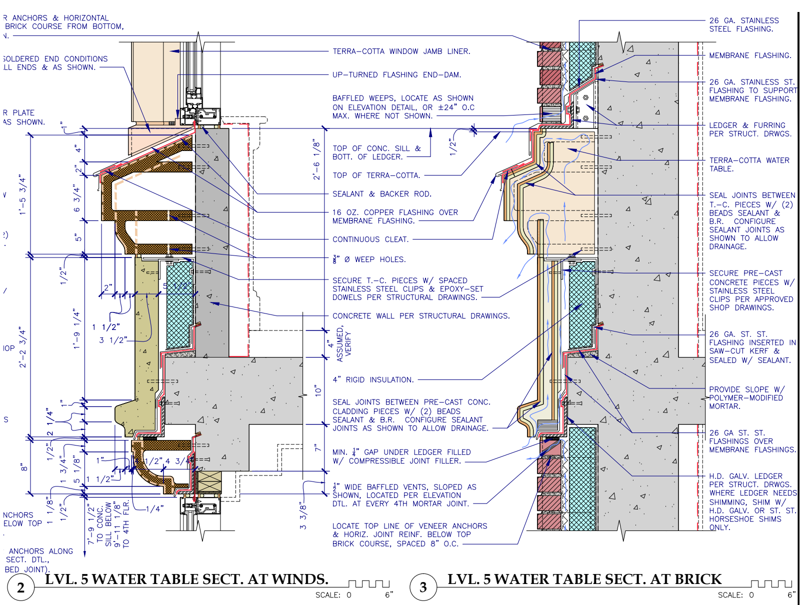


Fig. 4.16: Double-Stage, Drained Sealant Joints @ Terra-Cotta Water-Table Joints

The long-ago removed terra-cotta cornice band was replaced with a new pre-cast concrete cornice, essentially matching the original appearance, but incorporating a standing-seam copper roof to greatly reduce water-infiltration; drainage flashings to allow drainage from behind this band; double-stage, drained sealant joints to resist water entry at the joints; and exterior rigid and interior batt insulation to greatly enhance energy-efficiency.

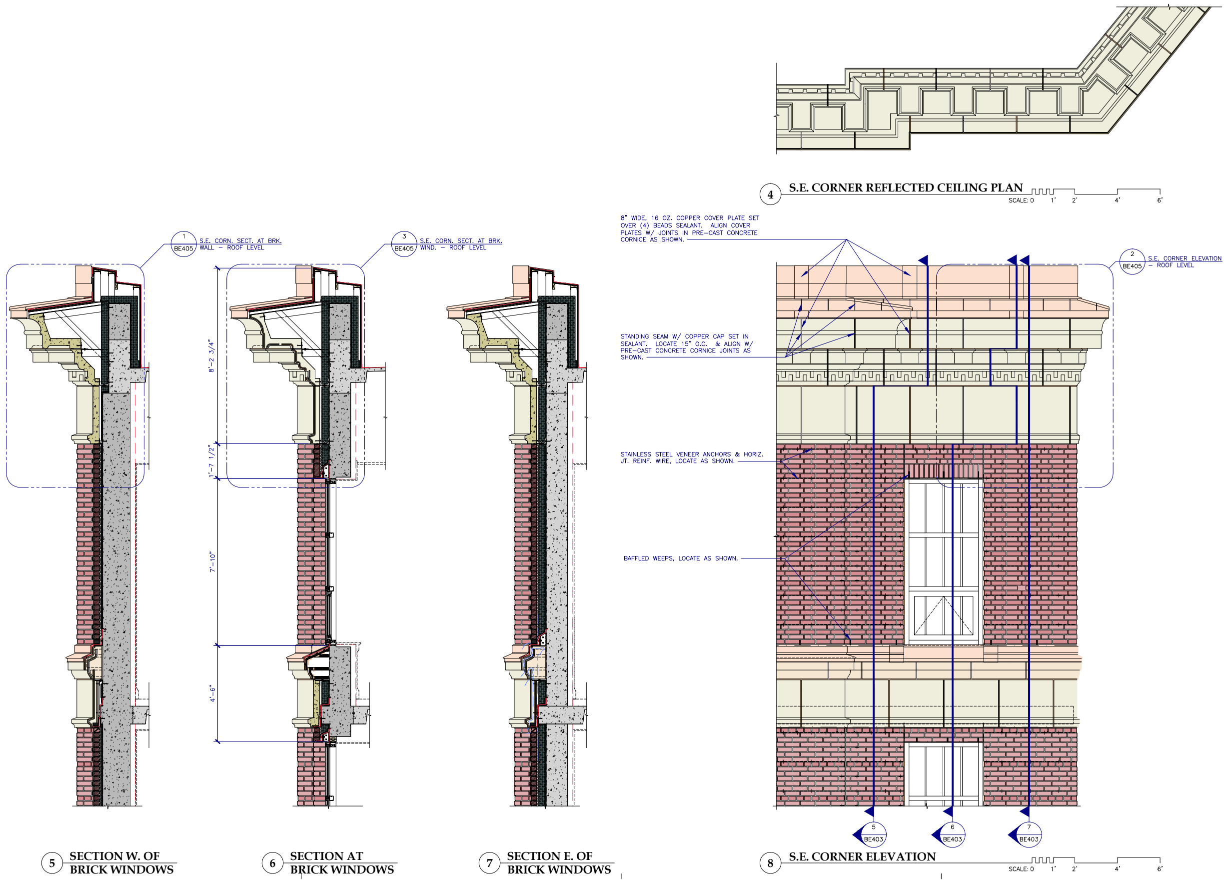


Fig. 4.17: Replacement of Original Terra-Cotta Cornice

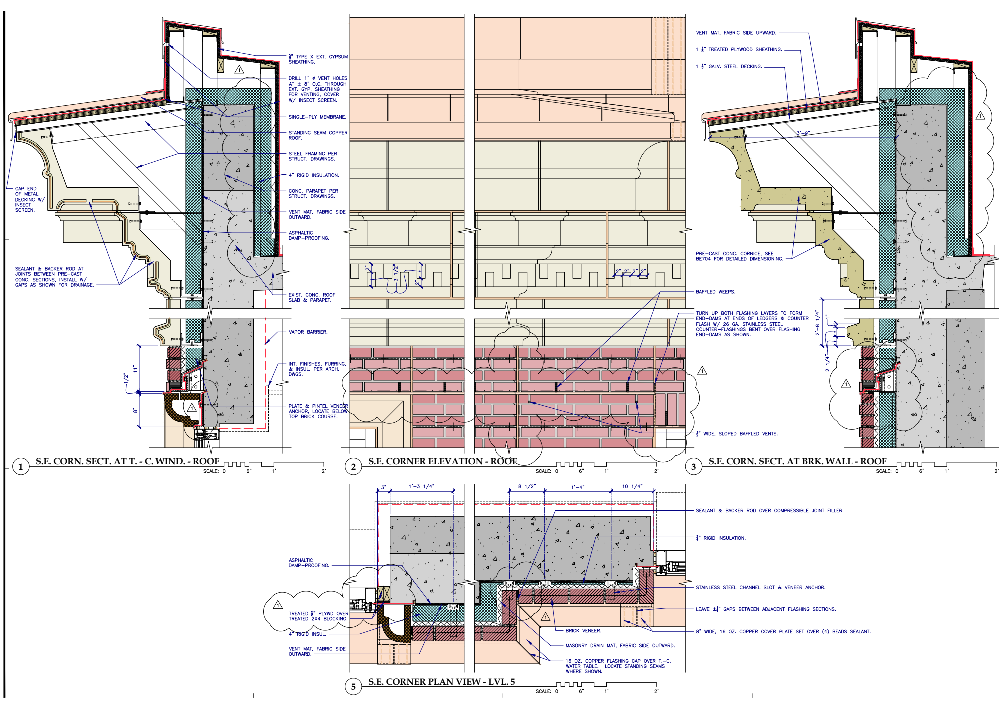


Fig. 4.18: Cornice Re-Construction Details

To allow the masonry to dry out as rapidly as possible following each rain, the masonry design incorporated both weeps at panel bottoms to drain water out and allow air to enter behind the cladding, as well as outward-sloping panel-top vents to allow air to exhaust out from behind the cladding, thus setting up a thermo-siphon drying effect.

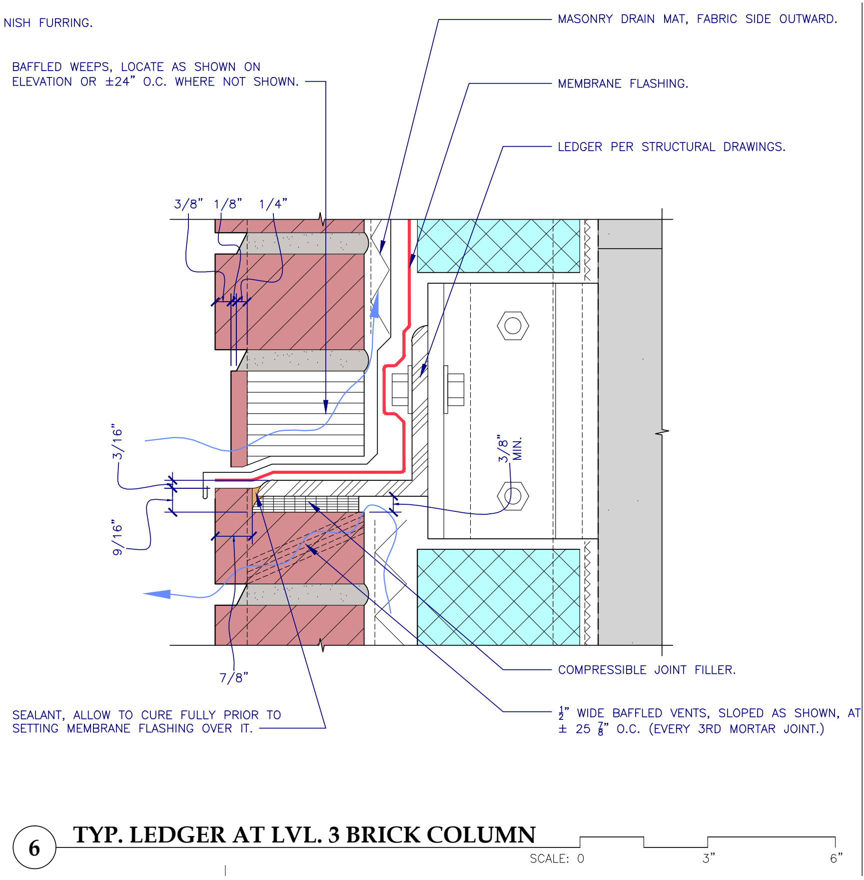


Fig. 4.19: Panel-Bottom Weeps & Panel-Top Vents to Optimize Drying of the Masonry

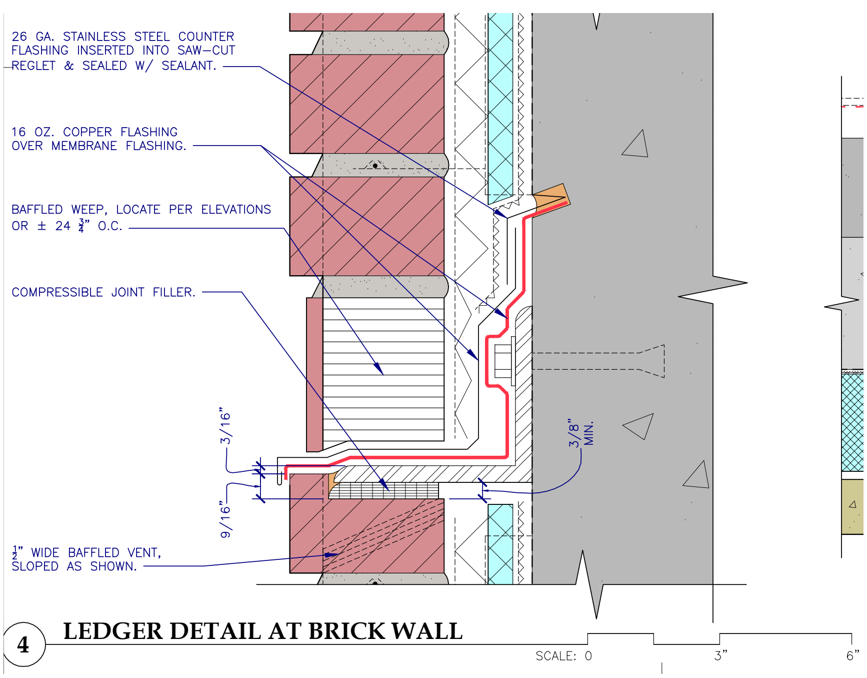


Fig. 4.20: Panel-Bottom Weeps & Panel-Top Vents to Optimize Drying of the Masonry

Phase 5: Corrective Construction, 2013-16

Phase 5.1 of the corrective work began with the Portico in 2013 to restore safety as quickly as possible. The work began by removal of all portions of its structure, except for its four marble columns, which were core-drilled through their entire height and reinforced with continuous, grouted-in steel strands to tie the separate marble sections together. A new concrete-frame structure of beams, pilasters, and a roof slab was cast atop these columns, with a temporary EPDM roof over this to protect the structure till the following year.

Fig. 5.1: Stairs Removed to Expose Found.

Fig. 5.2: Bracing the Marble Columns

Fig. 5.3: Bracing of Building Masonry

Fig. 5.4: Demolition of Portico Structure

Fig. 5.5: Demolition of Building Masonry

Fig. 5.6: Demolition of Building Masonry

Fig. 5.7: Demolition of Portico Beam

Fig. 5.8: Removal of Portico Beam

Fig. 5.9: Demolition of Portico Roof Slab

Fig. 5.10: Completed Portico Demolition

Fig. 5.11: Completed Portico Demolition

Fig. 5.12: Inner Wythes of Brickwork

Fig. 5.13: Inner Wythes of Brickwork

Fig. 5.14: Drilling of Portico Foundations

Fig. 5.15: Retrofitting Foundation Reinf.

Fig. 5.16: New Portico Stair Foundations

Fig. 5.17: New Portico Stair Foundations

Fig. 5.18: New Portico Formwork & Reinf.

Fig. 5.19: Core-Drilling Marble Columns

Fig. 5.20: Core-Drilling Marble Columns

Fig. 5.21: Removed Marble Column Cores

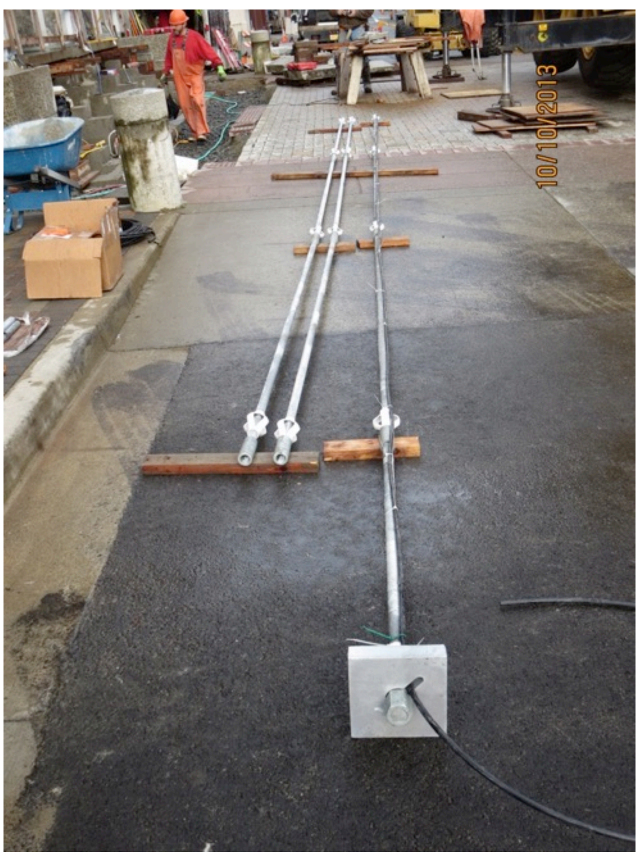


Fig. 5.22: Column Reinforcing Strands

Fig. 5.23: Column Reinforcing Stands

Fig. 5.24: Grouting Column Reinforcing

Fig. 5.25: Grouting Column Reinforcing

Fig. 5.26: Completed Portico Structure

Fig. 5.27: Re-Installed Capitol Steps

This initial phase of the corrective work also included excavation of the very wet crawlspace under the building, installing a crawlspace drainage system, and repairing the many concrete joists and piers which had become seriously damaged by corrosive spalling. This required cutting an access hole through the building's exterior foundation wall to allow equipment access, and the very short head-room and tight spaces necessitated use of Tonka-toy sized excavation equipment.


Fig. 5.28: Cutting Crawlspace Access Hole


Fig. 5.29: Cutting Crawlspace Access Hole


Fig. 5.30: Crawlspace Access Hole

Fig. 5.31: Excavating Crawlspace

Fig. 5.32: Excavating Crawlspace


Fig. 5.33: Crawlspace Drainage System

Fig. 5.34: Cleaned-Off Joist Reinforcing

Fig. 5.35: Shot-Creting Damaged Joist

Fig. 5.36: Repaired Floor Joist

Fig. 5.37: Repaired Concrete Pier

In 2014, corrective work began on the main portion of the building, starting with removal of the brick walls on the west and north sides, followed by installation of new concrete shear walls, damp-proofing, and windows, which is all Juneau's short construction season allowed, so the building was buttoned-up to make it through the winter, and the re-construction began on the west side in 2015.

Fig. 5.38: Scaffolding the West Side

Fig. 5.39: Demolition of Roof Parapet

For sake of installation simplicity, the new concrete shear walls were placed using the shot-crete method, wherein concrete is shot into place from a controlled hose nozzle.

Fig. 5.40: New Shear Wall Reinforcing

Fig. 5.41: Shot-Creting New Shear Walls

Fig. 5.42: New Concrete Shear Walls & Windows, W. Side

New concrete ledges were cast along the wall bases to support the new masonry veneer, and were capped with EPDM membrane, capped in turn by copper flashings with end-dams at typical end conditions. The damp-proofed concrete walls were then covered with a thin vent mat, followed by 4 ½" of polyisocyanurate insulation, overlaid with a ¾" thick, fabric-lined drainage mat to preclude clogging of the drainage cavity with mortar droppings.

Fig. 5.43: New Concrete Support Ledge

Fig. 5.44: EPDM Membrane Caps Ledges

Fig. 5.45: New Thr.-Wall Copper Flsh'gs.

Fig. 5.46: Brick Veneer Resembles Existing

Fig. 5.47: New Brick Veneer Cladding

Fig. 5.48: Insul. & Drain. Mat Layers @ Brick

For optimal seismic performance, the new brick veneer cladding was anchored to the primary structure with stainless steel seismic wire ties spaced roughly 16” O. C. in both directions, which were integrated with 9-gage horizontal reinforcing wire. In addition, all brick panel tops and vertical edges were anchored with wire ties spaced 8” apart. Vertical expansion joints were incorporated along all natural stress lines in the brick veneer to limit thermal and seismic stresses.

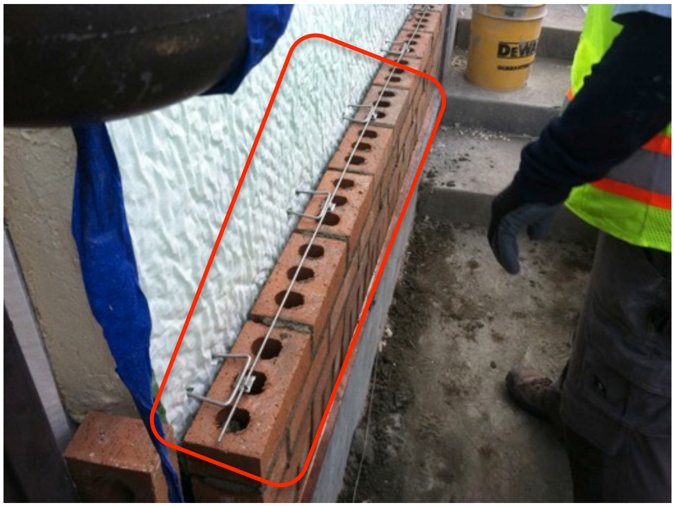


Fig. 5.49: Seismic Ties & Horiz. Jt. Reinf.

Fig. 5.50: Brick Exp. Joints @ Stress Lines

To accommodate dimensional variations in the existing concrete walls and the need to place the brick veneer 9” outward of these walls, an adjustable, double-angle furring system was designed to support the brick ledgers.

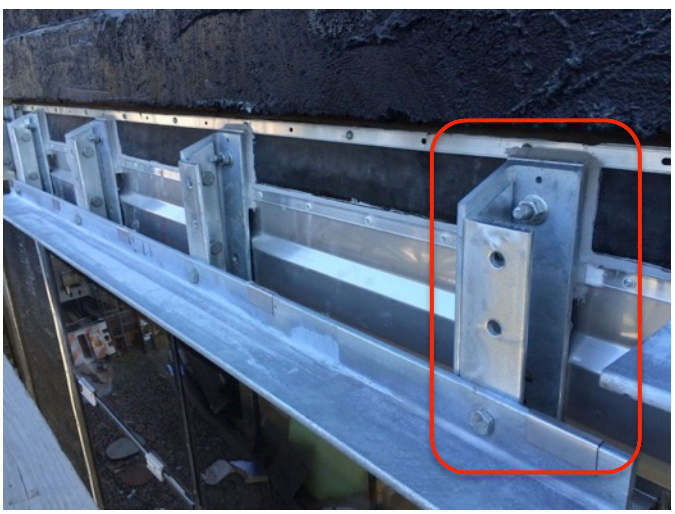


Fig. 5.51: Adjust. Dbl.-Angle Ledger Furring

Fig. 5.52: Adjust. Dbl.-Angle Ledger Furring

The ledgers were flashed with EPDM membrane, capped with 16 ounce copper flashings. All copper flashings were underlain with membrane to preclude water which may leak through joints in the copper flashings from reaching the ledgers, as well as to provide electrical isolation between the copper flashings and the galvanized steel ledgers to preclude electrolytic corrosion. To limit cost, concealed counter-flashings behind the masonry consisted of type 304 stainless steel, which is compatible with both copper and galvanized steel in most situations.

Fig. 5.53: EPDM Membrane Over Ledgers

Fig. 5.54: Copper & St. Steel Ledger Flsh'gs.

To allow good drainage from behind the brick veneer, baffled weeps were placed along all masonry panel bottoms, typically spaced 24" apart, and to accelerate drying of the masonry between rains, similar baffled vents were also installed along all masonry panel tops, sloped outward to drain water out while allowing moist air to exhaust outward. Placement of weeps, which also act as air intakes, along the brick panel bottoms and vents to exhaust air outward at the panel tops sets up a thermo-siphon drying effect, greatly accelerating drying, thus prolonging the masonry's lifespan.

Fig. 5.55: Installing Sloped Panel-Top Vents

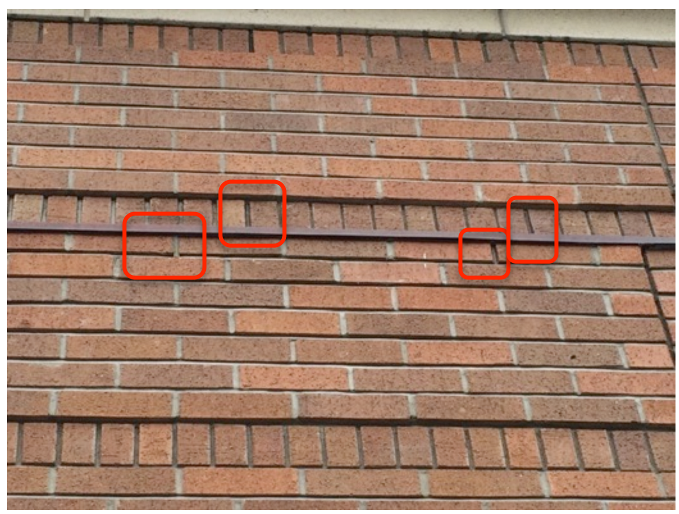


Fig. 5.56: Top Vents & Bottom Weeps

The level 2 water-table was rebuilt with pre-cast concrete, colored and textured to closely resemble the original stone. However, to protect its long-term integrity in Juneau's masonry-challenging climate and to preclude infiltration, it was capped with EPDM membrane overlaid with a thin vent mat and 16 ounce copper flashing caps, with type 304 stainless-steel counter-flashings used at concealed locations.

Fig. 5.57: Level 2 Water-Table In-Progress

Fig. 5.58: Level 2 Water-Table In-Progress

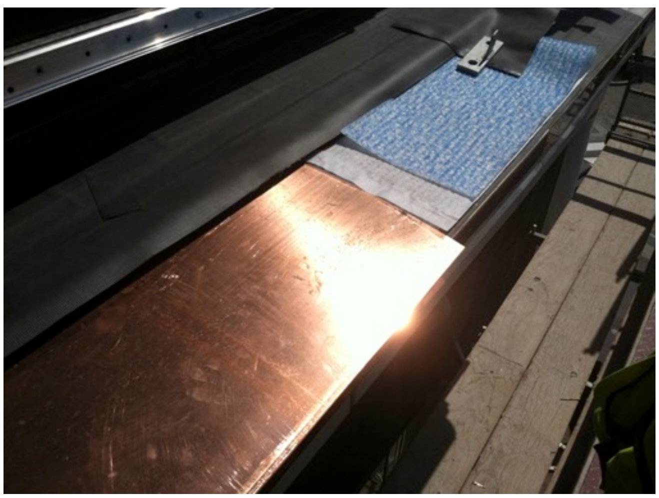


Fig. 5.59: Wtr.-Tbl. Vent Mat & Copper Cap

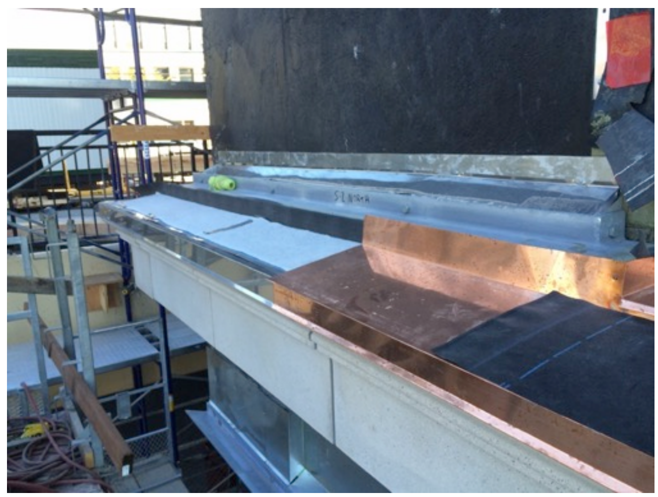


Fig. 5.60: Water-Table Flashings

Fig. 5.61: Water-Table Flashings

Fig. 5.62: Water-Table Flashings

Pre-cast concrete "sills", colored and textured to closely resemble the original stone "sills", were placed atop the level 2 water-table, with drainage weeps under these to allow water to drain out from behind the masonry.

Fig. 5.62: Pre-Cast Conc. Sill With Weeps

Fig. 5.63: Pre-Cast Conc. Sill With Weeps

The original multi-colored terra-cotta window surrounds were replicated in terra-cotta, but rather than being grouted in place, the new pieces were secured using type 304 stainless-steel pin-anchors, and were left hollow with drainage weeps to allow water to drain down. To accommodate thermal and moisture expansion and contraction, which on these 35-foot tall jambs could exceed ¼", the pieces were adhered to the anchors using a high-modulus silicone adhesive, rather than rigid epoxy.

Fig. 5.64: Replicated T.-C. Window Surround

Fig. 5.65: T.-C. Window Surround & Anchor

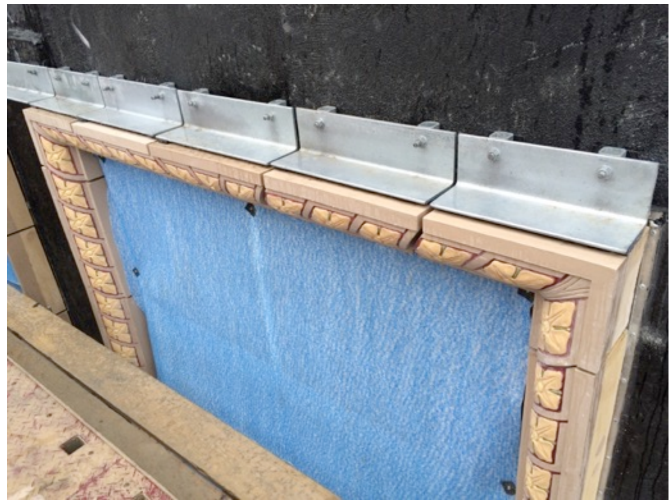


Fig. 5.66: Start of T.-C. Window Surr. Install. **Fig. 5.67: Terra-Cotta Window Head**

Similarly, the original terra-cotta window spandrels were also replicated in terra-cotta, and were secured with stainless-steel pin-anchors, again using a high-modulus silicone adhesive, rather than rigid epoxy.

Fig. 5.68: Start of T.-C. Spandrel Installation **Fig. 5.69: Terra-Cotta Spandrel Installation**

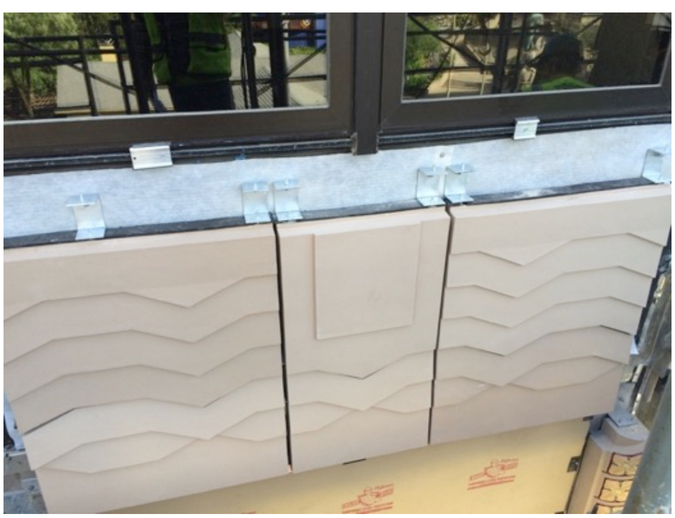


Fig. 5.70: Terra-Cotta Spandrel Installation **Fig. 5.71: Terra-Cotta Spandrel**

The original terra-cotta level 5 water-table was replicated using terra-cotta where multi-colored pieces were needed, and with pre-cast concrete at the wide monochromatic band. To preclude infiltration and protect the integrity of these pieces, this water-table was also capped with EPDM membrane and copper flashings.

Fig. 5.72: Level 5 Water-Table Band

Fig. 5.73: Level 5 Water-Table Band

The original terra-cotta roof-level cornice band, removed decades ago due to its degradation reflecting its inadequate design, was replicated using pre-cast concrete. To preclude infiltration and protect its integrity, this cornice was also capped with EPDM membrane, a thin vent mat, and a standing-seam copper roof.

Fig. 5.74: Lower Cornice Band

Fig. 5.75: Lower Cornice Band

Fig. 5.76: Upper Cornice Band

Fig. 5.77: Placing Upper Cornice Band

Fig. 5.78: Hoisting Upper Cornice Band

Fig. 5.79: Installed Upper Cornice Band

Fig. 5.80: Roof-Level Cornice Band

Fig. 5.81: Copper Roof Atop Cornice

Part 6: Concluding Remarks

In summary, this venerable capitol, perhaps Alaska's most architecturally and historically significant building, had been designed as many of its contemporary peers, which, however, proved woefully inadequate for Juneau's very masonry-challenging climate, and consequently, its exterior elements displayed a level of degradation far beyond the building's relatively young age. Various of these exterior elements had degraded to the point where they posed serious life-safety hazards to pedestrians below. Further, the building's overall structure was not designed to perform adequately in earthquakes of plausible magnitudes, had suffered seismic damage to various of its exterior masonry elements, and was at risk of complete collapse when the inevitable significant earthquake took place.

This building's many serious issues could have been addressed in the expected fashion, namely by exerting all effort to maintain its exterior masonry elements and installing interior shot-crete shear walls to enhance seismic performance. Based on past experience, I am very confident that this restoration pathway would have been mandated by many historic preservation boards.

Yet, this "preservation" approach would have proved very costly; would have produced a building whose exterior masonry would still continue to crumble away onto pedestrians below; would continue to consume inordinate amounts of heating energy each year; would have made the building yet heavier, thus requiring additional seismic upgrading to address the increased movement stresses; and would have reduced already tight interior space by thickening the exterior walls inward. Further, this approach would at best have extended the lifespan of the building's exterior by perhaps 40 years, at which point further attempts to preserve the existing masonry would have proved futile, requiring very costly replacement in any case in just a few decades.

In contrast, the "reconstruction" approach actually followed allowed the building to become lighter and seismically notably safer; made the exterior walls much more energy-efficient, reducing heat loss through the masonry by roughly 90%; and gave the building a new lease on life, probably extending the life-span of its exterior cladding to 100-150 years. It allowed the building to regain its originally-designed appearance while accommodating barely perceptible corrections of its technical errors. In short, the "reconstruction" approach vastly improved the building's seismic performance and safety, greatly extended its life-span, and improved its energy-efficiency immensely, at only marginally higher initial cost than the largely futile "restoration" approach would have cost.

I hope this approach can serve as a guide to the rational preservation of other historically significant buildings.

Fig. 6.1: In-Progress Re-Cladding of the Alaska Capitol, West Side
New cladding is left of corner, existing building is right of corner.

Fig. 6.2: New Cladding on West Side

Fig. 6.3: New Cladding on North Side

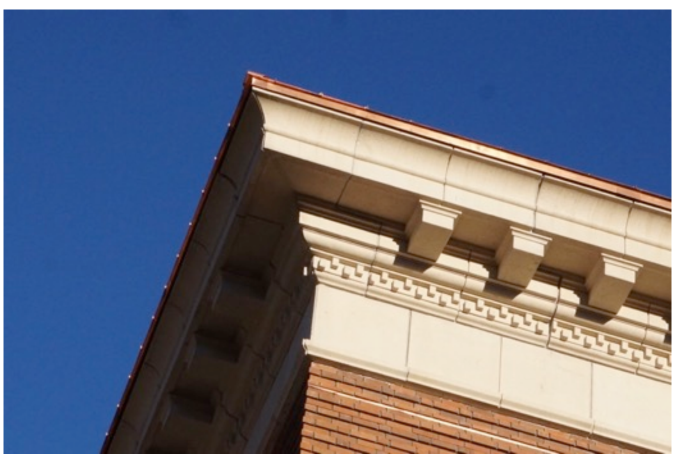


Fig. 6.4: Reconstructed Cornice Band

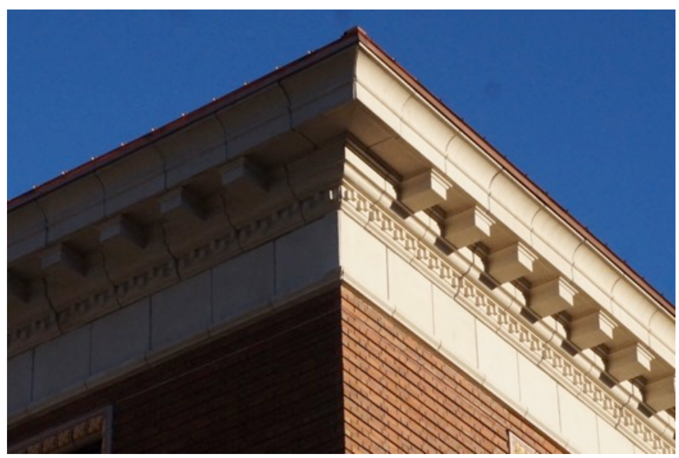


Fig. 6.5: Reconstructed Cornice Band

Fig. 6.6: In-Progress Re-Cladding of the Alaska Capitol, West Side

Fig. 6.7: New Concrete Shear Walls

Fig. 6.8: In-Progress Re-Cladding, West Side