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May 31, 2011

Mayor Peter W. Swiderski
Village of Hastings-on-Hudson
7 Maple Avenue
Hastings-on-Hudson, New York 10706

Re: **Building 52 Engineering Evaluation Report**

Dear Mayor Swiderski:

Attached is the Building 52 Engineering report for your information. The enclosed report generally describes the current condition of Building 52 and the suggested minimum requirements to stabilize the structure so that further deterioration is mitigated.

Please do not hesitate contacting me at 443.807.6233 if you wish to discuss further once you've had a chance to review the report.

Sincerely,

Eric J. Larson
Project Manager

Attachment

Village of Hastings-on-Hudson; Deven Sharma, AIA
Village of Hastings-on-Hudson; Chief James Drumm
Douglas Alligood
Village of Hastings-on-Hudson; Jerry Quinlan
Haley & Aldrich

Building 52
Stabilization Evaluation
One River Street
Hastings-on-Hudson, NY

27 May 2011

ROBERT SILMAN ASSOCIATES
STRUCTURAL ENGINEERS

Building 52
Stabilization Evaluation

One River Street
Hastings-on-Hudson, NY

Prepared for:
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RSA Project # 12900.02

ROBERT SILMAN ASSOCIATES
STRUCTURAL ENGINEERS

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NARRATIVE

Executive Summary

Introduction

Assumptions

Observations

- a. Floor Slab
- b. Roof
    1. Structural Slab
    2. Architectural
- c. Exterior Walls
    1. Column Bases
    2. Wall Conditions

Future Use Possibilities

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EXECUTIVE SUMMARY

Robert Silman Associates (RSA) has completed the stabilization evaluation for Building 52 and found that, with appropriate maintenance, the structure is capable of being stabilized for a period of at least ten years. This should provide adequate time to determine the appropriate future use. The work of this report does not include any hazardous material abatement issues; these are to be dealt with by others.

The stabilization is designed as and designed to achieve the following:
- Removal of safety hazards relative to structural condition of building
- Repair of conditions that, if left unrepaired, might cause further deterioration in the structure, including providing resistance against water infiltration

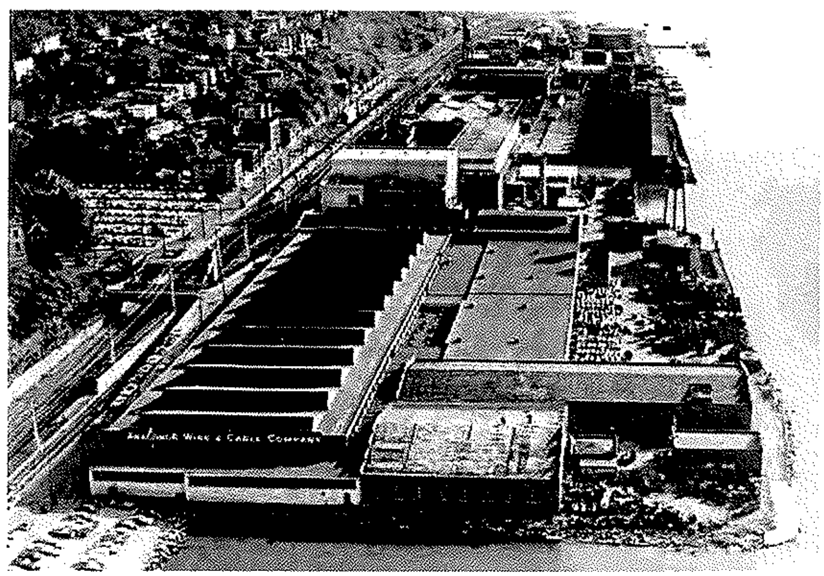
Further, the stabilization recommendations are not in basic conflict with permanent repairs that might be made once a permanent use is decided upon.

Basic major features of the stabilization include:
- A new roof with a minimum 20 year life
- Repairs to the roof slab
- Repairs to the end walls of the existing roof monitors
- Repair and repointing of the brick masonry perimeter walls

In addition, this report outlines the upgrades to the building that would be required by the New York State Building Code for three potential future use schemes.

INTRODUCTION

Building 52 is a former factory building, and was one of many buildings that made up the Anaconda Wire & Cable Plant. It is located in Hastings-on-Hudson, New York. It lies approximately 100 yards east of the Hudson River and directly to the west of the Hastings-on-Hudson Metro North train station. The building was most likely built in 1918 and was originally owned by The National Conduit & Cable Company and The National Brass & Copper Tube Company. It was used as a sheet mill. The plant, including Building 52, was later bought by Anaconda Wire & Cable and was used to produce cables. During World War II, Building 52 was used to produce fire-resistant copper cables to be used on US Naval vessels.



Historic site photo showing its many buildings (Building 52 is at lower left)

Building 52 is a one-story building that is 576 feet long in the north-south direction and 170 feet wide in the east-west direction. It consists of a concrete slab floor that is either a grade supported slab or possibly spans to piles. The roof is supported by steel columns, which run along the perimeter of the building at 16 feet on center in the east and west walls, and 17 feet on center in the north and south walls. There is another row of columns that runs from north to south down the middle of the building at 48 feet on center. The steel columns support steel trusses which run east to west. The trusses support smaller steel infill beams, which support a cinder concrete roof slab. The roof is a sawtooth roof that originally consisted of twelve roof monitors that contained concrete on the south facing slope and glass skylights on the steeper north facing slope. The exterior walls are masonry and do not appear to be load bearing.

Robert Silman Associates was retained by Haley & Aldrich to determine what would be necessary to stabilize Building 52.

ASSUMPTIONS

The observations of the steel structure were primarily limited to the existing condition of the column bases and their base plates. The steel columns, trusses, and filler beams were not documented nor assessed; they did not appear to exhibit structural distress or significant deterioration.

Limited structural drawings were available for RSA's use. There is a pile layout drawing, but it is not certain whether it represents what was actually installed on the site. There are also several drawings from the 1940s that show general equipment layout and plumbing layouts, but they contributed limited information for purposes of this study.

OBSERVATIONS

MODES OF OBSERVATION

In the months of August to November 2010, RSA visited the site several times in order to visually assess the conditions of Building 52. In many instances we were joined by James Gainfort Architects (JGA) who are the consultant for the roofing, skylights, and windows.

Additional visual observations were conducted by Abraham Joselow, PE, PC for electrical, plumbing, HVAC and fire protection systems and by Stephen Tilly, Architects, for comments on future potential uses of the building.

Our visual observations were aided by a nondestructive evaluation by GB Geotechnics (GBG) who visited the site in July 2010 and performed Infrared Thermal Imaging, Impulse Radar, and Metal Detection. See **Appendix D** for their full report.

RSA also requested floor and roof probes be performed so that we could better understand the make-up of the floor and roof slabs. We visited the site in October to observe these probes. See **Appendix C** for probe plan and documentation.

Our last method for obtaining information about the building was through concrete cores that were sent to Kemron Environmental Services to be tested. They were tested for compressive strength and chloride content. Two of the cores were petrographically analyzed. See **Appendix F** for full test results.

A. Floor Slab

The floor is exposed concrete and based on visual observations appears to be in fair to good condition [**photo 1**]. There are various trenches that may have housed pipes or rails, which have been filled in with concrete, most likely at a later date than when the slab was poured. [**photo 2**]. The surface is somewhat uneven across the entire floor. There is an existing pile layout drawing, which led us to assume that the majority of the floor consists of a spanning slab.


Photo 1: Overall interior


Photo 2: Trenches in floor

Using a combination of Impulse Radar and Metal Detection over 5 sample areas of the slab, GB Geotechnics (GBG) was able to determine that average thickness of the slab was 8 inches. In all areas investigated, they found one layer of reinforcing in each direction, which was typically closer to the bottom of the slab than the top. The spacing of the reinforcing varied from 6 inches to 12 inches on center. Bar size could not be determined. Our original assumption was that the slab was supported on pile caps that tied into the piles below. RSA requested that GBG try to locate possible pile cap locations. Because nothing was known about the pile cap thicknesses or reinforcing, GBG determined possible pile caps based on areas where data indicated thicker concrete or localized changes in reinforcement. GBG marked these areas on the slab with paint and included them in the report as well. RSA modified a few of the originally proposed probe locations based on GBG's results.

In addition to the work performed by GBG, five probes in the floor slab were made. Probes 1, 2, and 4 were performed towards the middle of the slab (away from columns), while probes 3 and 5 were performed closer to the base of steel columns. None of the probes was moved more than 10 feet from its original position. The locations of these probes can be found on **drawing SP-1** in **Appendix C**. Based on visual observations of the floor slab probes, RSA was able to confirm GBG's findings that the slab was typically 8 inches thick (+/- ½ inch) and that all reinforcing was found at the bottom of the slab only. The bottom layer of reinforcing was typically 1½ inch above the bottom of slab (+/- ½ inch). In three of the five probes, the reinforcing consisted of #6 bars at 10 inches on-center, each way (see **probe sketch FP-1** in **Appendix C**) [**photo 3**]. The two remaining probes were found to contain #4 bars at 6 inches on-center, each way (see **probe sketch FP-2** in **Appendix C**) [**photo 4**]. Neither pile caps nor piles were encountered at any of the five probe locations. This was inconsistent with data collected by GBG's non-destructive testing and therefore should be investigated further in the future.

At all probe locations, the concrete appeared to be in good condition. In general, there were no noticeable voids or cracks, nor were there any signs of separation between the paste and aggregate. Probe #4 showed the most signs of poor concrete placement with some voids and separation in the layer of concrete below the reinforcing [**photo 5**]. The reinforcing typically showed little signs of corrosion, however, at probe 4 the reinforcing had corroded slightly more. This may be a localized problem due to the above-mentioned concrete voids/separation.

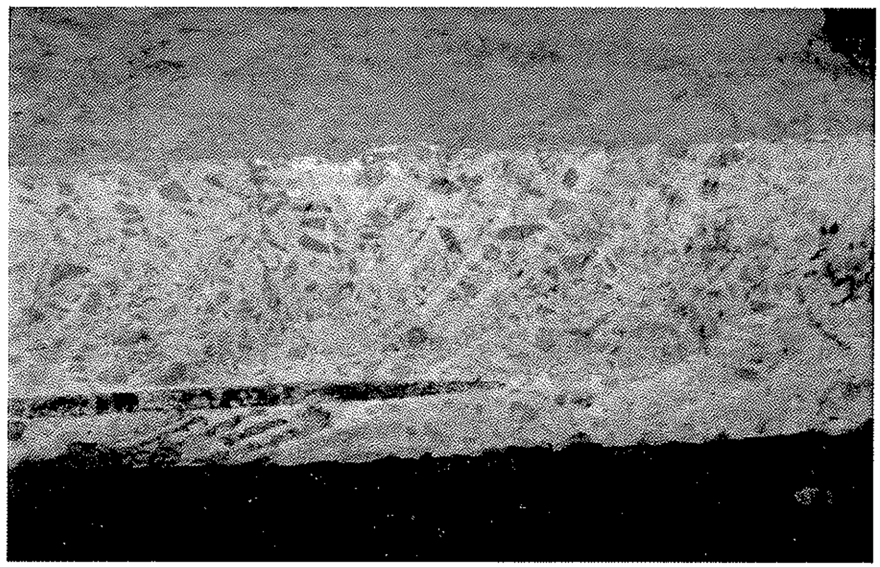


Photo 3 (top): Floor probe #2
cross section with reinforcing

Photo 4 (middle): Floor probe #5
cross section with reinforcing

Photo 5 (bottom): Floor probe #4

The original scope of work included the structural analysis of the ground floor slab to determine how much live load it might be able to support. This in turn would give a clue as to the allowable occupancies. The scope of work was created before any probes were conducted on site.

From an early original drawing called "Pile Layout", it was assumed that piles would be revealed in the probes spaced approximately 8 feet on center. Instead the probes found no indications of piles or pile caps anywhere. In addition, the reinforcement in the ground floor slab was limited to bottom bars only; there was no top reinforcement found in any of the four probes. In a normal pile supported slab, there are top bars present that are actually slightly larger than the bottom bars. This is because of the effects of continuity in the concrete slab, with the negative (or top) bending moment being larger than the positive (bottom) bending moment.

In the absence of observed piles, calculations were prepared (see **Appendix G**) to determine what the slab capacity might be if there were actually piles beneath it. The following assumptions were made:

1. The slabs were simply supported at every pile because there were no top bars to resist any negative bending moment that would result from continuity.

2. The slabs functioned as "slab bands" rather than as a two way flat slab. The width of the slab band was equivalent to the width of a column strip, had the slab been designed like a flat slab. The load of the middle strip was assumed to be carried 100% by the slab band in each direction, thus providing some redundancy.

3. Three different pile spacings were investigated: 6'-0", 8'-0" and 10'-0" in each direction. For each spacing, the slab capacity for two different reinforcing patterns was calculated, based on the findings in the probes.

The live load capacity of the slab for the three different pile spacings and two different reinforcing patterns is as follows:

Pile Spacing	**#4 @ 6" on center**	**#6 @ 10" on center**
6'-0"	481 psf	643 psf
8'-0"	238 psf	329 psf
10'-0"	125 psf	183 psf

It is recommended that in the early stages of any future adaptive reuse design that a much more comprehensive slab, pile, and subsurface exploration program be conducted. If piles are found, their capacity should be determined, their condition determined, and, if it is found to be necessary, repairs specified. If no piles are found, then an analysis of the allowable subgrade bearing capacity should be determined.

RSA was asked if the concrete slab of the ground floor could have two inches of concrete removed or scarified from its top surface. This would presumably allow the removal of any surface contaminants such as PCBs. In addition, the removal of the top two inches would eliminate the zone of carbonated concrete and would also eliminate portions of the slab that might be contaminated with chloride salts.

As explained above, no piles have been located as of the date of the writing of this report. However if piles were to be found, RSA has analyzed the slab and found that two inches could be removed from the top surface, leaving a remaining capacity to support superimposed temporary construction loads of about 70 psf for the widest pile spacing studied (10'-0" o.c.). The weight and distribution of the load of any machinery required to perform this concrete removal would have to be calculated to see if it met these loading restrictions.

In replacing the top two inches with new concrete, there would be real benefit to the ultimate load-carrying capacity of the repaired slab. If the new two inch topping were properly bonded to the remaining six-inch-deep base slab and if reinforcing bars were placed within this two inch layer of concrete, then the slab could be considered to be a continuous two way flat slab rather than a series of discontinuous simple spans as has been assumed in the analysis presented above. This sort of continuous slab will yield a much higher load-carrying capacity than the tabulated loads for the simply supported slab shown above.

All of this information is conjectural and needs to be verified at the time of a future adaptive resuse design.

Four concrete samples were cored from the floor slab and sent to a laboratory where they were tested for compressive strength and percent chloride content. An additional floor core was analyzed using petrographic examination (see **Appendix F**). The compressive strength of the four floor cores ranged from 4970 pounds per square inch (psi) to 5820 psi, resulting in an average compressive strength of 5380 psi. In contemporary practice, it is typical to design both spanning floor slabs and slabs on grade for a minimum compressive strength, so average compressive strength of the cores is acceptable.

The percent chloride in the cores ranged from .027% to .142%. Chloride concentrations greater than .050% greatly increase the possibility that the reinforcing steel in the concrete will corrode. Four out of the five concrete cores had chloride content higher than .050%. We recommend that, when additional testing is performed on the concrete roof slab prior to any adaptive reuse design, that a corrosion risk assessment be performed on the floor slab as well.

According to the petrographic analysis, the floor slab consists of normal weight concrete containing coarse aggregate in the form of crushed stone and fine aggregate in the form of sand. The cementitious paste in the concrete ranged in color, which is an indication that the concrete was not thoroughly mixed when placed in the field. Visual observation of the polished concrete sample and phenolphthalein staining indicated that the top of the concrete slab was carbonated. The carbonation extended ¾ inch to 1 inch into the slab. There did not appear to be indications of chemical attack in the slab. It did not appear that entrained air was added to the concrete mix. Air content was estimated at between 1% and 2%. There were a few small vertical cracks in the core that was petrographically analyzed. These cracks are not significant.

In **Appendix F,** Kemron's subcontractor—Testing, Engineering and Consulting Service, Inc.—ventures an opinion at the bottom of page six of their report. They state that "if the slab was to be structural,...it is not adequate because the reinforcing steel is not well embedded and corroded." RSA does not agree with this statement based on our observation of the probes cut into the floor where, at all locations observed, the reinforcing showed no signs of corrosion and was properly embedded. The testing lab had only a tiny sample of a core on which to base their conclusion and this was not representative of RSA's observations.

B. Roof

1. STRUCTURAL SLAB

During one site visit RSA was able to observe the underside of the roof slab from a scissor lift in various locations throughout the building. RSA personnel were not allowed to touch this surface due to the potential of environmental impact. RSA was only to observe it visually and direct the environmental contractor, Envirocon, to perform soundings on the slab. From the ground, looking up at the underside of the roof, the slab appeared mottled, as if there was possibly a large amount of moisture infiltration in many areas. Once up in the lift however, it was clear that what had looked like mold or other indicators of moisture infiltration was actually due to peeling and flaking of some sort of coating that had been applied to the underside of the slab [**photo 6**]. It is not clear what the coating was intended for but it is likely that moisture has caused it to flake off.

From the lift the concrete itself appeared to be in fair condition. No spalled concrete or areas of concrete that had cracked due to corroded reinforcing were observed. In some instances, the wire mesh was quite close to the bottom surface, and its outline was visible. In other cases, a portion of this exposed mesh was corroded [**photos 7 & 8**]. Around the roof drains, there were often more instances of water infiltration and the slab was generally in worse condition [**photo 9**]. There was at least one location where there was a large crack (up to one inch wide and approximately ten feet long) in the roof slab running in the east-west direction [**photo 10**]. The crack had been previously filled with a patching material, but still appeared to be a quick route for moisture into the building. This crack did not appear to be indicative of a global problem of the slab.

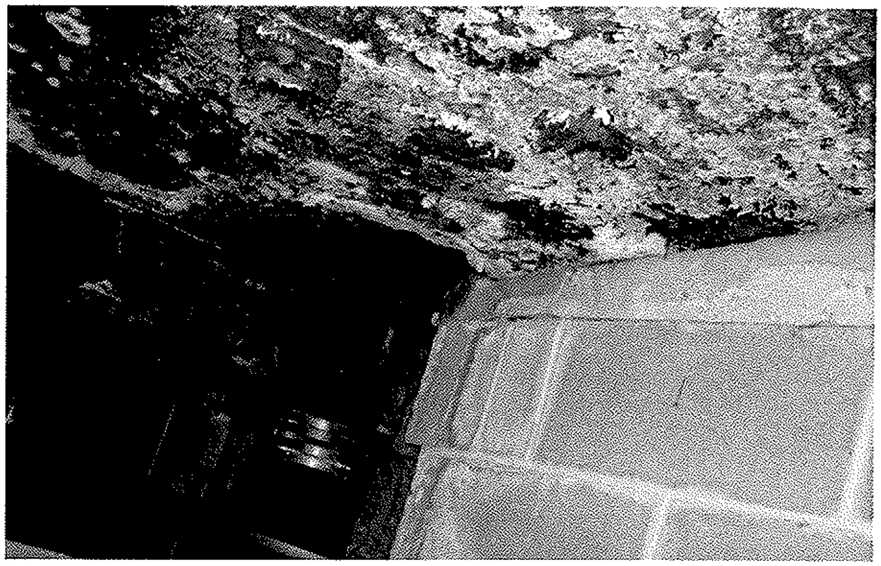
Along with looking at the slab from the lift, RSA also listened on the ground while two Envirocon employees tapped the underside of the roof slab with a hammer in a sampling of areas in the building [**photo 11**]. In general the tapping sounded consistent. There were no areas where tapping emitted a more hollow or dull sound that would have indicated spalled, delaminated, or deteriorated concrete. These findings agreed consistently with the visual observations.

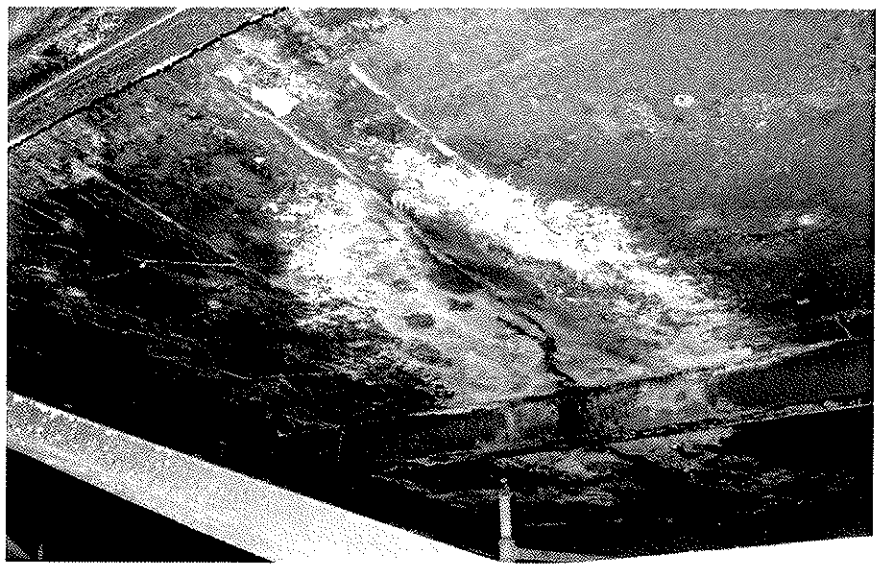
GBG conducted thermal imaging of the roof slab from different points along the floor in July 2010.

Photo 6: Underside of roof slab - surface

Photo 7: Underside of roof slab - mesh close to surface and corroded

Photo 8: Underside of roof slab - corroded mesh





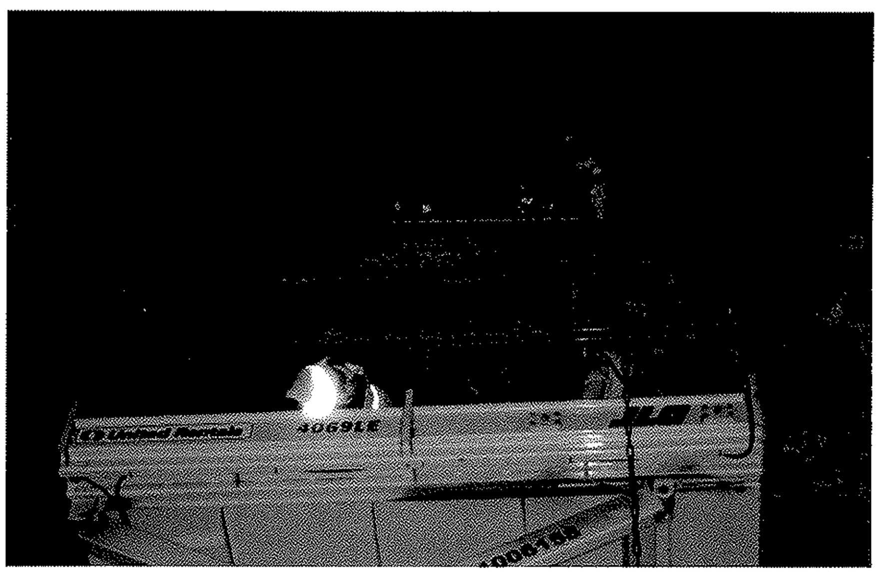


Photo 9 (top): Underside of roof slab - moisture by drain

Photo 10 (middle): Crack in roof slab

Photo 11 (bottom): Sounding in lift

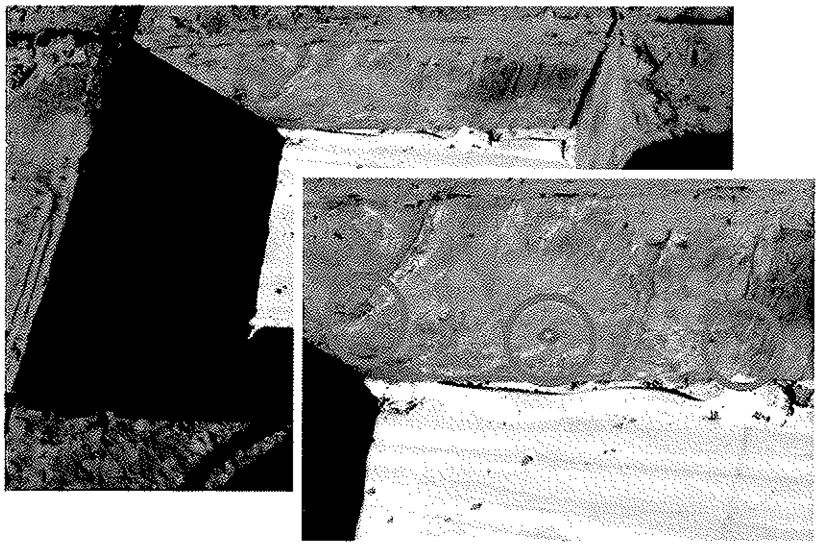


Photo 12: Roof probe #6 - *cross ection with reinforcing*

Photo 13: Roof probe #2 - *corrosion on reinforcing*

The goal of the thermal imaging was to determine areas with elevated moisture levels that might indicate corroded reinforcing, debonding concrete, and voiding. See **Appendix D** for a full explanation of their method and assumptions. In general their findings of elevated moisture corresponded to our visual observations made several weeks later. The infrared thermal camera detected colder areas around roof drains and at the crack mentioned above, which indicate moisture infiltration in those areas.

Seven roof probes were observed by RSA to further confirm the make-up, reinforcing and condition of the roof slab. From the probes, the thickness of the slab was measured at approximately 4 inches (+/- ½ inch) with a layer of W2 (0.159-inch diameter) wire mesh located ¾ inch (+/- ¼ inch) from the bottom side of the slab [**photo 12**]. The wire mesh spanning parallel to the slab (east-west) was spaced at 3 inches on-center. This spacing was consistent at all seven probes. The mesh perpendicular to the slab span (north-south) was minimal and a definitive spacing could not be determined as there was typically only one piece of wire per probe. This would indicate a spacing, for what is commonly called the temperature reinforcing, of 12 inches or larger. A sketch of the typical roof probe findings can be found on **RP-1** in **Appendix C**.

The probes at the roof also allowed for visual observation of the type of concrete used as well as the general condition of the slab. A cinder aggregate concrete, common for the age of this building, was found at all probes. The concrete was well consolidated and no real voids were noticed (above those that are typically found due to the cinder material being porous). The wire mesh generally showed little signs of corrosion, however, a few locations showed moderate corrosion. In particular, probe 2 had reinforcing that had moderate corrosion and rust staining of the concrete around the mesh. This was not surprising as this probe was performed near an existing roof drain where other visual observations had determined that water damage had been occurring for some time [**photo 13**].

Samples of the roof concrete were sent to a testing lab for structural analysis (see **Appendix F**).

The roof slab is constructed using a system that was very popular in its time because it was economical. Structural steel supports were provided for the roof slab approximately 7 feet-3 inches on center. The bottom of the slab was formed with wood boards hung from these steel members. Then wire mesh was draped over the top of the steel at the supports and permitted to curve down toward the bottom of the slab between the steel supports. Finally the concrete was poured into the forms, approximately 4 inches thick, for the total depth of slab.

The design of these slabs was empirical because the wire mesh was felt to be a continuous catenary. The coarse aggregate for the concrete in the New York area was often cinders obtained from the local utility company that burned coal in its power plants. Often, the cinder used for aggregate was free for the taking, so that the power company could get rid of it. The quality of the concrete, particularly its ultimate compressive strength was allowed to be very low, sometimes below 1,000 psi. The allowable live load was determined by an empirical formula and the surviving formula most often used is found, even today, in the latest version of the New York City Building Code. We applied this formula to the roof of Building 52 and found that the slabs were capable of supporting the code required snow loading.

However the results of the laboratory tests have cast some doubt on the ultimate quality of the roof slabs. The tests found the concrete to be fully carbonated. This means that atmospheric carbon dioxide has permeated for the full depth of the roof slab, four inches, and has reacted with moisture present in the slab. This reaction forms a weak acid and thus reduces the original highly base, alkaline environment found in the concrete surrounding the reinforcing mesh. It is this alkaline environment that provides a passive barrier against corrosion and once it has been destroyed, the slab is more vulnerable to ongoing corrosion of the reinforcing. In the roof slab of Building 52, because the reinforcing is such a small diameter, any corrosion might have a serious adverse effect. Thus additional testing is required before any stabilization at the roof is undertaken. A corrosion audit that can predict the remaining service life of the slab should be conducted prior to any adaptive reuse design. For purposes of this report, it is assumed that 25% of the roof slab would require replacement; this number is purely conjectural at this point.

2. ARCHITECTURAL

INTRODUCTION

The roof, a "saw-tooth" type commonly used on industrial buildings of the era, is an extension of the highly rational layout of the building's floor plan. North-facing glazed areas in the skylight "monitors" of the saw-tooth design allowed diffused, non-direct natural light to the interior. The height of the glazing area was divided between two lites; remnants of internal mechanical devices at the frames suggest the glazing was operable. Originally, the roof configuration had a dozen monitors spaced evenly across the length of the building. Each monitor occupies three structural bays of the building's length; the low side of every monitor coinciding with one of the twelve columns along the building's central column line. A typical monitor spans much of the building's width, with its triangular end set back about 24 feet from the east and west façade walls [**photo 14**].

Photo 14: Overall of roof looking south

There are three lines of drains on the roof, each parallel to the length of the building. One line is at the building's center; the other two each align with either east or west parapets. Between each monitor, low slope crickets formed in the roof deck direct water to a central drain or toward one of the other two drain lines. The remaining area along the long edges of the building consists of low slope roof, incorporating crickets to pitch water toward the drains. Drain locations coincide with the space between monitors, at every third column line.

The brick exterior walls of the building terminate in parapet walls with terra cotta coping. The top of coping is only inches above the roof deck along the long east and west elevations; at the north and south elevations, parapet height variations, typically a dozen brick courses or more above the roof surface, contribute definition to the facades.

The roof deck, including skylight monitor roof and curb, is board formed, poured in place concrete. End walls of skylight monitors are composed of light metal framing with cement plaster applied directly to metal lath. The primary membrane appears to be a coal tar built up roof (based on odor and appearance, without confirmation by testing). A limited amount of copper counter-flashing is visible beneath roofing material where the roof membrane terminates at the north parapet wall. Copper edging is typical along the raking edge of the monitor roofs.

OBSERVATIONS

The roof is in poor condition, the membrane partially blown off at several locations. Failure of the top layer of the membrane occurs across extensive areas near the middle of the building, from column lines 19 to 26 [**photo 15**], and at the southern end of the building. At a select number of locations, the concrete deck is completely exposed. There are hundreds of square feet of various sheet and trowel applied roof patches from multiple attempts to address localized membrane failures. These patches are primarily at low slope roof areas surrounding the monitors. Standing water was observed during an October 2010 site visit, when no precipitation had occurred during the previous 24 hours. Presently, there are several active leaks (these leaks, observed at the interior, are un-documented).

Photo 15: Roof membrane failure, column line 19, looking west.

The steeply sloped, formerly glazed portion of the skylight monitors presently have wood structure infill, oriented strand board (OSB) covering, building paper and asphalt roofing shingles. Shingles are approaching the end of their service life and are missing at limited areas. During a single up close observation, conducted by lift from the interior, the existing remaining metal skylight frames were examined. All glazing has been removed; broken remnants of glass are visible along the edges of the frames. Based on the observed skylight frame, we assume all steel frames to be deteriorated beyond repair [**photo 16**].

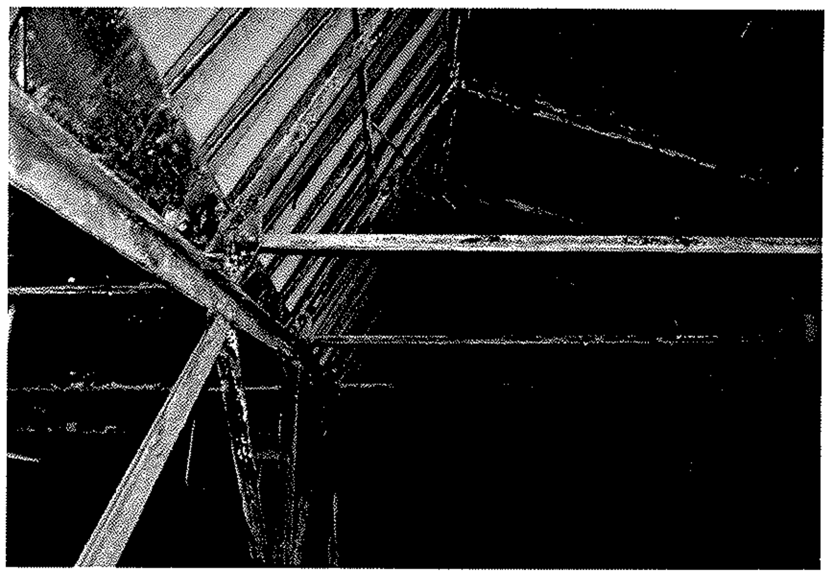


Photo 16: Interior view of skylight frame

The triangular end walls of the monitors are in very poor condition, deteriorated beyond repair. During a site visit, the construction of the side walls was observed through a hole that was formed by a recently fallen piece of cement plaster. These walls consist of 1 ½ inches x 1 inch "T"-shaped vertical light metal-framing members, spaced approximately 22 inches on center. Expanded metal lath, fastened with metal wire ties, had a 1 ½ inch application of cement plaster on each side; total wall thickness of approximately 3 inches. Various coating remnants observed on the exterior cement plaster surface suggest that at some time the walls were white. The presence of a black asphaltic top coat is indicative of a previous attempt to limit water infiltration. More recently, cracks have been dressed with trowel grade roof patching mastic.

Failures at these walls include loss of coatings, cracking, and deformation of the wall surface due to deterioration of the metal framing. Bulging typically occurs more toward the south end of the building, at both the east and west end walls. The deformation, increasing proportionally to the height of the triangular wall, is greatest at the bottom [**photo 17**]. A potentially dangerous condition exists where large pieces of cement plaster, having de-bonded from the metal lath, could fall into the building.

The roof monitor between column lines 22-25, including the concrete deck, much of the curb, and associated supporting steel, no longer exists. The monitor has been replaced by low slope roof on metal deck [**photo 18**]. The roof membrane over the metal deck differs from most of the building. At this area, the deteriorated membrane appears to be some sort of a glass fiber reinforced top sheet covered with a bitumen flood coat.

Photo 17: Cement plaster side wall at roof monitor

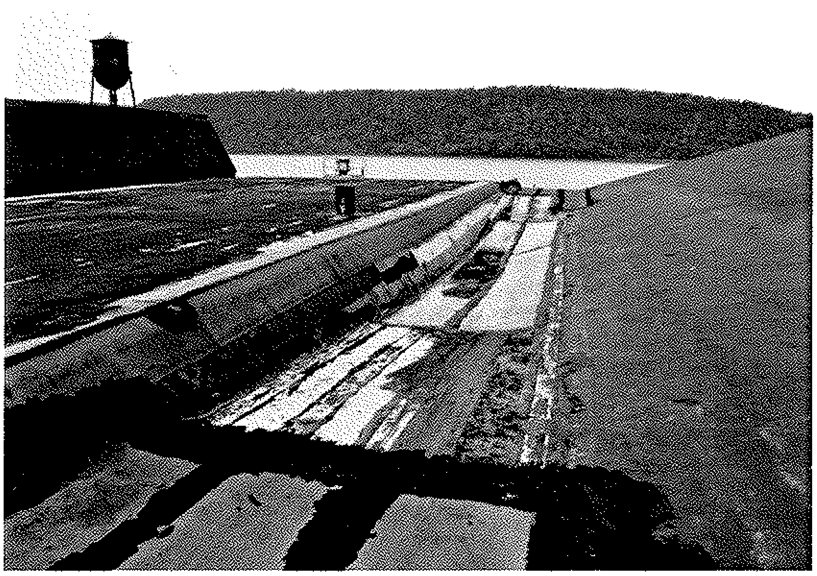


Photo 18: Low slope roof at former roof monitor

The parapet walls vary in height. At east and west façades, where the coping is just inches above the deck, the membrane terminates beneath the terra cotta coping [**photo 19**]. At north and south facades, where the parapet height varies between two to three feet above the roof deck, the roof membrane terminates at copper counter-flashing some eight inches above the roof deck. Most of the counter-flashing has been covered with successive layers of roofing mastic.

At east and west façade parapets, there are two shapes of terra cotta coping, both 16 inches wide, suggesting some coping units are replacements. At north and south facades, the terra cotta coping is 20 inches wide [**photo 20**]. The brick mortar joints on the roof side of these parapets are eroded. Coping is loose or missing at all parts of the building.

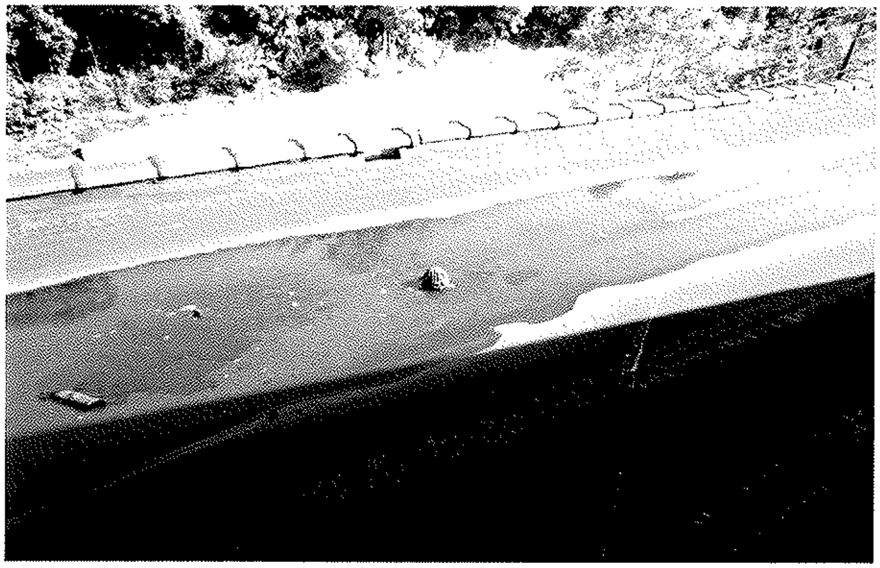
The existing drainage system is partially functional. The drains serve as the sole method for water to exit most of the roof surface; the roof is surrounded by parapet, with only two overflows both at the low East parapet near the northeast corner. More than half of the drains at the east line are covered with roofing. The center drain at column line 16 is clogged and holds water [**photo 21**].

CONCLUSIONS

The existing roof membrane, including perimeter and penetration flashings, cannot be effectively repaired. Roof replacement is mandatory. One approach to roof replacement first requires the total removal of the existing membrane system. This would then allow a detailed inspection and repair of the concrete decking before application of a new roof system. However, this approach is very expensive and is not required in order to stabilize the roof enclosure.

Instead, a new membrane system can be installed over the existing roof assembly without incurring much of the cost associated with removals and deck repair. Mechanically fastened polyisocyanurate insulation and cover boards under the membrane will bring the roof into compliance with prescriptive method requirements (continuous insulation, R-value of 20) of the current NYS energy code. Additional tapered insulation will be required to re-establish good drainage.

The choices for replacement roofing membranes should be limited to those that can be applied over tapered insulation to provide a lightweight, effective and durable protection against the elements. While a number of systems are available (modified-bitumen, built-up asphalt, spray-applied foam, and single-ply) we believe single-ply membranes offer the most value for money spent. Large sheets that minimize the number of field seams can be fully adhered to the cover board. Of the various forms of single ply membranes, thermoplastic membranes (TPO and PVC) offer welded seams, a white reflective color, and wide availability among applicators. We had good experience with a TPO membrane manufactured by Carlisle Syntec Systems.



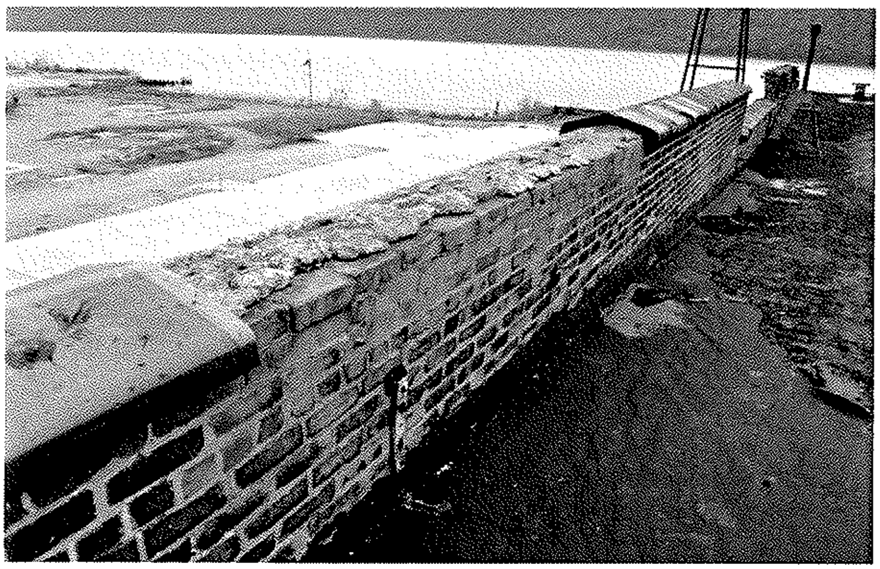


Photo 19 (top): Coping at east façade wall. Note overflow opening and presence of standing water.

Photo 20 (middle): Coping at south façade wall. Note missing coping.

Photo 21 (bottom): Standing water above center drain, column line 21

The skylight assemblies are too deteriorated to be renovated and reglazed. For purposes of stabilization, this report recommends that a stable condition over the current skylights can be achieved by removing the existing deteriorated shingles from the current OSB sheathing and recovering it with a new membrane. If a decision were ever made to reactivate the skylights, a wholly new metal and glass system would have to be installed in place of the current steel framing.

RECOMMENDATIONS

The following actions should be performed to stabilize the roof enclosure:

Main and Monitor Roofing Surfaces: First, scrape all loose and excessively built-up material from all roof surfaces, including the skylight monitors. Remove all flues, penetrations and miscellaneous pipes; patch all abandoned penetrations at concrete roof deck.

Install 4 inches of new rigid insulation over entire roof area consisting of two staggered layers of mechanically attached two-inch thick polyisocyanurate insulation. Additional tapered insulation boards will also be required at locations where the existing pitch to drains is insufficient.

Over the insulation, fasten a half-inch thick cement cover board, to which a new white, fully adhered TPO membrane (Spectro-Weld Reinforced TPO Membrane, by 'Carlisle Syn-Tec', 60-mil thickness) can be adhered.

Replace drain bodies and leader piping from the roof surface to existing interior storm water main laterals, which shall remain. Each existing drain must be broken out of the existing concrete deck, along with associated tailpiece, elbows and horizontal piping. The new drain body must be set slightly below the existing concrete roof surface and be "cast in" to the surrounding decking with new concrete. Reconnect the new drains to the main storm water laterals below.

At skylight portions of the existing monitors, remove all shingles, and all deteriorated OSB sheathing. Replace the deteriorated sheathing with new OSB, and then cover the entire skylight cover with rigid insulation, cover board and fully adhered TPO membrane following many of the procedures required for the main roof.

Demolish the two cement plaster end walls of each skylight monitor, including light metal framing. Remove existing metal flashings from the base and from the edge of the rake over these end walls. Install new metal framing and sheathing to accept base flashings from the new roof assembly. Cover the sheathing with insulating metal panels, and seal the top edges of these panels against the monitor roof rakes with new TPO-clad metal flashing.

If a decision is made to replace the skylights, then remove the entire existing wood cover. Cut out the existing metal skylight frames and the internal substructure used to operate these units. Inspect and repair the primary steel angles forming the sill and head of each skylight, as well as the exposed faces of existing steel truss members. Install new skylights, including frames, glazing, and associated flashing. Aluminum, thermally broken frames should be used, as should insulating glass with a maximum U-value of 0.30 Btu/sf/hr/degF. Flashing can be aluminum to match the windows. Note that skylight glazing is susceptible to damage from vandalism; consider providing protection for all glazed areas near public right of way.

If a decision is made to rebuild the missing monitor between column lines 22-25, provide new sheathing, vapor barrier, insulation, cover board and white TPO membrane on new metal deck.

Parapet Walls: Roof parapets require extensive masonry work to stabilize them. First, remove all existing terra cotta coping sections and inspect the exposed top courses of masonry. Reconstruct the top courses (assume top two courses of brick) as required to ensure their stability. Install a new plywood substrate to cap the repaired masonry. Provide new self-adhered sheet waterproof membrane and new TPO clad sheet metal coping over the plywood substrate, and tie each to the TPO roof membrane.

At the north and south façades, remove the coping, and inspect the remaining masonry. Reconstruct as much of the top courses of masonry as needed to establish a stable wall, then cap with plywood. Sheath the interior side of the parapet with cement board. Then extend fully adhered TPO membrane up the entire vertical face of cement board and tie this flashing into the main roof membrane. Cover the plywood coping sheathing with a waterproof membrane, then cap the wall with a TPO-clad sheet metal coping cover and heat weld it to the TPO wall flashing.

CLOSING REMARKS

The recommended roof membrane, available with a twenty-year warranty, should remain reliable for more than two decades. A maintenance program for the building should include a semi-annual examination of the roof membrane to check for damage and to verify all drains are clear of debris.

The white TPO membrane specified is an Energy Star qualified product that lowers the roof surface temperature and decreases the amount of heat transferred into the building. The four inches of insulation currently specified beneath the TPO membrane complies with the 2010 Energy Conservation Construction Code of New York State, under the prescriptive requirements of Section 502. As this report is prepared, the 2010 version of the Energy Conservation Code of New York State becomes effective at the conclusion of the 2010 calendar year.

The building's fenestration area, including the skylights, exceeds 40% of the total wall area, making the building not eligible for future compliance through a prescriptive path. For code compliance, a future change of use in the building would require compliance using "total building performance", which includes heating and cooling system, service water heating, fan system, lighting, process and plug loads for determination of the total building energy use. The path to code compliance for this building requires coordinated efforts of the future design team, including building enclosure, mechanical system and lighting designers. One approach to compliance may include installation of photovoltaic panels on the monitors; the south orientation of the sloped roof surfaces is an ideal location for such an installation.

Any building that is unconditioned (an unheated parking garage) need not comply with the requirements of the Energy Code. Determination of the building's future use in advance of stabilization would help to define exact roof insulation and skylight glazing requirements.

C. Exterior Walls

The observations of the exterior walls consisted of both probes and visual inspection over multiple visits to the site. The probes were performed at four column bases (see **SP-1** in **Appendix C** for locations). The probes were performed to observe the condition of the column base plates behind the masonry pilasters as these areas are prime locations for trapped moisture to collect and cause corrosion.

The visual inspection of the exterior wall consisted of personnel from RSA walking around the entire perimeter of the building and noting various conditions that need to be corrected in order to stabilize the building. These conditions typically consisted of re-pointing masonry, replacing masonry, protecting exposed steel from corrosion and helping to seal the building from further water damage.

1. COLUMN BASES

The four column base probes uncovered built-up steel columns atop base plates which were anchored into concrete piers (see **sketch WP-1B** through **WP-3** in **Appendix C**). The columns consisted primarily of angles and plates that were riveted together which is consistent with construction practices at the time the building was erected. Some of the columns have trapezoidal plates parallel to their flanges, however, these were not found at all locations [**photo 22**]. It is unclear what these additional plates were for, but they may have been used to transfer additional forces into the foundations.

The column bases observed generally showed little signs of corrosion. Any corrosion found was typically on the lower 12-18 inches of the column and did not appear to be aggressive. The column probe that had the largest amount of corrosion was wall probe 1A [**photo 23**]. The increased amount of corrosion was not found to be a surprise as this column was located behind a pilaster that had shown signs of deterioration (both cracking and separation from the wall) which would allow for increased amounts of water to reach the column. All of this being said, the corrosion of the column was not significant enough to cause concern for the stability of the building.

Photo 22: Wall probe #3 - *column base*

2. WALL CONDITIONS

As mentioned above, work on the exterior walls of the building also involved visual observation of the existing condition of the walls. All four walls presented a myriad of different conditions which are documented on drawings **S-100** through **S-102** in **Appendix B.**

WEST WALL

The west wall provided the most diverse range of conditions on the entire building and was mostly due to previous building extensions that have since been removed. The building previously had one shed-style addition that extended from grid 1 to grid 13. The remnants of this addition are still visible as T-shaped pieces of steel protruding from the masonry pilasters as well as painted masonry and a flashing reglet [**photo 24**]. In order to help seal the building and prevent deterioration of the exterior walls in this area, it is recommended that the steel T's be removed. At all sides, the steel lintels over windows should be scraped and painted and the reglet be removed.

Photo 23: Wall probe #1A - *column base*

Between grids 15 and 22 there is evidence of another extension which has since been mostly removed. It appears that this was once a bathroom for the building. Below the lowest windows, there were numerous abandoned beam pockets and paint and ceramic tiles on the masonry wall [**photo 25**]. There was also a large stretch of various tile materials adhered to the exterior slab on grade. As indicated on drawing S-100, the removal of all finishes (i.e., paint and tiles) is recommended from this area and any beam pockets should be filled with new masonry.

Photo 24: T-shapes at west wall

The last portion of the west wall (grid 23-37) had various exposed steel columns and painted CMU [**photo 26**]. In order to stabilize the building from further deterioration, it is recommended that all exposed steel (this includes steel lintels at all openings) be scraped and painted and that all CMU have existing paint removed and a breathable sealant be applied. There are also many areas over the surface where masonry needs to be repointed, or even replaced, and where other objects, such as conduit, should be removed (see drawing S-100 for full scope of work).

Photo 25: West wall beam pockets and paint

EAST WALL

Observation of the east wall found much more uniform conditions over the length of the wall. As the east side faces the train station and an elevated local roadway, there was no space for the original owner to construct building extensions, and thus, the required repairs are primarily masonry repair and replacement. All windows along this wall have been covered with plywood, which was most likely installed to prevent vandalism to the windows [**photo 27**]. All plywood should be removed and all existing windows be blocked in with CMU in order to provide a more long term solution to this problem.

The two other main repairs to this side of the building are to scrape and paint all exposed steel lintels to prevent further corrosion and to repoint large portions of the existing masonry. The repointing primarily occurs at the base of the wall and just below the upper windows. The base of the wall may need extensive repointing due to the fact that the "alley" created by the building and elevated roadway does not allow for a long window of time when sunlight can help dry out any trapped moisture (especially any snow drifts against the building) [**photo 28**]. A similar condition may be the cause of the deteriorated mortar joints below the upper windows.

NORTH WALL

Similar to the shed that had existed on the west wall, the north wall also once had a building extension. The only remaining pieces of this addition are T-shaped pieces of steel extending from the masonry pilasters along the wall and a flashing reglet just below the upper windows [**photo 29**]. Both the steel T's and flashing reglet shall be removed and the masonry repaired to help prevent additional moisture from entering the building. The remaining work on the north wall is primarily repointing of the existing masonry and the scraping/ painting of the exposed steel lintels. It is thought that, like the east wall, the north wall saw little sunlight and thus did not have the

Photo 26 (top): Exposed steel columns at west wall

Photo 27 (middle): East wall windows

Photo 28 (bottom): Repointing at base of east wall

Photo 29: T-shapes at north wall

Photo 30: Southeast pilaster

opportunity to dry out as well as the south or west face. This increased length of moisture exposure may have led to the mortar joints deteriorating more than elsewhere on the building.

SOUTH WALL

The south wall contained many of the same conditions found elsewhere on the building, but had a diversity similar to the west wall. Masonry repointing and replacement, as well as steel lintel scraping/painting, were the most common repair found on this wall. There was also some removal of various conduits and pipe penetrations, though these were minimal compared to the west wall. The south wall did contain two pilasters that needed significant rebuilding [**photo 30**]. These pilasters were in such poor shape due to water becoming trapped behind the brick and not only eroding the mortar but also expanding upon freezing and jacking the pilaster away from the rest of the building. As noted previously, this water infiltration did not have large detrimental effects on the steel columns behind the pilasters.

FUTURE USE POSSIBILITIES

RSA had been requested by Haley & Aldrich to consider what upgrades might be required by the New York State Building Code for three possible future uses for the building. Although this exercise is not strictly within the realm of "stabilization", the owner had requested a brief investigation. The three potential uses that RSA was asked to investigate are:

Use 1 Parking: covered, nonheated, non-occupied, single level
Use 2 Commercial: offices and/or retail occupants
Use 3 Commercial/assembly: offices, retail and community meeting spaces

All uses are to be one floor only, with no mezzanines or partial additional floors.

There are no special provisions required because of the building being located in a flood plain. Reference to the FEMA Flood Insurance Rate Map 36119C0307F. Panel 0307, Village of Hastings-on-Hudson, Number 360913 shows Building 52 to be in Zone X. This zone does not require any special provisions for resistance to floods.

For all potential uses except retail stores, if the columns are fire-protected to a height of twenty feet above the floor, there would be no requirement for a fire separation wall inside the building. The allowable floor area would become unlimited. The columns could be encased in concrete, masonry or spray on fireproofing (with an architectural finish applied as desired) at a very reasonable cost and there are relatively few of them. Therefore we recommend this for all columns for parking, office and community space use.

The maximum exit travel distances and the minimum number of exits will depend on the use classification, the number of occupants and the actual layout of the space. In general, it appears that for this building these requirements will not be overly restrictive regarding potential uses.

For retail store use, unless the entire roof structure as well as the columns were to be fire protected, there is a limit to the floor area between fire separation walls. However this floor area becomes quite large for a one story fully sprinklered building – 69,000 sq. ft. with possible additional increases depending on how much public frontage is planned for the final building.

For **Use 1** we have assumed that the garage would be classified as an Open Garage so that mechanical ventilation would not be required. For the building to qualify as an Open Garage the Code requires that at least 50% of the interior face of the exterior wall area on all four sides of the building be open and that the openings be distributed uniformly. Thus most of the present window openings would be converted to some sort of open entity – louvres or grating – that would permit natural ventilation.

For **Use 2** it is difficult to contemplate an architectural layout for one floor of offices in such a high ceilinged space. However if it were a large open area with the windows left in place but considered fixed glazing and the space fully conditioned, then offices might work. For retail we have assumed the possibility of large box retail or supermarket (an ideal use for this large building) and/or for smaller stores partitioned as in a small mall.

For **Use 3**, a mixed use possibility, the retail assumptions would be the same as Use 2 above. The community space portion might include large spaces such as a gym or an auditorium or a multi-function space or smaller meeting rooms. The two uses would be separated by a full height fire-rated partition.

For specific requirements, see the table that follows.

REQUIREMENTS FOR PROPOSED USES			
	Use 1: Parking Garage, Natural Ventilation, not heated	**Use 2: Commercial: Office and/or Retail**	**Use 3: Commercial/ Assembly: Office, Retail or Community Center**
Code Use Classification	S-2	M	M/A-3
Allowable Square Feet of Floor Area Between Fire Separations	Unlimited if columns are fireproofed for a height of 20 feet	69,000 sq. ft.	Use M: 69,000 sq. ft. Use A-3: Unlimited if columns are fireproofed for a height of 20 feet
Meet new Code Requirements for Accessibility, Egress, MEP	Required	Required	Required
Upgrade to meet new energy code	Not required	Required: Furr out walls and insulate to meet R-value; upgrade glazing.	Required: Furr out walls and insulate to meet R-value; upgrade glazing.
Seismic upgrade	Not required: no member receives more than 5% increase in seismic load nor has seismic resistivity reduced by more than 5%	Not required: no member receives more than 5% increase in seismic load nor has seismic resistivity reduced by more than 5%	Not required: no member receives more than 5% increase in seismic load nor has seismic resistivity reduced by more than 5%
Sprinklers	Dry automatic ordinary hazard	Wet automatic light hazard	Wet automatic light hazard
Electric Service	1000 ampere, 120/208 V 3 phase 4 wire	4000 ampere, 120/208 V 3 phase 4 wire	3000 ampere, 120/208 V 3 phase 4 wire
Water	Min. 1 ¼" service	Min. 2" service	Min. 2" service
Sanitary sewer	Lift station on south side of building force feeds main	Lift station on south side of building force feeds main; check to see if lift station and main have capacity for this increased use	Lift station on south side of building force feeds main; check to see if lift station and main have capacity for this increased use
HVAC	Not required	Roof mounted heating/ cooling units; allow 400 sq. ft./ ton of air conditioning	Roof mounted heating/ cooling units; allow 400 sq. ft./ ton of air conditioning
Other	At least 50% of wall area must be open for ventilation		Fire separation wall required between different occupancies
Live load capacity required by NYS Building Code	50 psf (for automobiles)	50 psf (+20 psf for partitions) for office 100 psf for retail	100 psf for community center 50psf (+20 psf for partitions for office) 100 psf for retail

CONCLUSIONS

In summary, based on our experience with similar types of buildings, RSA feels that Building 52 is a very good candidate for a future adaptive reuse. Minor repair issues do not affect this opinion. Thus, until it is determined what its future use might be, a ten-year stabilization effort is a logical choice to undertake at this time. We recommend that an annual inspection be conducted in the spring of each year to insure that no new defects have emerged. Since the building will require a new roof no matter what its future use, new roofing and flashing that will last at least 20 years is recommended. The other repairs will be shorter term in their effectiveness, but will allow time for decision making. And, should it become necessary to extend the stabilization period beyond ten years, there is no reason why a future assessment of the building, similar to this one, cannot be made at that time and further repairs recommended if required.

APPENDICES

A. Original Drawings

B. Field Observations

C. Probe Documentation

D. Non Destructive Evaluation
GBG

E. Roof Repair Drawings
James R. Gainfort, Architect

F. Concrete Testing Results
Kemron

G. Calculations

APPENDIX A

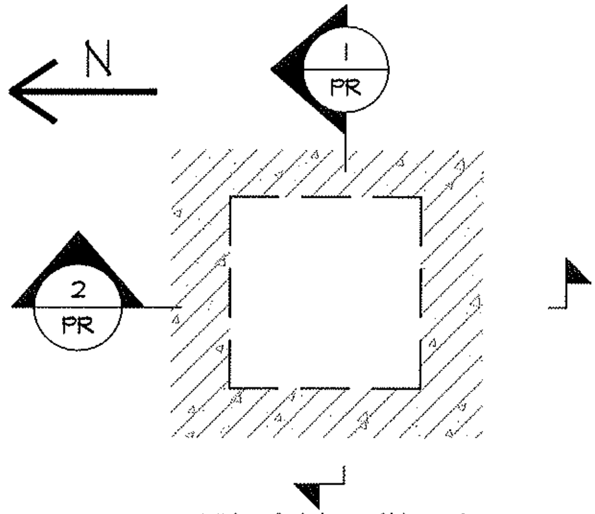
Original Drawings

APPENDIX B

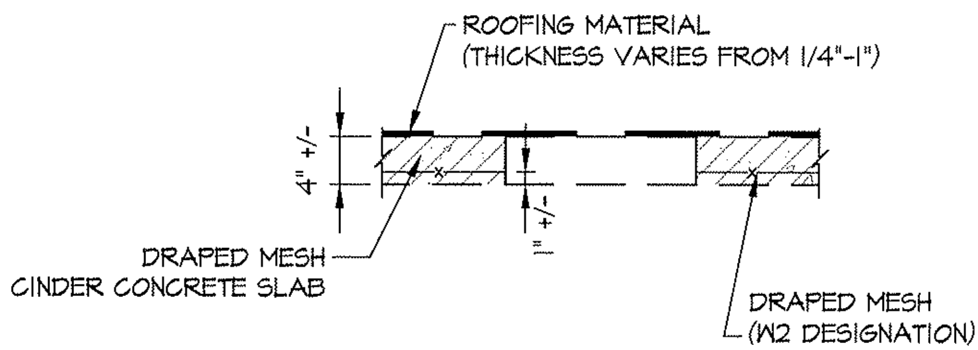
Field Observations

APPENDIX C

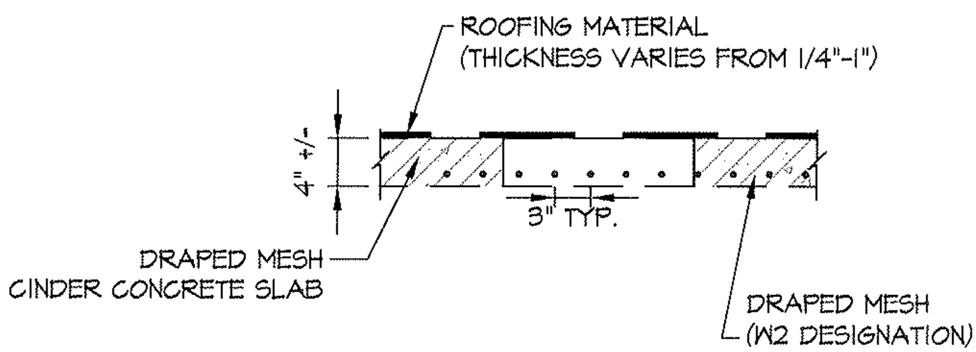
Probe Documentation



PLAN VIEW



SECTION 1



SECTION 2

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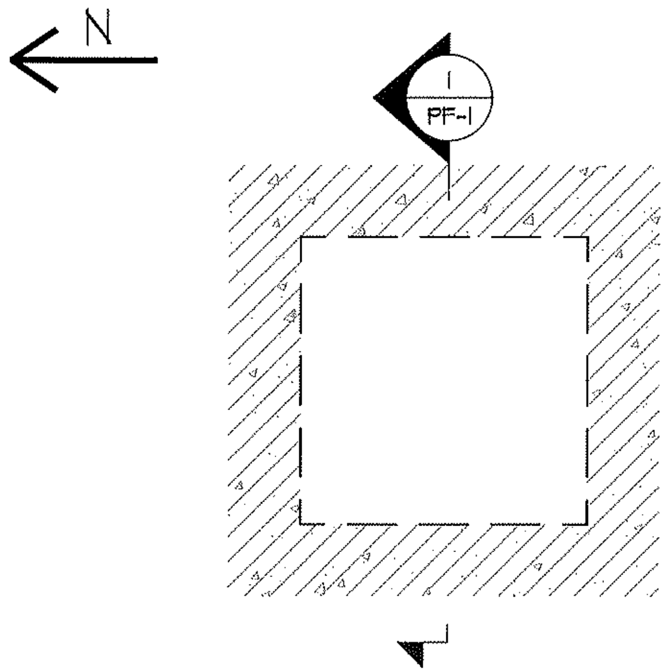
ROBERT SILMAN ASSOCIATES STRUCTURAL ENGINEERS 88 University Place New York, NY 10003 P 212.620.7970 F 212.620.8157 Reference number:	Job Title: BUILDING 52 STABILIZATION	Job Number: 12900.02 RP



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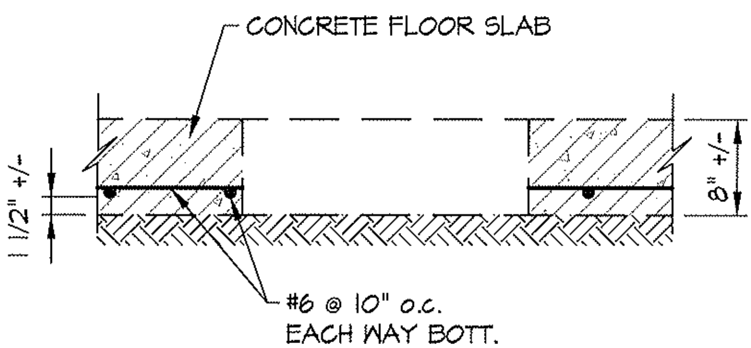

SECTION 1

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Date: 12/13/10

Scale: 3/4" = 1'-0"

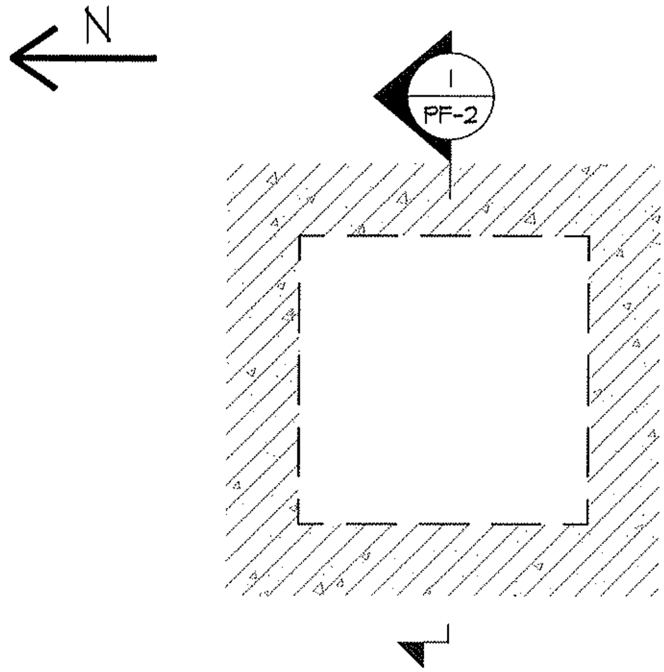
ROBERT SILMAN ASSOCIATES
STRUCTURAL ENGINEERS
88 University Place
New York, NY 10003
P 212.620.7970 F 212.620.8157

Reference number:

Job Title: BUILDING 52 STABILIZATION

Job Number: 12900.02

FP-1



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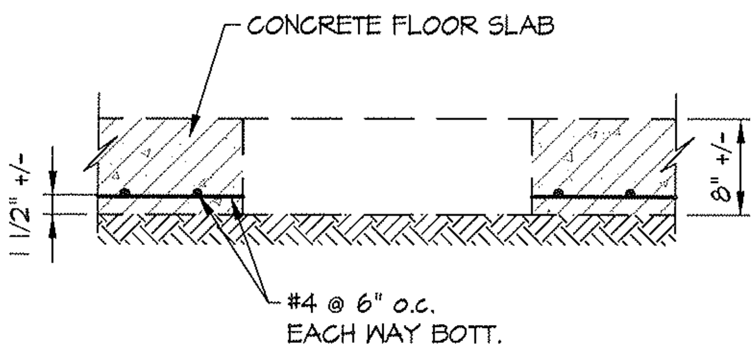

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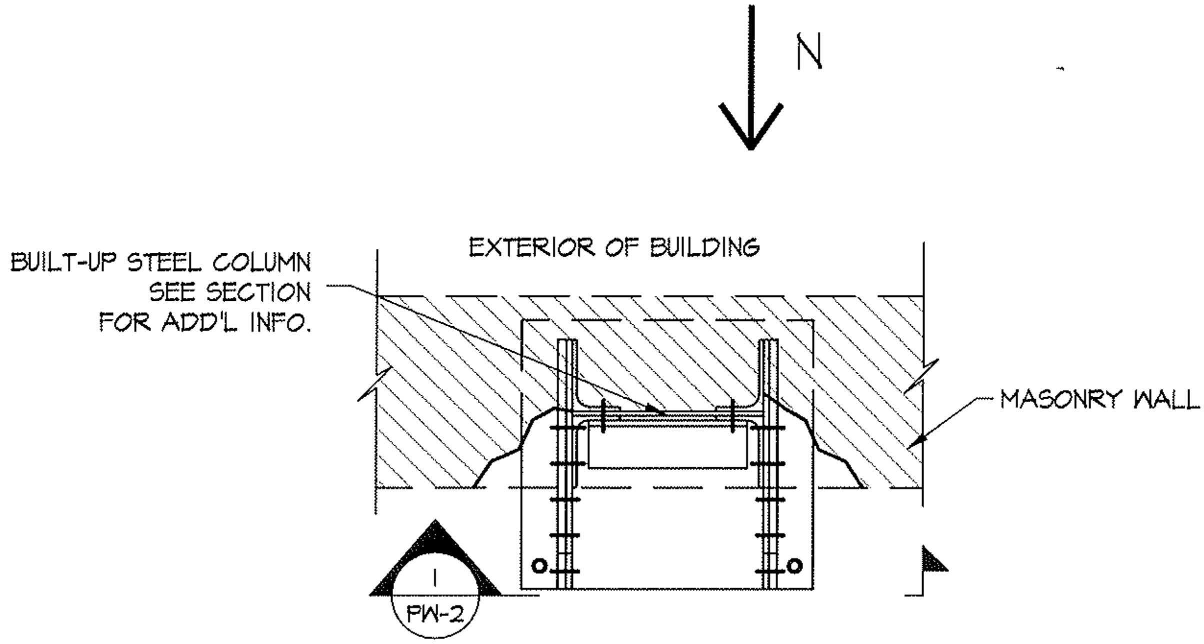
Job Number: 12900.02	FP-2



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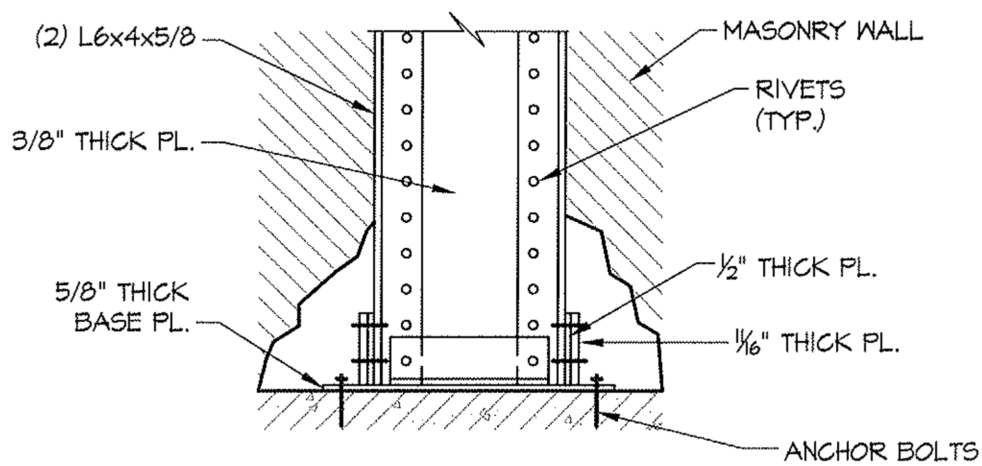

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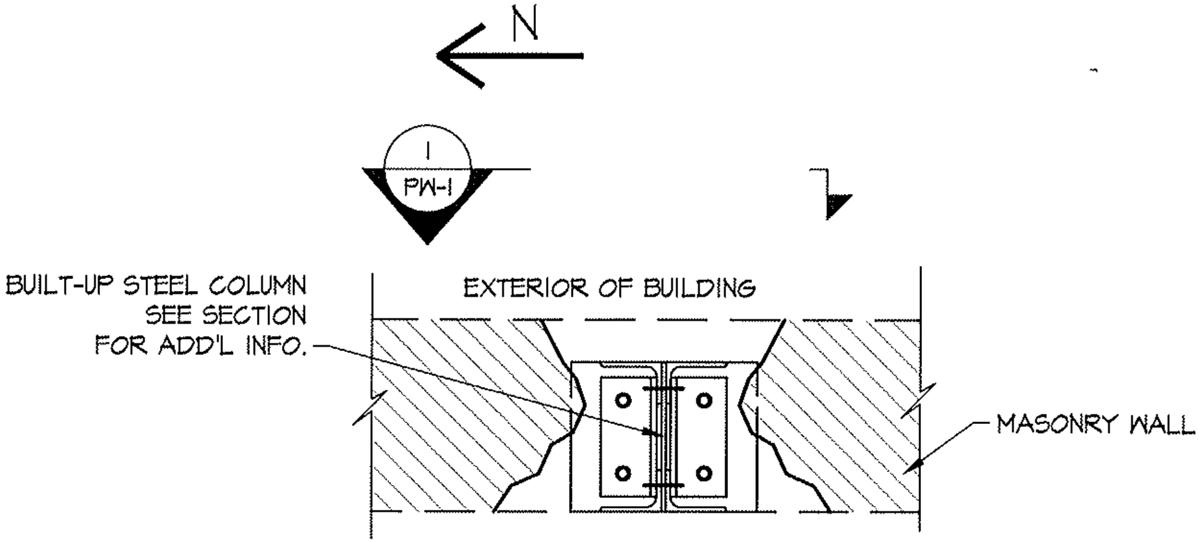
		WP-1B



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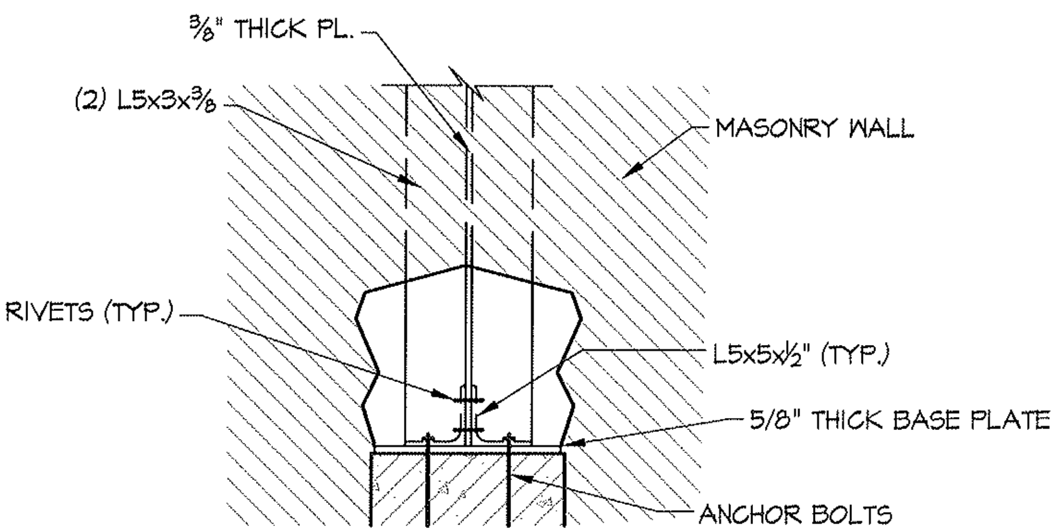

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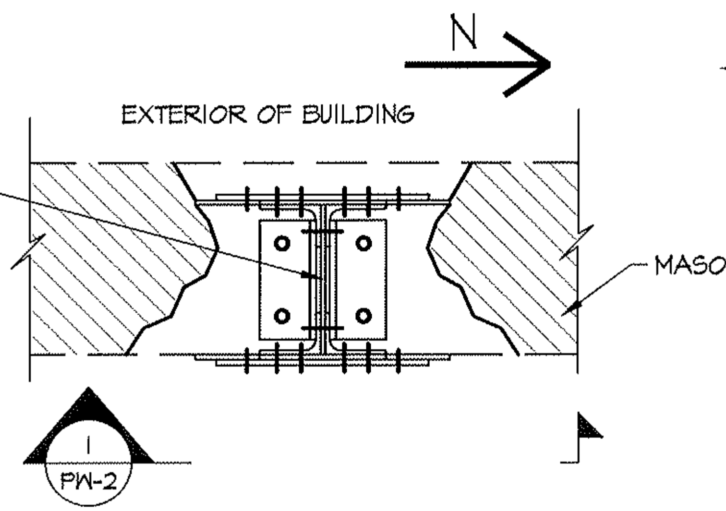

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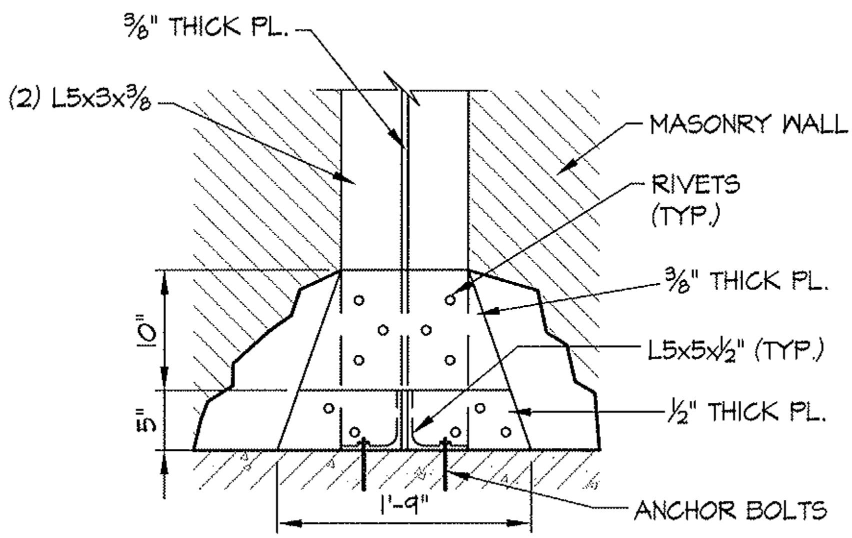

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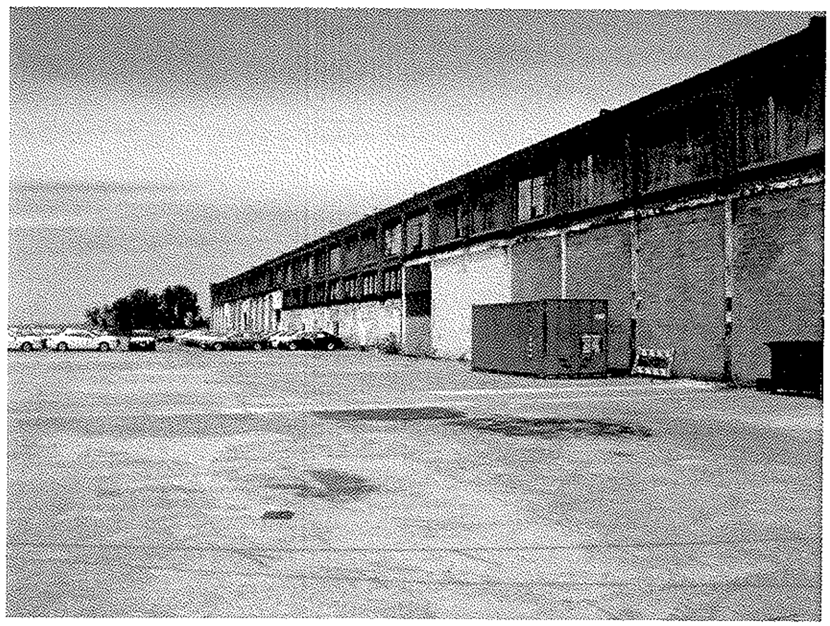
APPENDIX D

Non Destructive Evaluation

GBG Inc

GBG
Inc



Anaconda Building 52, Hastings-on-Hudson, NY

Non Destructive Assessment Of Floor and Roof Condition

Final Report – 10-030

Robert Silman Associates PC

PROJECT: Anaconda Building 52, Hastings-on-Hudson, New York

TITLE: Non Destructive Assessment of Floor and Roof Condition

CLIENT: Robert Silman Associates PC

GBG Report No: 10-030

Compiled By: A. White BEng &
C.S.A. Bransby-Zachary BSc MRICS

Issued on: 26th August 2010

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88 University Place, 9th Floor, New York, NY 10003 Tel: 212 777 3770
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GBG Inc

Anaconda Building 52, Hastings-on-Hudson, New York
Non Destructive Assessment of Floor and Roof Condition

1.0 INTRODUCTION

1.1 Terms of Reference

Structure:	Anaconda Building 52
Location:	Hastings-on-Hudson, New York
Consultants:	GB Geotechnics USA Inc. (GBG)
Instructed by:	Robert Silman Associates (RSA)
Survey Dates:	14th – 15th July 2010

1.2 General

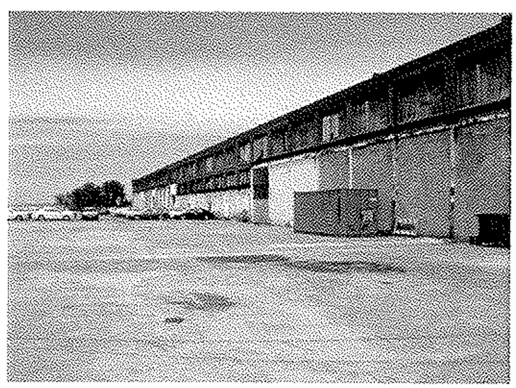
Further to your instructions, we attended the above referenced property to carry out a non-destructive evaluation (NDE) specifically to achieve the following:

1. Determine construction arrangement of the concrete floor in selected areas
2. Identify likely pile positions beneath the floor slab
3. Map the existence and extent of elevated moisture and/or delamination within the concrete roof slab

We have now completed analysis of the data collected to date and have pleasure in providing our report of the investigation (GBG ref: 10-030) which should be read in conjunction with GBG Drawings 10-030-01 and 02. Please note that this is the final report of our findings and therefore supersedes any previous reports whether written or oral.

1.3 Background Details

Anaconda Building 52 is one of the last remaining buildings on the former site of the Anaconda Cable Company. It is understood that the current owners are undertaking works to 'make the building good' before passing ownership to the township.


Building 52, viewed North-East

In order to determine the full scope of work required to bring the building up to the required standard, information is required regarding construction arrangement and condition.

Much of the information for such a basic feasibility study could typically be acquired through traditional probing / coring; however as a result of the heavy usage of PCB's and subsequent contamination of the site, and in an effort to understand the construction and condition on a more global basis throughout the site, data was also collected using non destructive testing methods.

GBG was commissioned to carry out a non destructive evaluation (NDE), which would establish the basic construction arrangement and condition of the floor and roof slabs, and would help determine the most appropriate locations for destructive probes (organised and documented by RSA).

2.0 THE SURVEY

2.1 General

Following an on-site safety training session on the 13th of July 2010 (attended by the GBG survey team) the NDE survey was carried out by GB Geotechnics USA Inc. over 2 survey sessions with a two person team on the 14th & 15th of July 2010.

In conjunction with the owner, the Client arranged permission for the survey team to access the building for the duration of the survey. Prior to our arrival on site the Client provided floor and roof plans (including preliminary RSA probe locations) for relocation on site and for use in the presentation of our results.

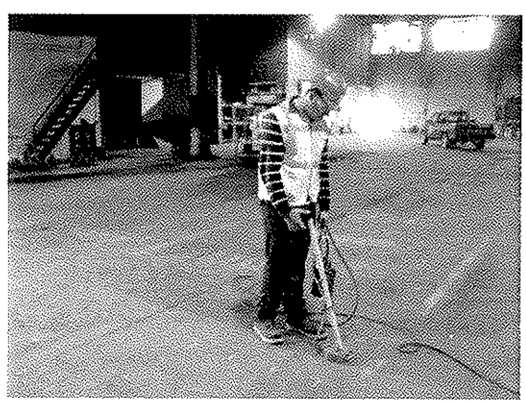
2.2 Methods

On site, the investigation was carried out using non destructive testing methods: these included Infrared Thermal Imaging, Impulse Radar and Metal Detection (Pachometer); a brief explanation of each NDE technique is given below, but further technical information is available on request.

As the main investigative techniques used are non-destructive, many of the findings given in this report are based on indirect measurements and the interpretation of electrical signals, electromagnetic signals and infra red thermal images. The findings represent the best professional opinions of the authors, based on their experience of similar investigations carried out on numerous other buildings over the past 30 years; and also the results of destructive methods of coring, drilling and probing carried out elsewhere on similar materials. Such tests have substantiated many of the conclusions that have been drawn.

2.2.1 Impulse Radar

Impulse radar was used to assess the construction arrangement of the floor (on a sample basis), specifically confirming slab thicknesses, reinforcement detailing and likely pile / pile cap locations beneath.



Impulse Radar Data Collection

The recording equipment was linked via a 60ft cable to the antenna and was powered by 12V DC batteries. Recovered signals were recorded both digitally and in analogue, as a paper record, enabling both on site interpretation and a more detailed analysis of the data off site.

All survey areas were investigated using various antennae with center frequencies between 1.6GHz and 400MHz; control settings were set to obtain information through the full thickness of the floor investigated and also near surface information.

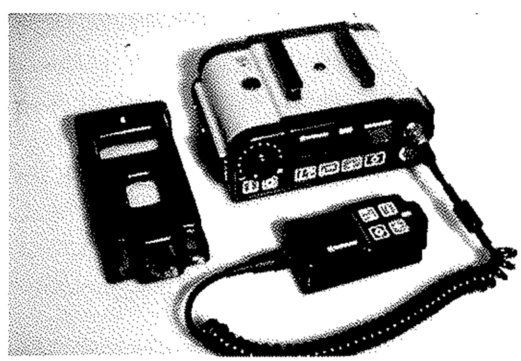
Survey grids marked in chalk on ground

Areas scanned using radar were selected by RSA, based on initially proposed probe locations and also the areas most likely to vary in construction or contain piles beneath. Typically, each area surveyed measured 15ft x 15ft; each was centered approximately over each the probe locations. A grid of measurements (1ft o.c.) were marked out in each survey area to reference all data collected.

2.2.2 Metal Detection / Pachometers

Metal detection was used in combination with impulse radar, primarily to confirm the existence and location of embedded metalwork (reinforcement) within the floor slabs surveyed in conjunction with impulse radar.

The method can positively identify that an object located is metallic and/or ferrous.



Typical Pachometer used

For the purposes of this survey it was used as a rapid scanner, allowing the presence of reinforcement or buried conduit to be found and therefore assisted in planning the radar profiles and helped to identify buried metallic objects. Metal Detectors are hand held and responses are noted by an audio signal, which is matched to a visual display of amplitude. Findings are recorded manually.

2.2.3 Thermal Imaging

A long wave infra red thermal camera was used to assess thermal variations over the exposed interior surface of the roof slab.

Changes in temperature identified through the use of thermography can be directly attributed to conditions such as:

- elevated moisture levels (damp),
- de-bonding concrete,
- voiding
- variations in construction

Ground Based Thermal imaging

The thermal output of the various surfaces was recorded in high-resolution, still thermo-graphic images; these were recorded in digital format and assessed both on site and off site.

For the purposes of this survey, we were specifically hoping to locate and map the extent of water ingress and resultant areas of elevated moisture ingress and, if possible, confirm whether spalling / delaminating concrete could also be resolved using this method.

3.0 FINDINGS

3.1 Overview

The findings and the conclusions reached have been derived from thorough data analysis using the various NDE techniques described above.

The findings are discussed briefly below are also presented on Drawings 10-030-01 and 02, which should be read in conjunction with this report. The two main phases of work (floor and roof slab surveys) are discussed separately below:

3.2 Floor Slab Survey

The results of the warehouse floor survey are presented on Drawing 10-030-1; this includes the location of all NDE survey areas, proposed probe locations, results in plan (for each survey area) and schematic sections through each section of floor surveyed.

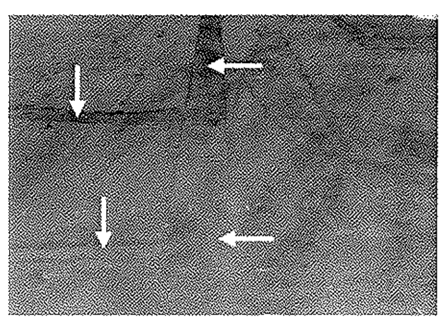
The floor slab construction has been assessed in detail over 5 areas (F-1 to F-5) using impulse radar and metal detection. Data was collected through the full thickness of the floor slab and into the supporting materials beneath.

Calibration through the concrete was not possible as the slab is ground bearing and, as such, no direct transmission could be taken through a known thickness of concrete. As calibration was not possible depth estimates through the slab and to reinforcement layers have been made using an assumed material wave speed of 10cm/ns which is typical for a well compacted concrete and normal levels of moisture.

Note: Once cores have been taken through the slab, they should be measured and the slab thickness compared to the radar data depth estimates. Any percentage error in material velocity identified and subsequent changes in concrete thickness estimates can then be adjusted for all survey areas.

3.2.1 Slab Construction

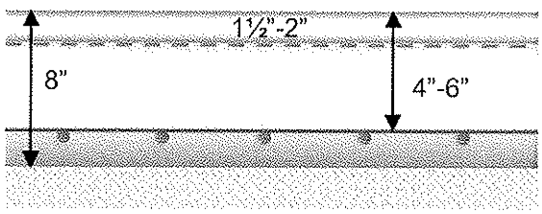
The overall floor slab construction is similar throughout; however variations in thickness and reinforcement placement were identified, which tend to relate to concrete filled service / pipe trenches (typically visible at the surface), repaired sections of slab (again typically visible at the surface), possible machine bases, changes in construction and likely pile cap locations beneath.



Visible service / pipe trenches

The typical floor slab arrangement appears to consist of an 8" thick reinforced concrete slab. The slab may incorporate a 1½ - 2" thick topping or screed; however any boundary between the two layers was poorly resolved therefore the screed could not be confirmed. The main slab contains reinforcement placed transverse and longitudinal to the warehouse walls at 9" o.c and at a depth of 4-6" deep. In some locations reinforcement is placed as tightly as 6" o.c. and as lightly as 24" o.c.; the placement depth also varies in places (See section below).

Note: bar sizes could not be provided as part of the NDE as the typical placement depth of 6" exceeded that required to obtain reliable sizing results using a Pachometer. As a result, bar sizes must be recovered during the probing phase.



Schematic section through floor slab showing typical Slab Construction

Schematic sections through the slab are provided for each area surveyed (Areas F1 to F5); these include concrete thickness, reinforcement spacing, conduits, trenches and likely pile locations (*See Sections A-A to G-G, Figure 1b*). The results from each area are also described in detail below:

- ***Probe (Survey Area) F-1*** – Data collected through Area F-1 identified 3 different slab designs. The central section of slab (extending NE to SW through survey area) represents the typical slab construction explained above. The NW and SE corners of the area however revealed thicker sections of slab, which appear to be at least 14" thick and contain reinforcement that is placed 12" o.c. in one direction and sparsely at 24" o.c. in the other.

  Other items resolved in the data were one near surface diagonal conduit or pipe and 4 possible pile locations (see 'X' symbols on drawing), which were selected due to an anomaly in the data (*See Section 3.2.2 for detailed explanation of pile cap data analysis*).

- ***Probe (Survey Area) F-2*** – Data collected through Area F-2 identified 2 different slab designs. Type A Construction (See Section B-B) represents the typical slab design; however Type B (See Section C-C), although being the same overall thickness (8" approx) contains reinforcement placed 12" o.c. in both directions. The bars are also placed deeper into the slab at approx 6-7" from the finished surface.

Visible changes in construction (See Red hatched areas) were confirmed as representing either a service trench (*note:* near surface pipe / conduit traced through center of diagonal trench) or occurred either side of a change in construction (between Type A and B construction).

Two potential pile cap locations were identified at Probe F-2. The data response at the SW corner identified a significant localised change in construction (reinforcement), therefore a probe was considered a likely pile location.

- ***Probe (Survey Area) F3*** – Data collected through Area F-3 identified only the typical slab design throughout.

  Three localised sections of slab were however confirmed as being thicker. One linear section is assumed to be a concrete filled trench; however the remaining two thicker sections could represent pile cap locations as one extends around a column and the other is coincident with the adjacent column line.

  One probe location has been recommended in the SW corner of Area F-3, where a pile cap may exist. Finally, a number of near surface linear features (pipes or conduits) extend along the west boundary of the survey area. *Note:* Care should be taken to avoid these features if adjacent probing is carried out as recommended.

- ***Probe (Survey Area) F4*** – Data collected through Area F-4 identified 2 different slab designs. Type A Construction (See Section E-E) & Type B (See Section F-F). The slab designs are essentially the same (typical slab design); however the transverse and longitudinal reinforcement for each area is placed in opposing directions.

  Area F-4 contained the only section of concrete which appears to contain increased moisture levels. *Note:* the apparent increase in moisture has been detected through analysis of radar data only. The response is restricted to this area only and is therefore unlikely to represent a widespread problem (probing in other locations will confirm whether a problem of moisture infiltration exists).

  One probe location has been recommended along the East boundary of Area F-4, where a pile cap may exist. At this location the slab did not appear to be thicker; however a localised increase in reinforcement might represent a pile cap and was therefore deemed to be an area of interest for probing purposes.

- ***Probe (Survey Area) F5*** – Area F-5 was much smaller than Areas F1-F4; the slab construction was found to be of typical design. One probe has however been recommended in this area where the concrete appears to thickens to approx 11"-12" around a column base and may therefore represent a pile cap location (See Section G-G, which illustrates the apparent thicker concrete at this location).

3.2.2 Pile Caps

We understand that the warehouse is built over reclaimed land (adjacent to the Hudson River) and is therefore likely to be supported over piles; these are believed (by RSA) to be spaced at approximately 8ft o.c. Documentation available is however only limited and only represents the original design; the actual location of the piles therefore remains unknown.

Typical red 'X' marked on site to denote the potential location of a pile cap

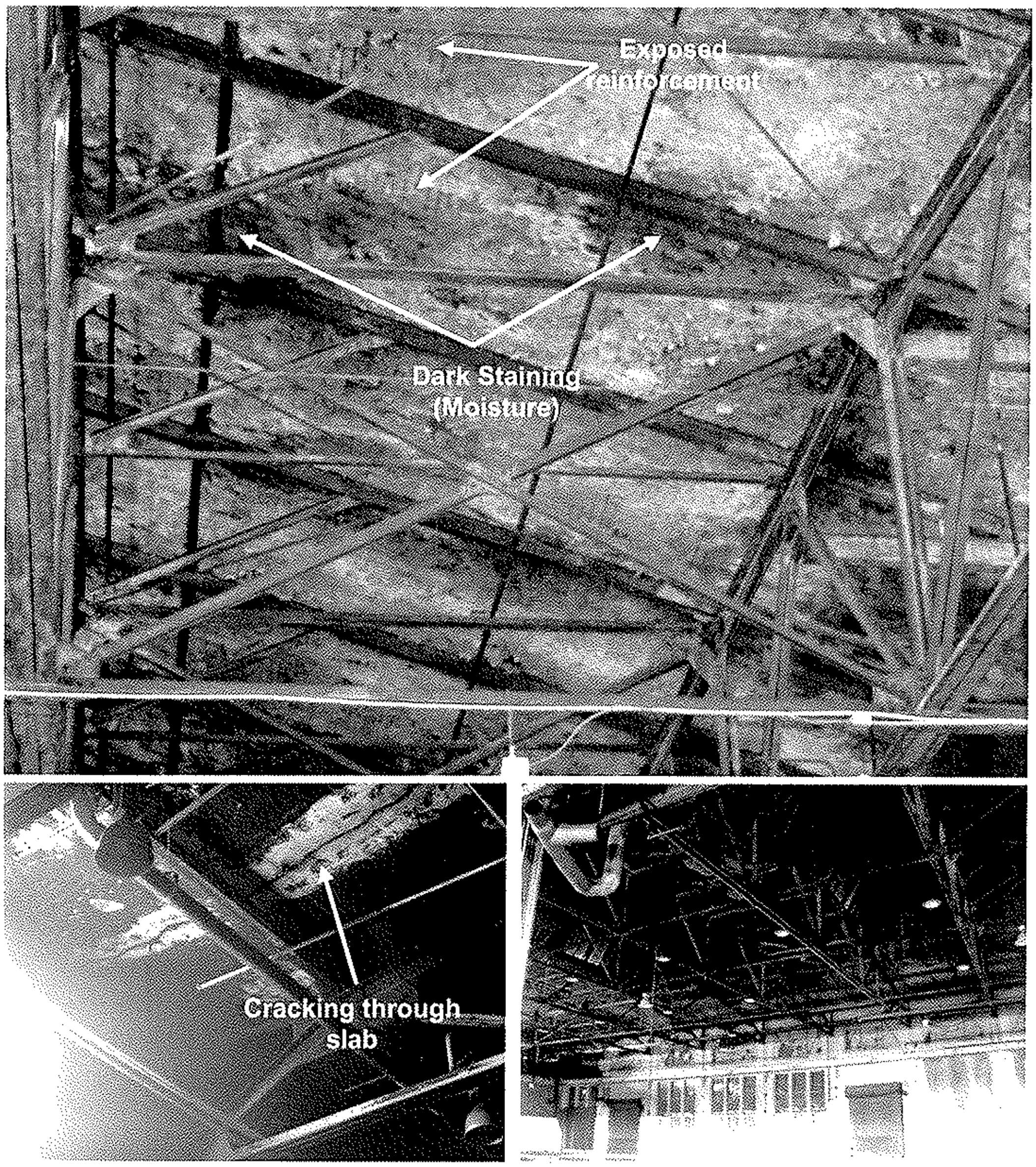
As part of this survey GBG were asked to locate pile caps; however as the pile cap design (assumed thicker concrete with timber pile beneath) is unknown, radar data was scanned for anomalies in the form of apparent localised variations in reinforcement (typically increases) or sections of slab which were thicker than others and did not appear to relate to services trenches or other construction changes / repairs.

The data collected in each of the areas has been analysed in detail, which has included plotting of any anomalies that could represent pile cap locations. Where a localised variation in construction has been identified in the data an 'X' has been placed on the drawings (and painted on site - See photo above) to denote a possible pile cap position.

Where the data responses and variations are significant a probe location has also been recommended. Taking the above description into consideration, there is no guarantee that a 'X' symbol and associated probe will reveal a pile cap; however without excavating the entire slab, they do currently represent potential positions.

3.3 Roof Slab Survey

The roof structure is a 'saw tooth' design with continuous troughs on each side of the building, extending North – South (rainwater discharge into drains). A system of exposed steel trusses provides support to the structure between the column locations indicated on the plans. The interior finish of the roof is generally exposed concrete, with some areas clad in metal sheet and some painted.



Images of Roof Soffit
Top – Showing spalling concrete and exposed reinforcement
Bottom Left – showing cracking through slab and section coated in black paint
Bottom Right – showing complex truss arrangement through which scanning took place

Visual inspection from ground level shows the warehouse roof slab to be in poor condition; although little is known about the embedded reinforcement condition. Parts of the soffit are covered in dark staining, suggesting an active water ingress problem (the roof is known to leak); other parts of the roof are cracked and in places the reinforcement is clearly visible suggesting a problem of spalling concrete most likely associated with long term water infiltration through the slab. *Note:* All these conditions are shown in the example photos of the soffit on the previous page.

Large sections of the soffit are also covered in small light patches, which could a number of things: including localised repairs, spalls or perhaps the remnants of an adhesive, which used to hold interior finishes to the exposed concrete. The origin or reasons for the patches is not known.

The soffit surface is therefore highly variable in its finish and visible condition. We understand that probes are to be taken through the roof slab (organised by RSA) to better understand its general construction and condition. In order to help target these probes and to provide more widespread information across the slab GBG carried out a thermal review of the soffit from ground level.

3.3.1 Thermal Review

The results of the thermal review are presented on Drawing 10-030-2; this includes thermal images taken at each of the recommended probe locations through the soffit and also annotation and explanation of each image used.

A long wave infra red thermal camera was used to assess thermal variations over the soffit. Variations in surface temperature can be attributed to a number of different factors such as retained moisture, damaged / spalling masonry and concrete, and also major changes in material thickness and voiding.

It is variations from the ambient temperature which are mapped as part of the thermal review, therefore an understanding as to the likely reasons for any variations and also the survey conditions are critical to the results collected and the analysis provided.

Mapping Moisture - For the purposes of this survey cooler (darker) responses were most likely to represent increased near surface moisture as the moisture itself would be cooler than the slab and evaporative cooling across the surface would increase the thermal contrast making them relatively straight forward to plot using this method.

Mapping Spalling - Typically identification of a spall relies on that spall cooling or heating more rapidly than the surrounding concrete as it becomes detached from the main body of the surrounding concrete. The heating and cooling cycle however relies on the slab heating or cooling. A secondary method of mapping spalls would be to look for discontinuities in thermal transfer through the slab, where sections of concrete that were delaminating or cracked should transfer heat through the slab at different rates to sections of good condition (well bonded and well compacted) slab.

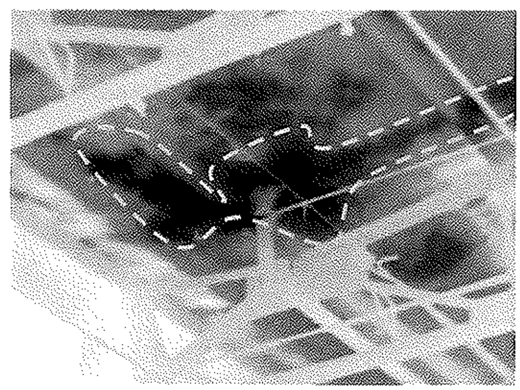
For the purposes of this survey, GBG was hoping to confirm whether tracking of a spall was indeed possible from ground level so that if the NDE results correlated with probe results then a more extensive scan (at closer proximity to the roof slab) might be of significant use to the client in mapping the extent of spalls throughout the warehouse, without the need for time consuming and logistically challenging sounding work.

Note: It is important to understand that the remote thermal scanning as described above cannot provide structural information regarding the slab (thickness, rebar arrangement etc); it can only provide a comparative condition assessment of surface condition that focuses on areas of increased moisture and potentially of delaminating and spalling concrete.

3.3.2 Increased Moisture / Water Ingress

Immediately prior to carrying out the site survey work a significant amount of rain had fallen (two days of heavy rain); this provided ideal conditions for mapping water ingress at the slab soffit.

An initial set of images were collected focussing only on the 7 locations (R-1 to R-7), which had been selected by RSA as potential probe locations (See Figure 2a for locations). Areas of increased moisture ingress were first mapped and a thermal image taken at each of these locations is presented and annotated on the drawings provided (See Fig 2b).



Example Thermal Image – Cooler (darker) responses across the soffit surface revealed increased moisture

Following the initial review of the probe locations, a more general survey was undertaken of the soffit. Thermal images were collected within each of the bays from a number of vantage points in order obtain the best possible coverage of the roof slab. Due in part to the previous heavy rainfall, identification of moisture was relatively straightforward. The areas have been plotted in plan on Drawing 10-030-02, Fig 2a.

Note: As areas of increased near surface moisture were widespread, only the most significant of these areas has been plotted onto the drawings. A more detailed thermal scan would be required to plot all areas of increased moisture.

The most significant areas of increased moisture were identified around roof drains suggesting that the waterproofing around the drain penetrations have deteriorated and failed. The design of the roof includes for longitudinal troughs, which channel rainwater towards the drains; standing water occurring over long periods, (perhaps due to blockages in the drains or build up of vegetation), also may have deteriorated the roof as increased moisture was also apparent along the trough lines (See Drawing 10-030-2 for extent of troughs and comments on likely problems associated with them).

One additional observation was that water ingress tends to occur around the columns along the center line of the building.

3.3.3 Mapping of Spalling / Delaminating Concrete

Identification of delamination and spalling using this technique was a more difficult task. The surface condition of the soffit was highly variable therefore it was not possible to identify smaller localised spalls from ground level. Instead, the analysis focussed on identifying larger areas of slab which looked like they might be spalling / delaminating.

A large proportion of the images collected contained sections of localised hotter responses; which could suggest a widespread problem of spalling and delaminating concrete.

It is also possible however that the hotter responses may also relate to changes in surface coating such as the black (presumably waterproof) coatings, the small patches observed almost everywhere or even sunlight reflecting back to the soffit from steel trusses and girders.

Example Thermal Image - of numerous hotter (lighter) thermal responses that 'could' represent spalling and

As a result of the various analysis considerations explained above, our recommendation is to confirm the accuracy of the thermal imaging results collected during the probing phase. Documentation of probing and ideally localised sounding work in the area surrounding the probes will identify whether delaminating or spalling sections of concrete exists. With this information available the thermal images for each of the probe locations R-1 to R-7 can be reanalysed to see whether the thermal variations correlate with the physical sounding assessment.

As the results were so variable and will require calibration through probing / sounding (as discussed above is necessary) the hotter responses have not been plotted in plan; only one example is shown (See Fig 2b, Probe R-1, response A) on the drawings provided.

4.0 SUMMARY

The program of NDE has provided construction related information for the concrete floor slab and comparative condition related information for the roof slab, which has helped to target planned probing work and has provided generalized information regarding the warehouse construction and condition that would not be possible if only probing had been carried out.

4.1 Floor Slab Construction

As expected the warehouse slab has a typical design throughout, but has numerous repaired sections, service trenches, construction changes (including variable reinforcement designs) and thicker sections, which might either relate to original machine bases or pile cap locations. A suitable sample of these different conditions will be included within the probing scope in order to better understand the floor slab arrangement, and hopefully to locate some of the piles to confirm both their design and approximate placement pattern.

If individual piles are located during probing (based on the NDE results), then this should provide calibration to the existing data and should ultimately allow for impulse radar to map the locations of other piles, should the client request this information.

Although the survey was limited to just 5 small areas, no evidence of any underground rooms or significant voiding was resolved. It should however be remembered that the warehouse is extremely large therefore a more extensive survey would be required to confirm whether any large open voids actually exist beneath the site.

Typically, the radar data collected was consistent throughout each of the survey areas scanned. The data transmission was relatively good through the slab and also of the reinforcement, suggesting that the concrete is likely to be typically well compacted through the full depth of the slab and around the reinforcement. Little evidence of any voiding was identified within the supporting materials.

Information recovered during the probing phase should be well documented and if requested GBG would be happy to adjust depth estimates (which are currently based on assumed material velocities of radio waves through concrete) and add an addendum to this report for explanation.

Should additional work be required to map more accurately the locations of piles (based on probe results) or to scan additional areas of slab, which may be of specific interest (perhaps to identify voiding within and/or beneath the slab, then GBG would be happy to provide proposals for this work.

GBG Inc

4.2 Roof Slab Condition

Infrared thermal images of the roof slab soffit have confirmed that water infiltration is a significant problem, which may have caused lasting damage to the embedded reinforcement and concrete. Active water leaks occur during heavy rainfall (confirmed at time of survey) and are focussed around the linear troughs, which extend along each side of the roof and which help channel water to the roof drains (which themselves leak). Water ingress (increased moisture in the concrete) was also identified around the columns located along the center line of the building.

Planned probing through the roof will provide additional information on construction arrangement and importantly will confirm the condition of both the concrete and the reinforcement. Close visual inspection should also resolve the reasons for the numerous small patches all over the roof surface.

In order to calibrate the thermal review results, specifically with regard to potential thermal responses to spalling and separation, sounding work is recommended adjacent to the planned probes. If documented during the probing phase, GBG would be happy to use this information and review the existing thermal data available. This process would help to calibrate the thermal data and provide a better understanding as to whether the existence and extent of spalling and delaminating concrete can be mapped using this method.

If the information can be collected remotely (using thermal imaging), then it should be considered for use in a more extensive survey across the soffit (*as an alternative to traditional sounding*) using a mobile scissor lift which would allow for more detailed thermal images to be collected. This would improve the quality of images collected and would allow for accurate mapping and plotting of all spalling and areas of increased moisture.

Finally, based on the results of the probes taken, additional NDE work could be considered in order to provide more widespread information regarding the construction and condition of the roof. On a comparative basis for example the condition of embedded reinforcement (including depth of cover, placement and size) can be established using non destructive methods. Hand access to the soffit however would need to be provided in order to achieve this.

APPENDIX E

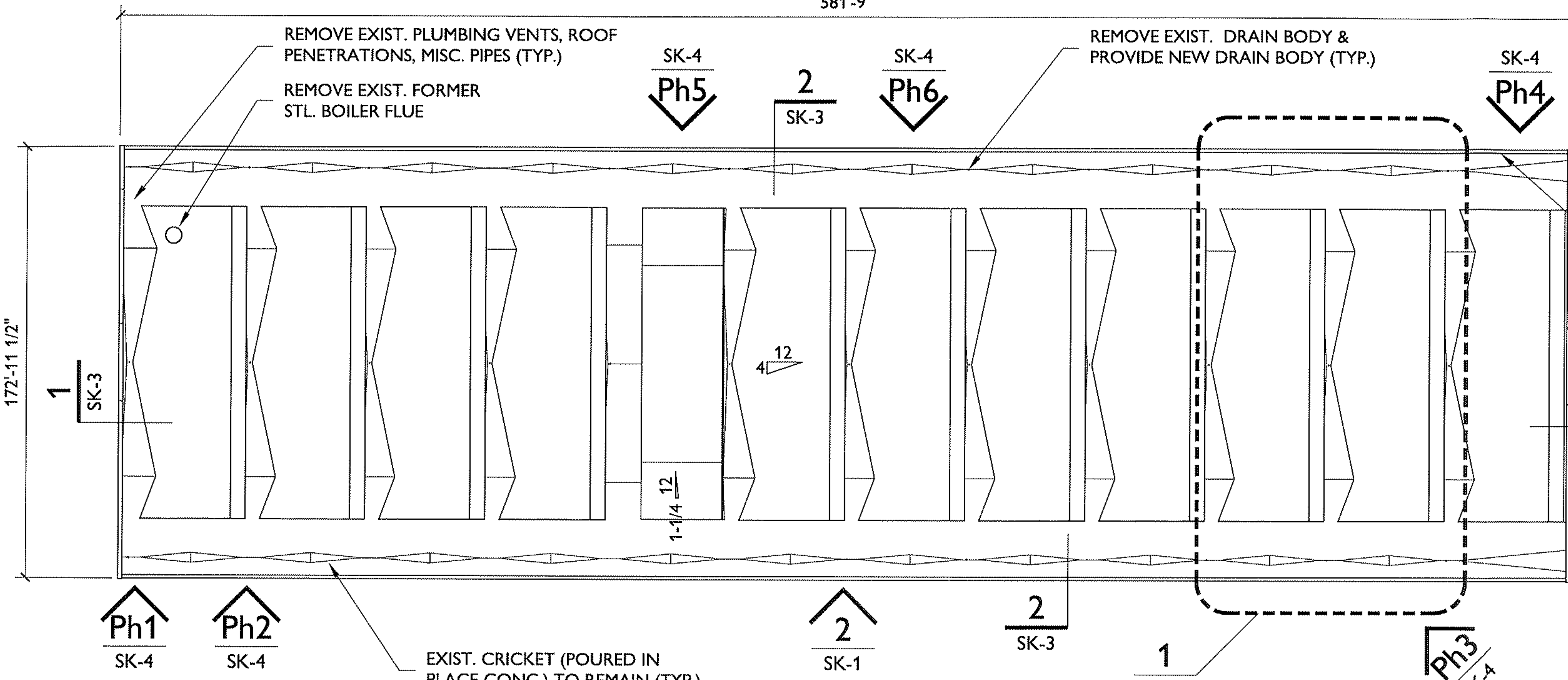
Roof Repair Drawings
James R. Gainfort, Architect

HASTINGS ON HUDSON - BUILDING 52
PRELIMINARY SCOPE OF WORK

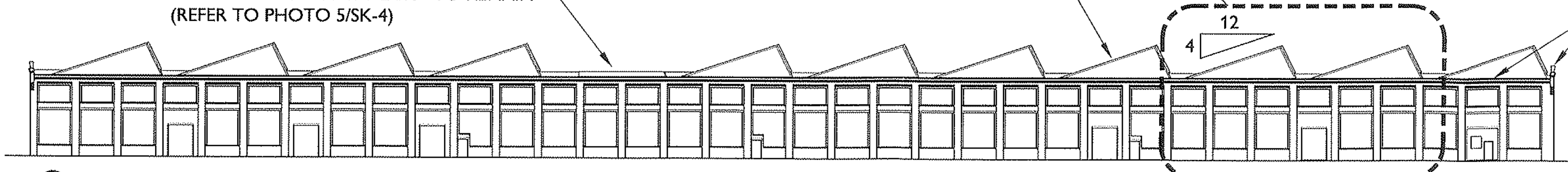
SK-1	ROOF PLAN & EAST ELEVATION
SK-2	ROOF & EAST ELEVATION DETAILS / S.O.W. NOTES
SK-3	COPING / ROOF DETAILS
SK-4	ROOF PHOTOS

ROOF AREA:

SKYLIGHT MONITOR	4,500 sf + 1,450 sf = 5,950 sf	x (11) =	65,450 sf
SKYLIGHT CRICKETS	350 sf + 350 sf = 700 sf	x (11) =	7,700 sf
TOTAL AREA (11) SKYLIGHT MONITORS / CRICKETS:			73,150 sf
AREA OF ROOF (NOT INCL. SKYLIGHT MONITORS):			35,800 sf
TOTAL ROOF AREA:			108,950 sf



1 ROOF PLAN
SK-1 SCALE: 1'=50'



2 EAST ELEVATION
SK-1 SCALE: 1'=50'

Title: Roof Plan & East Elevation - Preliminary S.O.W.

Project: Hastings on Hudson Building 52

Date: 8 Nov 2010
Scale: 1" = 50'
Job No. 1010

Dwg No. SK-1

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121 W 27th St, Suite 803 New York, NY 10001-6207
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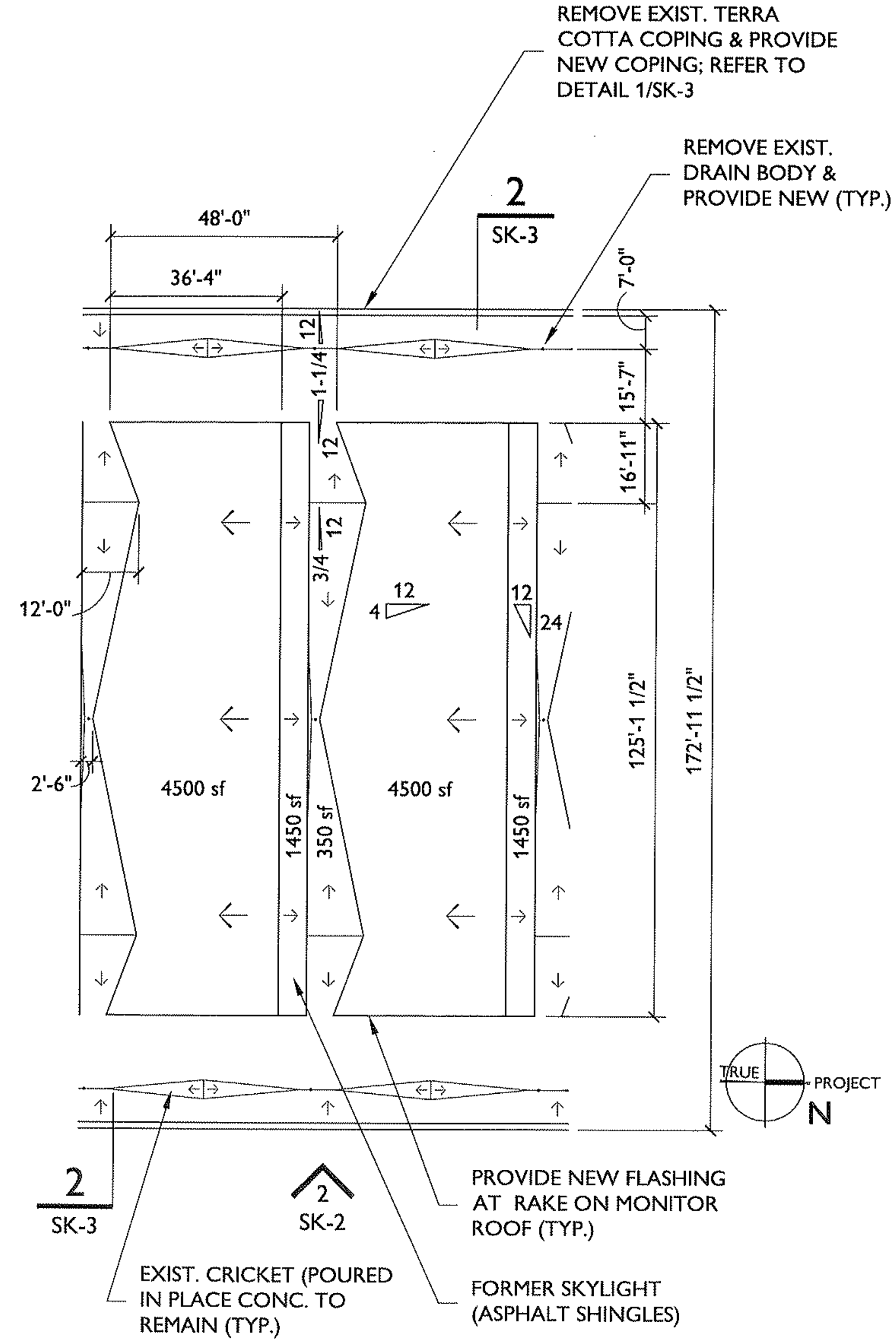


1 ROOF PLAN DETAIL

SK-2 SCALE: 1/32" = 1'-0"

ROOF S.O.W. NOTES:

DEMOLITION:

1. SCRAPE LOOSE / EXCESSIVELY BUILT-UP MAT'L AT MAIN ROOF & MONITOR SURFACES.
2. REMOVE ASPHALT SHINGLES AT EXIST. SKYLIGHT COVERS.
3. DEMO. CEM. PLASTER SIDE WALLS OF MONITORS, INCL. ASSOC. LIGHT MTL. FRAMING.
4. REMOVE ALL DRAIN BODIES, FLASHING, TERRA COTTA COPING, FLUES, PENETRATIONS & MISC. PIPEs.

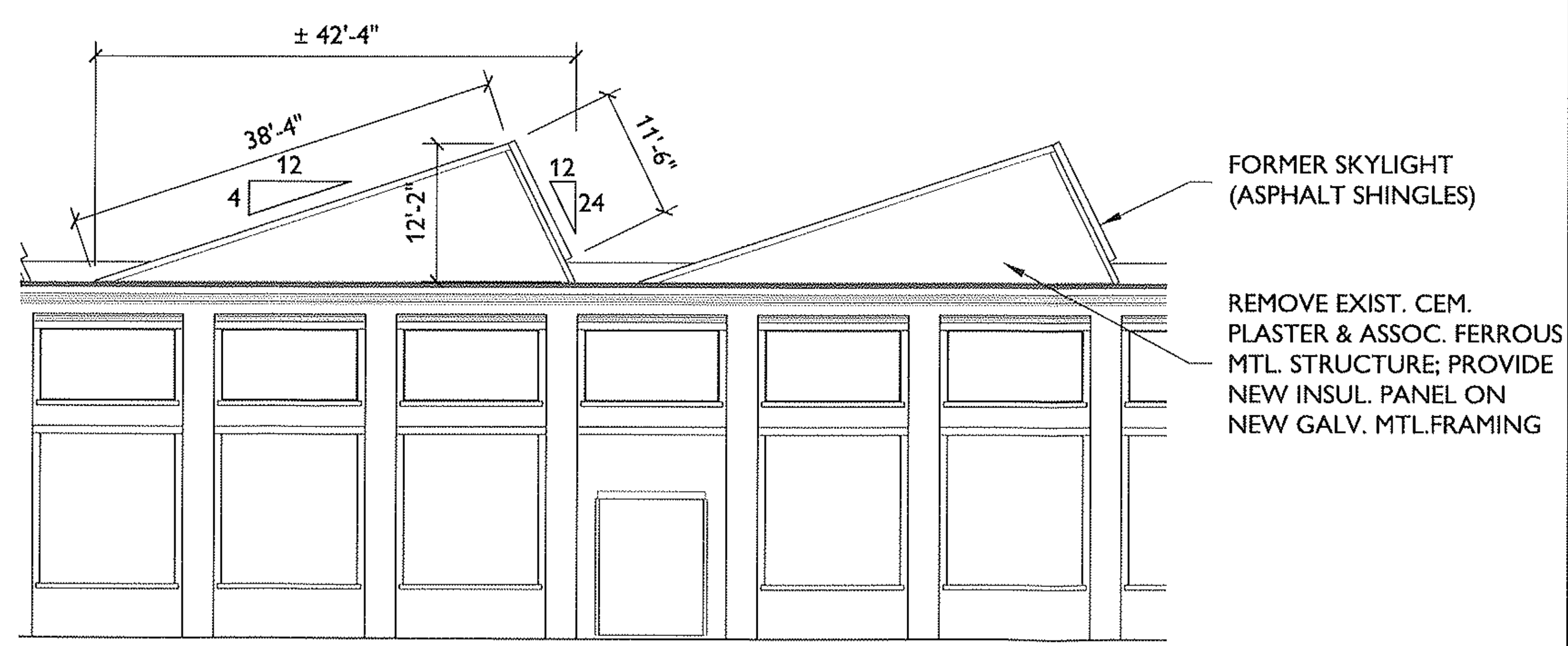
WHERE ROOF DECK BECOMES EXPOSED, REVIEW CONDITION OF CONC. & REPAIR IF REQ'D., REFER TO STRUCT'L ENGR'S S.O.W. WHERE CONC. MUST BE REPAIRED, ASSUME 100% REMOVAL OF EXIST. MEMBRANE.

MAIN & MONITOR ROOFS:

5. PROVIDE NEW (2) 2" LAYERS (4" TOT.) RIGID POLYISO. INSUL. BD., MECH'LY ATTACHED. STAGGER LAYERS & USE TAPERED INSUL. WHERE NEC. TO CORRECT / IMPROVE PITCH. PROVIDE NEW 1/2" CEM. BD. COVER OVER INSUL., MECH'LY ATTACHED. PROVIDE NEW WHITE, FULLY-ADHERED 60-MIL TPO (SPECTRO-WELD REINFORCED TPO MEMBRANE BY 'CARLISLE SYN-TEC').
6. PROVIDE NEW DRAIN BODIES; IF NEC., REPLACE LENGTHS OF DRAIN TAILPIECES / ELBOWS, ETC BELOW.
7. PROVIDE NEW RAKE FLASHING AT MONITOR SIDE WALLS.
8. AT EXIST. SKYLIGHT COVERS, PROVIDE NEW (2) 2" LAYERS (4" TOTAL) RIGID POLYISO. INSUL. BD., MECH'LY ATTACHED (STAGGER LAYERS). PROVIDE NEW WHITE, FULLY-ADHERED 60-MIL TPO (SPECTRO-WELD REINFORCED TPO MEMBRANE BY 'CARLISLE SYN-TEC').
9. PROVIDE NEW GALV. STL. FRAMING & INSUL. MTL PANELS AT MONITOR SIDE WALLS.

MASONRY PARAPETS:

10. REVIEW CONDITION OF MAS.; REBUILD TO INSURE STABILITY.
11. NORTH & SOUTH FACADE PARAPET WALLS: SHEATH ROOF SIDE FACE OF PARAPET W/ CEM. BD.; ADHERE TPO TO FACE OF BDS. PROVIDE NEW PLYWD., SELF-ADHERED SHT. WATERPROOFING & TPO-CLAD SHEET MTL. COPING; HEAT WELD TO VERT. TPO. REFER TO 1/SK-3.
12. EAST & WEST FACADE PARAPET WALLS: PROVIDE NEW PLYWD., SELF-ADHERED SHT. WATERPROOFING & TPO-CLAD SHEET MTL. COPING; HEAT WELD TO TPO FIELD MEMBRANE. REFER TO 2/SK-3



2 EAST ELEVATION DETAIL

SK-2 SCALE: 1/16" = 1'-0"

Title Roof & East Elevation Details / S.O.W. Notes

Project Hastings on Hudson Building 52

Date 8 Nov 2010
Scale As Noted
Job No. 1010

Dwg No. SK-2

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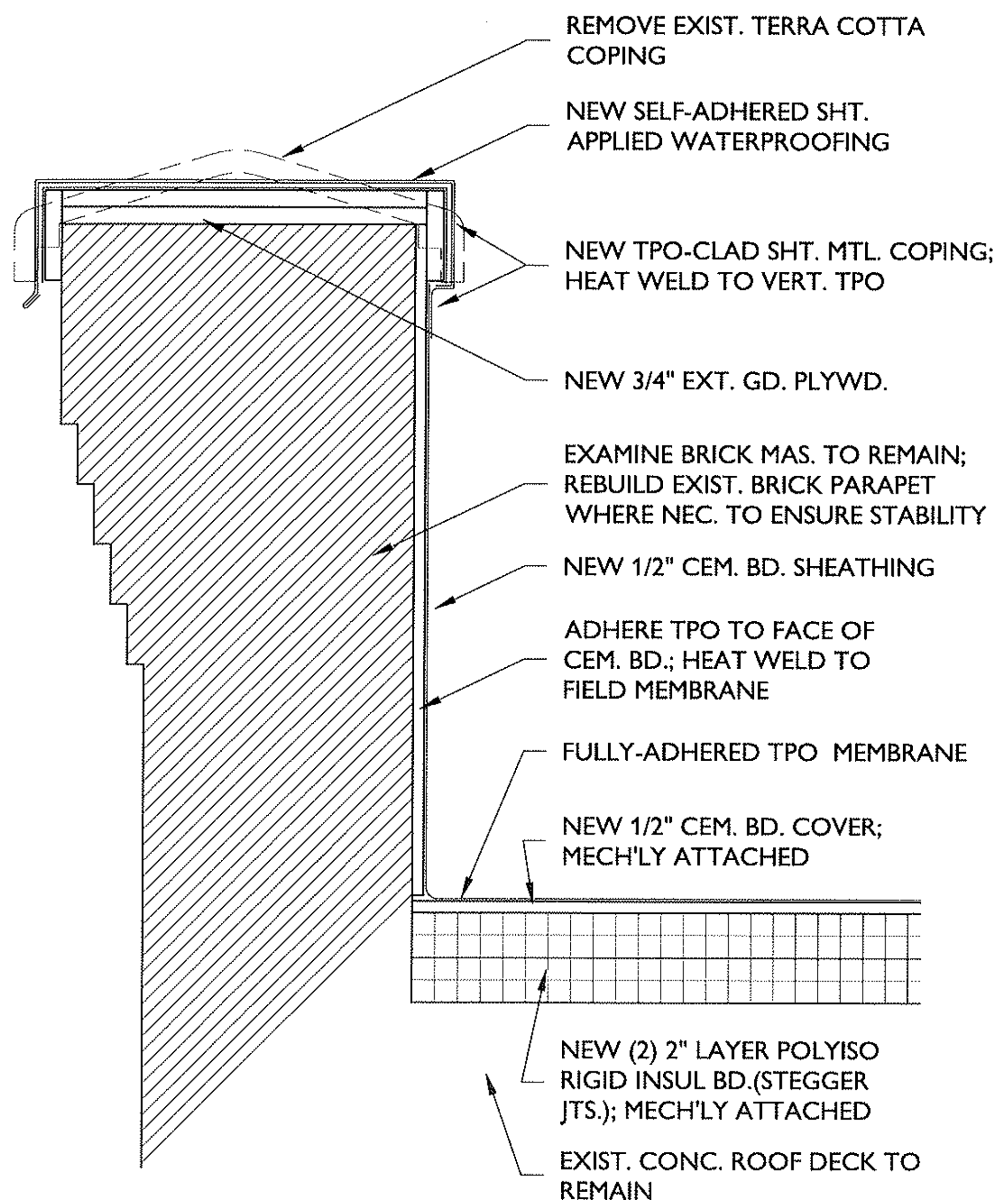
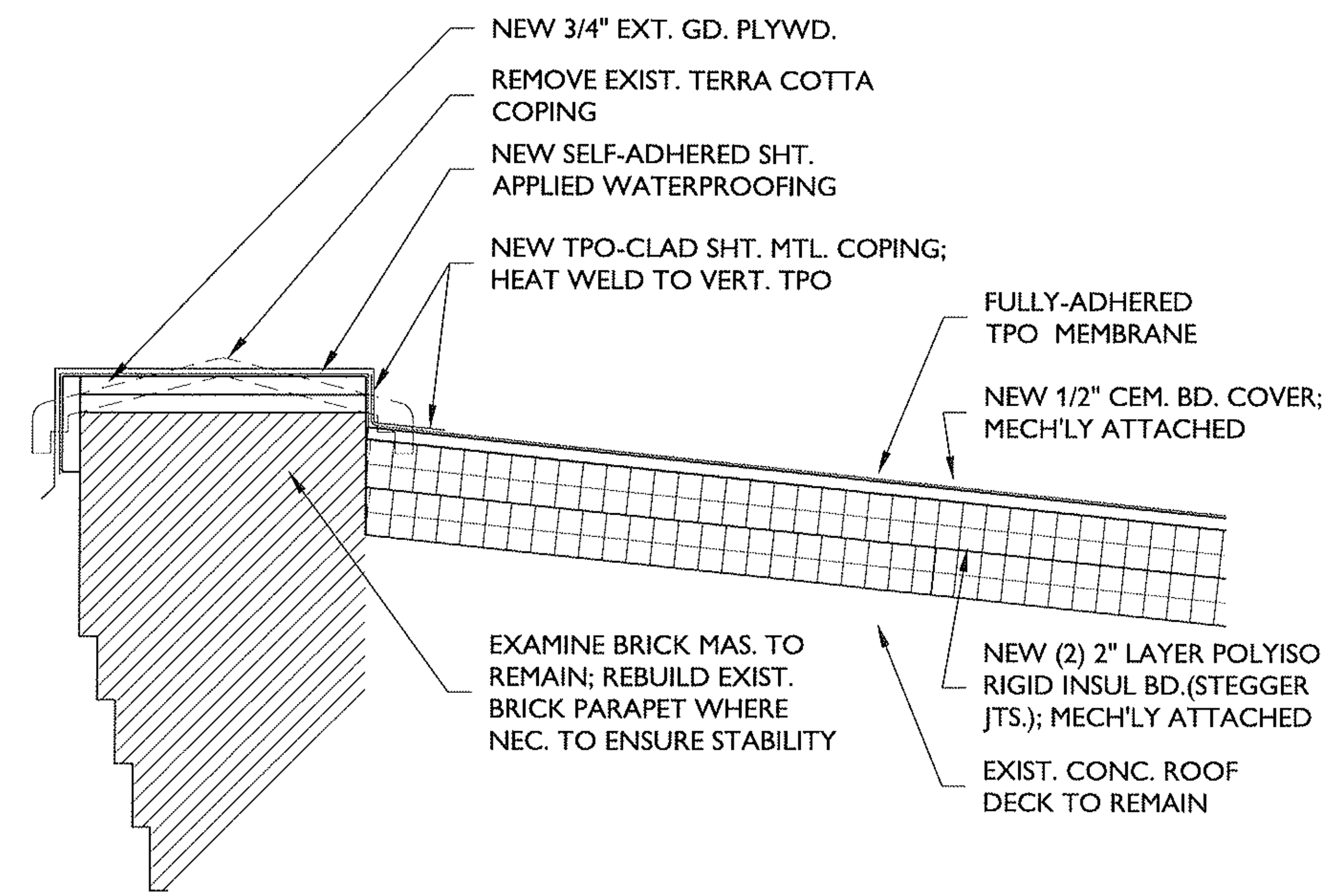


1 NORTH & SOUTH PARAPET DETAIL

SK-3 SCALE: 1-1/2" = 1'-0"



2 EAST & WEST PARAPET DETAIL

SK-3 SCALE: 1-1/2" = 1'-0"

Title Coping / Roof Details - Preliminary S.O.W.

Project Hastings on Hudson Building 52

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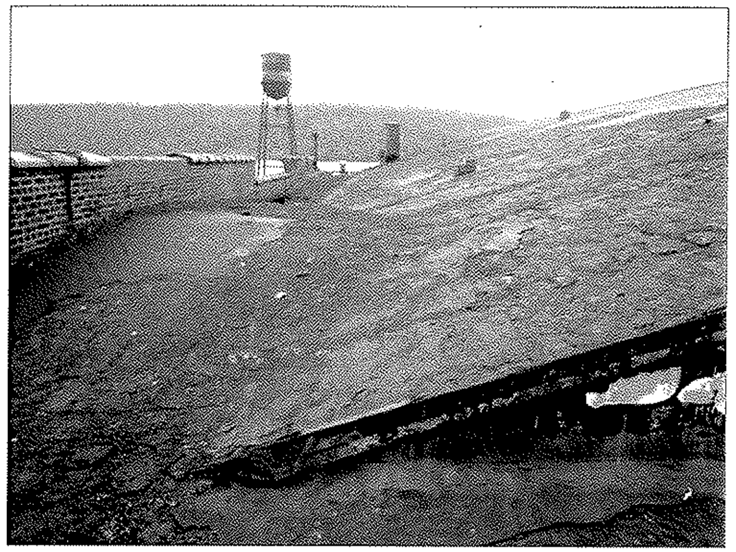
vox 212 736 3344 fax 212 736 4466

Date 8 Nov 2010

Scale 1-1/2" = 1'-0"

Job No. 1010

Dwg No. SK-3

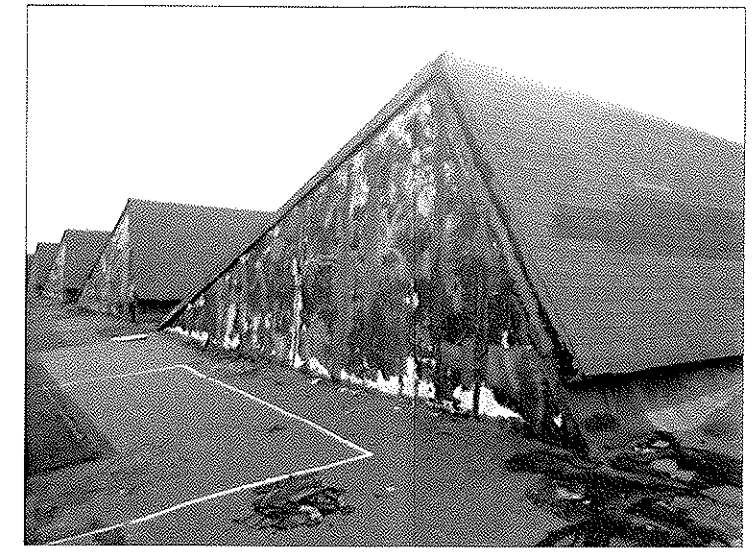


1 SOUTH PARAPET

SK-4

2 CRICKET
BETWEEN MONITORS

SK-4

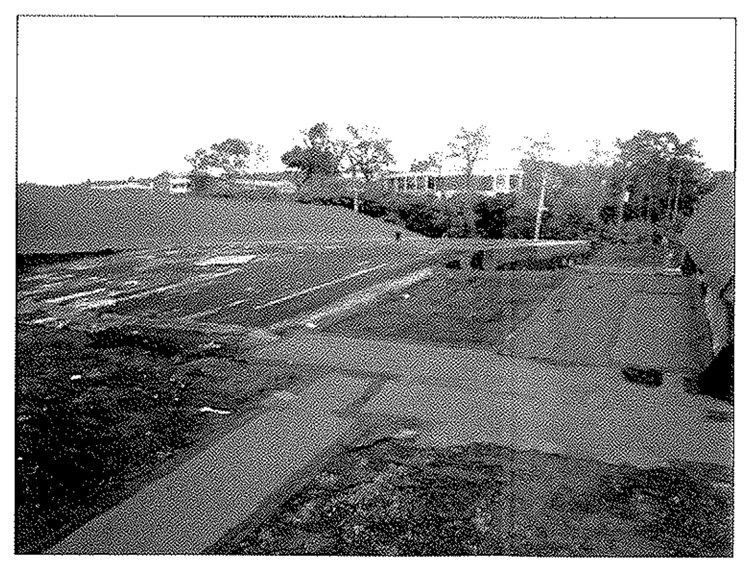


3 MONITOR SIDE WALLS
& SKYLIGHT COVERS

SK-4

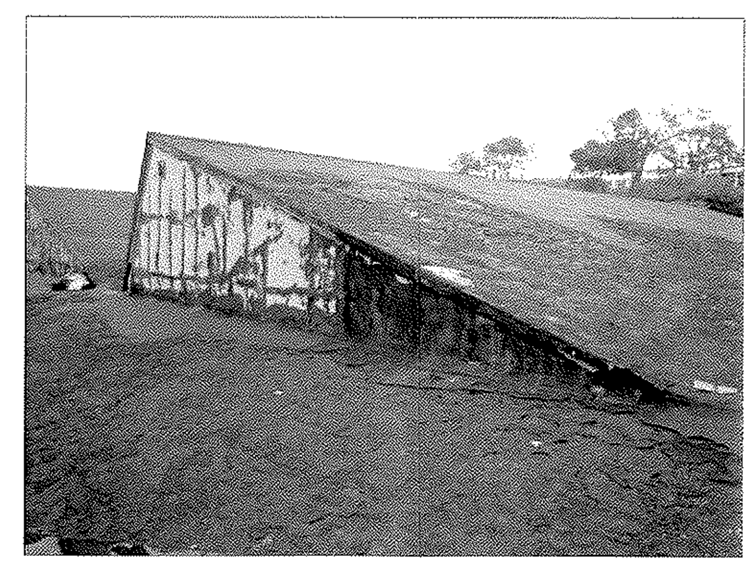
4 NORTH PARAPET

SK-4



5 AREA AT
FORMER MONITOR

SK-4



6 MONITOR SIDE WALLS
& ROOF MEMBRANE

SK-4

Title Roof Photos - Preliminary S.O.W.

Project Hastings on Hudson Building 52

James R. Gainfort AIA, Consulting Architects PC
121 W 27th St, Suite 803 New York, NY 10001-6207
vox 212 736 3344 fax 212 736 4466

Date	8 Nov 2010
Scale	N.T.S.
Job No.	1010
Dwg No.	SK-4

DWG name: I:\ACTIVE\2010\1010\Prelim-SOW\Temp-SK-1-Acad2000.dwg Saved: 08 Nov 2010, 6:00pm XREFs: x-24x36; x-sk11x17; x-sk-8x11; x-22x34; x-30x42; x-30x48; x-36x48; Ex-Roof-2010-10-29-1010; Ex-East-2010-11-01-1010 Printed by: sta-7 Layout: SK-4

APPENDIX F

Concrete Testing Results

Kemron

KEMRON
ENVIRONMENTAL SERVICES

1359-A Ellsworth Industrial Blvd • Atlanta, GA 30318 • TEL 404-636-0928 • FAX 404-636-7162

November 23, 2010

Sara Steele, P.E.
Robert Silman Associates
88 University Place
New York, NY 10003

Re: Letter Report
Hastings-on-Hudson Concrete Evaluation
KEMRON ATG Project #SE-0366-001

Dear Ms. Steele:

KEMRON is please to provide the attached report for the above reference property. The study consisted of performing Unconfined Compressive Strength (UCS) of the concrete by ASTM C42, petrographic evaluation by ASTM 856, and acid soluble chloride testing by ASTM 1152.

The attached report was prepared by Testing and Engineering Consulting Services, Inc. located in Lawrenceville, Georgia.

KEMRON Environmental Services, Inc. appreciates this opportunity to provide our services to Robert Silman Associates. If you have any questions, or require additional information, please contact us at (404) 601-6927.

Sincerely,

KEMRON Environmental Services, Inc.

Tommy A. Jordan, P.G.
Program Manager

M LE CH IN- F-CU T Y RECOR

ENVIRONMENTAL SERVICES

KEM N ENVIR NMENTAL SE VICES, INC.
1359-A Ellsworth Industrial Boulevard
ATLANTA, GEORGIA 30318
(404) 636-0928 FAX (404) 636-7162
WWW.KEMRON.COM

PROJECT NAME: Hastings-on-Hudson, NY

PROJECT #: H&A# 28612-248

TAT or DUE DATE: Standard

CONTACT: RSA:Sara Steele / H&A:Shawn Poff

PROJECT MANAGER: Tommy Jordan

PHONE#: 212.620.7970 / 860.989.9418 EXT.:

RECEIVING LAB: (404) 601 - 6908

PO#:

SAMPLED BY: PRINTED NAME: Shawn W. Poff (Haley & Aldrich)

SIGNATURE:

SAMPLE NUMBER	SAMPLE DESCRIPTION	SAMPLE DATE	Sample	Preservative	# / Size of Cont.	ANS 16.1 Modified 5 Day	Petrographic Analysis ASTM 856	Strength Testing ASTM C42	Chloride Analysis ASTM 1152					
C-5 (a)	Concrete slab core at F-2	10/2/2010		None	approx 4in x 8in		X							
C-5 (b)	Concrete slab core at F-2	10/4/2010		None	approx 4in x 8in			X						
C-6 (a)	Concrete slab core at F-4	10/2/2010		None	approx 4in x 8in			X						
C-6 (b)	Concrete slab core at F-4	10/2/2010		None	approx 4in x 8in			X						
C-7	Concrete slab core at F-1	10/2/2010		None	approx 4in x 8in			X						
C-1	Concrete roof core at R-1	10/9/2010		None	approx 4in x 4in			X						
C-2	Concrete roof core at R-2	10/9/2010		None	approx 4in x 4in			X						
C-3	Concrete roof core at R-5	10/8/2010		None	approx 4in x 4in			X						
C-4	Concrete roof core at R-7	10/2/2010		None	approx 4in x 4in		X							

SAMPLES RELINQUISHED BY:	DATE/TIME	SAMPLES ACCEPTED BY:	DATE/TIME
Shawn Poff [signature]	10/18/2010 1630	[signature]	10/19/10 11:15
[signature]			

COMMENTS: Samples contain low levels of PCBs - wear disposable gloves and safety glasses when handling

TEC Services
Testing • Engineering • Consulting

November 19, 2010

Mr. Tommy Jordan
Kemron Environmental Services, Inc.
1359A Ellsworth Industrial Boulevard
Atlanta, Georgia 30317

Phone: 404-601-6908
Fax: 404-636-7162
e-mail: tjordan@kemron.com

Subject: **Report of Concrete Materials Testing**
Concrete Floor and Roof Slab Cores
Hastings-on-Hudson, New York
TEC Services Project No. TEC 10-0808.02

Dear Mr. Jordan:

Testing, Engineering and Consulting Services, Inc. (TEC Services), is pleased to submit this report of our concrete materials testing. Our testing was performed on concrete cores obtained from a facility located in Hastings-on-Hudson, New York. The purpose of our services was to perform materials testing to determine the general quality of the concrete. Our report includes background information, test results, petrographic observations and conclusions. Our services were performed in accordance with the terms and conditions of our Service Agreement dated May 29, 2009.

BACKGROUND INFORMATION

The following background information was obtained from Kemron Environmental Services, Inc. (Kemron) representative Mr. Tommy Jordan. Please contact us if this information is incorrect so that we may revise our report as deemed necessary.

The provided concrete cores were obtained from a facility in Hastings-on-Hudson, New York. The facility was previously used to produce cable and wire and is approximately 90 years old. During World War II the facility produced cables coated in pcbs which were used on Navy vessels. Cores were obtained by others from the concrete roof deck and floor slab. The top surface of the roof slab is covered by a roof membrane. The floor slab was placed on grade, but it is unknown at this time if the slab is structural or was to be supported by the grade. For the past few years the facility has been vacant and has been exposed to the environment via roof leaks and windows missing glass.

Testing performed by Kemron indicates that the concrete contains pcbs. The pcbs are likely a result of the previous manufacturing practices of the cable factory. These pcbs require slab remediation. Prior to performing the slab remediation the ultimate client requested that Kemron determine the quality of the concrete. Kemron provided TEC Services with 4 cores from the roof slab and 5 cores from the floor slab of the facility. Kemron requested TEC Services test the cores to determine the compressive strength, chloride content and quality.

COMPRESSIVE STRENGTH TESTING

Compressive strength testing was performed on Cores 1, 2, 5B, 6A, 6B and 7 in accordance with ASTM C42, *Standard Test Method for Obtaining and Testing Drilled Cores and Sawn Beams of Concrete*. Core 3 fractured into pieces and did not remain intact when we sawcut the ends. As a result we were unable to test Core 3. However, the fracturing of the sample from sawcutting indicates a low compressive strength. Cores 1 and 2 were from the roof slab. Cores 5B, 6A, 6B and 7 were from the floor slab. The results of our testing indicate that the 90 year old concrete from the roof slab has an average compressive strength of 2380 psi with a range from 1640 to 3120 psi. The floor slab has an average compressive strength of 5380 psi with a range from 4970 to 5820 psi. The results of our compressive strength testing are reported within Table 1 at the end of our report.

ACID SOLUBLE CHLORIDE TESTING

Acid-Soluble chloride testing was performed on (9) powder samples from portions of the provided cores in accordance with ASTM 1152, *Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete*. The testing was performed by Wyoming Analytical Laboratories. The chloride test samples from Cores 1, 2, 3, 5B, 6A, 6B and 7 were obtained by crushing portions of the cores into a powder after the compressive strength testing was performed. Both ends of the cores were sawcut and the ends were not included in the powder samples. Cores 4 and 5A were not tested in compression, but the ends were also sawcut. The middle portion of the cores was then sawn in half perpendicular to the top surface. One half of this middle portion from each core was crushed into the testable powder sample. In summary the tested powder samples represent a blend of the middle portion of each core.

The results of our chloride testing indicate chloride contents in the roof slab cores which vary from 0.004 to 0.011% per mass of concrete. The chloride contents in the roof slab cores are insignificant. The chloride contents of floor slab cores 5A, 5B, 6A, 6B and 7 were 0.142, 0.122, 0.027, 0.134 and 0.123% per mass of concrete respectively. The chloride contents in 4 of the 5 floor slab cores are excessively high. Chloride concentrations of 0.050% and higher per mass of concrete significantly increase the potential for reinforcing steel corrosion. The results of our testing are provided within Table 2 at the end of our report.

PETROGRAPHIC EXAMINATIONS

Core 4 from the roof slab and Core 5A from the floor slab were selected for petrographic examination by Kemron. The cores had a diameter of 3¾". The ends of both cores were inadvertently sawcut parallel to the top surface. Approximately ¼ to ½" was removed from each end of Core 4. These thin portions were saved, but were too thin to cut and polish. Approximately ⅛ to ¼" was removed from the top of Core 5A. This sawcut was so close to the top surface that it simply shaved the top surface not yielding a thin removed portion. The sawcut at the other end of Core 5A removed the bottom 2" of the core. This portion was cut and polished. The examined polished planes were obtained by sawcutting perpendicular to the top surface of the slabs. One half of each core was ground and polished in preparation for petrographic examination (Photos 1 – 2). The prepared polished plane sections were examined in accordance with the applicable sections of ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete*, using a digital microscope at magnifications from 20X to 200X. Our significant petrographic observations are provided below. Our conclusions for each core are based on the provided background information, petrographic observations and our experience with similar evaluations.

Core 4: *(Roof Slab)*

General: The side of the core was labeled by others in its as received condition. The labeling indicated that the top of the core was at the end closest to the wire reinforcing inclusion. This appears to be incorrect. We observed a roof membrane on the top of one sawn end portion (Photo 3). The roof membrane indicates the top surface of the core. Beneath the roof membrane the paste of the concrete is a light tan (Photo 4). This light tan paste could be matched up with paste at the end of the core labeled as the bottom. This did not match the paste labeled as the top. As a result the sample appears labeled incorrectly. Also, it is not often that wire reinforcing is located so close to the top surface. It is typically located near the bottom of the slab. Our report will note that the end closest to the wire reinforcing is the bottom.

Coarse Agg: The coarse aggregate was a lightweight aggregate comprised of boiler slag and bottom ash (Photo 5). Boiler slag and bottom ash were commonly used to produce lightweight concrete prior to the development of the modern lightweight shale and clay aggregates. The maximum aggregate size typically appeared to be approximately ¾", but we observed a 2" lightweight aggregate at the side of one core. The surfaces of the coarse aggregate varied from angular to subrounded. The coarse aggregate particles were typically equidimensional in shape. The coarse aggregate was evenly distributed in the cores with no indications of segregation. We observed some paste with the voids of the lightweight aggregate (Photo 6). This is an indication that the lightweight aggregate was not adequately saturated prior to mixing. We did not observe indications of deleterious aggregate reactions.

Fine Agg: The fine aggregates appeared to be a natural sand comprised primarily white and tan colored quartz. The maximum fine aggregate size was approximately ¼ inch. The fine aggregate was evenly distributed within the core. The surfaces of the fine aggregate varied from subangular to subrounded. The fine aggregate particles were equidimensional in shape. We did not observe indications of deleterious aggregate reactions.

Paste: The matrix (hardened cement paste) of the cores was light gray in coloration (Photo 7). The overall coloration appeared relatively uniform with the exception of the previously noted tan paste beneath the roof membrane. The light tan paste is likely a result of carbonation. A portion of the sample which was not polished or tested for chlorides was freshly sawn and the sawn surface was sprayed with phenolphthalein. The phenolphthalein test indicates that the full depth of the core is carbonated. Carbonation of concrete occurs when carbon dioxide in the air reacts with the calcium hydroxide in the concrete to produce calcium carbonates. This reaction results in lowering the pH (alkalinity) of the concrete. The high alkalinity of concrete provides passive protection for the reinforcing steel from corrosion. Without the high alkalinity of the concrete the reinforcing steel will corrode readily in the presence of oxygen and moisture.

Core 4: *(Roof Slab) continued...*

Paste: The matrix does not appear to contain supplemental cementitious materials such as fly ash or slag. We observed some occasional particles resembling fly ash cenospheres, but these were likely from the bottom ash lightweight aggregate (Photo 8). The paste of the concrete was very soft as it could be easily scratched and gouged with a steel point. The paste also eroded significantly during sample preparation (Photo 9 & 10). We estimate the water to cement ratio to be in the range of 0.60 to 0.65. The porosity of the paste varied, but generally appeared to be high. The soft eroded paste appeared to simply be a result of a high water cement ratio and not some form of deterioration such as aggregate reactions or chemical attack.

Air Voids: The concrete does not appear to contain entrained air. We estimate the total air content of the concrete to be in the range of 1 to 2%. We observed numerous voids at the periphery of the aggregate particles, but it is difficult to determine if these voids are a result of the eroded soft paste or bleed water channels and trapped voids. Bleed water channels and trapped voids beneath the aggregate typically indicate an excess water content.

Surfaces: As mentioned previously the ends of the core were sawn, but both thin portions were saved. A black roof membrane material was observed on the top surface of the core. The thickness of the membrane varied from approximately 1/16" to 3/16" thick. The bottom surface of the core was difficult to interpret. It did not appear to be wood formed (Photo 11). The bottom surface also appeared to have been painted 2 or 3 different colors.

Void Deposits: We observed occasional secondary deposits within the voids of the concrete. The deposits appeared to be secondary ettringite formations (Photo 12). These formations are not detrimental to the concrete and are common in concrete subjected to wetting and drying cycles. This is an indication that the roof membrane may not have been in place for the life of the structure.

Reinforcing: We observed a piece of wire reinforcing near the bottom of the core. The diameter of the wire was approximately 3/16". We estimate the bottom cover to be approximately ½ to ¾". The wire reinforcement appeared to be corroding. The corrosion of the wire reinforcement appeared relatively significant (Photo 13). The corrosion bleed into the paste, but it had not produced cracking in the concrete.

Cracking: We did not observe significant cracking within the paste of the sample.

Conclusions: The concrete within the core from the roof slab is of poor quality and may have exceeded its useful life as a structural slab. Phenolphthalein staining indicates the concrete has carbonated through the full section. As a result the wire mesh has corroded and will continue to corrode in the presence of oxygen and moisture. Additionally, our compressive strength testing indicates low strengths (1640 psi).

Core 5A: *(Floor Slab)*

General: The direction to the top surface of the core was labeled correctly on the side by others. As mentioned previously the removed bottom portion was also polished. The portion missing from the top surface of the core was approximately ⅛ to ¼" thick.

Coarse Agg: The coarse aggregate appeared to be a crushed stone comprised primarily of limestone. The limestone did not react excessively with dilute hydrochloric acid, but it did dull the surface of the aggregate. This indicates the limestone is likely dolomitic. The maximum aggregate size was approximately ¾". The surfaces of the coarse aggregate varied from subangular to subrounded. The coarse aggregate particles were typically equidimensional in shape with a few elongated particles. The coarse aggregate was evenly distributed in the cores with no indications of segregation. We did not observe indications of deleterious aggregate reactions.

Fine Agg: The fine aggregates appeared to be a natural sand comprised primarily white and tan quartz. The sand appeared similar to the sand within the roof slab. The maximum fine aggregate size was approximately ¼ inch. The fine aggregate was evenly distributed within the cores. The surfaces of the fine aggregate varied from subangular to subrounded. The fine aggregate particles were equidimensional in shape. We did not observe indications of deleterious aggregate reactions.

Paste: The matrix (hardened cement paste) of the cores varied significantly in coloration. The majority of the middle portion of the core was gray, but the gray coloration was not uniform. Zones of relatively darker and lighter gray paste were observed (Photo 14). This is a result of not thoroughly mixing the concrete and is not uncommon in concrete from this era. The non-uniform paste coloration however, does not appear to have significantly affected the strength of the concrete (5380 psi AVG). Additionally, the upper ½ to ¾" of the paste was light tan in coloration (Photo 15). This is a result of carbonation. If we add the approximate ¼" missing from the top surface the depth of carbonation is approximately ¾ to 1" deep. This amount of carbonation is excessive and is likely a result of the manufacturing process of the factory. Utilizing phenolphthalein staining we confirmed that the carbonation ended at the transitions from tan to gray paste. We also observed isolated light tan zones within the middle of the core and near the bottom surface. The isolated zone of carbonation within the middle of the core appears to have been a result of interconnected voids providing a pathway for the carbon dioxide beneath the slab (Photo 16).

The matrix does not appear to contain supplemental cementitious materials such as fly ash or slag (Photos 17 & 18). The hardness of the paste varied with the coloration of the paste. In general it appeared relatively hard. We estimate the water to cement ratio to be in the range of 0.45 to 0.50. The porosity of the paste varied, but generally appeared to be relatively high. We did not observe indications of chemical attack.

<u>Core 5A</u>: *(Floor Slab) continued...*

Air Voids: The concrete does not appear to contain entrained air (Photo 19). We estimate the total air content of the concrete to be in the range of 1 to 2%. We observed some occasional trapped voids and bleed water channels, but the concrete did not appear to have a high water content. Large voids, which appear a result of poor consolidation, were observed near the bottom at the unpolished side of the core. These voids were more excessive at the bottom of the other floor slab cores around the reinforcing steel (Photo 20).

Reinforcing: We observed a steel reinforcing bar at the side of the core with approximately 1¼" bottom surface cover. ACI recommends 3" of cover for concrete placed against a subgrade. The rebar was not included in the polished section. The reinforcing bar appeared to be a #6 bar and showed signs of significant corrosion. The corrosion bleed into the nearby paste, but had not produced cracking in the concrete. The bottom portion of the other floor slab cores also contained reinforcing with excessive corrosion. The corrosion in these cores has progressed to the point of producing exfoliation (corrosive layering) of the steel (Photo 21). The excessive corrosion in the other cores is a result of the reinforcing not being embedded within the concrete. The diameter of the bars with excessive corrosion is approximately 0.6". The diameter of a clean bar with no corrosion was measured to be 0.75" (Photo 22). This indicates either significant section loss in the corroded bars or different bar sizes.

Surfaces: The top surface of polished core was sawn, but we observed the remaining top portion of another floor slab core. Indications of minimal surface erosion were observed, but the top surface generally appeared to be in good condition considering the age and previous use (Photo 23). This top portion from another slab core was cut perpendicular to the top surface and polished (Photo 24). The portion was approximately ½" thick. We did not observe indications of freezing damage, cracking or other detrimental microstructural features. The bottom surface appeared to have been placed on a stone base.

Cracking: We observed a few vertical cracks beneath the top surface of the core (Photo 25 & 26). These cracks likely extend from the top surface. The cracks were not prevalent across the top surface and do not appear to be significantly detrimental.

Conclusions: The paste of the concrete within the floor slab cores appears to be of relatively good quality. We did not observe indications of significant deterioration excluding the corroding reinforcing steel. The corroding reinforcing steel is likely a result of the high chloride contents, carbonation and insufficient embedment. The slab will likely remain effective if it is designed as a slab on grade. However, if the slab was to be structural, as the large reinforcing bars indicate, it is not adequate because of the reinforcing steel is not well embedded and corroded.

Closing

Testing, Engineering and Consulting Services, Inc. appreciates the opportunity to provide our professional services for this important project. If you have any questions regarding this report, or if we can be of further assistance please contact us at 770-995-8000.

Sincerely,

TESTING, ENGINEERING AND CONSULTING SERVICES, INC.

Brian J. Wolfe
Project Engineer

Robert S. Jenkins, P.E.
Senior Concrete Petrographer

Attachments: Photo Pages (Photos 1 – 26)
Table 1: Results of Compressive Strength Testing of Concrete Cores C42
Table 2: Results of Acid-Soluble Chloride Testing C1152

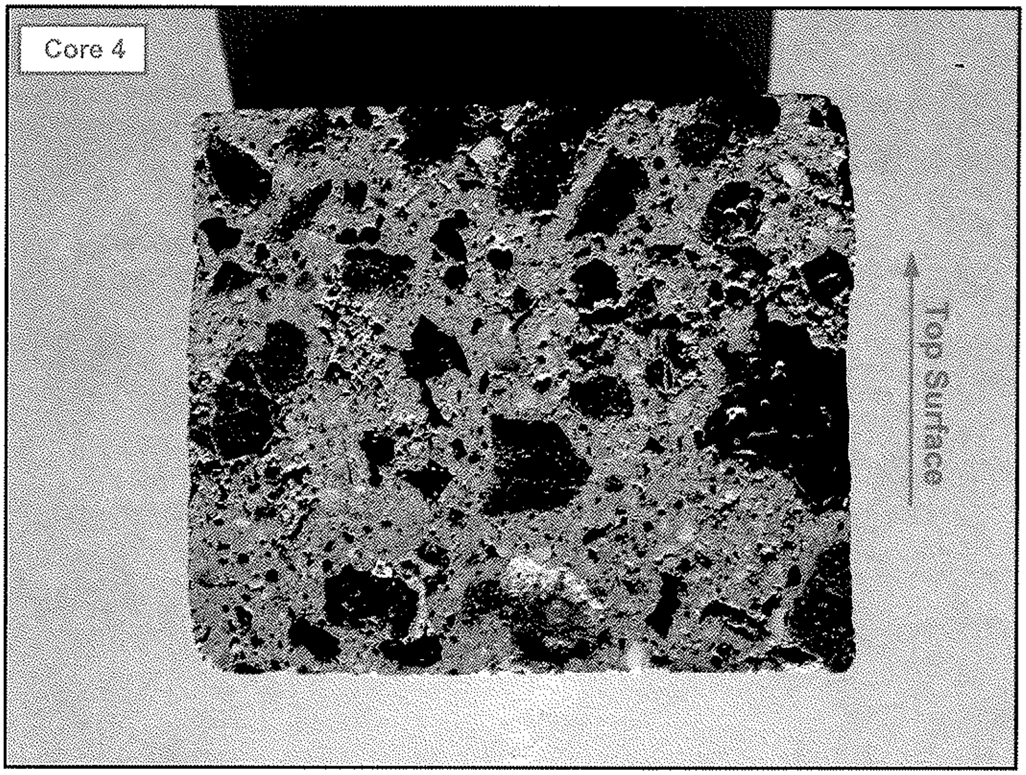


Photo 1. Overall of polished section of Core 4 in natural light.

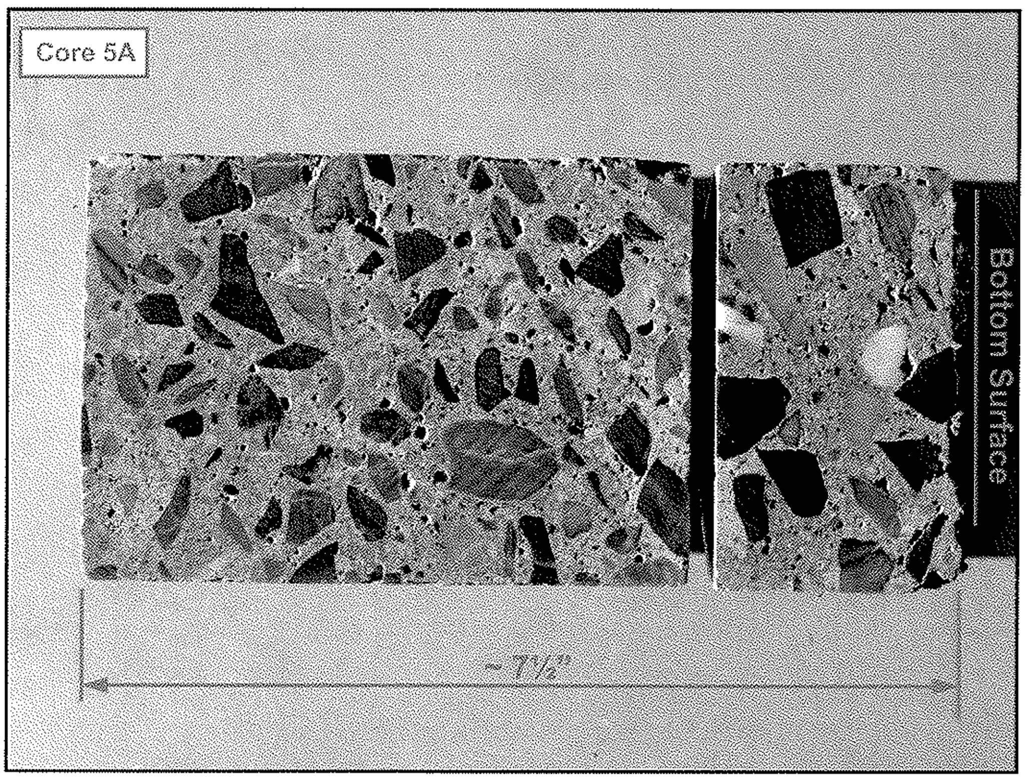


Photo 2. Overall of polished section of Core 5A in natural light.

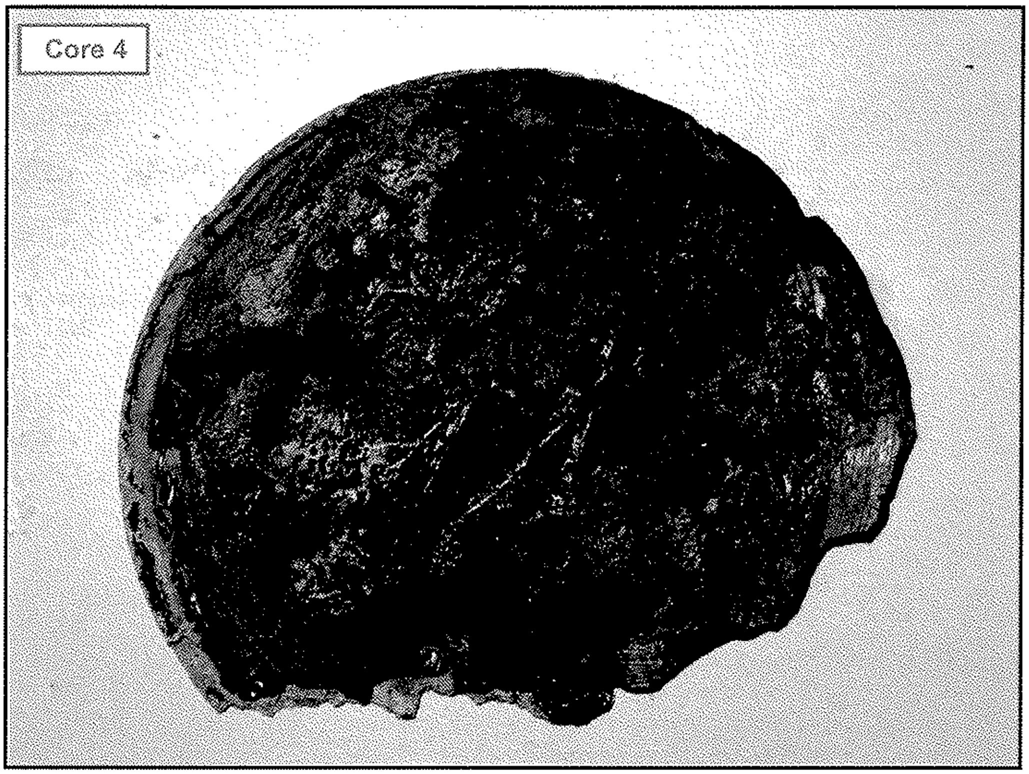


Photo 3. Roof membrane observed on top surface of Core 4.

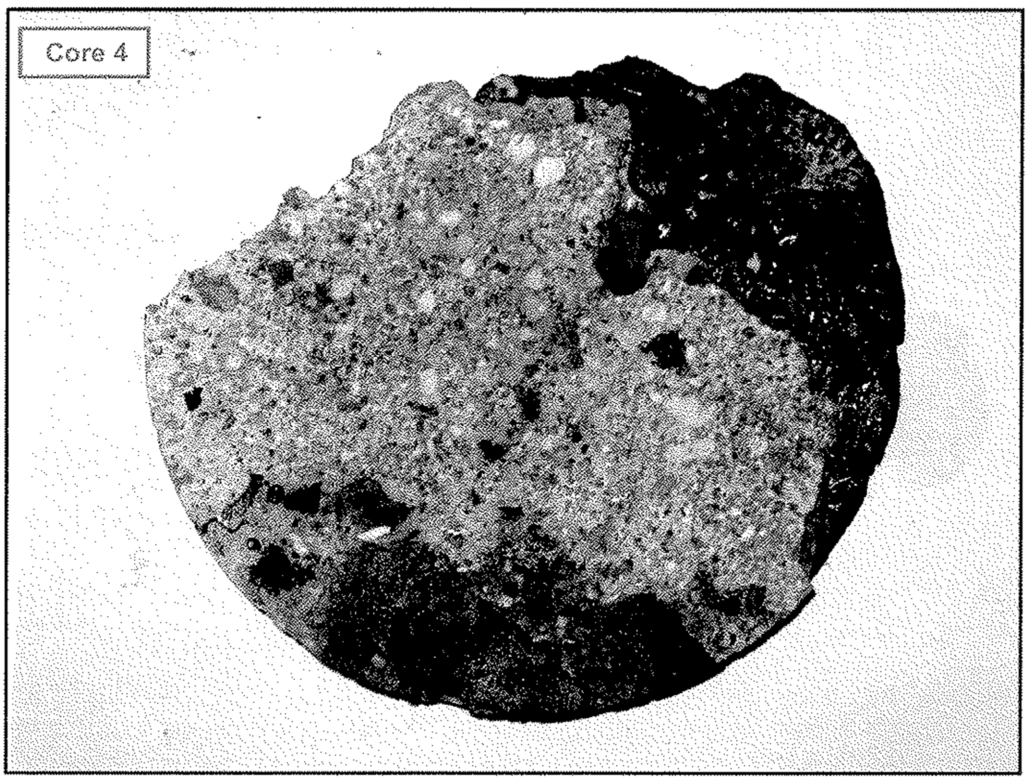


Photo 4. Light tan paste observed at underside of roof membrane.

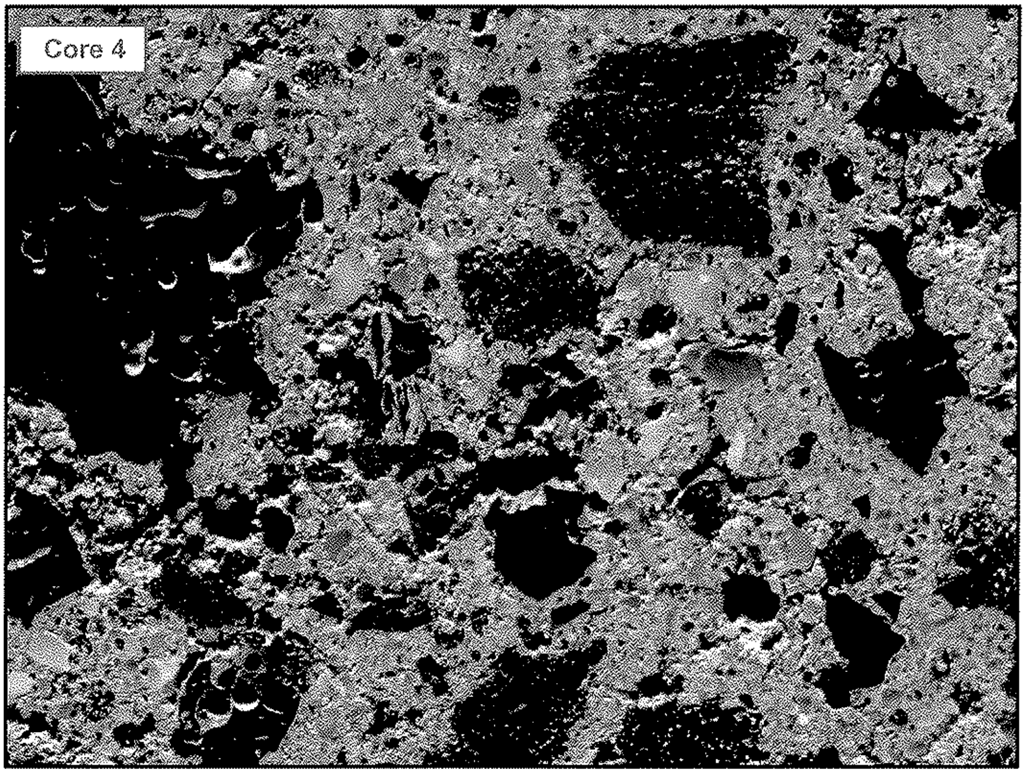
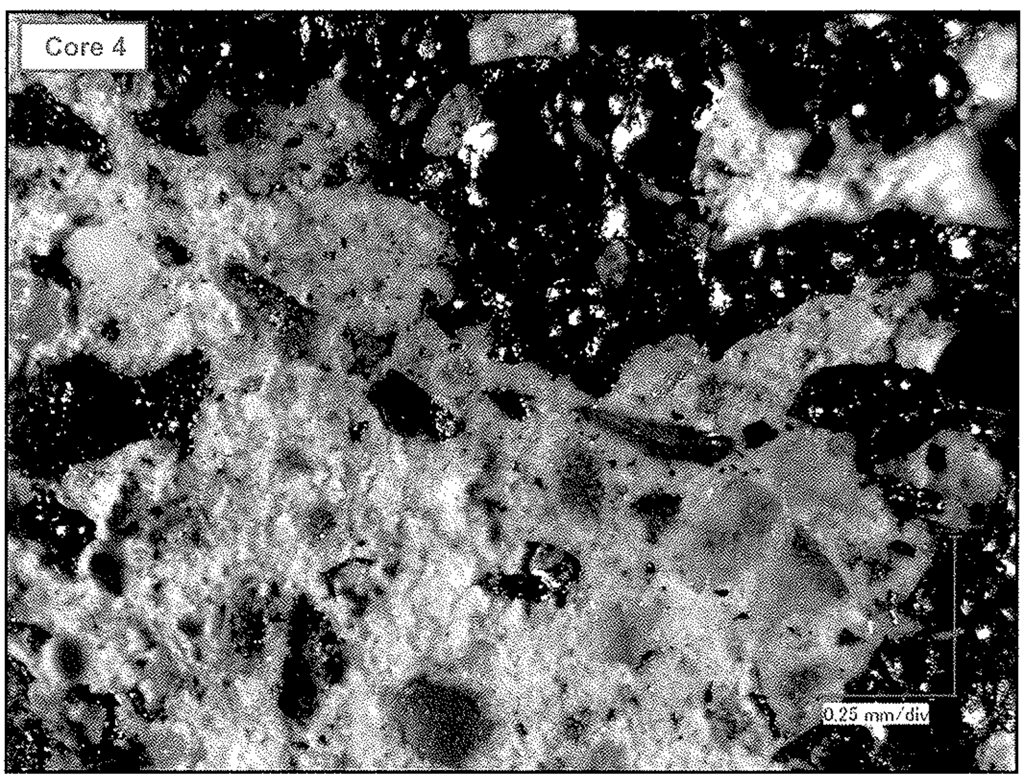
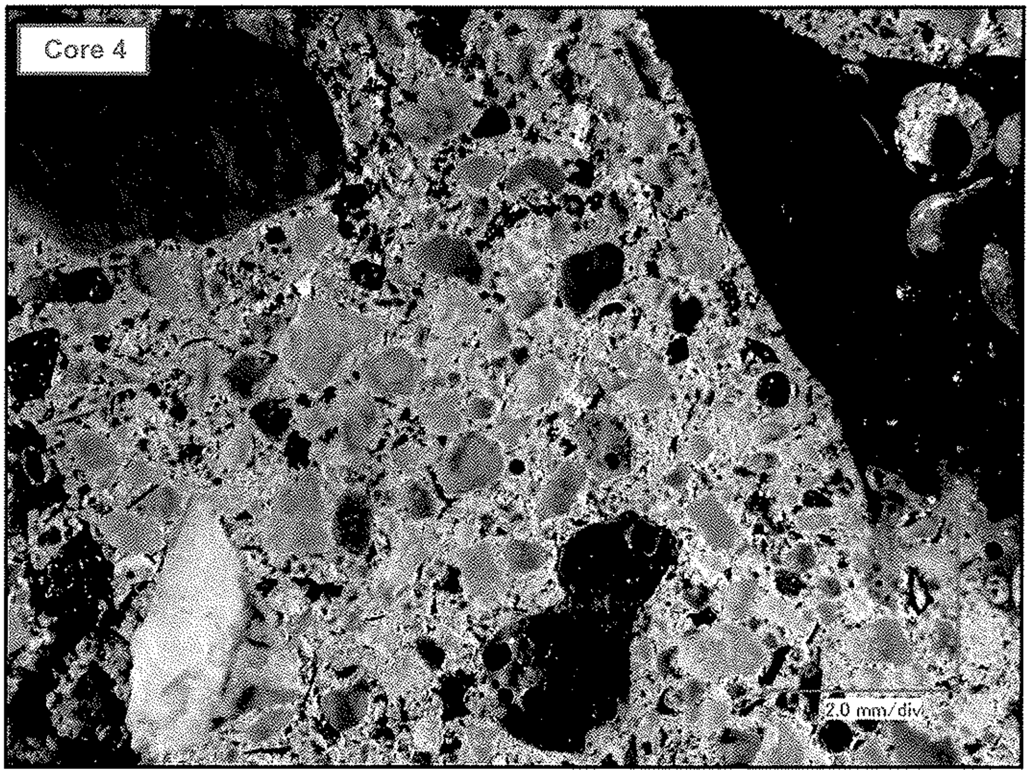


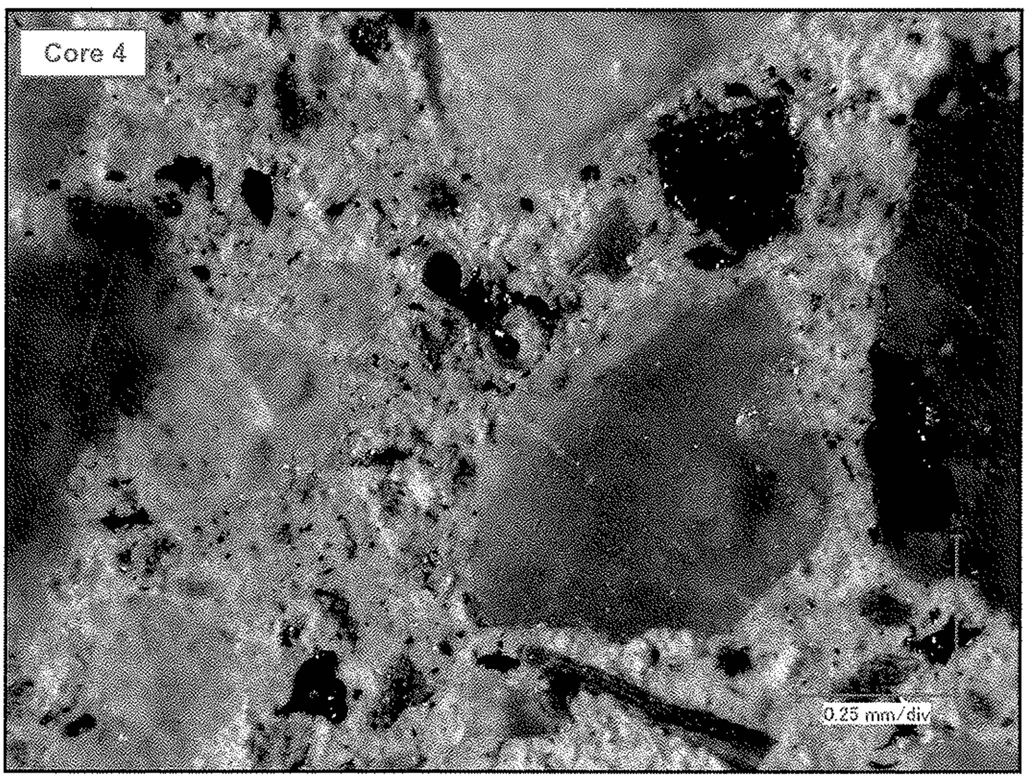
Photo 5. Boiler slag and bottom ash lightweight aggregate observed in Core 4.



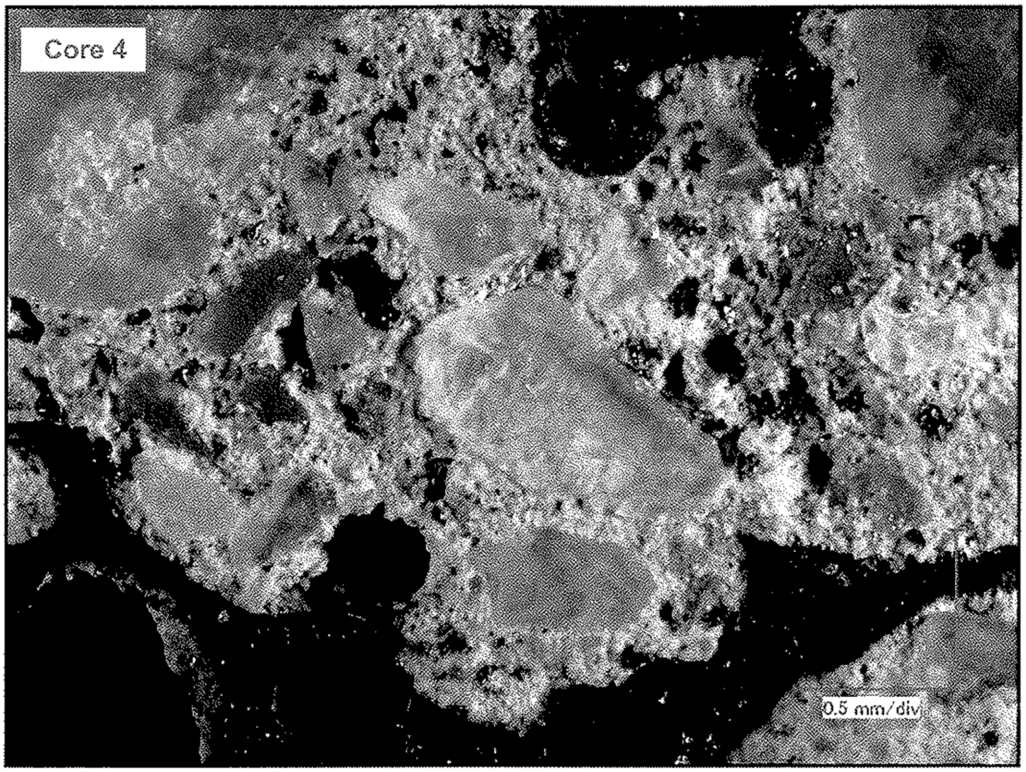
Photomicrograph 6. Paste observed within lightweight aggregate.



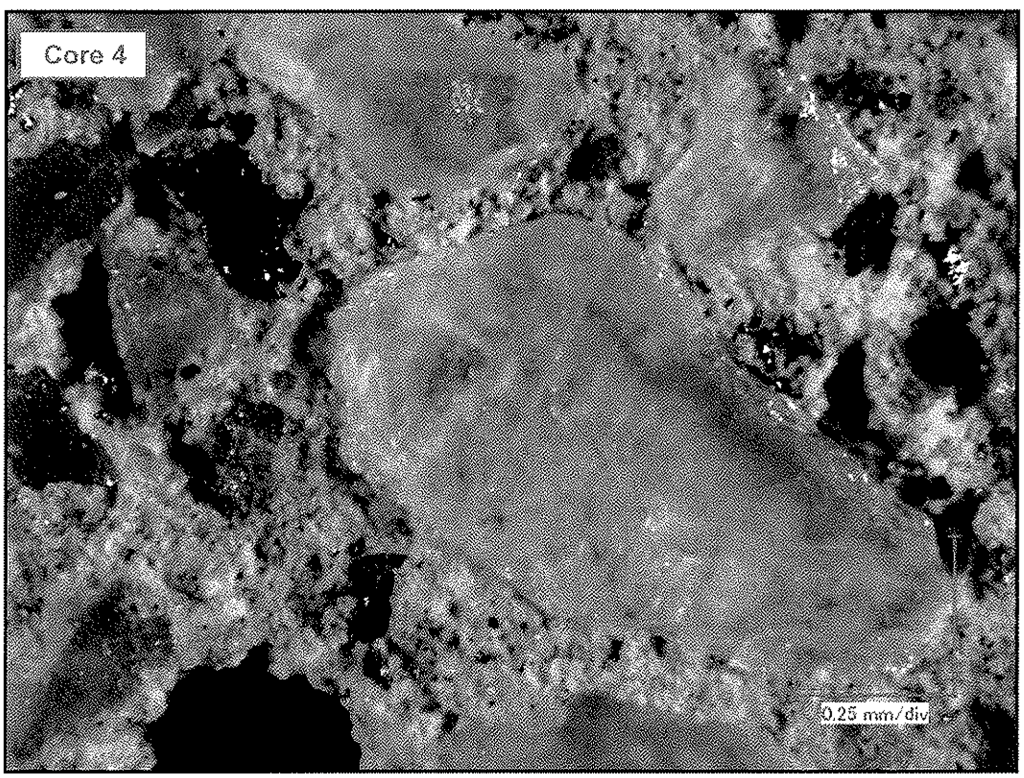
Photomicrograph 7. Light gray paste observed in Core 4.



Photomicrograph 8. Fly ash cenospheres observed in paste.



Photomicrograph 9. Eroded paste observed in Core 4.



Photomicrograph 10. Eroded paste observed in Core 4.

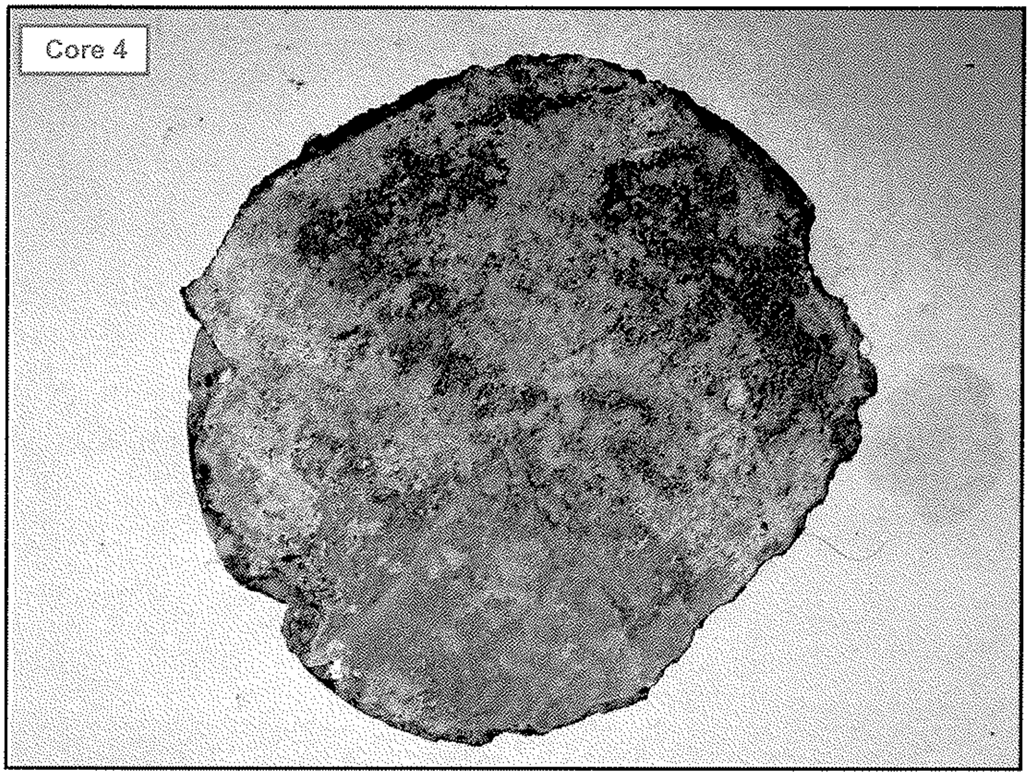
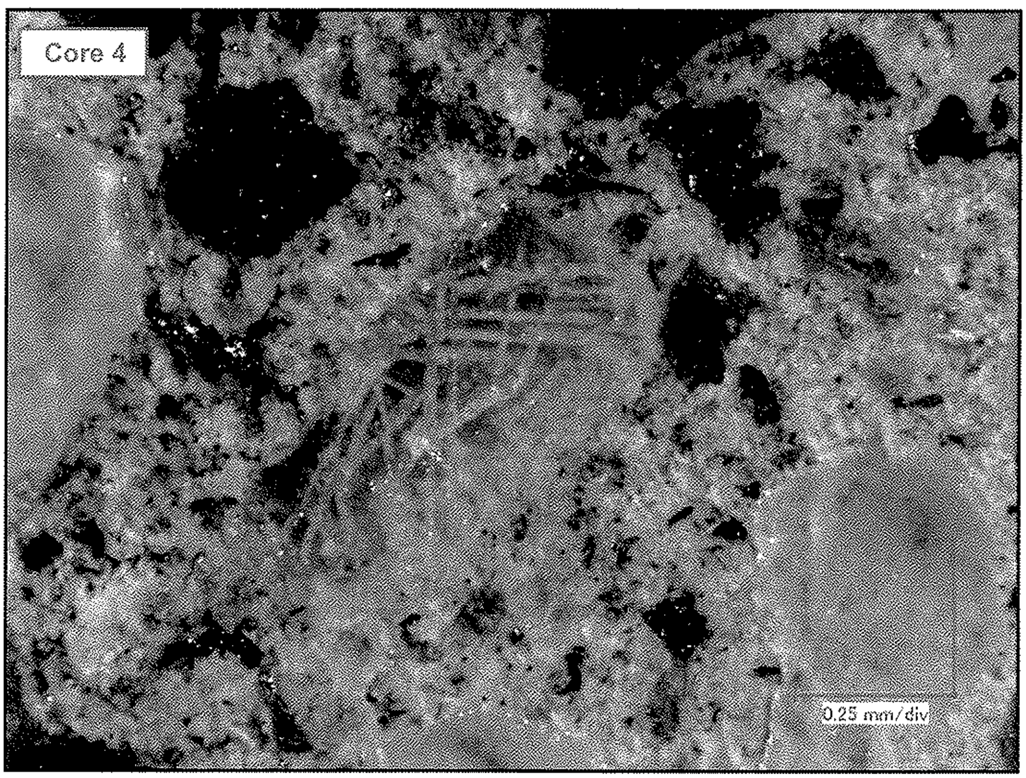
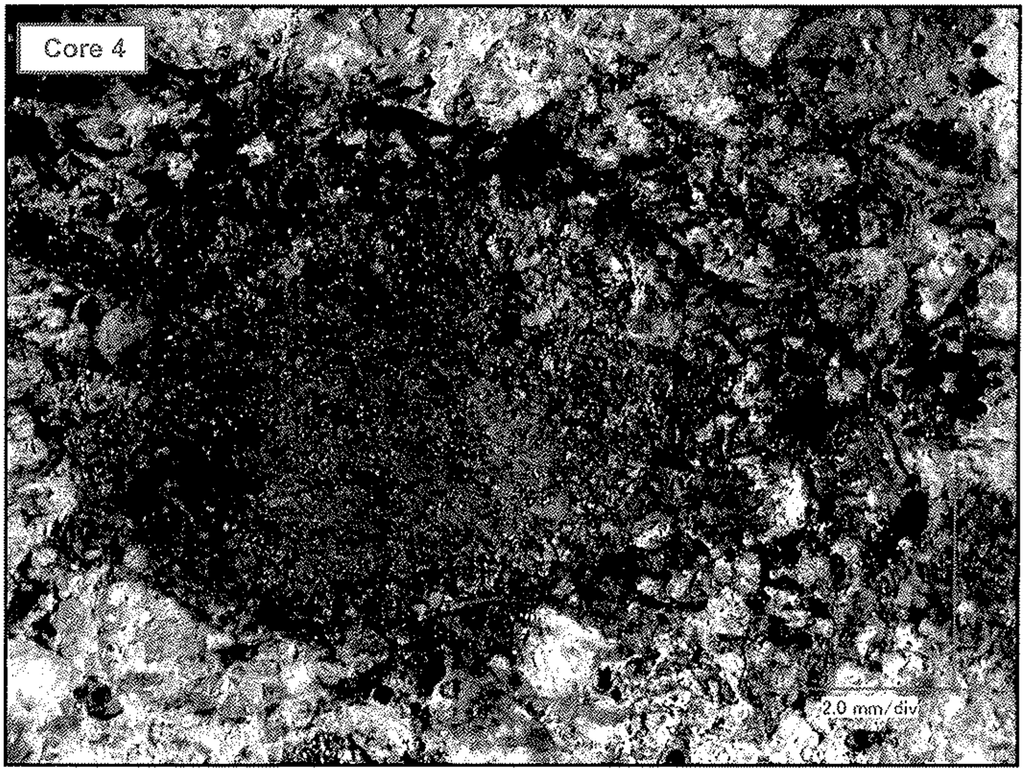


Photo 11. Bottom surface of Core 4.



Photomicrograph 12. Secondary deposits observed within void.



Photomicrograph 13. Close up of corroded wire reinforcing. Corrosion observed in nearby paste.

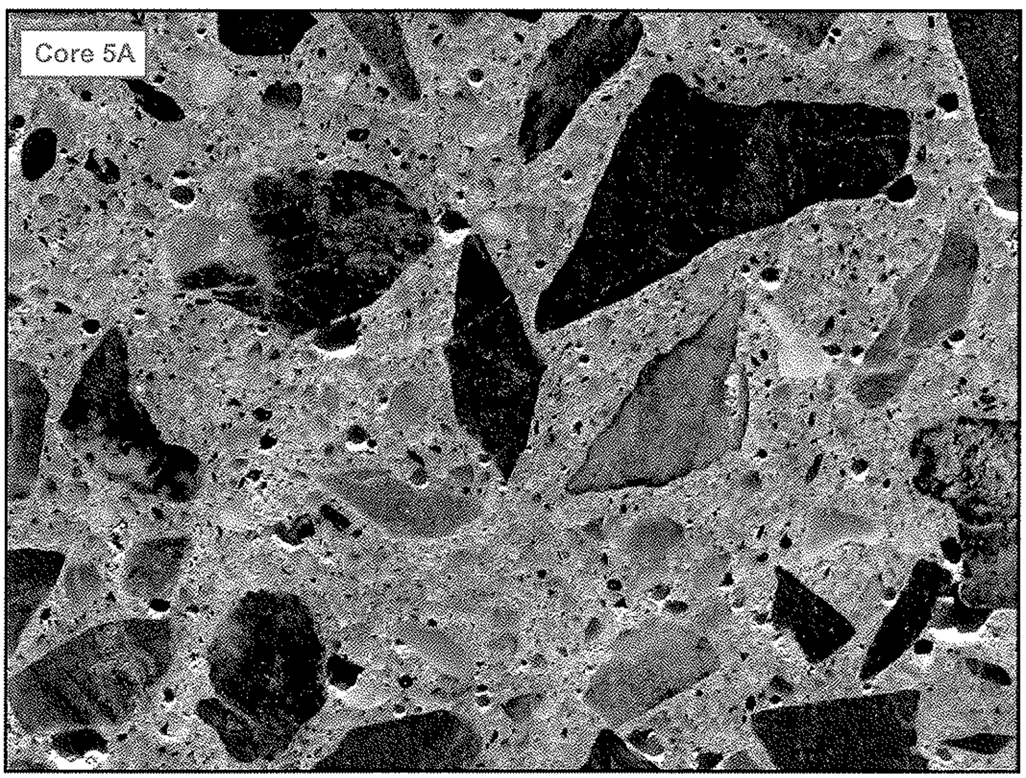


Photo 14. Uneven distribution of paste color observed.

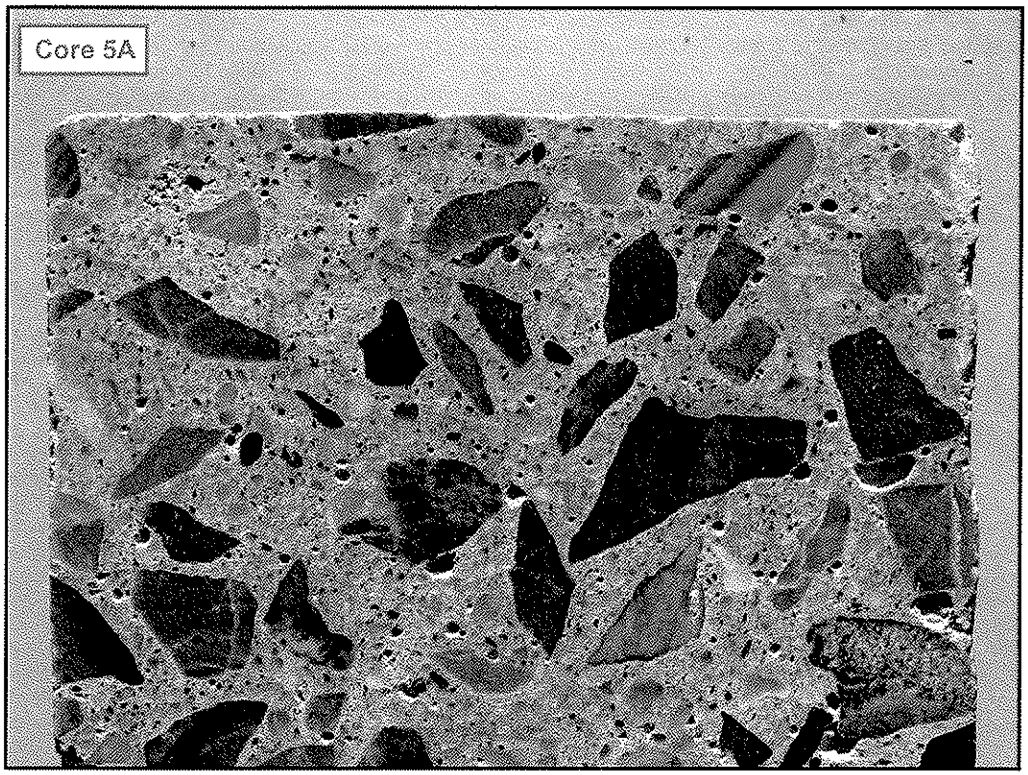


Photo 15. Depth of carbonation approximately ¾ to 1" deep.

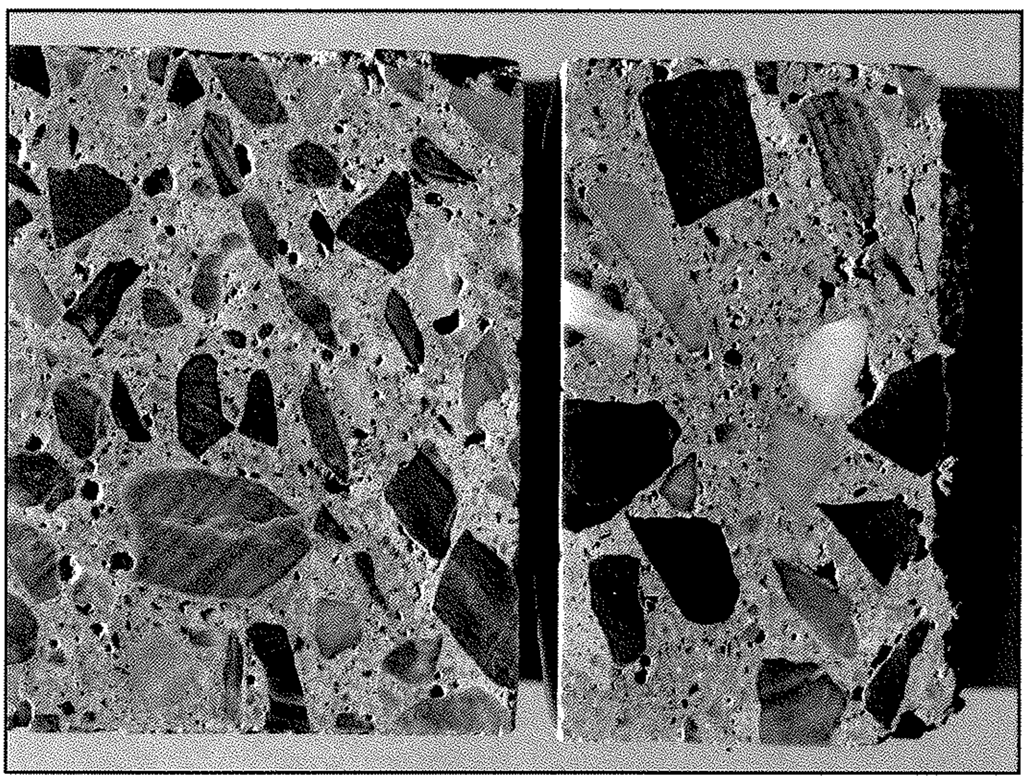
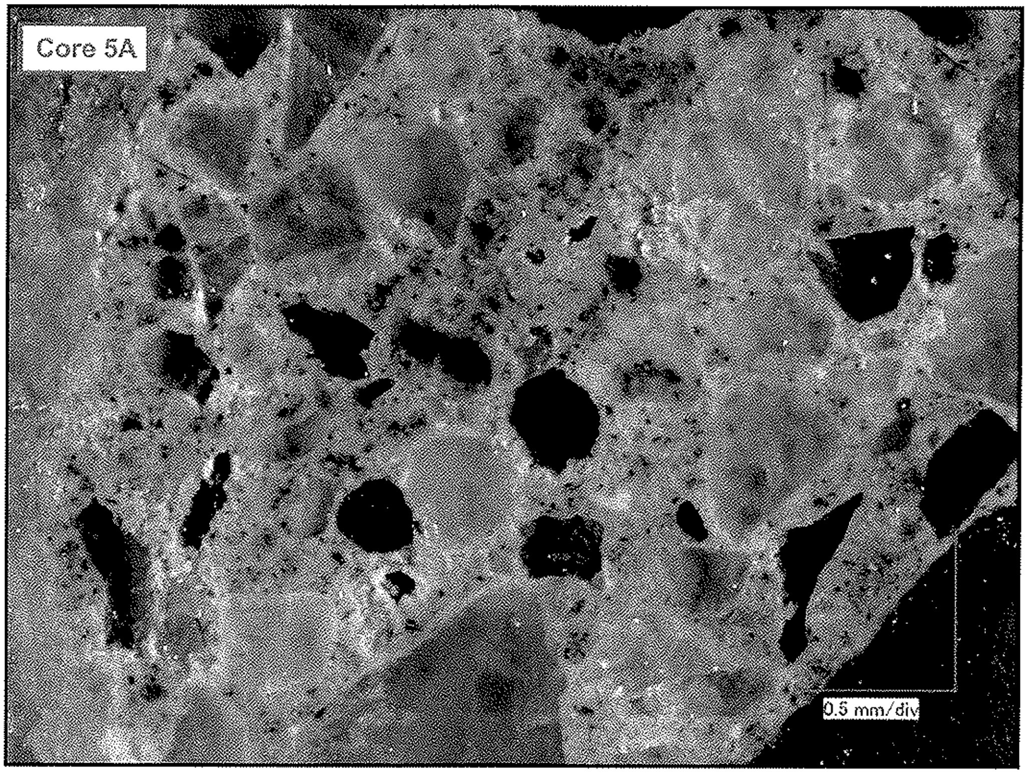
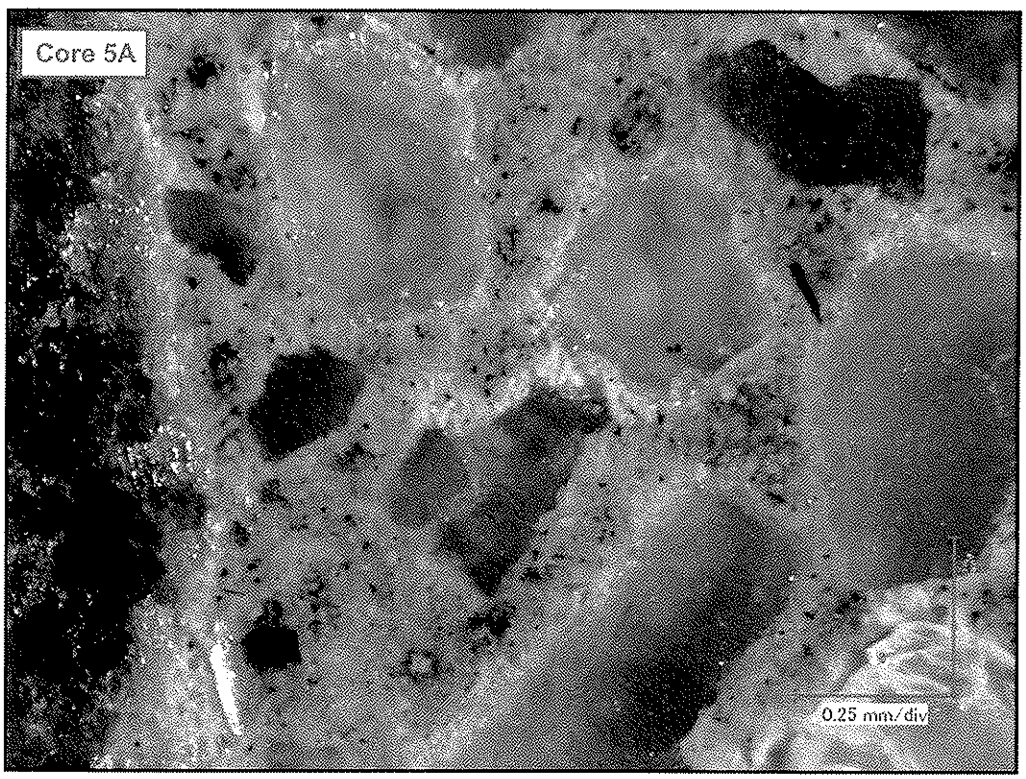


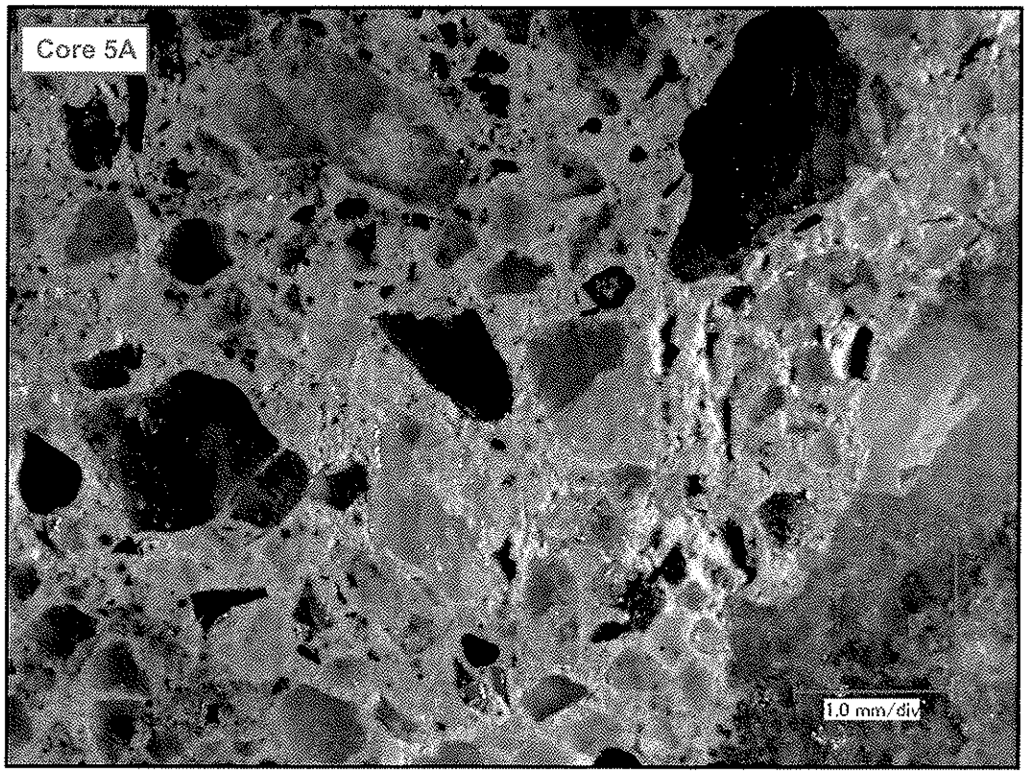
Photo 16. Isolated zone of carbonated paste observed within Core 5A.



Photomicrograph 17. Paste of floor slab Core 5A.



Photomicrograph 18. Paste of Floor slab Core 5A.



Photomicrograph 19. No entrained air voids observed in Core 5A.

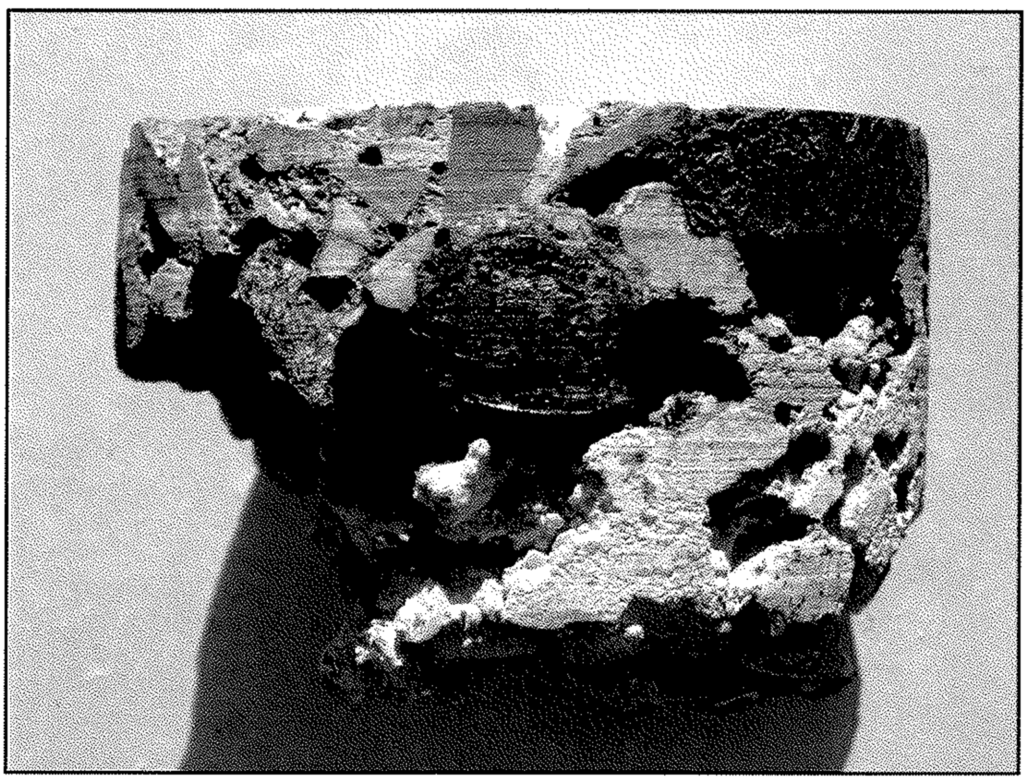


Photo 20. Poor consolidation of concrete observed around reinforcing steel bars.

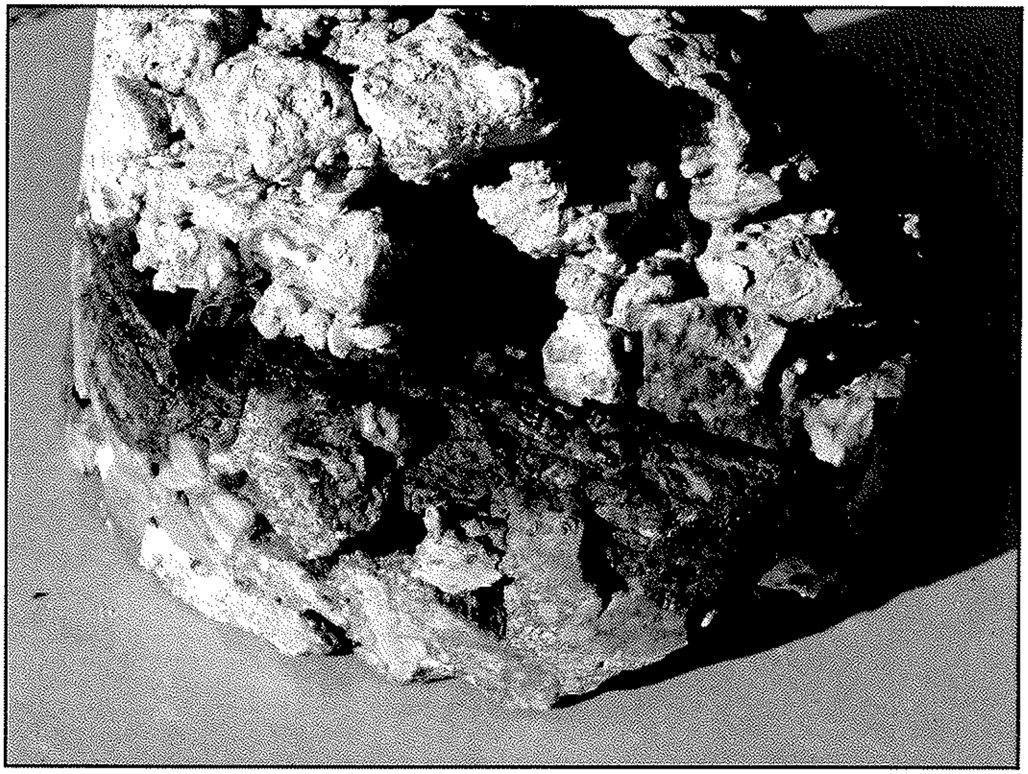


Photo 21. Excessively corroded rebar observed at bottom of core.

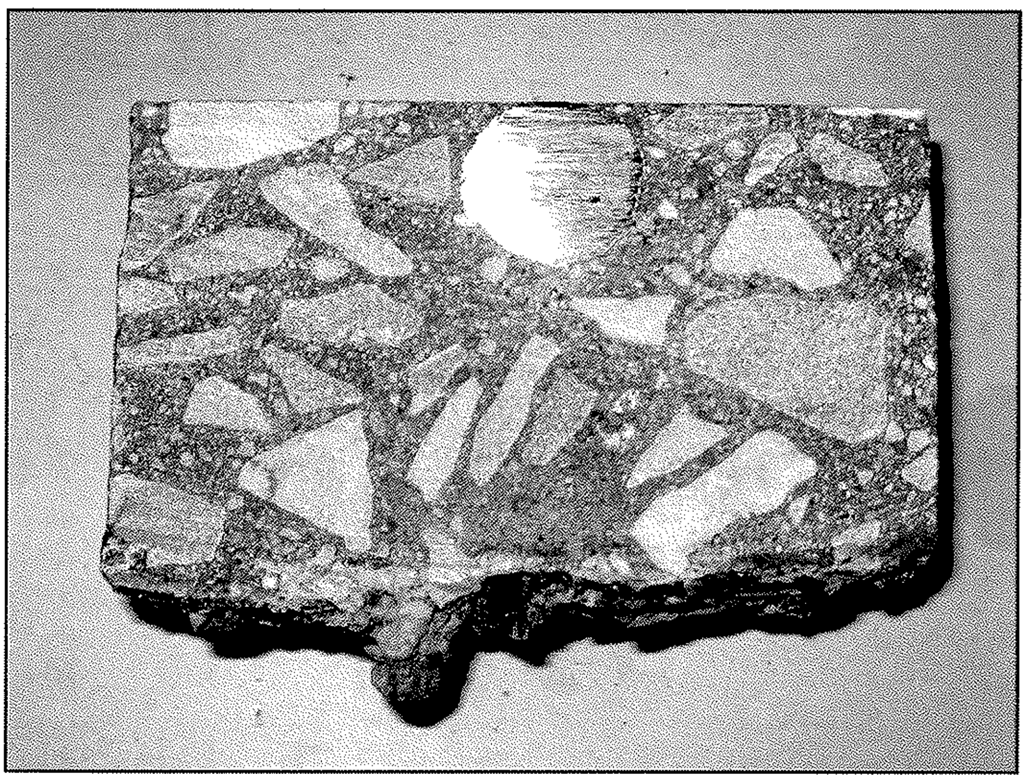


Photo 22. No corrosion observed in steel, paste stained with phenolphthalein (pink staining = high pH).

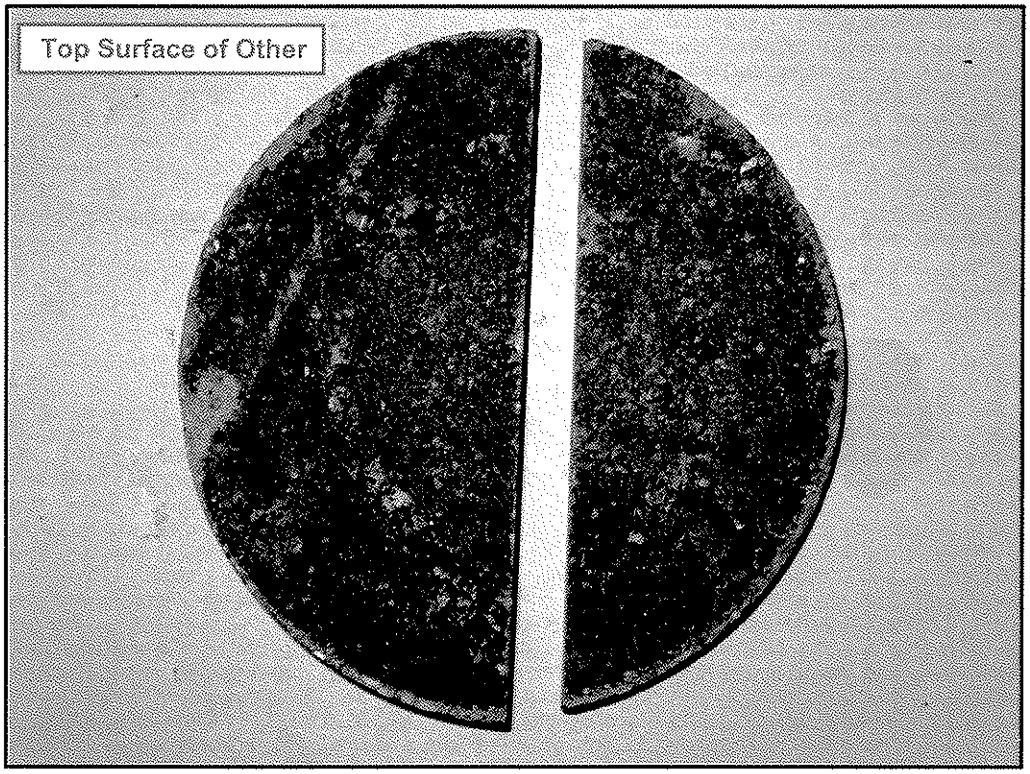


Photo 23. Top surface of other core.

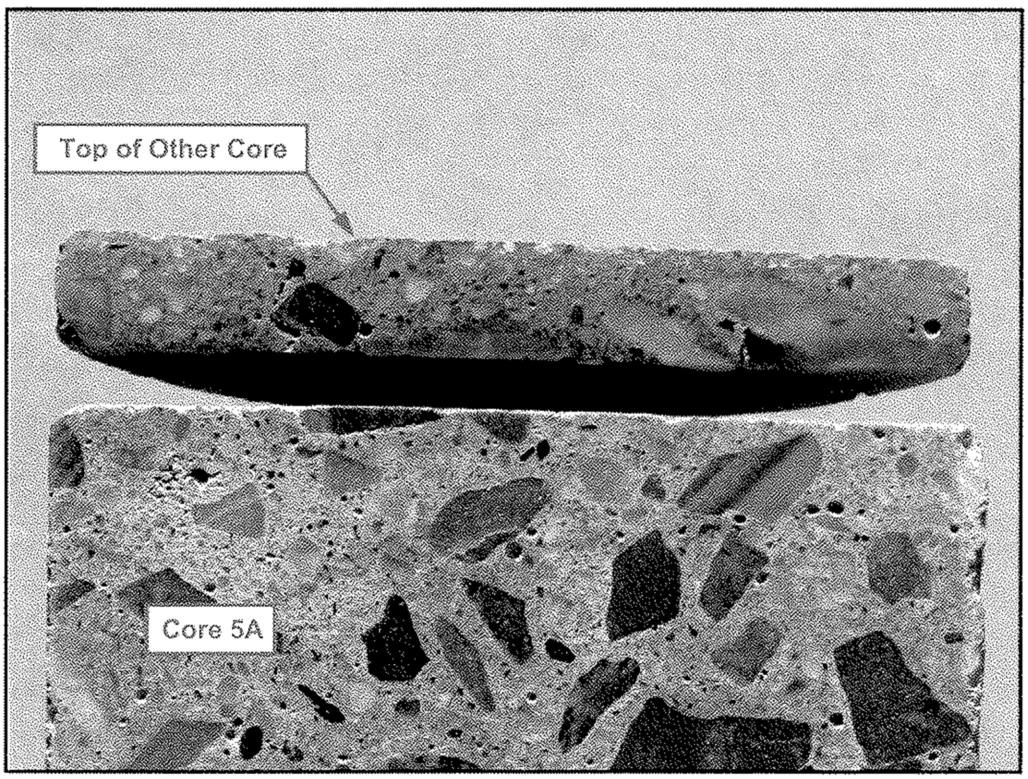
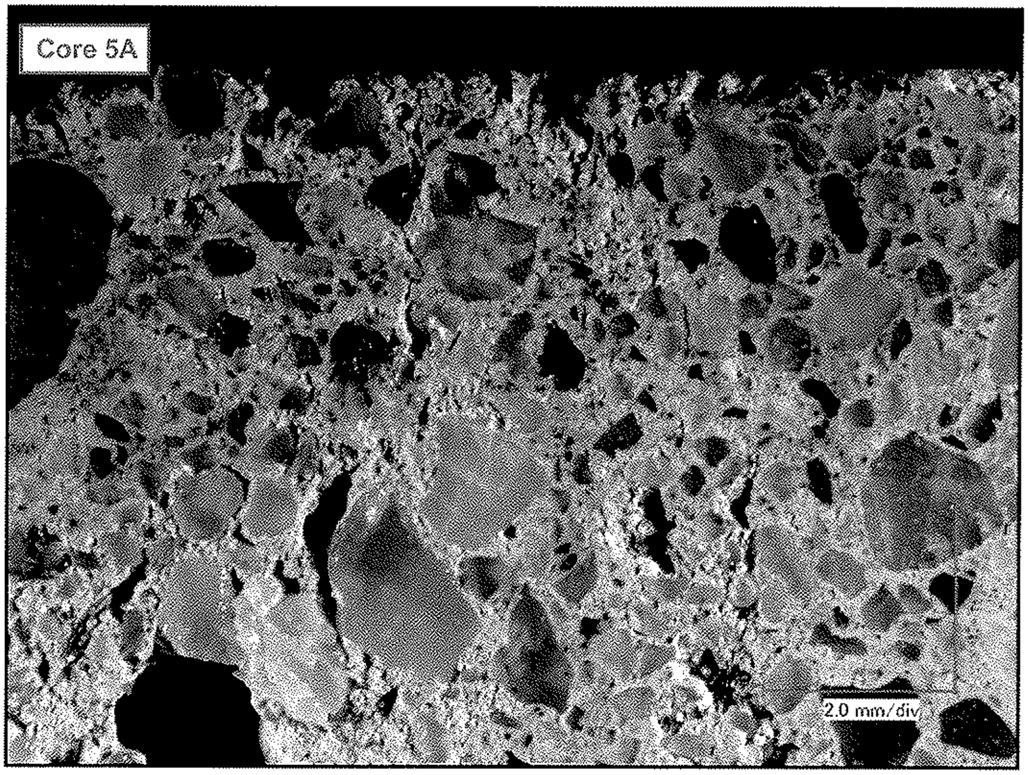
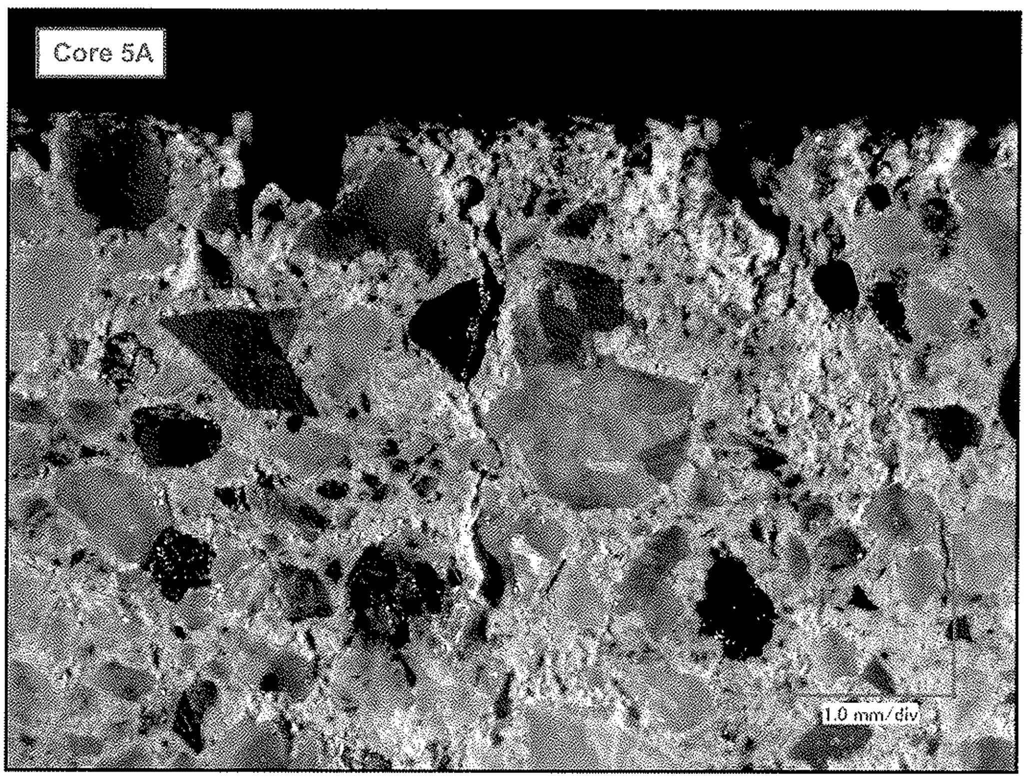


Photo 24. Polished section obtained from top ½" of other floor slab core.



Photomicrograph 25. Vertical cracks observed in Core 5A.



Photomicrograph 26. Vertical cracks observed in Core 5A.

Table 1: Results of Compressive Strength Testing of Concrete Cores ASTM C42

CORE NO.	Age *(years)*	Diameter *(in)*	Sawn Length *(in)*	Capped Length *(in)*	Area, *(in^2)*	Maximum Load *(lbs)*	Fracture Type	L/D Ratio	Correction Factor	Adjusted Compressive Strength *(psi)*
1	~90	2.75	2.9	3.0	5.94	20,770	2	1.09	0.892	3120
2		2.75	3.0	3.1	5.94	10,820	2	1.13	0.901	1640
3		Sample crumbled and did not remain intact when ends were sawcut								
Roof Slab Average										**2380**
5B	~90	3.70	6.4	6.5	10.75	61,440	3	1.76	1.000	5720
6A		3.70	5.1	5.2	10.75	56,300	3	1.41	0.949	4970
6B		3.70	4.8	4.9	10.75	66,680	3	1.32	0.938	5820
7		3.70	5.7	5.8	10.75	55,730	3	1.57	0.966	5010
Floor Slab Average										**5380**

Notes:
(1) The cores were tested on 10/26/10. Slabs were constructed in the 1920s.
(2) The cores were drilled and loaded perpendicular to the top surface of the slabs.
(3) The maximum aggregate size was approximately ¾" in the floor slab cores.
(4) The coarse aggregate in the roof slab cores was comprised of bottom ash and boiler slag.
(5) The maximum aggregate size was typically about ¾", but we observed a 2" piece in one core.
(6) No reinforcing was included in the tested portions.
(7) No significant defects were observed in the tested portions.

Table 2: Results of Acid Soluble Chloride Testing ASTM C1152

Core No.	Depth	Slab Construction	% chlorides (2)
1	Blend of tested middle portion (1)	Roof Slab	0.005
2			0.004
3			0.007
4	Middle portion of petrography (2)		0.011
5A		Floor Slab	***0.142***
5B	Blend of tested middle portion (1)		***0.122***
6A			0.027
6B			***0.134***
7			***0.123***

Notes:
(1) The chloride test samples were obtained by crushing portions of the cores into a powder after the compressive strength testing was performed. The ends were sawcut and were not included in the tested portion.
(2) Cores 4 and 5A were selected for petrographic examination. The chloride test sample was obtained from the middle portion, of the other half of the core, which was not polished. The ends of the cores were sawcut and were not included in the
(3) The percentage of chlorides is provided per mass of concrete.
(4) The testing was performed by Wyoming Analytical Laboratories

APPENDIX G

Calculations

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PROJECT ANACONDA JOB NO. 12900.02 PAGE 1

SUBJECT ROOF SLAB CAPACITY BY GMS DATE 11/12/10

- ASSUME W2 MESH @ 3" O.C.
  - → $A_s = .08$
- SLAB SPAN = 7.25'
- ASSUME C = 20,000

$$w = \frac{3CA_s}{L^2} = \frac{3(20{,}000)(.08)}{(7.25')^2} = 91.3 \text{ PSF}$$

4" SLAB	=	36 PSF
BUILT UP ROOF	=	10 PSF
DL	=	46 PSF

→ LL_{CAP} = 45 PSF

PROBES @ GROUND FLOOR WERE INCONCLUSIVE -

1. NO PILES WERE LOCATED
2. NO THICKENED SLABS OR PILE CAPS WERE LOCATED
3. NO TOP SLAB REINFORCING WAS FOUND

HOWEVER, HISTORIC DRAWING WWB 00994, "PILING LAYOUT," SHOWS PILES BENEATH SLAB @ ≈8'-0" o.c. FOR MOST OF BLDG. 52

FOLLOWING ANALYSIS IS BASED ON ASSUMPTIONS:

1. THERE ARE PILES (INVESTIGATE SPACINGS OF 6'-0", 8'-0", 10'-0")
2. THERE ARE NO TOP BARS. THEREFORE ALL SPANS ARE DESIGNED AS SIMPLY SUPPORTED, WITH CRACKING PERMITTED OVER ℄ OF SPANS.

RESULTS OF FLOOR PROBES:

$t = 8"$ SLAB THICKNESS

COVER TO BOTTOM OF REINF. = 1½", $d = 6"$ MEAN ℄ OF 2 LAYERS

TWO REINFORCING PATTERNS:

A. #4 @ 6" o.c. E.W., $A_s = 0.40\ in^2/FT$, $\rho = .0056$

B. #6 @ 10" o.c. E.W., $A_s = 0.53\ in^2/FT$, $\rho = .0073$

ASSUME: $F_y = 40{,}000\ psi$, $F'_c = 4000\ psi$

$$\left.\begin{aligned} M_{U_A} &= 7.02^{1k} \\ M_{U_B} &= 9.06^{1k} \end{aligned}\right\}$$ FROM ACI DESIGN HANDBOOK

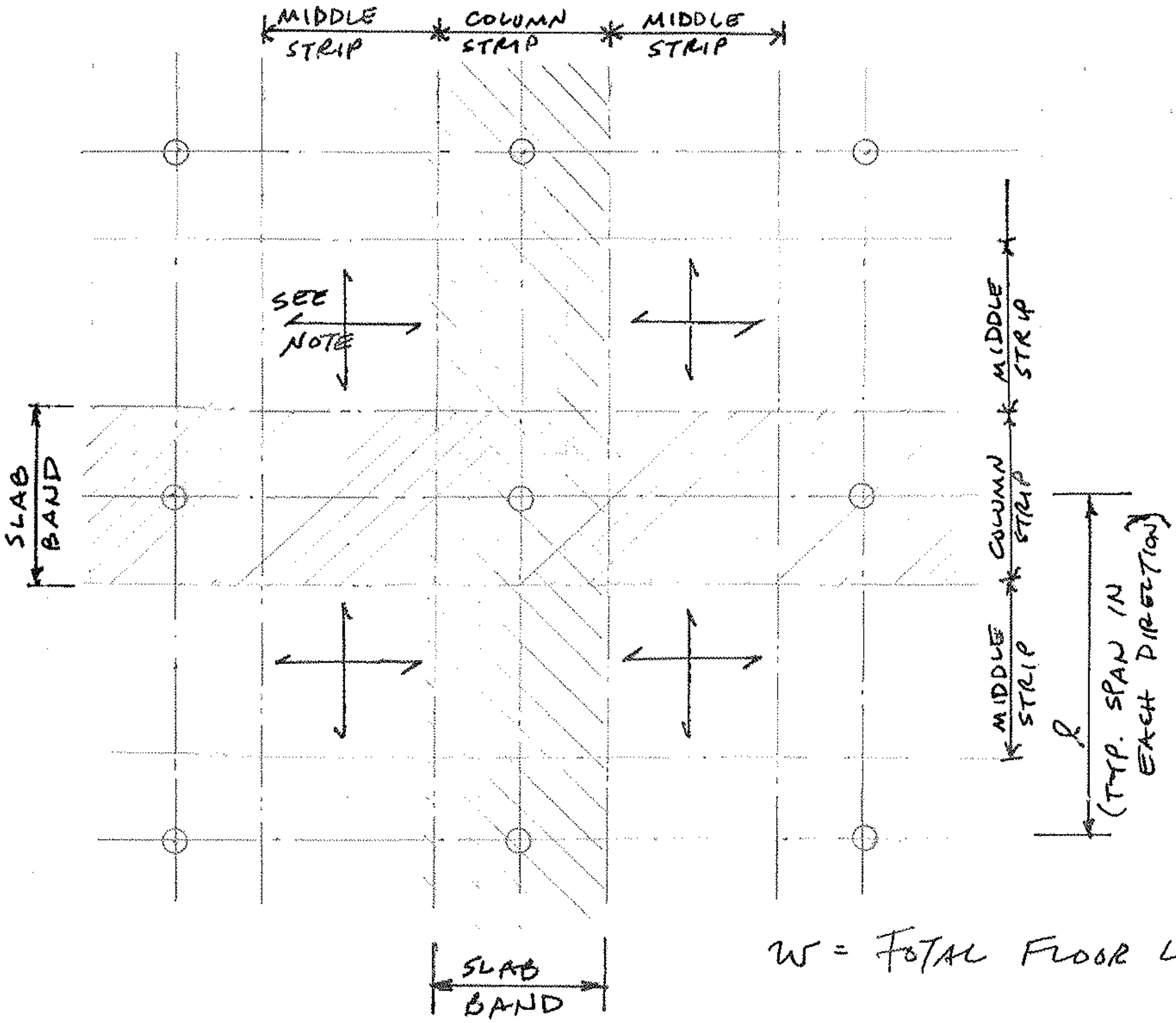
ASSUME: BECAUSE ~~THERE~~ IS NO TOP REINFORCEMENT, THIS DOES NOT BEHAVE AS A TWO-WAY SLAB AS DEFINED BY ACI.

∴ FLOOR IS ANALYZED AS A 4'-0" WIDE "SLAB BAND" SIMPLY SUPPORTED ON PILES. SLAB BAND SUPPORTS, IN ADDITION TO IT SELF, 2'-0" OF SLAB ON EACH SIDE FROM THE MIDDLE STRIP ONLY. THAT PORTION OF SLAB ⊥ TO SLAB BAND THAT FORMS PART OF COL. STRIP OR SLAB BAND RUNNING ⊥ TO SLAB BAND BEING DESIGNED, IS ASSUMED TO BE SUPPORTED BY SLAB BAND IN THE ⊥ DIRECTION.

(SEE PG. 1-2 FOR DIAGRAM)

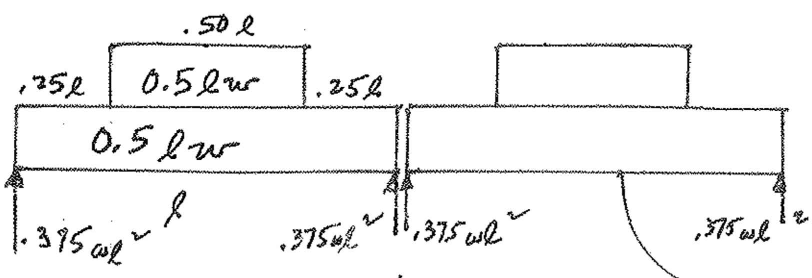
PROJECT BUILDING 52 JOB NO. 12980.02 PAGE 1-2

SUBJECT GROUND FLOOR BY M DATE 12/2010



w = TOTAL FLOOR LOAD (psf)

NOTE: LOAD FROM MIDDLE STRIP SUPPORTED BY SLAB BAND.
USE 100% OF " " LOAD IN EACH DIRECTION



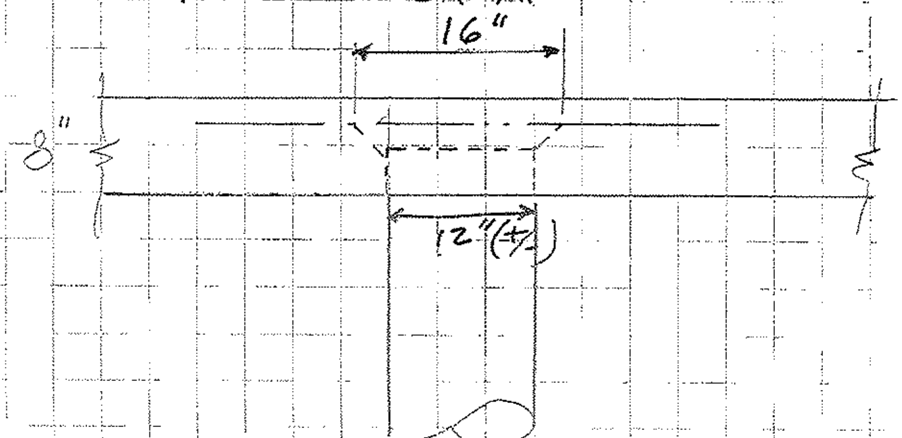
PILE LOAD = wl^2

$M_{max} = .375wl^2 \times \frac{l}{2} - .5wl \times .5l \times .25l - .5wl \times .25l \times .125l =$
$= .1094\ wl^3$

M_{CAP} (BASED ON REINF.) = $\phi b d^2 f'_c \omega (1 - 0.59\omega)$ [SEE ACI DESIGN MANUAL, TABLE 6.2]
for $d = 6''$, #4@6", $\rho = .0055$, $M_{CAP} = 7.02^{1K}$
#6@10", $\rho = .0073$, $M_{CAP} = 9.06^{1K}$

PILE SPACING	SLAB BAND/FT. $M_U = \frac{.1094 w l^3}{l/2} = .2188 w l^2$	$M_U = M_{CAP}$, $w_{TL} = \frac{M_{CAP}}{.2188 l^2}$		w_{LL} (FACTORED) = $w_{TL} - 0.120$		w_{LL} (UNFACTORED) = w_{LL}(FACT.)/1.6	
		A (7.02) #4@6"	B (9.06) #6@10"	A #4@6"	B #6@10"	A #4@6"	B #6@10"
6'-0"	7.8768 w	.891	1.150	.771	1.03	.481	.643
8'-0"	14.0032 w	.501	.646	.381	.526	.238	.329
10'-0"	21.88 w	.320	.414	.200	.294	.125	.183

CHECK PUNCHING SHEAR:



$V = 4\sqrt{f'_c}\, b_o d = 4 \times \sqrt{4000} \times \pi \times 16 \times 4 =$
50.83^K (FACTORED) $\div 1.5 \approx 34^K$ (UNFACTORED)
6'-0" SPACING $= (.643 + .100) \times 6^2 = 26.7^K$
8'-0" " $= (.329 + .100) \times 8^2 = 27.5^K$
10'-0" " $= (.183 + .100) \times 10^2 = 28.3^K$
ALL $< 34^K$

ALSO ALL LESS THAN 40^K, LIKELY CAPACITY OF A TIMBER PILE.

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PROJECT BUILDING 52 JOB NO. 12900.02 PAGE 1-4

SUBJECT GROUND FLOOR BY RS. DATE 12/2010

CAN 2" BE TEMPORARILY REMOVED FROM TOP SURFACE OF GROUND FLR. SLAB - SCARIFY TO REMOVE PCBs (ALSO WILL REMOVE CARBONATED CONCRETE & ANY CHLORIDE CONTAMINATION).

THIS WILL REDUCE "d" FROM 6" to 4" (SEE TABLE 6.2, ACI)

#4 @ 6" o.c. = A_s = .40 in², ρ = .40/4x12 = .00833, M_u = 4.46 ^k

#6 @ 10" o.c., A_s = .53 in², ρ = .53/4x12 = .011, M_u = 5.80 ^k

CHECK LOWEST LOAD, PILES @ 10'-0" o.c.

$$w_{TL} = \frac{M_{CAP}}{.2188 \ell^2} = \frac{4.46}{21.88} = .204^{KSF}$$

LESS FACTORED DL of 6" SLAB = -.090 = .075 x 1.2

FACTORED v_{CAP} = .114 KSF

UNFACTORED ÷ 1.6 = .071 KSF

SLAB WILL SUPPORT CONST. LOAD OF 71 psf MIN. DURING WORK. HOW HEAVY IS SCARIFYING MACHINE?

ADVANTAGE IS THAT NEW 2" TOPPING SLAB CAN BE BONDED TO BASE 6" SLAB & CAN HAVE TOP REINF. INSTALLED. THUS SLAB CAN BE DESIGNED AS CONTINUOUS 2-WAY FLAT PLATE (IF THERE ARE INDEED PILES) WHICH WILL HAVE A MUCH GREATER LOAD CARRYING CAPACITY.