VIRTUAL MEETING OF THE ADVISORY PARKING COMMITTEE
WEDNESDAY, June 2, 2021 @ 7:30am
https://zoom.us/j/98209276859 or dial: **877 853 5247 US Toll-free,**
Meeting ID: 982 0927 6859

1. Roll Call
2. Introductions
3. Review of the Agenda
4. Approval of Minutes, May 5, 2021
5. Electric Vehicle Charging Stations
6. Meeting Open to the Public for items not on the Agenda
7. Miscellaneous Communications
    a Parking Structural Assessment Report Recommendations
8. Next Meeting – August 4, 2021
9. Adjournment

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City of Birmingham
Advisory Parking Committee
Regular Meeting

Held Remotely Via Zoom And Telephone Access
Wednesday, May 5, 2021

Minutes

These are the minutes of the Advisory Parking Committee ("APC") regular meeting held on Wednesday, May 5, 2021. The meeting was called to order at 7:30 a.m. by Chair Al Vaitas.

1. Rollcall

Present: Chair Al Vaitas
Vice-Chair Richard Astrein
Aaron Black (arrived 7:45 a.m.)
Judith Paskiewicz
Mary-Claire Petcoff
Lisa Silverman
Jennifer Yert (left 8:00 a.m.)

(all members were located in Birmingham, MI except Mr. Astrein, who was located in Huntington Woods, MI, Mary-Clare Petcoff, who was in Hilton Head, SC, and Chair Vaitas, who was in Bloomfield Hills, MI.)

Absent: Steven Kalczynski
Lisa Krueger
Anne Honhart

SP+ Parking: Catherine Burch
Sarah Burton

Administration: Scott Grewe, Patrol Commander
Mike Albrecht, Police Commander
Eric Brunk, IT Manager
Laura Eichenhorn, City Transcriptionist

2. Introductions

None.

3. Review of the Agenda

4. Approval Of Minutes: Meeting Of April 7, 2021

Motion by Mr. Astrein

Seconded by Dr. Silverman to approve the minutes of the regular APC meeting of April 7, 2021 as submitted.

Motion carried, 4-0.

ROLL CALL VOTE
Yeas: Astrein, Silverman, Vaitas, Petcoff
Nays: None
Abstain: Yert, Paskiewicz

5. Parking Structure Internet Upgrade

IT Manager Brunk reviewed the item.

Dr. Silverman noted that by her calculations the City has only experienced internet outages 1.6% of the time, which she said was not necessarily a significant enough figure to merit switching providers.

IT Manager Brunk said internet downtime causes frustration for Staff and lost revenue on the days it is down. He said the proposed technology upgrades in the parking structures also require more reliable internet connection.

Dr. Silverman asked if now was the appropriate time to be making this investment after about a year of lost revenue for the parking fund due to Covid-19.

IT Manager Brunk said he and Ops. Cmdr. Grewe determined that completing the internet installation before July 1, 2021, when the City will begin charging again, will present the internet upgrades from causing any inconvenience to the parking users.

Dr. Silverman asked if this fiber upgrade could eventually be used to broaden internet access to Birmingham residents in the future.

IT Manager Brunk said that while Crown Castle tends to work more with businesses, it could potentially install wireless access points throughout the City if the City chose to pursue that.

Motion by Mr. Astrein
Seconded by Ms. Yert to recommend upgrading the internet connections at all (5) parking garages to the Managed Fiber Ethernet from Crown Castle for a 36-month term.

Motion carried, 7-0.

ROLL CALL VOTE
Yeas: Astrein, Yert, Black, Silverman, Vaitas, Petcoff, Paskiewicz
Nays: None

6. Credit Card Processing Fees

Cmdr. Albrecht reviewed the item.

Both Mr. Astrein and Dr. Silverman recommended that the City look into whether other vendors might have lower credit card processing fees for the on-street parking meters. They noted that many of the charges are negotiable in general.

Cmdr. Albrecht said that when the Parking Manager joins the City they might pursue finding the same credit card processor for the structures and the on-street meters. He said that at that point the Parking Manager might also look into whether reduced processing fees might be available.

Cmdr. Grewe noted that while some credit card processing fees might be negotiable in general, these contracts coming in through the RFP process might limit the City's ability to negotiate somewhat. He noted that finding a credit card processor for the City is contingent on the rates offered during the RFP process. Ops. Cmdr. Grewe concurred with Cmdr. Albrecht however that the new Parking Manager could look into potentially reduced processing fees in the future.

7. Meeting Open to the Public for items not on the Agenda

Ops. Cmdr. Grewe informed the APC that the Commission would be receiving a legal overview of how the Parking Assessment District works at their May 10, 2021 meeting.

A number of APC members said they would try and attend the Commission workshop.

Mr. Astrein said it would be worthwhile for the City to consider increasing the cost for repeated on-street parking violations. He said that it seemed that the current costs may not be acting as an effective-enough deterrent.

8. Miscellaneous Communications

9. Next Meeting: Wednesday, June 2, 2021

10. Adjournment

No further business being evident, the meeting adjourned at 8:28 a.m.

____

Patrol Commander Scott Grewe

MEMORANDUM

Police Department

DATE: **May 25, 2021**

TO: **Advisory Parking Committee**

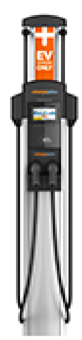
FROM: **Scott Grewe, Operations Commander**

SUBJECT: **Electric Vehicle Charging Station**

INTRODUCTION:
As electric vehicles become more popular, the need for Electric Vehicle (EV) charging stations is on the rise. Currently the City has no charging stations in any of the parking structures or at the on-street meters. While multiple automakers have promoted a shift to increasing production of electric vehicles the City has begun to review the installation of EV charging stations.

BACKGROUND:
Staff contacted multiple vendors researching several options for EV stations, including speaking with local communities where EV stations are already in use.

Chargepoint
Staff spoke with Ian McGill of Chargepoint. Chargepoint provides 24/7/365 access for customer support and complete the install of their units. With Chargepoint, the equipment is leased which includes the software to operate the charging units. Chargepoint recommended their CT4000 level 2 model which costs $100 per port per month. Each unit is a dual port unit totaling $2,400 per year or $12,000 over a 5-year lease per unit with two ports.



EV Connect
Staff spoke with Brice Burman of EV Connect. Burman advised EV Connect is the software provider and the City would purchase a charging station and they would provide the software to operate the station. Burman stated they work with BTC charging stations ($4,200 per unit) or Evo Charge ($2,800 per unit). He advised the software is $60 per port or $120 per unit totaling $1,440 per year for the software. Under the same 5-year plan the BTC station would cost $11,400 and the Evo Charge would cost $10,000.

Eco Green / Enel X
Staff spoke with Amanda from their e-mobility division. Amanda stated that they are a software provider and recommend the Juice Box Station be purchased to operate with their software. The Juice Box Pro costs $1,419 per unit (one port each) and needs either their stand ($499) or their pedestal ($1,100) for installation. Software is $120 per port per year. Estimated cost for one pedestal with two ports over 5 years is $5,183. These units run off of Wifi, however if there is poor service a data plan may need to be added at $300 per year for each location (up to 16 ports per location). This could add an additional $1500 per location over the 5 year comparision.

Comparison
Two of the companies, EV Connect and Eco Green, are software companies so the equipment is purchased and owned by the City. The City will be responsible for service and maintenance of the units, using any available warranties, and may see increased cost savings if the units stay in operation beyond the five year examples above depending on potential repair costs. However, Chargepoint leases the equipment to the City and handles all equipment issues to include software upgrades but comes at a higher price as a result. Both the Chargepoint and BTC charging stations include a digital touch screen control at each charging stand. All companies offer 24/7/365 support and programmable units. Additionally, the Chargepoint unit is the only unit that comes standard with a retractable cord which others offer at an extra expense. Staff also contacted Volta and Tesla and have not heard back from them at this time.

Staff reviewed the website, Chargehub.com that includes a map list of charging locations and limited information about the location. While most locations listed were located at private businesses the cities of Ann Arbor and Royal Oak showed charge locations at public parking lots. Staff contacted representatives from each of those two locations.

City of Ann Arbor
Staff spoke with Jada Hahlbrock, Manager of Parking Services, from the Downtown Development Authority for the City of Ann Arbor. Hahlbrock stated the City of Ann Arbor first put in charging stations approximately 10 years ago with a federal grant that specified the type of charging stations to be used. Ann Arbor has Clippercreek charging units which are currently free to users. Hahlbrock stated the units have been very reliable, with only minor repairs. She also advised they have started to look at more sophisticated units as their current Clippercreek models have no options. Hahlbrock stated they are leaning towards Chargepoint since the equipment is leased and the responsibility for maintenance is on the vendor not the City. She also stated they are looking to charge users in the future and stated they should have done that from the beginning as they expect backlash if and when they change. Additionally, she added retractable cords are great to reduce the chance of damage to them from being driven over and a system to enforce time limits. Hahlbrock stated the only complaint they typically receive is vehicles parked all day at the charging stations.

City of Royal Oak
Staff spoke with Chris Annetta, Parking Systems Manager, regarding the charging stations located in the City of Royal Oak. Annetta advised the City of Royal Oak has four charging stations located in two of the their parking structures. He stated all four are Chargepoint models and each are dual port for a total of eight ports. Annetta stated the units have been very reliable and the cloud system to monitor their Chargepoint stations was very informative and useful. He stated they do charge for use of the stations and are currently charging $1.30 per hour and stated they have collect approximately $300 from the stations for May, though the 26th. Annetta also stated they are reviewing options for customizing their system further to encourage more change over. The only complaint they have received was regarding people parking at the space for the charging station who were not using the station. Annetta stated the stations were installed in first floor locations to be seen which resulted in the use of high demand locations which was part of the problem. They are currently reviewing additional signage to discourage the use of the spot when not using the charging station.

FISCAL IMPACT:
DTE is currently offering "Charger Infrastructure Incentives" that fund the EV charging stations for business and commercial electric customers. Level 2 stations can receive $2,500 per port. There must be a minimum of two ports per site to ensure charging availability, a maximum of 20 port rebates per site and a maximum of 100 port rebates per business or commercial customer.

Incentives from DTE may provide up to $5000 per stations (2 ports). Depending on the charging station selected additional costs to the City will vary. These costs will include electrical installation and use, painting and signage for each location as well as potential wayfinding signage within the structures. Operation costs of each station can be offset or potentially covered by charging users. In the Royal Oak comparison, they collected approximately $300 for the month of May. Assuming that's the average, they will collect $3,600 for the year. Costs of their equipment based on figures provided by Chargepoint is $9,600 per year (4 stations) leaving them with a $6,000 deficit.

SUMMARY:
There is a nationwide push to increase the infrastructure for electric vehicles. While there is a rise in the use and production of electric vehicles, the City has no charging stations. Currently the parking structures are around 50% capacity during any given time and future demand is unknown, however, expected to be lower than pre-pandemic levels. As a result, there is an ability to assign dedicated parking spaces for special use without adding stress to the current demand levels.

Staff reviewed the mobile app for Chargepoint and found that many charging stations are often available and have limited use. For example, in Royal Oak, of their eight ports, most were only used every other day and only by one car per day. Staff has asked Royal Oak for a more detailed use report, however this would suggest that installation of two units (four ports) in two of the five structures (one unit per structure) maybe a first step to review usage before installing system wide.

SUGGESTED RECOMMENDATIONS:
Staff is looking for recommendations on the following items.

Discussion and selection of vendor for charging stations. Staff recommends proceeding with the Chargepoint CT4000 unit.

To install one charging station at ___________ structure and one at __________ structure and evaluate usage over time for future installations.

Staff suggests Pierce and Park Structures.

Or

Install one charging station at each one of the five parking structures and evaluate usage over time for additional stations at each location.

APPENDIX B. OPINION OF PROBABLE COSTS

Immediate Recommendations (within 1 Year)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Investigation and repair of two Lower Level columns ǂ	1	LS	$ 50,000	$ 50,000
Immediate Recommendations Total				**$ 50,000**
Near-Term Recommendations (within 1 to 2 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Localized concrete repairs in slabs, full depth	300	SF	$ 80	$ 24,000
Localized concrete repairs in slabs, partial depth topside	500	SF	$ 45	$ 22,500
P/T slab tendon splice and materials - allowance	1	LS	$ 50,000	$ 50,000
Replace construction joint sealant*	1,500	LF	$ 6	$ 9,000
Rout and seal cracks in elevated slabs and replace failed sealant at isolated cracks	500	LF	$ 6	$ 3,000
Replace expansion joint seals*	150	LF	$ 125	$ 18,750
Install traffic bearing membrane at construction joints, occupied areas, and vehicle entrance lanes	25,000	SF	$ 5	$ 125,000
Apply concrete sealer at all elevated levels	147,500	SF	$ 0.40	$ 59,000
Inspect and clean drain lines*	1	LS	$ 15,000	$ 15,000
Subtotal				**$ 326,250**
General Conditions, Overhead and Profit (15%)				$ 48,938
Project Contingency (15%)				$ 48,938
Engineering/Testing/Construction Period Services (10%)				$ 32,625
Near-Term Recommendations Total				**$ 456,750**
Long-Term Recommendations (within 3 to 5 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost**
Concrete Structure Repairs				
Localized concrete repairs in slabs, full depth	150	SF	$ 80	$ 12,000
Localized concrete repairs in slabs, partial depth	100	SF	$ 45	$ 4,500
P/T slab tendon splice and materials - allowance	1	LS	$ 25,000	$ 25,000
Partial depth concrete repairs at beams, columns, foundation walls, and stairs and isolated crack repairs at beam-column intersections	1,000	SF	$ 90	$ 90,000
Waterproofing Repairs				
Install traffic bearing membrane at drains and concrete repairs	2,500	SF	$ 8	$ 20,000
Replace cove sealant at roof level, install cove sealant at other isolated locations	2,500	LF	$ 6	$ 15,000
Modify stair tower roof downspouts	1	LS	$ 2,500	$ 2,500
Facade, Stair Tower, and Miscellaneous Repairs				
Repair brick masonry cladding - allowance	1	LS	$ 250,000	$ 250,000
Repair stairwell storefront assemblies	1	LS	$ 50,000	$ 50,000
Stairwell handrail repairs, clean and paint metal surfaces	1	LS	$ 150,000	$ 150,000
Subtotal				**$ 619,000**
General Conditions, Overhead and Profit (15%)				$ 92,850
Project Contingency (15%)				$ 92,850
Engineering/Testing/Construction Period Services (10%)				$ 61,900
Total				**$ 866,600**
* Highest priority of near-term repair items.				
** Prices based on current (2021) dollars, and do not include increases for inflation (recommended 3% per year).				
ǂ Pending further analysis during repair design phase; includes engineering, shoring, and masonry allowances.				

APPENDIX B. OPINION OF PROBABLE COSTS

Immediate Recommendations (within 1 year)						
Item Description	**Est. Qty.**	**Units**	**Unit cost**		**Est. Cost***	
Loose concrete removal	2	work day	$	1,000	$	2,000
Replace damaged/missing drain covers	24	each	$	350	$	8,400
Reset displaced and loose stair tower metal cover plates	10	each	$	200	$	2,000
Immediate Recommendations Subtotal					**$**	**12,400**
Near-Term Repair Recommendations (within 1 to 2 years)						
Item Description	**Est. Qty.**	**Units**	**Unit cost**		**Est. Cost***	
Concrete						
Partial-depth topside slab concrete repairs	5,000	SF	$	45	$	225,000
Partial-depth underside slab concrete repairs	4,500	SF	$	100	$	450,000
Waterproofing and Drainage Improvements						
Rout and seal cracks and joints in slab	25,000	LF	$	6	$	150,000
Traffic bearing membrane on Level 5	41,000	SF	$	5	$	205,000
Inspect and clean drain lines	1	each	$	15,000	$	15,000
Masonry Repairs						
Replace concrete masonry units at stair towers	50	SF	$	20	$	1,000
Subtotal					**$**	**1,046,000**
General Conditions, Overhead and Profit (15%)					$	156,900
Project Contingency (15%)					$	156,900
Engineering/Testing/Construction Period Services (10%)					$	104,600
Near-Term Recommendations Total					**$**	**1,464,400**
Long-Term Repair Recommendations (within 3 to 5 years)						
Item Description	**Est. Qty.**	**Units**	**Unit cost**		**Est. Cost***	
Concrete						
Partial-depth topside slab concrete repairs	5,000	SF	$	45	$	225,000
Partial-depth underside slab concrete repairs	4,500	SF	$	100	$	450,000
Concrete column repairs	150	SF	$	110	$	16,500
Concrete wall repairs	60	SF	$	100	$	6,000
Waterproofing Improvements						
Traffic bearing membrane on Level 2, 3, and 4	123,000	SF	$	5	$	615,000
Replace sealant at cove seal joints	1,000	LF	$	6	$	6,000
Masonry Repairs						
Localized repointing of clay masonry veneer	100	SF	$	20	$	2,000
Replacement of clay brick masonry units	60	EA	$	15	$	900
Steel lintel clean and coat	40	LF	$	350	$	14,000
Subtotal					**$**	**1,335,400**
General Conditions, Overhead and Profit (15%)					$	200,310
Project Contingency (15%)					$	200,310
Engineering/Testing/Construction Period Services (10%)					$	133,540
Long-Term Recommendations Total					**$**	**1,869,560**
Grand Total					**$**	**3,346,360**
*Prices based on current (2021) dollars and do not inlcude increases for inflation (recommended 3 percent per year)						

APPENDIX B. OPINION OF PROBABLE COSTS

Near-Term Recommendations (within 1 to 2 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Localized concrete repairs in slab, full depth	1,500	SF	$ 80	$ 120,000
Localized concrete repairs in slab, partial depth	500	SF	$ 45	$ 22,500
P/T slab tendon repairs - allowance	1	LS	$ 75,000	$ 75,000
Replace pre-molded expansion joint seals (Tier A through Tier 3), including expansion joints near stairs*	1,200	LF	$ 125	$ 150,000
Replace control joint sealant at intermediate PT anchorages (N-S joints)*	2,000	LF	$ 6	$ 12,000
Rout and seal cracks in elevated slab and replace failed sealant at isolated cracks	750	LF	$ 6	$ 4,500
Install traffic bearing membrane at control joints, expansion joints, and PT tendon repair areas	36,000	SF	$ 5	$ 180,000
Apply concrete slab sealer on elevated levels	195,000	SF	$ 0.40	$ 78,000
Install waterproofing and flashing improvements at pedestrian bridges	2	LS	$ 8,000	$ 16,000
Replace deteriorated horizontal lines at floor drains and associated components*	150	LF	$ 90	$ 13,500
Inspect and clean lines as part of repair effort*	1	LS	$ 15,000	$ 15,000
Remove loose brick coping fragments and verify all brick coping units are secure (not loose)*	1	LS	$ 1,500	$ 1,500
Repair brick distress within east stair towers (vertical cracking and outward displacement). Coordinate with waterproofing efforts at pedestrian bridges.	2	LS	$ 24,000	$ 48,000
			Subtotal	**$ 736,000**
			General Conditions, Overhead and Profit (15%)	$ 110,400
			Project Contingency (15%)	$ 110,400
			Engineering/Testing/Construction Period Services (10%)	$ 73,600
			Total	**$ 1,030,400**
Long-Term Recommendations (within 3 to 5 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Structural Repairs				
Localized concrete repairs in slab, full depth	250	SF	$ 80	$ 20,000
Localized concrete repairs in slab, partial depth	100	SF	$ 45	$ 4,500
Partial depth concrete repair at beams, columns, walls, spandrels	1,000	SF	$ 90	$ 90,000
Repair composite steel decking and supporting steel framing elements	1	LS	$ 7,500	$ 7,500
Waterproofing Repairs				
Replace winged expansion joints at Tier 4 (roof)	275	LF	$ 125	$ 34,375
Install traffic bearing membrane outside elevators at Tier 1 and at column bases at inclined columns (small areas)	700	SF	$ 8	$ 5,600
Install traffic bearing membrane at drains (small areas)	350	SF	$ 8	$ 2,800
Replace remaining joint sealant on elevated levels, including perimeter cove seal.	5,000	LF	$ 6	$ 30,000
Crack repairs at foundation walls and perimeter walls where active water infiltration is present	150	LF	$ 35	$ 5,250
Add drain in region of standing water on northeast end of Tier 1	1	LS	$ 4,000	$ 4,000
Facade, Stairwell and Miscellaneous Repairs				
Replace deteriorated brick coping units in-kind ǂ	900	LF	$ 100	$ 90,000
Repoint deteriorated brick mortar	900	SF	$ 50	$ 45,000
Clean efflorescence (including exterior facade access)	1	LS	$ 4,000	$ 4,000
Rout and seal cracks on slab on ground and replace failed joint sealant (Tier B)	1,500	LF	$ 6	$ 9,000
			Subtotal	**$ 352,025**
			General Conditions, Overhead and Profit (15%)	$ 52,804
			Project Contingency (15%)	$ 52,804
			Engineering/Testing/Construction Period Services (10%)	$ 35,203
			Total	**$ 492,835**
* Highest priority of near-term repair items.				
**Prices are based on current (2021) dollars.				
ǂ See report discussion for alternative repair options.				

OPINION OF PROBABLE COSTS

Near-Term Recommendations (within 1 to 2 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Replace construction joint sealant*	900	LF	$ 6	$ 5,400
Repair column stiffener and moment connection plates*	24	EA	$ 1,000	$ 24,000
Inspect and clean drain lines*	1	LS	$ 15,000	$ 15,000
Traffic bearing membrane - complete replacement or new installation	142,000	SF	$ 4	$ 568,000
Traffic bearing membrane - add'l top coat only	72,000	SF	$ 2.50	$ 180,000
Rout and seal cracks in elevated slabs	1,500	LF	$ 6	$ 9,000
Replace expansion joint seals at stair towers	100	LF	$ 125	$ 12,500
Localized concrete repairs in slab, partial depth topside	2,500	SF	$ 45	$ 112,500
Localized concrete repairs in slab, full depth	11,000	SF	$ 80	$ 880,000
P/T slab tendon and anchor repair - allowance, approx. 50 repairs	1	LS	$ 250,000	$ 250,000
			Subtotal	**$ 2,056,400**
			General Conditions, Overhead and Profit (15%)	$ 308,460
			Project Contingency (15%)	$ 308,460
			Engineering/Testing/Construction Period Services (10%)	$ 205,640
			Near-Term Recommendations Total	**$ 2,878,960**
Long-Term Recommendations (within 3-5 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Concrete Structure Repairs				
Localized concrete repairs in slab, partial depth topside	100	SF	$ 45	$ 4,500
Localized concrete repairs in slab, full depth	150	SF	$ 80	$ 12,000
Structural Steel Repairs				
Repair column base plates and/or anchorages	10	EA	$ 250	$ 2,500
Repair exterior pipe column bases	44	EA	$ 750	$ 33,000
Repair beam-to-column shear connections	70	EA	$ 500	$ 35,000
Repair intermediate vehicle barrier post connections, properly clean and paint	115	EA	$ 250	$ 28,750
Replace intermediate vehicle barrier post connections, in-kind	50	EA	$ 500	$ 25,000
Repair intermediate vehicle barrier post bases ǂ	18	EA	$ 10,000	$ 180,000
Replace vehicle barriers, in kind	5	EA	$ 400	$ 2,000
Facade Repairs				
Replace facade panels and posts impacted by vehicles	1	LS	$ 5,000	$ 5,000
Replace missing anchors at facade base plates	1	LS	$ 2,500	$ 2,500
Reattach facade panel tie-backs ǂ	1	LS	$ 200,000	$ 200,000
Miscellaneous				
Repair stair landings, tread/risers, CMU walls, and brick headers	1	LS	$ 20,000	$ 20,000
			Subtotal	**$ 550,250**
			General Conditions, Overhead and Profit (15%)	$ 82,538
			Project Contingency (15%)	$ 82,538
			Engineering/Testing/Construction Period Services (10%)	$ 55,025
			Long Term Recommendations Total	**$ 770,350**
			Grand Total	**$ 3,649,310**

* Highest priority of near-term repair items.

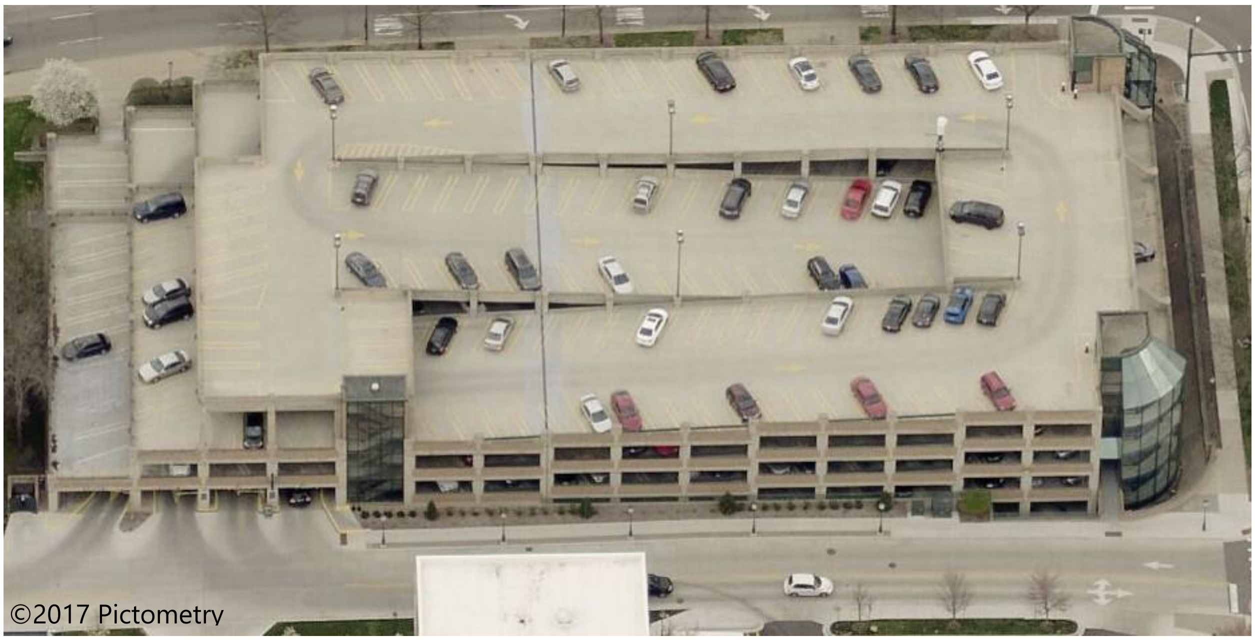
** Prices based on current (2021) dollars, and do not include increases for inflation (recommended 3% per year).

ǂ Pending further analysis during repair design phase.

City of Birmingham
Parking Garage Structural Assessment Program

Chester Parking Structure

180 Chester Street
Birmingham, MI 48009



FINAL REPORT

April 30, 2021

WJE No. 2019.6318.0

PREPARED FOR:

Mr. Scott Grewe
Operations Commander - Birmingham Police Department
City of Birmingham
151 Martin Street
Birmingham, Michigan 48009

PREPARED BY:

Wiss, Janney, Elstner Associates, Inc.
30700 Telegraph Road, Suite 3580
Bingham Farms, Michigan 48025
248.593.0900 tel

City of Birmingham
Parking Garage Structural Assessment Program

Chester Parking Structure

180 Chester Street
Birmingham, MI 48009

Matthew E. Lewis, PE
Project Manager & Senior Associate

Sarah V. Rush, PE
Senior Associate

Derek R. Hibner, PE, CDT
Project Associate

FINAL REPORT

April 30, 2021
WJE No. 2019.6318.0

PREPARED FOR:

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Operations Commander - Birmingham Police Department
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151 Martin Street
Birmingham, Michigan 48009

PREPARED BY:

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CONTENTS

1.0 INTRODUCTION

As requested, Wiss, Janney, Elstner Associates, Inc. (WJE) has completed limited condition assessments of the North Old Woodward, Park Street, Peabody, and Chester parking structures. These assessments were performed with the intent to determine the current and future infrastructure needs in support of a capital improvement plan; the intention of the plan is to extend the useful life of the structures and to maintain the structural integrity to ensure the structure can support the code-prescribed loadings. This report summarizes our observations at the Chester Parking Structure, located at 180 Chester Street in Birmingham, Michigan, and provides recommendations for your consideration.

2.0 STRUCTURE DESCRIPTION

Constructed in 1988, the Chester parking structure features six levels of parking. Tiers A and B are below grade, with Tier B consisting of a reinforced concrete slab on ground, while the Ground Tier (Tier 1) and Tiers 2 through 4 are above grade. The three-bay side-by-side structure is rectangular-shaped, extending 167 feet in the north-south direction and 330 feet in the east-west direction for a total parking area of approximately 282,500 square feet. The central and south bays are sloped to serve as circulation ramps, while the north bay has only a minor transverse slope for drainage. The floor plans for Tiers 2, 3, and 4 are set back in increments at the garage perimeter (terraced). Stair towers with elevator shafts project from the building plan on the east corners of the parking structure and are connected to the adjacent sidewalk with pedestrian bridges at grade. A third stair tower is present to the west of the vehicle entrance.

The structure consists of cast-in-place conventionally reinforced concrete columns supporting cast-in-place post-tensioned (PT) concrete beams and slabs. The PT tendons consist of single 7-wire strands in plastic sheathing with bonded reinforcement. The one-way structural slab consists of tendons extending in the east-west direction spanning between the PT beams, while the temperature and shrinkage tendons run in the north-west direction. One expansion joint is present on the east end of the garage, while multiple construction joints are present in each bay, which corresponds to locations of intermediate PT anchorages. The PT beam tendons are draped and continuous between aligned bays. A portion of the floor slab near the elevator shafts of each level (approximately 18 feet by 8 feet), consists of a composite reinforced concrete slab with a corrugated steel deck. This slab portion is independent of the adjacent post-tensioned slab and is separated by an expansion joint.

The perimeter walls generally consist of conventionally reinforced concrete supported on the concrete slab, which are clad with brick veneer. Concrete spandrel panels are present at the roof level and recessed areas on the north facade. A traffic coating is present at the pedestrian bridges and outside the elevators on Tier 1, around perimeter column bases on Tier 3, and at some of the expansion joints on Tier 3 and all expansion joints on Tier 4. The stair towers are clad in brick masonry veneer and curtainwall assemblies. The vehicle barrier system at interior column lines consists of post-tensioned cables.

2.1 Document Review

Construction drawings, produced by Luckenbach, Ziegelman and Partners, Inc. and dated May 27, 1988, were provided by the City of Birmingham for WJE's review. The construction drawings are similar to, but do not represent exactly, the as-built conditions. Key differences represented in the construction drawings include the following:

- A second vehicle entrance located at the south end of Tier A, which is not represented in the as-built construction.
- Brick pavers outside the elevators (at the composite slab), which are not represented in the as-built construction.
- Office spaces at the southwest corner of the Tier 1 and Tier A, encompassing approximately 800 and 1,200 square feet in plan, respectively, are present but are not indicated on the drawings.

Other pertinent information obtained from the reviewed drawings is discussed within the observation sections below. However, one item of note is that the uniform design live loads used in the design of the deck are based on older code requirements and are higher than the current code-required loads (2015 *Michigan Building Code* (MBC)). Floor slabs and beams were designed for a live load of 50 pounds per square foot (psf), while columns were designed for a live load of 40 psf. The roof level slab and beams were designed for a live load of 67.5 psf (including snow), while columns supporting the roof level were designed for a live load of 54 psf. For comparison, the MBC requires 40 psf live load if this same garage was constructed new. Based on our site visit observations, several past restoration projects have occurred at the building; however, documentation related to these efforts was not provided to WJE for review.

3.0 FIELD ASSESSMENT

WJE visited the site on several occasions in January and February 2020 to perform visual assessments of the accessible and exposed portions of the structure and facade. WJE returned to the site in May 2020 to perform a delamination survey at representative locations. WJE returned to the site on multiple occasions throughout February 2021 to perform non-destructive evaluation measures, review inspection openings, extract concrete cores for materials testing, and complete additional assessment efforts. The deck's offices were not accessible and have been excluded from our assessment.

WJE's scope included a limited sounding survey of the supported levels in accordance with *ASTM D4580 - Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding*. For this survey, areas of delamination were identified using the chain-drag method, localized hammer sounding, and use of a delamination wheel at select underside locations. In areas of sound concrete, these methods produce a clear, ringing sound, and when a delamination is encountered, a hollow, drum-like sound is produced. Between 25 and 75 percent of the total area for each floor was surveyed, including all construction joints where intermediate PT anchorages are located. Sounding of the underside of the slab with a delamination wheel was primarily limited to locations of previous repair and visible indications of potential concrete deterioration (e.g. at visible cracks, spalls). Similarly, sounding of the beams and columns was generally limited to locations of previous repair and visible indications of potential concrete deterioration.

A summary of pertinent observations follows.

3.1 Structural Slabs

The elevated PT slabs are typically 5-1/2 inches thick with draped PT tendon reinforcing. Bonded mild reinforcing steel is present at the maximum and minimum vertices of the draped tendon profile, as well as the drains, slab edges, and tendon anchorages. Outside the elevators, the reinforced concrete slab is 4-1/2 inches thick with 1-1/2 inch galvanized steel composite decking, which is supported by steel framing members that are bolted to the stair walls or supported by steel pipe columns.

All elevated floor slabs are generally in serviceable condition with localized areas of distress, as described below:

1. The steel deck typically exhibits isolated regions of corrosion, ranging from surface corrosion to loss of the full deck cross-section (Figure 1). The supporting steel framing elements exhibit similar signs of corrosion, including the bases of the steel pipe columns. In some locations, the framing members are deformed due to the buildup of corrosion byproduct between elements. Peeling and blistered paint are also typically present. The composite slab above exhibits isolated cracking and delamination.
2. Localized areas of spalled and unsound concrete were identified throughout the elevated slabs during the visual and delamination surveys. In general, less than 1 percent of the areas surveyed were unsound.
    a. At less than 20 locations, localized areas of spalled and incipient spalled concrete are present, less than 5 square feet in size each, primarily concentrated at Tier 2. The spalls typically expose corroded mild reinforcing steel (Figure 2 and Figure 3). At some spalls, the concrete cover over the bar was less than 1/4 inch (low cover).
    b. PT tendons were not readily visible at these spalls; however, as part of the inspection opening work (described in further detail below), WJE had the three spalled locations on the top side of the slab repaired to mitigate potential damage to the PT system.
    c. Isolated cracks align with locations of mild reinforcing steel in similar areas, as determined by corrosion staining, exposed bar, and non-destructive evaluation methods, though the concrete in these areas was generally sound. Some cracks have previously been routed and sealed; however, in some regions, cracking propagates beyond what was previously routed and sealed.
    d. Delaminations elsewhere in the slab are primarily concentrated near drains and the PT anchorages (deck perimeter, construction joints, and expansion joints).
3. At approximately 10 locations, isolated cracks (<0.010 inches wide) extend diagonally up to 3 feet away from columns. This distress was primarily noted in the north bay of Tier 2, central bay of Tier 3, and the south bay of Tier 4. The concrete on either side of the cracks is sound and a majority of the cracks have previously been routed and sealed.
4. Isolated cracks are present in line with transverse temperature and shrinkage reinforcing steel and penetrate previous repair areas (Figure 4). This distress was primarily noted at Tier 2 and Tier 3. Other isolated cracks are located at miscellaneous locations throughout the deck, generally less than 10 linear feet each. Most of these cracks have previously been routed and sealed without readily apparent crack propagation.
5. Approximately 10 PT tendon anchorage repairs are located along the expansion joint on Tier 3 and Tier 4 (Figure 5). These repair areas are sound with isolated cracks, efflorescence, and corrosion staining present. These cracks often align with the location of PT tendons or are transverse to the repair area and are attributed to shrinkage and/or restraint.
6. Isolated grease stains were noted within previous repair areas but were not observed elsewhere within the deck.
7. Isolated locations of grout that was placed in the formed stressing pockets at slab tendon anchorages are missing (Figure 6). Minor surface corrosion of the exposed tendon is visible in these regions.

8. Previous concrete repairs not associated with PT tendons are in isolated regions elsewhere in the structure and are typically located near drains and in areas of shallow concrete cover. The formed-and-poured concrete repairs are generally in serviceable condition with isolated cracking, efflorescence, and corrosion staining similar to that observed within the PT repair areas. Previous repair areas that used trowel-applied patch material were generally unsound.

3.1.1 PT Inspection Openings

Based on the slab observations noted above, locations throughout the elevated levels of the structure were selected for inspection openings to permit the evaluation of the embedded PT tendons. WJE retained a local concrete restoration contractor, Pullman SST, to create and repair the inspection openings specified by WJE. Refer to Figure 7 for the locations of inspection openings.

1. Six inspection openings were created throughout the upper levels of the structure at low points of tendon drape (slab underside):
    a. **Inspection Openings 1, 2, and 4:** One tendon was exposed at each inspection opening location below and in-line with an area of spalled concrete. The tendon sheathings were intact, the tendons were holding tension, grease filled the sheathing voids, and the tendons were uncorroded (Figure 8).
    b. **Inspection Openings 3 and 5:** Two tendons were exposed at each inspection opening location in-line with a previously repaired tendon that exhibited signs of distress (cracks, corrosion, and grease staining). The tendon sheathings were intact, and the tendons were holding tension. At one of the inspection openings, corrosion byproducts and moisture had emulsified the grease. Section loss and pitting of the individual wires were not observed. The grease and tendon were in good condition and uncorroded at the other opening (Figure 9 and Figure 10).
    c. **Inspection Opening 6:** Two tendons were exposed in a sound concrete area adjacent to a previous PT repair. The tendon sheathings were intact, and the tendons were holding tension. Corrosion byproducts and moisture had emulsified the grease (Figure 11). Section loss and pitting of the individual wires were not observed.

3.2 Superstructure Elements

The beams supporting the decks are post-tensioned and are supported by columns or foundation walls. The beams at pedestrian bridges are conventionally reinforced. The spandrel beams at the roof level and recessed areas of the north facade are conventionally reinforced. The majority of columns in the structure are conventionally reinforced concrete, with the exception of inclined columns supporting Tier 4, which are post-tensioned. The beams and columns are generally in serviceable condition with concentrated areas of distress, as follows:

1. Evidence of water infiltration is present on the majority of beams at the expansion joints (Figure 12).
2. Localized areas of delaminated concrete and cracking are present within PT beams at each level (Figure 13). At approximately 20 locations, the locations of unsound concrete range in size from approximately 5 to 40 square feet.
    a. The observed beam distress is concentrated at expansion joints and stair towers at each level of elevated slab, with more severe distress concentrated at Tier 1 and Tier 3.

   b. A majority of the previous concrete beam and column repairs are unsound on Tier 1 and Tier 4 (Figure 14). Previous concrete repairs at other levels are generally serviceable.
3. Isolated locations of grout that was placed in the formed stressing pockets (grout pockets) for the PT beam reinforcing have debonded, though corrosion or grease staining was not observed on the anchorages.
4. Arched beams supporting the pedestrian bridges exhibit areas of unsound concrete, corroded exposed reinforcement, cracking, and corrosion and efflorescence staining (Figure 15).
5. Honeycombing in the concrete surface was noted at isolated columns, though these areas were found to be sound (Figure 16).
6. Approximately 30 columns across the structure, particularly on columns supporting Tier 2 and Tier 4, have localized areas of delaminated concrete that are primarily near the bottom of the columns, which are an average size of 2 to 4 square feet each (Figure 17). This includes some of the inclined columns (Figure 18). Other areas of concrete distress were observed at columns located below failed expansion joint seals, pedestrian bridges, and at some beam-to-column intersections.
7. Several of the columns and spandrel beams have narrow cracks, which primarily align with stirrup reinforcement, and are particularly concentrated on the exterior facing surfaces (Figure 19). Corrosion staining was generally not observed, except in isolated regions where reinforcing bars were exposed due to low concrete cover (less than 1/8 inch to 1/4 inch).
8. Evidence of water infiltration and restraint cracking is present at several locations where the spandrel beams bear on the columns (Figure 20). Water staining, corrosion staining and/or efflorescence were observed during WJE's site visit. The cracked concrete areas were generally sound.

3.2.1 Foundation and Perimeter Walls

Reinforced concrete foundation walls are present at Tiers A and B. The perimeter walls at each elevated slab are primarily comprised of reinforced concrete that bear on the slab and are clad with brick veneer. The following conditions were observed:

1. Vertical cracks are present at isolated locations of the foundation walls at Tier B (Figure 21). These cracks are largely concentrated at wall openings, penetrations, and vertical construction joints. Some of these regions exhibit signs of active water infiltration or staining, efflorescence, and corrosion staining. The cracked areas contain regions of delaminated concrete in some areas.
2. Isolated cracking on perimeter walls, with crack widths measuring up to 0.025 inches, is present at Tier 1, Tier 3, and Tier 4, with isolated areas of efflorescence staining (Figure 22). These cracks largely propagate from embedded steel handrails, or appear to be due to shrinkage and/or low cover. At one location in the north bay of Tier 1, a small region of unsound concrete is present (Figure 23).

3.2 Waterproofing and Drainage Components

The expansion joints in the deck typically consist of pre-molded expansion joint seals, except for the roof level which are winged expansion joint seals. A traffic-bearing waterproofing membrane (traffic coating) is not typically present on slab surfaces, except at the Tier 4 and skyward-facing Tier 3 expansion joints and at the pedestrian bridges and outside the elevator shafts on Tier 1.

Localized areas of traffic-bearing membrane are also present around the bases of the inclined columns at Tier 3. The following pertinent conditions were observed:

1. Most of the pre-molded seals for expansion joints have tears in the sealant or adhesion failures of the sealant to the concrete substrate (Figure 24). Active water penetration was noted at a majority of these locations during WJE's assessment. The expansion joint seals between the composite slab outside the elevators, pedestrian bridges, and adjacent post tensioned concrete slab exhibit similar distress (Figure 25).
2. Though the roof level, winged expansion joint seals exhibited isolated distress (e.g. split, adhesively failed sealant, or active water penetration); they are in better condition than the widely failed expansion joint seals on the floors below.
3. Most of the sealant at the construction joints throughout the structure has adhesively failed, including the perimeter cove seals at Tier 3 and Tier 4, and slab-on-grade joints.
4. Efflorescence and water staining are present at the bottom of the slab at approximately 10 percent of the construction joints, indicating prolonged moisture exposure in these regions (Figure 26).
5. Where miscellaneous cracks have previously been routed and sealed, the sealant is generally in serviceable condition, though some areas exhibit adhesive failure. Some cracked areas were actively leaking at the time of WJE's assessment and contained effloresce and water staining on the bottom surface, including areas in, and surrounding, the composite steel deck slab.
6. Much of the traffic coating that exists at the Tier 3 and Tier 4 expansion joints is worn or delaminated (Figure 27).
7. The traffic coating at the pedestrian bridges is delaminated (Figure 28).
8. The traffic coating around the bases of inclined columns at Tier 3 has tears and areas of delamination (Figure 29).
9. One area of standing water was noted on the northeast end of Tier 1 (Figure 30). The area of standing water encompasses approximately 1,000 square feet. Smaller isolated areas of standing water are present within Tier 3, which are approximately less than 100 square feet each.
10. Most of the floor drains throughout the structure were clear of debris at the time of WJE's site visits; however, nearly all the horizontal rainwater collection pipes connecting the floor drains to the risers are corroded, including areas of full section loss and fractured steel (Figure 31 and Figure 32). In a few locations, the extent of corrosion extends to the drain bowls (Figure 33).

3.3 Stairs, Facade, and Miscellaneous

The stairwell walls are nominally 12 inches thick with clay brick masonry exterior wythes. The drawings indicate that the central cavity is fully grouted and reinforced. The brick masonry cladding the perimeter walls, stair towers, and pedestrian bridge and site walls is generally in serviceable condition, with areas of minor distress. The stairwell curtainwalls and roofs are generally in serviceable condition. The slab on ground (Tier B) contains isolated regions of distress. The following pertinent conditions were observed within these systems and elsewhere within the garage:

1. A partial-depth repair at the slab on ground, encompassing approximately 125 square feet in the north bay, is unsound and cracked (Figure 34).

2. Spalled and cracked brick coping units at the perimeter walls are typically present at Tiers 2 through 4 (Figure 35).
3. Areas of efflorescence, as well as eroded mortar joints between brick veneer units, are present on the north, east, and west elevations, particularly concentrated at Tier 1 (Figure 36 and Figure 37). Similar distress was noted within the stairwells, pedestrian bridges, and site walls with active water infiltration observed in some areas during WJE's assessment.
4. At the common wall between the southeast stairwell and the deck, a large vertical crack, approximately 1/4-inch wide, extends from the slab on ground to the top of Tier A (Figure 38). The crack is visible on both surfaces of the wall and is approximately 20 feet in length. Portions of the wall contain additional cracking and isolated regions that are delaminated and outwardly displaced by approximately 1/4-inch on each surface of the wall.
5. All stairwells exhibit scaling and erosion of the concrete treads and landings to varying degrees, which is attributed to exposure of deicing salts and cyclical freeze-thaw damage (Figure 39). Stairwell landings also contain isolated cracks.
6. Large quantities of deicing salts were present on each level and area of the garage, including within parking stalls (Figure 40).
7. The slab-on-grade contains isolated cracks, largely stemming from column corners and attributed to shrinkage and restraint. A majority of these locations have been previously sealed, though some sealed regions exhibit adhesive failure.
8. The concrete curbs at the vehicle entrance exhibit scaling and erosion, which is attributed to exposure of deicing salts and cyclical freeze-thaw damage.
9. The vehicle barrier cables along the interior column lines are generally in serviceable condition, with isolated areas exhibiting cracked coating and minor surface corrosion. Isolated grout pockets are debonded or have been previously repaired with a trowel-applied patch repair, and grease stains were noted at a few locations (Figure 41).

4.0 REPAIRS COMPLETED TO-DATE

In an effort to take advantage of reduced occupancy during the COVID-19 pandemic, the City of Birmingham approved a limited scope of repairs on May 18, 2020 to be performed by DRV Contractors, LLC. As of the issuance of this report, the following repairs have been performed:

- Removal of loose concrete on the underside of slabs throughout the garage
- Localized concrete repairs in the stair towers

5.0 MATERIALS TESTING

Five concrete cores were extracted from various locations in the structure and sent to WJE's Cleveland laboratory for materials testing. The locations of the cores are provided in Table 1 below. The lab studies included petrographic examinations, water-soluble chloride analysis, water absorption testing, and carbonation depth measurements. One concrete mix was observed within all cores extracted from the slabs. A summary of the findings is presented in this report section. See ***Appendix A*** for more testing information and figures.

Table 1. Core Locations

ID	Core Location	Location Description
C1	Tier 1 South Bay	In drive lane at vehicle entrance
C2	Tier 3 South Bay	In drive lane
C3	Tier 2 Northwest End	In parking stall within covered region of deck, away from drains and outside of drive lane
C4	Tier 4 (Roof) Northeast End	In drive lane, near stair tower
C5	Tier 1 Northeast End	In drive lane, within area of ponded water near deck exit route

The concrete slab materials are generally in serviceable condition. The concrete mix consists of limestone, siliceous sand, and portland cement. The concrete is air entrained, which improves the concrete's freeze-thaw durability. No distress due to internal deleterious reaction mechanisms, such as alkali-silica reaction, was observed.

WJE did not find indications of secondary distress as a result of external factors (e.g. chlorides, moisture, freeze-thaw damage, etc). This indicates that, although isolated areas of standing water were observed, the deck does not appear to have experienced sustained long-term moisture over the course of its service life thus far. It is important to repair and maintain the damaged waterproofing components within the deck to further protect the concrete from progressive distress. WJE also visually observed surface erosion on the top, exposed concrete surfaces. However, when analyzed in the lab, we did not find microcracking or other indications of distress beneath the eroded top surfaces.

5.1 Water Absorption

Water drop testing was performed to test the hydrophobicity (water repellency) of the top surface. Refer to Table 2 of ***Appendix A*** for a summary of the test results for each core. Water drops applied to the core surfaces spread and were absorbed into the concrete surfaces of Cores 2, 3, and 4. However, hydrophobic properties of the paste were observed to a maximum depth of 3/8 inch in these three cores. These findings indicate a penetrating sealer may have been applied to the deck and penetrated into the concrete, but has likely worn from the top surface over time, significantly reducing its effectiveness. These cores pertain to Tiers 2, 3, and 4 (Roof).

Water drops applied to the surfaces of the cores taken on Tier 1 (Cores 1 and 5) loosely beaded and were eventually absorbed. The paste exhibited hydrophobic properties in Core 1 but not in Core 5 with depth from the top surface. These findings suggest that a sealer may have been applied on Tier 1 with currently limited performance on the top surface. Based on the expected service life of penetrating sealers, an additional sealer application on Tiers 2, 3, and 4 should be anticipated to maintain the probable existing waterproofing system. While limited benefits currently exist for Cores 1 and 5, consideration should be given to the application of a sealer on Tier 1. The presence and condition of an existing sealer on Tier A is unknown at this time, but can be determined as part of future design and construction phases, if desired.

5.2 Carbonation Testing

The high pH of uncarbonated concrete provides protective passivation of the embedded steel reinforcement. Carbonation is a chemical process that occurs in the cement paste due to penetration of atmospheric carbon dioxide and lowers the pH of the concrete. The depth of carbonation increases over time and is accelerated at cracks or joints. When the carbonation front reaches the depth of reinforcing steel, the steel becomes more susceptible to corrosion because the passivation layer from the high pH of the concrete is no longer present. The depth of the carbonation for each core is shown in Table 2 of ***Appendix A***.

The maximum depth of paste carbonation from the top of the cores is 1/2 inch. The maximum depth of complete paste carbonation is 1/2 inch, and partial carbonation is 3/4 inch from the bottom surfaces. The relatively uniform depth of carbonation on both slab surfaces suggests that the probable installation of a penetrating sealer on the top surface occurred later in the deck's service life, after some level of carbonation had occurred. The depth of carbonation is less than the depth of the typical embedded reinforcing steel; thus, the increased potential for corrosion due to carbonated concrete is not a concern currently. However, embedded steel elements in areas of shallow cover would be expected to experience an increased potential due to carbonated concrete at these depths, which may result in deterioration of the surrounding concrete.

5.3 Water-Soluble Chloride Testing

The purpose of the chloride analysis was to determine the current water-soluble chloride ion content at various depths of the slab. The results are contained within Table 2 of ***Appendix A***. The water-soluble chloride content by mass of concrete at the typical depth of reinforcing steel was found to be negligible. However, significantly elevated chloride content was measured at the top surface of Cores 2, 3 and 4. High levels of chloride contents at the slab surface are indicative of externally applied chloride sources, such as the application of deicing salts. The test findings also suggest that a chloride-containing admixture was not used in the concrete mixture during the garage's construction. Localized areas of greater chloride contamination may occur at cracks and joints. With continued use of chloride-containing deicing salts, the chloride concentration and depth would be expected to increase if the probable existing penetrating sealer is not maintained.

6.0 DISCUSSION

Overall, the parking structure is in serviceable condition with localized areas of deterioration. Waterproofing repairs are recommended in the near future to maintain the condition of the parking structure.

6.1 Concrete - General

Concrete parking structures in Michigan are susceptible to deterioration due to their exposure to moisture, deicing salts, and temperature changes (i.e., cyclic freezing and thawing, thermal expansion and contraction, etc.). The primary causes of concrete deterioration in concrete parking structures is corrosion, typically due to chloride contamination and carbonation as both conditions can promote corrosion of embedded steel reinforcement.

Because steel corrosion product occupies a larger volume than the native steel, it is common for distress in the form of cracks, delaminations, or spalls to develop when the embedded steel corrodes and expands, placing expansive forces on the surrounding concrete.

Post-tensioned structures efficiently combine steel, which is strong in tension, and concrete, which is strong in compression, to utilize the full cross section of a structural element at all points along its length. Compared to conventionally reinforced concrete, post-tensioned concrete typically offers greater durability, particularly due to its ability to minimize cracking and to protect the tendons from corrosion. The benefit of post-tensioned concrete over conventionally reinforced concrete depends heavily on adequate protection of the tendons from moisture. Locations that are most susceptible to moisture exposure include tendon anchor points, where the sheathing or anchor may not be protected, and construction joints or concrete repairs, where the tendon sheathing is made discontinuous for stressing or possibly damaged during repair, respectively. These locations of discontinuous sheathing at cracks or joints can allow water to directly reach the tendons. Deterioration of PT tendons, particularly corrosion leading to section loss, can result in failure of that tendon. If an unbonded tendon becomes de-tensioned for any reason, that tendon no longer carries load at any point along its length.

6.2 Structural Slabs

The majority of the elevated concrete slabs in the structure are in sound condition, with the exception of a few areas of localized distress, which are largely concentrated at Tier 2. Areas exhibiting localized delamination and spalled concrete are generally concentrated near mild reinforcing steel with shallow concrete cover and areas of prolonged moisture exposure, such as concrete near drains, and failed construction and expansion joint seals.

Based on our findings, the isolated spalls and delaminated areas in the slab are associated with shallow cover of bonded reinforcement and were not directly related to PT damage or deterioration. Although tendon damage was not observed at the inspection opening locations, the loss of concrete cover potentially exposes the corresponding tendon to moisture, chlorides, and sheathing damage, accelerating that tendon’s rate of deterioration. Thus, repair of the concrete and improved water management is recommended to protect the PT reinforcement.

Diagonal cracking within the slab that extended from some columns is attributed to shrinkage and restraint and does not constitute a structural concern. Similarly, isolated miscellaneous cracks within the slab are attributed to shrinkage and restraint, as well as low concrete cover to bonded mild reinforcing. Cracks in the elevated slabs should be routed and sealed on the top slab surface to mitigate water penetration, and failed sealant materials should be replaced.

Most of the previous concrete repairs are in serviceable condition, except where trowel-applied patch repairs were performed. WJE has found that the form-and-pour repair technique results in more durable repairs than trowel-applied repairs. Therefore, we encourage using this technique instead of trowel-applied repairs in the future. Full-depth concrete repairs are anticipated in most locations of concrete distress that are not associated with the PT tendons; however, partial-depth repairs may be possible in areas of mild reinforcing with shallow concrete cover. PT sheathing repairs should be anticipated wherever PT tendons are exposed within a concrete repair area.

6.2.1 Post-Tensioning

Of the nine PT tendons inspected, no failed tendons were found. Minor corrosion was observed within the grease at four inspection locations, though the exposed tendons were in good, serviceable condition, with no corrosion or section loss observed for the exposed strand. Previous PT repair efforts include restressing tendons, anchorage repairs, and sheathing repairs. The previous PT repairs were sound when struck with a hammer, though a few repair areas exhibit isolated shrinkage cracking with some areas containing signs of active water infiltration, such as efflorescence and corrosion staining. Each exposed tendon located in line with or near prior PT repairs exhibited signs of moisture infiltration in the form of corrosion by-products and emulsified grease. Such moisture exposure may have occurred either prior to or since the repair effort. It is not anticipated that these areas will require near-term repair, though installation of a traffic-bearing waterproofing membrane is recommended to protect the existing PT tendon repair areas.

Although the exposed tendons and previous repair areas were in serviceable condition, due to the limited nature of the inspection openings and to account for concealed areas of distress, we recommend that budgetary cost estimates assume some PT repairs are required during concrete repair efforts. Installation of a traffic-bearing membrane is also recommended along PT anchorage zones (at construction joints and expansion joints) for improved durability and protection of the existing PT anchorage components, which can be costly to repair.

6.2.2 Composite Concrete Deck

The corrosion and peeling paint noted on the composite steel deck and framing supporting the elevated slab at stair and elevator landings is attributed to the ingress of chloride-contaminated water through the failed expansion joints and perimeter sealant, and deteriorated concrete above. WJE did not note widespread deterioration of the concrete over the steel deck, though isolated cracked and delaminated regions should be repaired. The failed expansion joint seals and sealant should be replaced, and the exposed steel deck and framing elements should be cleaned and painted with a corrosion-inhibiting coating.

6.3 Waterproofing and Drainage Components

In WJE's experience, the long-term durability of parking structures subjected to chloride-laden moisture from de-icing salts is extended when moisture is well-managed throughout and prevented from absorbing into the structure.

6.3.1 Expansion Joint Seals and Joint Sealant

The failed pre-molded expansion joint seals are allowing water to penetrate the deck assembly and deteriorate the beam and column members below. All of the pre-molded expansion joint seals (on Tiers A through Tier 3) are recommended for replacement. The winged expansion joint seals on the roof level are in relatively better condition than the pre-molded expansion joint seals below, but they are exhibiting initial signs of distress and should be considered in budgetary planning estimates.

The construction joints in elevated slabs of this garage generally correspond to intermediate PT anchorage points. Like the expansion joint seals, the adhesively failed construction joint sealant is resulting in deterioration of the structural systems below. Deterioration at construction joints and expansion joints is of particular importance due to the presence of unprotected PT anchorages in these regions. Thus, maintaining the expansion joint seals and construction joints is a cost-effective method to mitigate the need for more costly future repairs. The sealant in other joints or isolated cracks around the parking structure show signs of aging and failure and are recommended for replacement.

6.3.2 Standing Water on Slabs

The isolated areas of standing water are attributed to poor finishing during original construction and are not believed to be related to insufficient floor drains and/or excessive structural deflection. The concentration of standing water creates a condition of extended exposure of the concrete structure to chloride-contaminated water and potential cyclic freeze-thaw deterioration if the concrete was not properly air-entrained. Furthermore, standing water in the winter months will freeze, which may cause slippery walking surfaces. Based on the materials test results and our observations, a penetrating sealer or traffic-bearing membrane is recommended to be installed in areas of standing water. At the large area of standing water noted at the northeast end of Tier 1, installation of a supplemental drain is recommended.

6.3.3 Traffic Coatings and Penetrating Sealers

Traffic-bearing membrane systems are a very common waterproofing system used to extend the life of a parking structure. The typical service life for a new traffic-bearing membrane in low traffic areas can easily exceed 10 years. In high traffic areas and in areas with significant turning, maintenance of a traffic-bearing membrane to address wear can be necessary in less than 5 years. Silane sealers, which in WJE's experience have proved to be an effective penetrating sealer for concrete, are also common. However, silane sealer does not have the capability to bridge cracks. Silane sealers wear over time and are generally reapplied at regular intervals varying between 5 and 7 years.

The existing areas of deteriorated traffic coating are attributed to natural aging, wear from automobile traffic, damage from snowplow blades, and/or deferred maintenance. In most cases, both the topcoat (wear course) and base coat are worn and damaged. The topcoat serves as a protective wearing surface while the base coat is more flexible and serves as the primary waterproofing layer. Where both coats are damaged, the existing membrane should be removed and replaced. However, in areas where the base coat remains undamaged and only the wear course is worn (i.e. at some of the composite steel deck areas), an additional compatible wear course may be applied to the existing system to extend the service life of the existing membrane.

The traffic coating observed at the base of the inclined columns supporting Tier 4 was likely installed at these locations to protect the PT anchorages. Localized repair of the delaminated concrete at these column bases are recommended, as well as removal and reapplication of the traffic-bearing membrane. Considering the use of deicing salts and elevated chloride levels at multiple levels, applying a silane sealer throughout the deck is recommended. A traffic-bearing membrane is recommended at drains due to their elevated exposure and to bridge isolated cracks in these regions. The slab-on-ground areas do not require membrane or sealers. Installation of new coating or sealer materials should occur in conjunction with localized concrete and sealant repairs in areas of cracked and unsound concrete.

6.3.4 Pedestrian Bridge Waterproofing

The delaminated membrane, poor edge detailing, and failed cove sealant on the pedestrian bridges are resulting in the observed deterioration of the arched reinforced concrete beams below. Replacement of the traffic coating and cove sealant at the base of brick masonry wall are recommended, as well as flashing improvements in the brick wall. We recommend that conceptual repairs also include installation of counter-flashing near the bottom of the brick masonry walls on each side of the pedestrian bridges in conjunction with the recommended brick and coping repairs, discussed in detail below.

6.3.5 Rainwater Collection Pipes

Deterioration of steel drainage pipes noted throughout the garage is attributed to the low slope of the horizontal pipe, in combination with transmission of chloride-contaminated water from the slab surfaces, freezing, and deferred maintenance. Replacement of the deteriorated horizontal sections of pipe that connect the floor drain to the riser is recommended. Replacement efforts should aim to improve the slope of the pipe wherever possible. Note that this work will also involve replacement of some of the deteriorated floor drains and vertical drain connectors as well. The risers are in serviceable condition.

6.4 Superstructure Elements

6.4.1 Concrete Beams

The distress observed at the PT beams is typically concentrated at the expansion joints and near the composite steel deck areas. The deterioration primarily consists of partial-depth concrete delaminations due to corrosion of the embedded steel elements and failed waterproofing elements above. Most of the previous repair areas are exhibiting signs of distress, which is largely attributed to continued moisture exposure. If left unmitigated, the embedded reinforcing steel in the beams, including the PT reinforcement, may become damaged, leading to more costly repairs. Most of the beams elsewhere in the structure are in good condition.

Localized partial-depth concrete repairs are recommended to address the noted PT beam distress. PT tendon repairs are not anticipated at these areas, and repairs should not use trowel-applied patch materials. Note that during beam repairs, shoring of the slab may be required at some locations due to the extent of distress.

The arched beams supporting the pedestrian bridges contain localized areas of unsound concrete, that require partial-depth concrete repairs in conjunction with the recommended waterproofing work above.

6.4.3 Concrete Columns, Foundation Walls, and Perimeter Walls

The majority of columns in the structure are in good condition, with localized areas of unsound concrete largely attributed to restraint and differential movement or moisture exposure at failed expansion joint seals. The isolated regions containing concrete honeycombing do not require repair. Most of the previous column repairs exhibit signs of continued deterioration. Partial-depth repairs are recommended in unsound areas. Shoring of supported elements is not anticipated, but if the concrete removal extends too far into the column cross-section, it may become necessary.

The foundation and perimeter walls are in good condition, with minimal areas of unsound concrete and isolated cracks. The areas of unsound concrete are attributed to restraint and differential movement near construction joints and corrosion of the embedded reinforcing steel. The cracks in the foundation walls generally do not exhibit signs of moisture infiltration, and do not require repair. Where water staining or efflorescence was observed at isolated cracks and at member intersections, waterproofing measures are recommended to mitigate distress of the concrete, such as installation of grout or sealant.

6.5 Stair Towers, Facade, and Miscellaneous

The vertical cracks observed in the brick masonry wall adjacent to the southeast stair entryway at Tier A and Tier B are likely related to the formation of corrosion product on the embedded vertical reinforcing bars, which is evidenced by the outward displacement noted on each side of the wall. Such corrosion is likely due to the failed waterproofing of the pedestrian bridge above. Anticipated repairs include removing the cracked brick units at each side of the wall, repairing the corroded reinforcing and grout, and replacing the brick in-kind.

Deterioration of the brick coping units is attributed to moisture exposure, cyclic freeze-thaw damage, and deferred maintenance. The observed deterioration within the brick veneer is largely due to moisture infiltration within the wall cavity because of the failed coping units, curtainwall sealant, or waterproofing membranes above. The efflorescence deposits, staining, and eroded mortar joints at the base of walls near pedestrian areas are also related to the application of deicing salts to walking surfaces. Where water infiltration within the brick assemblies occurs at perimeter slab areas, prolonged moisture exposure could lead to deterioration of the concealed PT slab anchorages, resulting in costly PT repairs.

Since many of the existing coping units exhibit signs of distress, and due to the unique size and shape of the coping bricks, conceptual repairs may include removing and replacing all of the brick coping in-kind. This option will maintain the original design of the wall assembly and aesthetic, which has performed well for its almost 35-year service life. Alternatively, repairs may include removal of all the brick copings, installation of a stainless steel through-wall flashing, and installation of new copings with an improved brick, dimensioned limestone, or sheet metal coping. This option will not maintain the current aesthetic and may be more costly, but will improve the water management and durability of the assembly for a longer estimated service life. In either option, repairs should include cleaning the wall cavity of debris and ensuring all drainage weeps are clear and can adequately drain. WJE can provide further information regarding these repair options during future design and repair phases, if desired. In the near-term, the City of Birmingham should consider engaging a contractor to remove any loose brick units or spalled brick material.

The efflorescence deposits and staining on the brick masonry facade elements may be cleaned, and the eroded mortar joints may be re-pointed, with some localized brick units requiring replacement. The scaled concrete observed at the vehicle entrance curb and stair landings is attributed to deicing salts and do not require repair at this time. The isolated corroded surfaces of the vehicle barrier cables may be cleaned and coated as part of the overall maintenance plan for the deck.

7.0 RECOMMENDATIONS

Based on our observations and our experience with similar parking garages, WJE offers the following categorized recommendations for your consideration.

7.1 Near-Term Repair Recommendations

WJE recommends that the following repair items be completed in the near future (within the next 1 to 2 years). These recommendations are intended to minimize water infiltration and drainage issues and extend the service life of the parking structure.

1. Concrete Structure Repairs
   a. Localized repairs (both partial and full depth) of unsound concrete at the elevated slabs.
   b. Isolated slab post-tensioned tendon and anchorage repairs.
2. Waterproofing Component Repairs
   a. Replace pre-molded expansion joint seals (at Tiers A through Tier 3). *
   b. Replace construction joint sealant at intermediate PT anchorages. *
   c. Rout and seal isolated cracks and replace failed sealant at previous cracks in the elevated slabs.
   d. Install traffic-bearing membrane over internal PT anchorages located at expansion joints and construction joints, and over isolated PT tendon repair areas.
   e. Apply concrete slab penetrating sealer at elevated levels.
   f. Install improved waterproofing and flashing systems at the pedestrian bridges.
   g. Replace deteriorated horizontal lines at floor drains. *
   h. Inspect and clean drain lines as part of drain work. *
3. Facade and Miscellaneous Repairs
   a. Remove loose brick fragments at copings and verify all coping units are secure (not loose). *
   b. Repair brick masonry distress at east stair towers (vertical cracking and outward displacement). Coordinate with waterproofing efforts at pedestrian bridges.

These repairs may be phased if needed to accommodate occupancy, schedule, or budgetary concerns. The highest priority repair items are indicated with an asterisk (*).

7.2 Long Term Repair Recommendations

WJE recommends that the following repairs be completed within the next 3 to 5 years. These recommendations are intended to address localized structural deterioration, minimize water infiltration, and improve the durability of the parking structure.

1. Structure Repairs
   a. Localized repairs (both partial and full depth) of unsound concrete at the elevated slabs, beams, columns, walls, and spandrels.
   b. Repair composite steel decking and supporting steel framing elements, including cleaning and painting steel elements, isolated concrete repairs, and replacing failed joint sealant and perimeter expansion joint seals.

2. Waterproofing Repairs
    a. Replace winged expansion joint seals (at Tier 4 roof level).
    b. Repair or replace existing traffic-bearing membranes, including areas outside elevators on Tier 1 and at the bases of the inclined columns.
    c. Install traffic-bearing membrane near drains.
    d. Replace the remaining construction joint sealant within the garage, including perimeter cove seals.
    e. Perform crack repairs at foundation walls where active water penetration is present.
    f. Install a supplementary drain in the large region of standing water (northeast end of Tier 1).
3. Facade, Stairwell, and Miscellaneous Repairs
    a. Replace/repair deteriorated brick coping units.
    b. Repoint deteriorated brick mortar and clean efflorescence staining on brick surfaces.
    c. Rout and seal isolated cracks within slab on ground (Tier B).

7.3 Maintenance Recommendations

WJE recommends that the following maintenance items be completed on a regular basis, or as indicated.

1. Utilize snowplows with shoes, rubber tips, or small skis to prevent damage to the traffic-bearing membrane and perform the plowing in a manner that minimizes impacts. Do not store plowed snow on the supported levels.
2. Avoid *excessive* de-icing salt applications.
3. Assess the traffic-bearing membrane on an annual basis in the spring to identify and repair de-bonded areas and scrapes related to snow plowing operations from the previous years.
4. Periodically assess the penetrating sealers and reapply as needed.
5. Remove accumulated debris and clean floor drains on a bi-annual basis.
6. Each spring, power wash and clean the deck surfaces to remove debris and the accumulation of deicing salts.
7. Periodically assess vehicle barrier cables and repair as needed.
8. Periodically inspect overhead concrete surfaces and remove loose or unsound concrete.
9. Periodically assess and perform concrete repairs, as needed.

8.0 OPTION OF PROBABLE COSTS

8.1 Repair Project Cost

As shown in Appendix A, the probable construction cost to address the near-term repair recommendations (within the next 1 to 2 years) is on the order of $1,030,000. In addition, the probable cost to implement the remaining long-term recommendations (within the next 3 to 5 years) is approximately $500,000. This estimate includes a 15 percent contingency and a 10 percent budget for engineering, testing, and inspection. Based on our experience with similar repair projects, WJE believes it is prudent to include a contingency to accommodate unforeseen conditions that are encountered during repair construction.

The majority of the unit costs contained in the construction cost estimate are based on costs for similar work on previous concrete repair projects located in the Midwest region. Repair quantities are based on the current level of deterioration, and unit prices are in current dollars. Both are subject to increase over time. With regard to construction costs specifically, an increase of 3 percent per year is recommended to account for inflation. Actual costs will depend on a number of factors, including the bidding environment and owner-provided constraints. Please also keep in mind that the COVID-19 pandemic has made construction pricing and scheduling less predictable, and its influence is not accounted for in this cost estimate.

These cost estimates assume that all of the work recommended for each phase (near-term and long-term) will be performed during one construction project each (i.e., one large project to address the near-term items and one large project to address the long-term items). It is possible, and may be preferable to the owner, to perform the repairs in smaller work areas and over multiple years, or in a prioritized manner, in the event that funding is limited, or parking spaces are not available. While smaller work areas occupy fewer parking spaces, an increase in both the duration and overall cost for the repair project should be anticipated. Similarly, cost efficiencies may be realized if all the recommended repairs are performed within large one near-term project.

8.2 Expected Maintenance Costs

This parking structure is nearly 35 years old. Given the exposure to moisture and deicing salts, concrete distress related to corrosion of the embedded reinforcement should be expected throughout the life of the parking structure. In particular, loose concrete removal, periodic sealant and expansion joint seal replacement, and traffic-bearing membrane repairs should be anticipated. Regular repairs and maintenance can decrease the rate of deterioration and increase the longevity of the parking structure. Therefore, WJE recommends that an annual budget be established for such repairs and maintenance. In addition, a significant concrete repair and waterproofing project should be anticipated every 5-10 years for the remaining life of the parking structure.

Maintenance and repair costs of parking structures increase exponentially over time due to exposure to aggressive environments. Maintenance of the structural steel, concrete, and waterproofing components of this garage should be expected. For this 280,000 square foot parking structure, we recommend a budget of approximately $300,000-400,000 every 5 years, increasing as the structure ages.

9.0 CLOSING

WJE performed an assessment of the Chester Street parking structure in Birmingham, Michigan, including a visual survey, investigative openings of the post-tensioned system, and materials testing. Based on the findings WJE provided repair and maintenance recommendations and presented our opinion of the probable repair costs for budgeting purposes. At your request, and under separate authorization, WJE can prepare construction documents to implement the recommended repairs.

We appreciate the opportunity to be of continued service to The City of Birmingham. If you have any questions, please feel free to contact us.

FIGURES

Figure 1. Peeling paint and corroded steel deck on underside of slab at elevator entryway, Tier 2. Note concrete beam distress near region of water penetration.

Figure 2. Spall exposing corroded mild reinforcing steel.

Figure 3. Spall exposing corroded mild reinforcing steel.

Figure 4. Isolated cracks in line with transverse temperature and shrinkage reinforcing steel, and penetrate previous repair areas.

Figure 5. Isolated cracks, efflorescence, and corrosion staining at PT tendon anchorage repairs.

Figure 6. Isolated missing grout from grout pocket for slab PT anchorage.

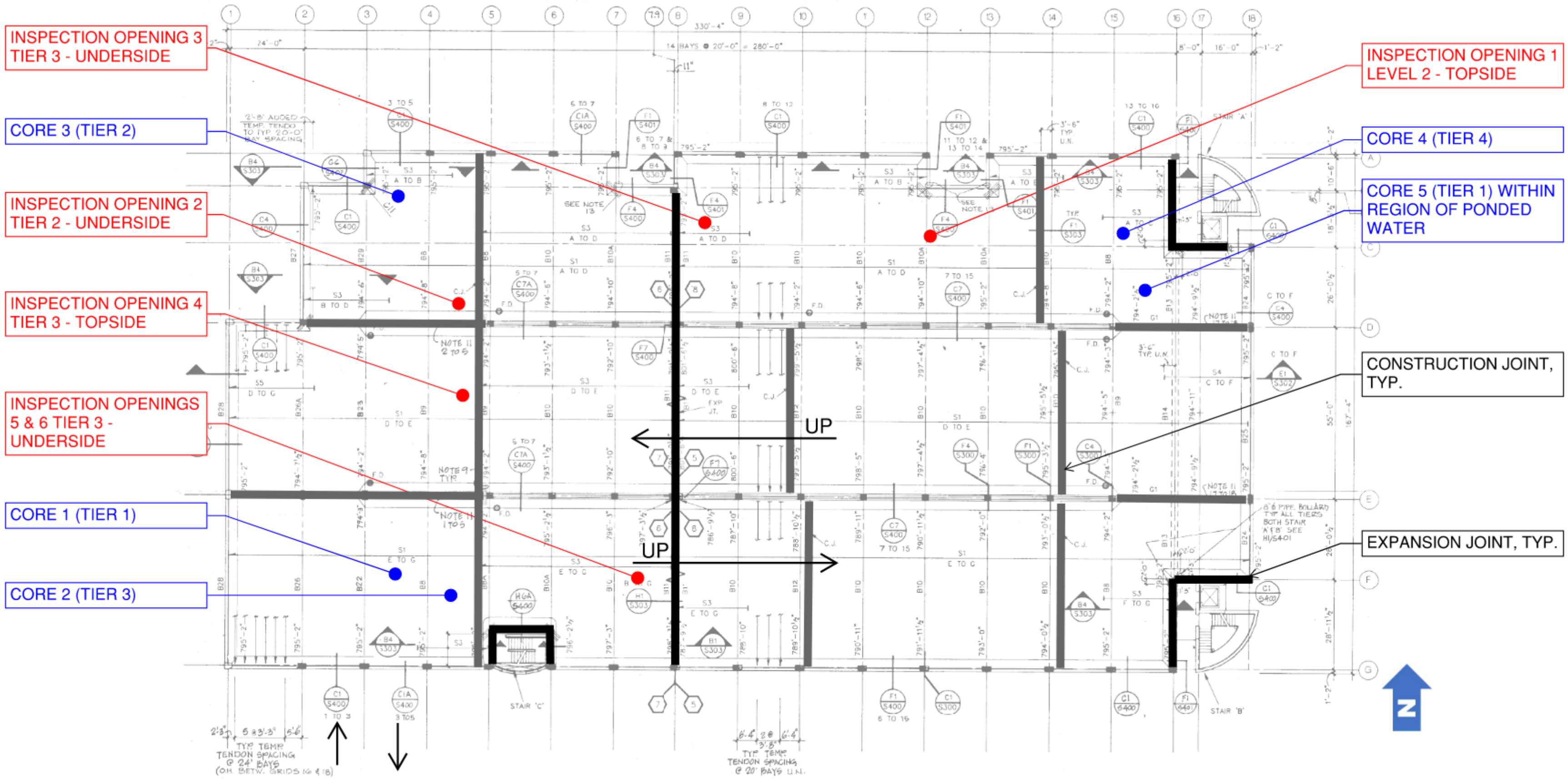

Figure 7. Approximate inspection opening locations.

Figure 8. Typical good tendon condition at Inspection Openings 1, 2, and 4.

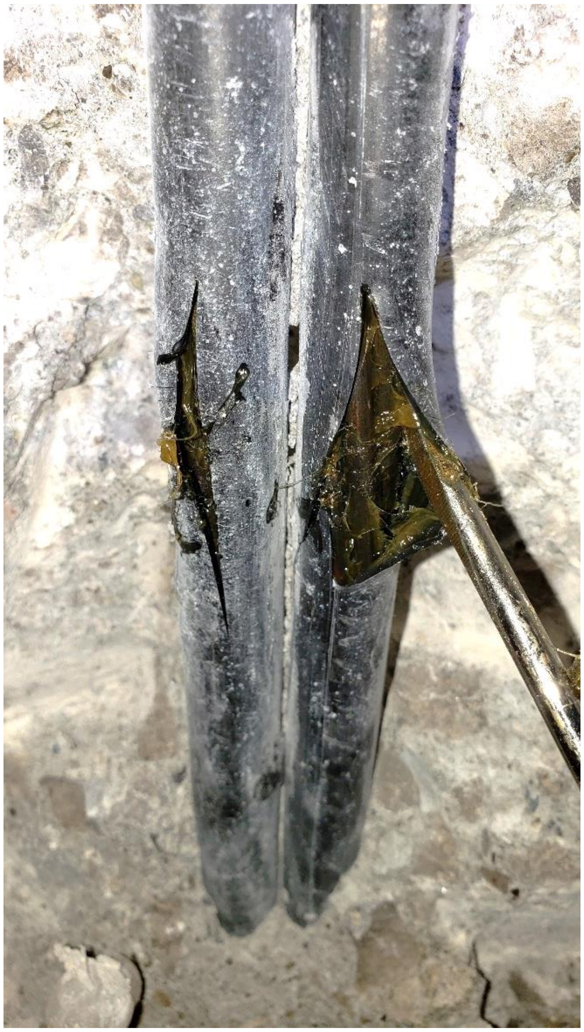

Figure 9. Exposed tendons at Inspection Opening 3.

Figure 10. Exposed tendons at Inspection Opening 5 with corrosion byproducts and emulsified grease.

Figure 11. Exposed tendons at Inspection Opening 6 with corrosion byproducts emulsified in the grease.

Figure 12. Water infiltration and corrosion staining at the underside of a deteriorated expansion joint.


Figure 13. Arrow indicates delamination on beam.


Figure 14. Unsound concrete patch at top of column, Tier 3.

Figure 15. Typical area of unsound concrete, cracking, and corrosion and efflorescence staining on arched beam supporting pedestrian bridge.

Figure 16. Isolated concrete honeycombing in concrete column.

Figure 17. Arrow indicates crack with adjacent unsound concrete near column bottom.

Figure 18. Delaminated coating and unsound concrete at base of inclined column supporting Tier 4.

Figure 19. Shrinkage cracks on exterior column, Tier 2.

Figure 20. Active water infiltration where spandrel beam bears on column.

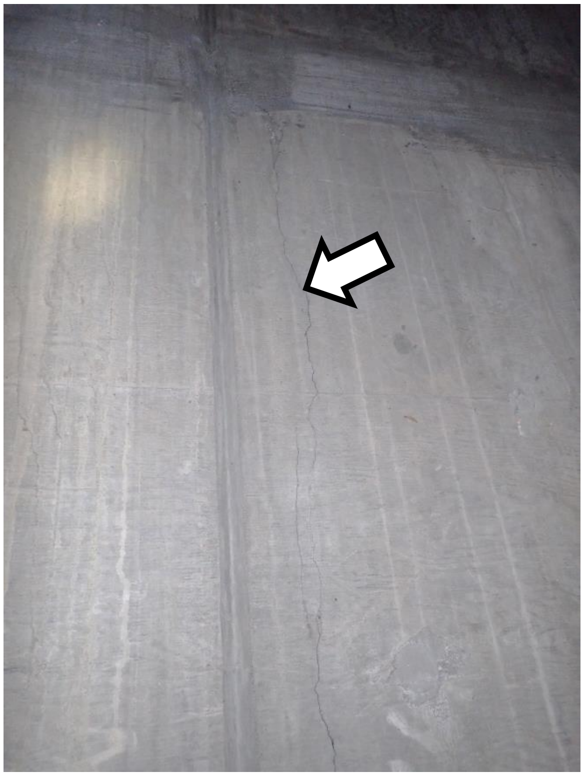

Figure 21. Arrow indicates vertical crack in foundation wall next to construction joint, Tier B.

Figure 22. Crack with efflorescence staining on perimeter wall.

Figure 23. Crack and unsound concrete on perimeter wall, north bay of Tier 1.

Figure 24. Adhesively failed edges of pre-molded expansion joint seal.

Figure 25. Arrow indicates adhesively failed pre-molded expansion joint seal near elevator shaft.

Figure 26. Efflorescence, moisture, and corrosion staining on the underside of a construction joint.

Figure 27. Worn and delaminated coating at Tier 4 expansion joint.

Figure 28. Delaminated coating at pedestrian bridge.

Figure 29. Delaminated coating at inclined column base, Tier 3.

Figure 30. Dashed line indicates standing water observed at Tier 2.


Figure 31. Through-corroded floor drainage pipe.


Figure 32. Cracked floor drainage pipe with corrosion product.


Figure 33. Corrosion product from floor drain bowl above.


Figure 34. Unsound and cracked partial-depth concrete repair at Tier B.


Figure 35. Cracked and spalled brick coping on concrete spandrel beam, Tier 2.


Figure 36. Efflorescence staining and eroded mortar joints at site wall.


Figure 37. Efflorescence, staining and eroded mortar joints, west elevation.


Figure 38. Vertical crack between southeast stairwell common wall and deck, Tier A.


Figure 39. Cracked and eroded concrete stair landing.


Figure 40. Typical de-icing salt placement noted during WJE's assessment.

Figure 41. Arrow indicates grease staining from grout pocket for vehicle barrier cable.

APPENDIX A. MATERIALS TESTING REPORT

City of Birmingham
Parking Garage Structural Assessment Program

Chester Parking Structure

180 Chester Street
Birmingham, MI 48009

Karla Salahshour
Senior Associate, Petrographer

LABORATORY REPORT

March 28, 2021

WJE No. 2019.6318.0

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CONTENTS

INTRODUCTION

Wiss, Janney, Elstner Associates, Inc. (WJE) completed laboratory testing on five concrete cores extracted from the Chester Parking Structure located at 180 Chester Street in Birmingham, Michigan. The Chester structure was constructed in 1988 and features six levels of parking. The structure consists of cast-in-place conventionally reinforced concrete columns supporting cast-in-place post-tensioned (PT) concrete beams and typical 5-1/2-inch thick slabs. Laboratory testing was completed on concrete cores that were extracted from the PT concrete slabs to characterize the material. The laboratory testing was completed as part of a larger investigation of the parking structure being performed by WJE's Detroit, Michigan office. The findings from this laboratory report will be used to assist in the repair recommendations for the parking structure.

SAMPLING

Five concrete cores were extracted throughout the parking structure and sent to WJE's Cleveland, Ohio laboratory for material testing. A summary of the core extraction locations is provided in Table 1. Photographs of the cores are provided in Figure 1. The cores were extracted vertically through the full thickness of the PT slab, and they ranged in length from 5-1/4 to 6-1/2 inches. The tops of the cores represent the exposed, wearing surface of the slab. The bottoms of all five cores are formed surfaces. No reinforcement was intersected by the cores.

Laboratory testing was performed on all five cores. A petrographic examination was only requested on Core 2 to characterize the concrete. Chloride ion content, water absorption, and carbonation tests were conducted on all five concrete cores. A summary of the testing performed is provided in Table 1.

Table 1. Summary of Chester Parking Structure Concrete Cores

Core ID	Core Extraction Location	Location Description	Testing Performed			
			Petrographic Examination	Chloride Ion Content	Water Absorption	Carbonation
1	Level 1 (Ground)	Drive lane, near entrance to parking structure		X	X	X
2	Level 3	Drive lane	X	X	X	X
3	Level 2	Outside of drive lane (in parking stall), away from drains, in covered region of slab		X	X	X
4	Roof (Level 4)	Drive lane		X	X	X
5	Level 1 (Ground)	Drive lane, within region of previously observed ponded water, return route out of garage		X	X	X

MATERIALS TESTING

Petrographic Examination

Methodology

Cursory examinations of the as-received core samples and saw-cut cross-sectional surfaces prepared for other laboratory testing were performed on all of the cores. A petrographic examination involving a more detailed examination of the material was conducted on Core 2 as part of the materials testing program. The petrographic studies were conducted in accordance with the procedures described in ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete*. Microscope examination and various tests conducted during the petrographic examination are designed to elicit specific information about the composition and condition of the concrete. The observations are interpreted to derive conclusions about quality, performance, and probable cause of various types of distress.

A 3/4-inch thick slab was cut along the longitudinal axis from the middle of Core 2 using a water-cooled, continuous-rim, diamond saw blade. The saw-cut surfaces of the slab were then lapped using discs of progressively finer abrasives to achieve a fine, matte finish suitable for examination with a stereomicroscope. Lapping exposes textural features such that the edges of air voids, cracks, and aggregate constituents can be more easily identified. A lapped cross-section of the core is shown in Figure 2. Fresh fracture surfaces were also prepared to study the physical characteristics of the concrete. Lapped and fracture surfaces were examined at magnifications up to 90X using a stereomicroscope. A thin section was prepared from the top 1-1/2 inches of the core to further assess paste and aggregate characteristics. The thin section was examined at magnifications ranging from 50X to 630X using a petrographic (polarized-light) microscope.

Unit weight was measured for representative portions of each core according to Section 9, Unit Weight and Loss of Free Water, of ASTM C1084, *Standard Test Method for Portland-Cement Content of Hardened Hydraulic-Cement Concrete*. The results are provided in Table 2.

Findings

The concrete in all of the cores appears compositionally similar based on a visual inspection of the saw-cut surfaces. The petrographic examination was conducted, however, only on Core 2. The cores represent a crushed limestone coarse aggregate and blended fine aggregate in a portland cement, air-entrained paste.

The coarse aggregate consists of crushed limestone particles with a maximum size of 1/2 to 3/4 inch. The particles are uniformly distributed and well graded. The particles are tan, brown, and gray in color, angular to sub-rounded in shape, and occasionally porous. A trace amount of expanded shale particles was observed on the lapped surfaces. The fine aggregate consists of a blend of calcareous and siliceous aggregates. A minor amount of fine aggregates, primarily chert particles, contain a darkened rim around their perimeter. While the majority of these rims are judged to be a naturally-occurring feature, a few rims were discontinuous adjacent voids and suggest their formation after mixing into the concrete (Figure 3). These rims may be a result of alkali-silica reaction. However, no distress such as cracking was associated with the rimmed chert fine aggregates.

The paste in the body of Core 1 is medium gray in color. The paste is hard and cannot be scratched using a copper probe in the body of the core. In thin section, residual portland cement was observed in the paste (Figure 4). Cement-sized limestone fines were also observed in thin section that may be a result of the use of calcareous aggregates. Textural features observed microscopically are consistent with a moderately low to low water-to-cementitious materials ratio. The paste is air-entrained, and voids were observed as both small, spherical entrained air voids and irregularly-shaped, entrapped voids. The total air content was estimated to be 7 to 9 percent (Figure 5). No secondary deposits were observed within the air voids. Bleed water voids were observed near the top of Core 2 (Figure 6). Irregularly-shaped voids up to 1/8-inch wide were observed throughout the body of the core, but they were not interconnected. Small (up to 1/8-inch wide) bugholes were observed on the bottom formed surfaces.

The top of all five cores are eroded to varying degrees and contain partially exposed coarse and fine aggregate. The fracture of near-surface porous aggregates and associated loss of overlying paste (i.e. pop-outs) was observed on the top surface of Core 2. Thin, elongate voids within the paste were observed along the top of Core 2 (Figure 7). No distress was observed beneath the top surface on the lapped cross sections. The paste within the top 1/2 to 1 inch of Core 2 is lighter gray in color than in the body of the core, suggesting a local increase in water-to-cement ratio.

Chloride Ion Content

Methodology

The water-soluble chloride ion contents were determined for the five cores at multiple depths. These depths were selected near the top surface (1/4 to 3/4 inch from the top) to determine if deicing salts, either applied directly to the slabs or carried in by vehicular traffic over time, penetrated into the concrete slab. The next depth (1-1/2 to 2 inches from the top) is located near the top level of mild reinforcing steel. A mid-depth (3 to 3-1/2 inches) slice was selected to serve as a baseline for the concrete. A depth near the bottom of the slab (depth varies due to slight differences in core lengths) was selected to determine if chlorides from spray from vehicular traffic or other sub-base conditions beneath the slab have penetrated into the concrete. These slices of concrete were saw-cut from one-half of each of the cores to be used for the testing.

The water-soluble chloride analysis was performed essentially according to ASTM C1218, *Standard Test Method for Water-Soluble Chloride in Mortar and Concrete*. The results are provided in Table 2.

Findings

Studies have shown that chloride contents above approximately 0.03 percent by mass of concrete, depending on the cement content, can promote corrosion of embedded uncoated steel in non-carbonated normal weight concrete in the presence of sufficient moisture and oxygen. Levels below this threshold may accelerate corrosion in carbonated concrete. The chloride contents measured at all five cores for the bottom four depths are all below this threshold. The chloride contents at the top of all five cores are in excess of this threshold.

The elevated chloride at the top surface of the cores indicates that the source of the chlorides is external to the concrete, such as from deicing salts. The chloride content is significantly elevated at the surfaces of Cores 2, 3, and 4 and only slightly elevated compared to the approximate corrosion threshold in Cores 1 and 5. Interestingly, Cores 1 and 5 were extracted near the entrance and exit to the parking structure at the ground level (Level 1).

Only a slight increase in chloride ion content was measured along the bottom of Cores 1, 4, and 5. This indicates that chloride penetration from the underside of the slabs is minimal.

Water Absorption

Methodology

During the laboratory testing, an assessment of the absorptivity of the top surface was requested to aid in the determination of a repair design for the parking structure. During this testing, water drops were applied to the as-received surface of each of the cores, and the shape and absorption of the water drop were recorded. Water drops were also applied at several locations on a laboratory-prepared fresh fracture surface of each core oriented perpendicular to the top surface, prepared for one of the core halves of each core. The absorptivity of each of the water drops was also recorded with depth from the top surface. Results are provided in Table 2.

Findings

The water drops applied to the surfaces of Cores 1 and 5 loosely beaded, but given time, the water drops were eventually absorbed. Loosely beaded refers to the water which remains in cohesive drop of water, rather than spreading on the surface, but not specifically a spherical shape. Water drops applied to the surfaces of Cores 2, 3, and 4 spread upon application and were absorbed, albeit at various rates (Figure 8). The absorptivity of the paste on the surface of Core 2 is likely related to the lighter gray colored paste, and elevated water-to-cement ratio, observed on the surface of that core during the petrographic examination.

On laboratory-induced fracture surfaces, a difference in absorptivity of the paste was observed to a maximum depth of 3/8 inch for Cores 1 through 4. For these four cores, water drops beaded and were noted to not be absorbed, either loosely or tightly, within the top portion of the core but not to greater depths. No difference in absorptivity along the fracture surface was observed for Core 5.

Water drops that bead and are not absorbed by the concrete paste may indicate the presence of a sealer that had been surface applied and penetrated into the concrete. The beaded nature of the water drops on the surfaces of Cores 1 and 5 suggests that remnants of such a material may remain on the surface of both cores, which may be related to the darker surface paste observed on the tops of Cores 1 and 5 compared to the other three cores. The presence of hydrophobic paste to a depth of 3/8 inch in Core 1 suggests that the material was able to penetrate into the concrete, whereas similar properties were not observed with depth in Core 5 suggesting a possible lack of penetration in that core, which may be related to differences in water-to-cement ratio as well as many other factors.

The lack of the beading of the water drops applied to the surfaces of Cores 2, 3, and 4 combined with the hydrophobic nature of the paste with depth from the surface in the three cores suggests a material had penetrated into the near-surface region of the concrete but has since deteriorated from the top surface in these areas.

Carbonation Depth

Methodology

One half of each of the five cores was fractured longitudinally in the laboratory for the carbonation studies. The fracture surface was blown free of debris using compressed air and treated with phenolphthalein indicator solution. The indicator solution will turn non-carbonated paste purple; carbonated paste will remain unchanged. Paste that exhibits a light purple color is judged to be partially carbonated. Carbonated paste loses its natural passivation of the embedded, uncoated reinforcing steel due to the reduction in pH of the paste. In the presence of moisture and oxygen, the steel is susceptible to corrosion. The depth of paste carbonation from the top and bottom surfaces are provided in Table 2.

Findings

The maximum depth of carbonation from the top surface of the concrete slab was 1/2 inch. The depth of carbonation from the top surface has not yet reached the assumed depth of mild reinforcing steel (at least 1-inch deep).

The maximum depth of fully carbonated paste from the bottom surface was 1/2 inch in Cores 2 and 4, although a depth of 3/4 inch of partially carbonated paste was measured for Core 1. The depth of cover for the PT strands along the length of the slabs was not reported to the laboratory, but it is assumed that the depth of carbonation from the bottom surface is not yet nearing the reinforcement. The location and condition of the PT strands are being investigated by WJE's Detroit office.

Table 2. Summary of Material Testing

Core ID	Core Length (inch)	Unit Weight (pcf)	Chloride		Water Absorption Description[1]	Carbonation	
			Depth from Top Surface (inch)	Water-Soluble Chloride (% by mass of sample)		From Top Surface (inch)	From Bottom Surface (inch)
1	6	140	1/4 - 3/4	0.032	Top - water drop loosely beaded, moderately absorptive	3/8	3/4 (partial)
			1 1/2 - 2	0.007			
			3 - 3 1/2	<0.003			
			5 1/4 - 5 3/4	0.008	FF - hydrophobic to 3/8 inch		
2	5-1/4	144	1/4 - 3/4	0.242	Top - water spread, slowly absorbed	1/2	1/2
			1 1/2 - 2	0.008			
			3 - 3 1/2	<0.003			
			4 1/2 - 5	0.004	FF - hydrophobic to 1/4 inch		
3	5-3/4	145	1/4 - 3/4	0.102	Top - water spread, absorbed	5/16	3/8
			1 1/2 - 2	<0.003			
			3 - 3 1/2	<0.003			
			4 3/4 - 5 1/4	<0.003	FF - hydrophobic to 3/16 inch		
4	5-1/2	148	1/4 - 3/4	0.184	Top - water spread, rapidly absorbed	3/8	1/2
			1 1/2 - 2	0.007			
			3 - 3 1/2	<0.003			
			4 3/4 - 5 1/4	0.007	FF - hydrophobic to 3/8 inch		
5	6-1/2	148	1/4 - 3/4	0.038	Top - water drop loosely beaded, slowly absorbed	1/8	0
			1 1/2 - 2	<0.003			
			3 - 3 1/2	<0.003			
			5 1/2 - 6	0.008	FF - not hydrophobic		

[1] FF = fresh fracture surface prepared in the laboratory to which water was applied

FIGURES

Figure 1. The as-received appearance of the tops (upper left), bottoms (upper right), and sides (lower) of Cores 1 through 5 are pictured. In the lower image, Cores 1 through 5 are pictured from the left to right, respectively.

Figure 2. Lapped surface of Core 2. Residual copper left from a paste scratch hardness test is circled on the image.

Figure 3. A chert fine aggregate contains a discontinuous reaction rim adjacent the void (arrow).

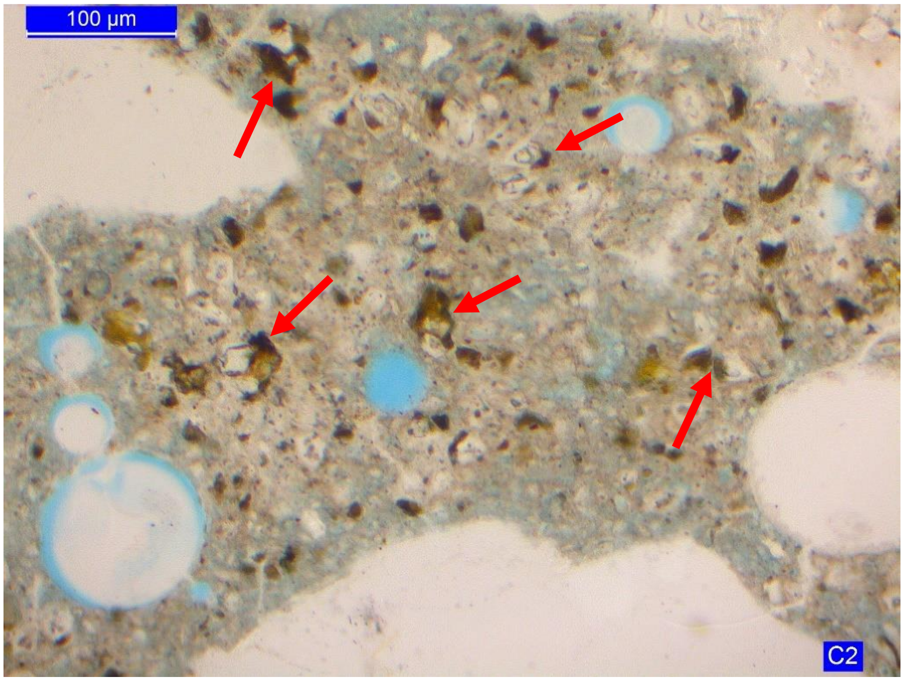

Figure 4. Residual portland cement particles (arrows) within the paste in Core 2.

Figure 5. Air void system in Core 2. Entrained air voids appear primarily as small, black circular areas due to the use of low-angle light illumination.

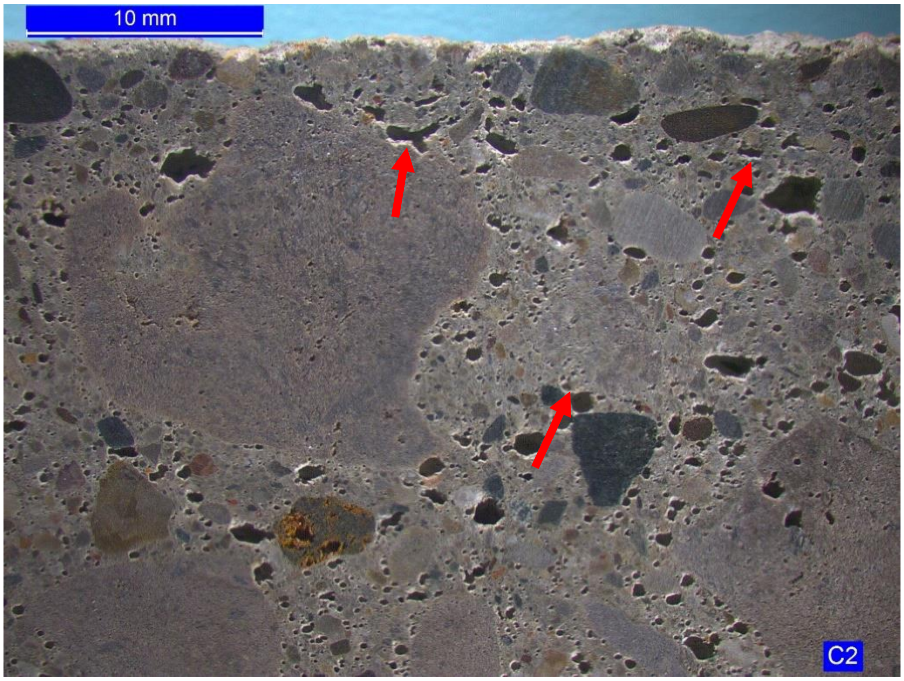

Figure 6. Irregularly-shaped bleed water voids (arrows) near the surface of Core 2. No distress is associated with the presence of these voids near-surface.

Figure 7. Thin, elongate voids on the top surface of Core 2.

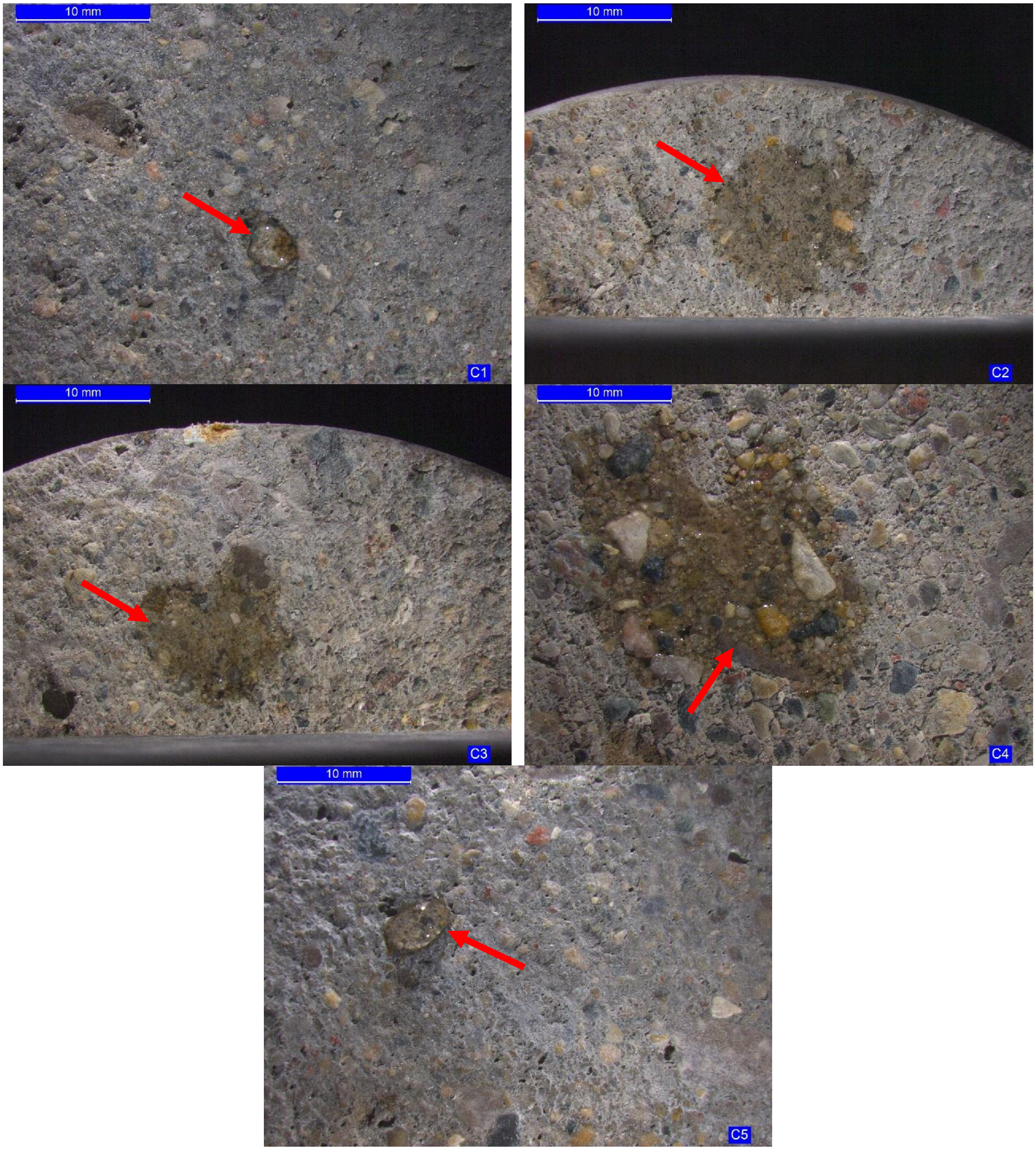

Figure 8. Water drops (arrows) applied to the surface are pictured for all five cores. Water drops beaded on Cores 1 and 5 and spread on Cores 2, 3, and 4.

APPENDIX B. OPINION OF PROBABLE COSTS

Near-Term Recommendations (within 1 to 2 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Localized concrete repairs in slab, full depth	1,500	SF	$ 80	$ 120,000
Localized concrete repairs in slab, partial depth	500	SF	$ 45	$ 22,500
P/T slab tendon repairs - allowance	1	LS	$ 75,000	$ 75,000
Replace pre-molded expansion joint seals (Tier A through Tier 3), including expansion joints near stairs*	1,200	LF	$ 125	$ 150,000
Replace control joint sealant at intermediate PT anchorages (N-S joints)*	2,000	LF	$ 6	$ 12,000
Rout and seal cracks in elevated slab and replace failed sealant at isolated cracks	750	LF	$ 6	$ 4,500
Install traffic bearing membrane at control joints, expansion joints, and PT tendon repair areas	36,000	SF	$ 5	$ 180,000
Apply concrete slab sealer on elevated levels	195,000	SF	$ 0.40	$ 78,000
Install waterproofing and flashing improvements at pedestrian bridges	2	LS	$ 8,000	$ 16,000
Replace deteriorated horizontal lines at floor drains and associated components*	150	LF	$ 90	$ 13,500
Inspect and clean lines as part of repair effort*	1	LS	$ 15,000	$ 15,000
Remove loose brick coping fragments and verify all brick coping units are secure (not loose)*	1	LS	$ 1,500	$ 1,500
Repair brick distress within east stair towers (vertical cracking and outward displacement). Coordinate with waterproofing efforts at pedestrian bridges.	2	LS	$ 24,000	$ 48,000
			Subtotal	**$ 736,000**
			General Conditions, Overhead and Profit (15%)	$ 110,400
			Project Contingency (15%)	$ 110,400
			Engineering/Testing/Construction Period Services (10%)	$ 73,600
			Total	**$ 1,030,400**

Long-Term Recommendations (within 3 to 5 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Structural Repairs				
Localized concrete repairs in slab, full depth	250	SF	$ 80	$ 20,000
Localized concrete repairs in slab, partial depth	100	SF	$ 45	$ 4,500
Partial depth concrete repair at beams, columns, walls, spandrels	1,000	SF	$ 90	$ 90,000
Repair composite steel decking and supporting steel framing elements	1	LS	$ 7,500	$ 7,500
Waterproofing Repairs				
Replace winged expansion joints at Tier 4 (roof)	275	LF	$ 125	$ 34,375
Install traffic bearing membrane outside elevators at Tier 1 and at column bases at inclined columns (small areas)	700	SF	$ 8	$ 5,600
Install traffic bearing membrane at drains (small areas)	350	SF	$ 8	$ 2,800
Replace remaining joint sealant on elevated levels, including perimeter cove seal.	5,000	LF	$ 6	$ 30,000
Crack repairs at foundation walls and perimeter walls where active water infiltration is present	150	LF	$ 35	$ 5,250
Add drain in region of standing water on northeast end of Tier 1	1	LS	$ 4,000	$ 4,000
Facade, Stairwell and Miscellaneous Repairs				
Replace deteriorated brick coping units in-kind ǂ	900	LF	$ 100	$ 90,000
Repoint deteriorated brick mortar	900	SF	$ 50	$ 45,000
Clean efflorescence (including exterior facade access)	1	LS	$ 4,000	$ 4,000
Rout and seal cracks on slab on ground and replace failed joint sealant (Tier B)	1,500	LF	$ 6	$ 9,000
			Subtotal	**$ 352,025**
			General Conditions, Overhead and Profit (15%)	$ 52,804
			Project Contingency (15%)	$ 52,804
			Engineering/Testing/Construction Period Services (10%)	$ 35,203
			Total	**$ 492,835**

* Highest priority of near-term repair items.

**Prices are based on current (2021) dollars.

ǂ See report discussion for alternative repair options.

City of Birmingham
Parking Garage Structural Assessment Program

North Old Woodward Parking Structure

333 North Old Woodward Avenue
Birmingham, Michigan 48012

FINAL REPORT
May 5, 2021
WJE No. 2019.6318

PREPARED FOR:
Mr. Scott Grewe
Operations Commander - Birmingham Police Department
City of Birmingham
151 Martin Street
Birmingham, Michigan 48012

PREPARED BY:
Wiss, Janney, Elstner Associates, Inc.
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WJE

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Parking Garage Structural Assessment Program

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CONTENTS

1.0 INTRODUCTION

As requested, Wiss, Janney, Elstner Associated, Inc. (WJE) completed limited condition assessments of the North Old Woodward, Park Street, Peabody and Chester parking structures. These assessments were performed with the intent to determine the current and future infrastructure needs in support of a capital improvement plan; the intention of the plan is to extend the useful life of the structures and to maintain the structural integrity to ensure the structure can support the code-prescribed loadings. This report summarizes our observations at the North Old Woodward Parking Structure, located at 333 North Old Woodward Avenue in Birmingham, Michigan, and provides recommendations for your consideration.

1.2 Background

WJE originally visited the site in July 2019 as a part of a previous limited structural assessment project. The City of Birmingham was considering modifications to the property on which the parking structure is located and, as a result, was interested in determining preliminary cost estimates to repair the existing parking structure. During an approximate 8-hour site visit, the assessment included a limited visual inspection of the accessible and exposed portions of the structural components, a limited visual inspection of the facade, and a limited sounding survey of portions of the structural components as summarized in WJE's *NOW Parking Structure Preliminary Assessment* report, dated July 5, 2019.

During the limited July 2019 assessment, WJE observed conditions that we identified as "Immediate Recommendations". Specifically, the recommendations pertained to loose overhead concrete and precast concrete facade panels of immediate concern. We recommended removing loose overhead concrete throughout the parking structure and facade to minimize the potential for concrete pieces to dislodge and impact pedestrians or vehicles. Additionally, we recommended stabilizing, repairing, or replacing the facade panels of immediate concern. Subsequently, WJE was requested to further document the facade panels, develop a temporary stabilization repair design and sketch, and provide a letter report to summarize our observations and to present our recommendations (*NOW Parking Structure Recommendations* dated July 26, 2019). The stabilization repairs were then performed by a contractor at select facade panels.

Following the implementation of the stabilization repairs, an existing facade panel connection, without supplemental stabilization installed, failed, resulting in the facade panel falling onto a drive lane below. It was observed that the existing connection failed due to advanced corrosion within the faced panel, unobservable and concealed when in its original installed position. Because of this, The City of Birmingham proactively elected to remove all existing precast concrete panels that comprise the facade of the parking structure. The facade panels served as the vehicle barrier system for the perimeter of each above ground level. Thus, the City of Birmingham requested that WJE design a new vehicle barrier system to be installed in order to maintain the existing level of safety within the garage with the facade panels removed. A steel cable-based barrier system was the method preferred by the City of Birmingham for this application, and the installation of the barrier system was completed in July 2020. As a part of the installation of the new barrier system, concrete repairs were performed at the slab edges along the entire structure perimeter.

2.0 STRUCTURE DESCRIPTION

The parking structure was constructed in 1966 and has five levels of parking with a centralized ramp system. The structural system on the supported levels consists of a two-way slab system comprised of reinforced concrete flat slabs supported on columns with drop panels. Level 1 is a reinforced concrete slab on ground, and Level 5 is uncovered rooftop parking. The supported slabs are approximately 15 inches thick, including a concrete topping that varies in thickness from three to six inches. The structure is square in plan with approximate dimensions of 200 feet by 200 feet, for a total area of 200,000 square feet of floor space between all levels. The facade at the corner towers is primarily brick masonry cladding with concrete masonry unit (CMU) backup; additionally, precast concrete units with an exposed aggregate finish extends from grade to the top of the corner towers, surrounding the windows and doors. At the time of the initial visual portion of the assessment, the remaining portions of the facade consisted of exposed aggregate precast concrete panels individually attached at each slab level; now, the panels have been removed and a prestressed cable vehicle barrier system is in place.

2.1 Document Review

WJE reviewed relevant sheets of the original construction drawings dated November 16, 1966 and authored by O'Dell, Hewlett & Luckenbach Associates and Architects as part of our assessment. Pertinent information is discussed within the observation sections below.

Based on our site visit observations, several past restoration projects have occurred at the building. WJE reviewed relevant sheets of repair construction drawings prepared by Walker Restoration Consultants dated March 11, 1991; February 16, 2004; and February 18, 2010. Pertinent information is discussed within the observation sections below. Documentation related to any other efforts was not provided to WJE for review.

3.0 FIELD ASSESSMENT

As part of the current structural assessment program, WJE visited the site multiple times in January 2020 to perform visual inspections of the accessible and exposed portions of the structure and the facade. WJE returned to the site in May 2020 to perform a delamination survey at representative locations. WJE returned to the site on February 26, 2021 to extract concrete cores for materials testing. A summary of pertinent observations follows.

WJE's scope included a limited sounding survey of the supported levels in accordance with *ASTM D4580 - Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding.* For this survey, areas of delamination were identified using the chain-drag method, localized hammer sounding, and use of a delamination wheel at select underside locations. In areas of sound concrete, these methods produce a clear, ringing sound, and when a delamination is encountered, a hollow, drum-like sound is produced. Between 25 and 50 percent of the total area for each floor was surveyed. Sounding of the underside of the slab with a delamination wheel was primarily limited to locations of previous repair and visible indications of potential concrete deterioration (e.g. at visible cracks, spalls).

A summary of pertinent observations follows.

3.1 Structural Components

3.1.1 Structural Floor Slabs

The condition of the structural floor slabs varied throughout the parking structure. The slabs were generally in serviceable condition with localized areas of distress as described below.

1. Spalled, loose, and unsound (delaminated) concrete is common on the underside of the elevated concrete slabs (Figure 1). The reinforcing bars exposed at locations of spalling are typically corroded and have low concrete cover (Figure 2).
2. With respect to the concrete sounding survey:
   a. During the sounding survey, WJE noted that the delaminations often extend beyond the region where the concrete deterioration is readily visible. Additionally, the sounding survey identified areas of unsound concrete that occur where there are no visible indications of concrete deterioration (Figure 3 and Figure 4).
   b. Based on the quantity of delaminated concrete found in the survey areas, the amount of delaminated concrete throughout the structure was estimated by extrapolation. The extrapolated results of the visual and sounding survey are shown in Table 1 below.

Table 1. Slab Top Surface Survey

Level	Total Area (Sq. Ft.)	Total Estimated Area of Delaminations (Sq. Ft.)	Total Estimated Percent of Delaminations
5	40,560	2,700	7
4	40,560	1,600	4
3	40,560	1,900	5
2	40,560	1,600	4

3. Cracks are present in many locations on the topside of the slabs, and in some locations, on the underside of the slabs.
   a. Typically, the cracks on the slab underside are relatively narrow (i.e., less than 0.015 inches wide). Corrosion staining and efflorescence (salt deposits) are present at many of the cracks (Figure 5). In some locations, where cracks commonly extend perpendicularly to the slab edge at the cantilevered portions of the slabs, spalled concrete and/or previous repair areas are in line with the cracks (Figure 6).
   b. Multiple cracks exist throughout the topside of the slabs including radial cracks near columns and crazing cracks throughout the structure (Figure 7 and Figure 8). Previously sealed cracks are present at many of the cracks on the topside of the slabs; however, the sealant is typically torn or exhibits adhesive failure (Figure 9).
4. There are many previous repairs on the topside and underside of the supported slabs.
   a. Of the 73 underside repair areas noted, approximately 33% of the repair areas are unsound, debonded, and/or have cracks with corrosion staining and efflorescence (Figure 10). The localized

repairs on the underside typically consist of shallow, partial depth, trowel-applied mortar. Unsound concrete also extends beyond the repair areas at some locations (Figure 11).

b. The topside repairs are generally in better condition than the underside repairs. Most of the topside repair areas are sound. However, most of the repair areas have shrinkage related cracks.

5. The edge of the concrete floor slab is exposed along the perimeter of the structure and on the interior at the ramps; the exterior edges were repaired as a part of the cable barrier system installation project. Spalls and cracks are commonly present at the interior slab edges, some of which are at failed previous repairs (Figure 12).

3.1.2 Columns

Rectangular columns with a rectangular drop panel support the elevated slabs. The columns and drop panels are generally in serviceable condition with localized concrete distress as outlined below.

1. Localized areas of unsound and cracked concrete exist on approximately 35 percent of the columns, typically occurring at the bottom corners at locations of shallow concrete cover (Figure 13). The areas of unsound concrete are typically less than two square feet each.
2. Previous repairs were found at several columns. Many of the column repairs are unsound or deteriorated (Figure 14).

3.1.3 Walls

Reinforced concrete walls exist along the perimeter of the structure on Level 1 and Level 5. Additionally, interior reinforced concrete walls exist surrounding three sides of the ramp on Level 5.

1. Limited, localized areas of spalled concrete and failed previous repairs exist in several locations. Corroded reinforcing steel was typically exposed at the spalled locations (Figure 15).
2. At approximately 30 locations on Level 5, exposed reinforcing steel exists at areas where there is low concrete cover (Figure 16).
3. Cracking with efflorescence exists at the wall above the ramp on Level 5 (Figure 17).

3.2 Waterproofing Components

WJE was on site during a rain event and documented active water leaks and water ponding (Figure 18, Figure 19, and Figure 20). Additionally, the following distress to waterproofing components were noted:

1. A traffic-bearing membrane is present on the curbs on level 5. The coating appears to be well bonded to the concrete, but there are localized tears and abrasions (Figure 21).
2. Sealant is present at the joints of the concrete slab topping and at previous crack repairs. Most of sealant exhibits adhesive failure (Figure 22).

3.3 Facade

The following pertinent observations were noted:

1. The steel lintel above the snow shoot vehicular door is corroding, and some of the brick masonry units above the corroding lintel at the ends are cracked (Figure 23)

2. Small areas of brick masonry, approximately four-square feet on average, are spalled near grade in approximately three locations (Figure 24)
3. Areas of the precast concrete panels were spalled and cracked in approximately six locations, primarily concentrated near the stair tower door thresholds (Figure 25).
4. The cove seals at the and sealant joints at the precast concrete panels were weathered and exhibit cohesive and adhesive failure (Figure 26).

3.4 Miscellaneous

WJE noted the following miscellaneous conditions:

1. The CMU blocks exhibits freeze-thaw deterioration in some locations (Figure 27).
2. Some of the stair tower cover plates are loose or not level with the surface of the slab, creating a potential tripping hazard (Figure 28)
3. The lighting in the parking structure in generally dim (Figure 29). Many areas of low light exist where lights contain burned out bulbs or the light is not functioning.
4. Several of the floor drains exhibit distress including cracked, displaced, or missing gratings. Additionally, some of the drains are clogged causing ponded water as noted above.

4.0 REPAIRS COMPLETED TO DATE

In an effort to take advantage of reduced occupancy during the facade removal construction, vehicle barrier installation, and the COVID-19 pandemic, the City of Birmingham approved a limited scope of repairs on May 18, 2020 completed by DRV Contractors, LLC, in addition to the previously approved construction related to the facade. At of the issuance of this report, the following repairs were completed:

- Removal of existing precast concrete facade panels on all elevations.
- Installation of new vehicle barrier system.
- Removal of loose concrete on the underside of slabs throughout the garage.
  - Note that there may be additional loose concrete since the removal of the loose concrete in 2020.
- Localized concrete repairs in the stair towers.
- Removal and replacement of the concrete drive at the southwest entrance.
- Removal of interior full height precast members, and replacement with new full height steel posts incorporated into the new vehicle barrier system.

5.0 MATERIALS TESTING

Four concrete cores were extracted from various locations in the structure and sent to WJE's Cleveland laboratory for materials testing. The locations of the cores are provided in Table 2 below. The lab studies included petrographic examination, water-soluble chloride analysis, water absorption testing and carbonation depth measurements. A summary of the findings is presented in this report section. See ***Appendix A*** for more testing information, figures, and discussion.

Table 2. Core Locations

Core	Core Location	Location Description
1	Level 2 (first supported level)	In drive lane in front of the top of the first ramp.
2	Level 3	In parking stall.
3	Level 4	In parking stall at a location of typical craze cracking.
4	Level 5 (roof level)	In drive lane, near a drain.

5.1 Petrographic Examination

Cursory examinations of the core samples were performed, and saw-cut, cross-sectional surfaces were prepared for laboratory testing. A petrographic examination involving more in-depth studies of the concrete material was conducted on Cores 3 and 4 as part of the materials testing program.

5.1.1 Concrete Overlay (Topping)

Each of the cores contained a topping concrete that comprised 2-1/4 to 6 inches of the top portion of the core. The topping concrete is compositionally similar in Cores 1, 2, and 3 and dissimilar to Core 4 based on a visual inspection of the saw-cut surfaces. The topping concrete represented a crushed limestone coarse aggregate and blended fine aggregate in an air-entrained paste. The topping concrete had complete initial contact with the substrate concrete and remained bonded during coring and sample preparation. The topping concrete had been applied to an irregular profile of the substrate concrete. No obvious bruising was observed in Core 3, but horizontal voids and separations and fractures within coarse aggregates were observed immediately below the bonding surface in Core 4. A core sample (Core 3) was extracted at the Level 4 slab where the typical cracking was observed. Cracks were oriented perpendicular to the top surface in the topping in this core. One of the cracks is 3 inches long, discontinuous along its in length, and passes around aggregates along the first 1-1/2 inches of the crack.

5.1.2 Concrete Substrate

The concrete substrate in all of the cores appears compositionally similar based on a visual inspection of the saw-cut surfaces. The cores contain blended river gravel coarse aggregate and siliceous sand fine aggregate in air-entrained Portland cement paste. The concrete substrate is in good overall condition due to the lack of distress observed in the examined cores.

5.2 Water Absorption

Water drop testing was performed to test the hydrophobicity (water repellency) of the top surface. Water drops applied to the surface of Cores 3 and 4 spread on the surface, although their absorption into the paste of the topping varied considerably. The water was slowly absorbed for Core 3 and rapidly absorbed for Core 4. The hydrophobic properties near the surface of these two cores suggests the penetration of a penetrating-type sealer that may have been applied to the surface. However, the differences in absorption on the top surface suggest such a material may have deteriorated over time.

In Cores 1 and 2, water drops applied to the surface loosely beaded and did not absorb. Loosely beaded refers to the drop of water which remains a cohesive drop of water, rather than spreading on the surface, but not specifically a spherical shape. A beaded water drop would be expected for a surface that had been treated with a sealer-like material (that remains on the surface).

5.3 Carbonation Testing

The high pH of uncarbonated concrete provides protective passivation of the embedded steel reinforcement. Carbonation is a chemical process that occurs in the cement paste of the concrete and lowers the pH of the concrete. The depth of carbonation increases over time and is accelerated at cracks or joints. When the carbonation front reaches the depth of reinforcing steel, the steel becomes more susceptible to corrosion because the passivation layer from the high pH of the concrete is no longer present. The depth of the carbonation for each core is shown in Table 2 of ***Appendix A***.

The maximum depth of complete paste carbonation from the top surface of the cores, within the topping, is 1/8 inch. Paste that is partially carbonated was measured to a maximum depth of 1/2 inch for Core 3. The depths of carbonation from the top surface, both fully and partially carbonated depths, have not yet reached the depth of reinforcing steel in the topping, measured to be 3-3/4 inch in Core 2.

The maximum depth of complete paste carbonation from the bottom surface was 1/4 inch. A depth of 5/16 inch was measured for partially carbonated paste in Core 2. This depth of carbonation has not yet reached the depth of intersected reinforcement, which has a clear cover distance of 1 inch in Core 3 and 7/8 inch in Core 4.

The depth of carbonation is less than the depth of the typical embedded reinforcing steel; thus, the increased potential for corrosion due to carbonated concrete is not a concern at this time. However, embedded steel elements in areas of low cover would be expected to experience and increased potential due to carbonated concrete at these depths, which may result in deterioration of the surrounding concrete.

5.4 Water-Soluble Chloride Testing

The purpose of the chloride analysis was to determine the current chloride content at various depths of the slab. The results are contained within Table 2 of ***Appendix A***. Studies have shown that chloride contents above approximately 0.03 percent by mass of concrete, depending on the cement content, can promote corrosion of embedded uncoated steel in non-carbonated normal weight concrete in the presence of sufficient moisture and oxygen. Levels below this threshold may accelerate corrosion in carbonated concrete. The chloride ion contents measured for the two top-most depths, except for Core 1, exceeded this threshold. The chloride ion contents measured at the surface were significantly elevated over the chloride ion content at the second depth, indicating a decrease in chloride with depth from the surface. This gradient suggests an external source of chloride, such as from deicing salts on the slab surface, as would be expected. The chloride ion contents along the bottom of the cores are below this threshold, although the content measured for Core 2 is nearing the threshold.

6.0 DISCUSSION

Overall, the parking structure is in serviceable condition but in need of repair, with multiple areas of distress and deterioration observed. The distress and deterioration have progressed such that repairs are warranted in the near future to maintain the condition of the parking structure. Durability improvements are also worth considering to extend the service life and improve the durability of the structure.

6.1 Concrete Deterioration

Concrete parking structures in Michigan are susceptible to deterioration due to their exposure to moisture, deicing salts, and volume changes. The primary cause of concrete deterioration in concrete parking structures is corrosion of embedded steel reinforcement. Over time, if moisture is allowed to penetrate cracks in the concrete, the embedded reinforcing steel may begin to corrode. Because the steel corrosion byproduct (i.e. rust) occupies a larger volume than the original steel, it is common for distress in the form of cracks, delaminations, or spalls to develop when the embedded steel corrodes. Both chloride contamination and carbonation of the concrete can increase the potential for corrosion of embedded reinforcing steel. While the carbonation testing indicates that the depth of carbonation has not yet reached the depth of reinforcing steel, the chloride testing performed indicates significantly elevated chloride ions at the tops of all of the cores, and the chloride content of the concrete exceeds the threshold to promote corrosion in the presence of sufficient moisture and oxygen. Additionally, chloride-laden moisture traveling through untreated cracks can increase corrosion of the embedded steel. Deterioration of exposed concrete elements as a result of corrosion can be reduced by protecting them from water exposure.

The concrete deterioration in this parking structure is primarily located at the underside and topside of the concrete slabs where reinforcing bars are located. This distress is associated with long-term exposure to moisture and chlorides that have penetrated cracks in the concrete. Areas with low concrete cover, such as many areas noted on the slab underside, are also more susceptible to corrosion of the embedded reinforcing steel. Continued corrosion of the embedded reinforcing steel and deterioration of the concrete could lead to additional spalls on the undersides of the slabs which present a potential hazard of loose concrete falling from above.

6.2 Slab Cracking

The cracks observed in the slabs are relatively common in reinforced concrete structures. In a two-way slab system, cracks on the slab underside generally occur at the middle of the slab span, and the cracks on the slab topside occur near or around columns. These cracks are typically attributed to restrained volume change of the concrete (i.e. initial shrinkage and/or temperature changes), flexural stresses and/or deflection of the slabs.

Cracks were commonly observed around the columns, and sometimes oriented in a radial direction (cracks originating at the column extending outward). Radial cracking in the negative moment region of the slab over columns is common and the result of flexural stresses in the top of the slab. This type of cracking is often observed in two-way reinforced concrete slab structures and does not present a significant structural concern at this time. While some of the cracks were in the radial direction, most of the cracks near the columns were crazing cracks similar to the cracks observed away from the columns.

The crazing cracks observed within Core 3 are consistent with an early-age cracking attributed to restrained volume change of the concrete. While the observed cracks are not of a structural concern, untreated cracks provide a path for moisture infiltration within the cracks, which can lead to freeze-thaw deterioration of the concrete. Additionally, untreated cracks provide a path for moisture infiltration within the cracks which can lead to the embedded reinforcing steel, further promoting corrosion of the reinforcing steel and subsequent spalling of the concrete.

6.3 Previous Structural Repairs

Based on information provided by the City of Birmingham to WJE, it has been about 12 years since the last large repair effort at the parking structure. Given that this structure has been in service for over 55 years, ongoing deterioration is expected, despite previous repair efforts to maintain the structure, and a repair effort to maintain and restore the structure should be expected every 5 to 10 years. It should be noted that during the field assessment, WJE observed locations of concrete distress that were noted in the 2010 repair documents prepared by Walker Restoration Consultants, but had not been repaired, indicating some of the proposed repairs may not have been performed at that time.

Based on our experience with similar structures and with the current condition of the parking structure, concrete distress identified during our field assessment could be addressed using typical concrete repair processes that have become standard in the repair industry. With regard to the concrete slabs, columns and walls, the extent and severity of deterioration is consistent with expectations for a fifty-year-old parking structure that has not undergone repairs in the last 12 years.

Areas of previous repair exist throughout the structure. Many of the repairs are cracked, debonded, and/or exhibit corrosion staining. Corroded steel within concrete repairs can be an indication of improperly performed repairs, such as not removing the corrosion from the reinforcing steel or not removing the chloride-laden concrete from around the reinforcing steel.

Over the past two decades, the fundamentals of concrete repair practices to address corrosion-related distress and provide durable concrete repairs have been well-documented by the American Concrete Institute (ACI) and the International Concrete Repair Institute (ICRI). These fundamentals include chipping concrete around partially exposed and corroded steel reinforcing bars, removal of corrosion products and chlorides, applying corrosion-inhibiting coatings to protect embedded steel, adequately preparing the concrete surface, and using suitable repair materials, among other considerations. Additionally, WJE has found that the form-and-pour repair technique results in more durable repairs than trowel-applied repairs. Therefore, we encourage using this technique over trowel-applied repairs in the future.

6.4 Waterproofing Components

In WJE's experience, the long-term durability of parking structures subjected to chloride-laden moisture from de-icing salts is extended when moisture is well managed throughout and prevented from absorbing into the structure. Good moisture management generally includes adequate drainage, installation of a protection system, and quickly addressing or preventing deficiencies in the protection system and sealants.

6.4.1 Joint Sealants

The typical expected service life for new sealants in parking structure applications is 5 to 10 years. The majority of the joints in the surface overlay (i.e. concrete topping) have sealant adhesive failures. The sealant in the previously treated cracks is also aged and failing. Based on these observations, the joint sealants are nearing, or in most locations have exceeded, their service life. The aged and failed joint sealants in the parking structure should be removed and replaced. Given the overall condition of the sealants, it would be prudent to replace all of the joint sealants rather than perform localized repairs at areas of failed sealant only.

6.4.2 Silane Sealer and Traffic-Bearing Membrane

Based on the water absorption testing results, it appears that a penetrating sealer was present on all levels but has deteriorated throughout the slabs in the structure. Furthermore, during the water absorption testing, water drops retained a cohesive bead and were not absorbed on the surface of Cores 1 and 2 but spread and were absorbed (although at different rates) on the surface of Cores 3 and 4. The paste in all four cores was observed to exhibit hydrophobic properties with depth from the surface, indicating that a penetrating material, such as a silane-sealer, may have been applied to the slabs in the past. The difference in absorption on the top surface indicates that the sealer has deteriorated at some locations of the parking structure. Although sealers make the concrete water-resistant, they do not completely stop the penetration of water. Sealers are not flexible materials and, as such, cannot bridge cracks. Narrower cracks can be made essentially watertight by making both faces of a crack hydrophobic, but some of the wider cracks would require routing and sealing with a flexible sealant to be made watertight.

Traffic-bearing membrane systems are the most common waterproofing system used on parking structures to extend the life of the structure. A traffic-bearing membrane has already been installed over the concrete curbs on Level 5, but localized areas of the membrane are in need of repair.

Currently, the supported slabs of the parking structure do not have vehicular traffic-bearing waterproofing systems, and the previously applied silane-sealer is deteriorated. If moisture can be kept out of the slab concrete, future corrosion activity should be reduced. Preventing or prohibiting bulk moisture from entering the concrete will minimize future concrete deterioration and extend the service life of the parking structure. Therefore, a waterproofing system is an important measure commonly used to greatly reduce the amount of moisture and chlorides that can enter the concrete.

Traffic-bearing membrane systems, also known as traffic coating systems, typically consist of multi-layer polyurethane or epoxy coating with integral aggregate broadcast for slip resistance. The bottom layer of the system generally provides the waterproofing and the upper layers contain the aggregate and protect the bottom layer and are considered wear courses. A membrane would provide a waterproof barrier on the top surface of the topping slab and would be flexible enough to bridge the cracks in this topping. Considering the distress observed throughout the supported slabs, it would be prudent for the City of Birmingham to consider installing a traffic coating on the supported levels to extend the life of the structure and increase the durability of the repairs, thereby extending the time between significant repair projects.

6.3 Facade

Some of the brick masonry units are cracked or spalled. The cracked masonry above the corroded steel lintel is indicative of rotation at the ends of the lintel, causing distress to the masonry. The spalled masonry is typically concentrated near grade and primarily attributed to freeze-thaw deterioration. The sealants at the concrete panel to masonry wall joints is beyond their useful service life and warrant replacement.

7.0 RECOMMENDATIONS

Based on our observations and our experience with similar parking garages, WJE offers the following categorized recommendations for your consideration.

7.1 Immediate Recommendations

The removal of loose overhead concrete throughout the parking structure and facade to minimize the potential for concrete or masonry pieces to dislodge and impact pedestrians or vehicles should be performed. This is generally performed with hand tools or small electric chipping hammers with the intent of removing loose concrete, rather than concrete chipping hammers used for concrete demolition. Additionally, the replacement of broken grates and refastening of stair tower cover plates should be performed to limit the potential tripping hazards.

7.2 Near-Term Repair Recommendations

WJE recommends that the following repair items be completed in the near future (within the next 1 to 2 years). These recommendations are intended to minimize water infiltration, address concrete distress, and extend the service life of the parking structure.

1. Concrete Repairs
   a. Partial-depth repairs on the top surface of the slabs and ramps.
   b. Partial-depth repairs on the underside of the slab and ramps.
2. Waterproofing and Drainage Improvements
   a. Rout and seal cracks and joints in slabs
   b. Install traffic bearing membrane on Level 5
   c. Inspect and clean drain lines
3. Masonry Repairs
   a. Replacement of deteriorated concrete masonry units in the stair towers.

7.3 Long-Term Repair Recommendations

WJE recommends that the following repairs be completed within the next 3 to 5 years. These recommendations are intended to address structural deterioration, improve the waterproofing systems, and address observed distress within the facade.

1. Concrete Repairs
   a. Partial-depth repairs on the top surface of the slabs and ramps.
   b. Partial-depth repairs on the underside of the slab and ramps.
   c. Partial-depth repairs at columns.
   d. Partial-depth repairs at concrete walls.
2. Waterproofing Improvements
   a. Install traffic bearing membrane on Levels 2, 3 and 4
   b. Replace sealant at cove seal joists
3. Masonry Repairs
   a. Localized repointing of the clay masonry veneer.
   b. Replacement of cracks and spalled brick masonry units.
   c. Cleaning and coating of the corroded steel lintels.

7.4 Maintenance Recommendations

Due to their use and exposure, it is inevitable that parking decks in Michigan will deteriorate with time and require maintenance and repairs. In our experience, development of a long-term maintenance plan, including periodic inspections and repairs, has many benefits. Developing an understanding of the "health" of the structure and how that "health" changes over time allows you to better anticipate and control the flow of the repairs. Addressing deteriorated conditions early in their life span significantly reduces the risk of unexpected (and costly) large scale repairs in the future.

WJE recommends the following maintenance items to be completed on a regular basis or as indicated.

1. Do not store plowed snow on the supported levels and utilize snowplows with shoes, rubber-tipped blades, or small skis to prevent damage to the waterproofing system.
2. Remove accumulated debris and clean floor drains on a bi-annual basis.
3. Periodically assess the penetrating sealers and re-apply as needed.
4. Each spring, power wash and clean the deck surfaces to remove debris and the accumulation of deicing salts.
5. Periodically inspect overhead concrete surfaces and remove loose or unsound concrete.
6. Periodically inspect the cable barrier system for corrosion, deflection of the cables, or other distress conditions.
7. Periodically assess and perform concrete repairs, as needed.

8.0 OPINION OF PROBABLE COSTS

8.1 Repair Project Cost

As shown in Appendix B, the probable construction cost to address the immediate and near-term repair recommendations (within the next 1 to 2 years) is on the order of $1,500,000. In addition, the probable cost to implement the remaining long-term recommendations (within the next 3 to 5 years) is approximately $1,900,000. This estimate includes a 15 percent contingency and a 10 percent budget for engineering, testing, and inspection. Based on experience with similar repair projects, WJE believes it is prudent to include a contingency to accommodate unforeseen conditions that are encountered during repair construction.

The majority of the unit costs contained in the construction cost estimate are based on costs for similar work on previous concrete repair projects located in the Midwest region. Repair quantities are based on the current level of deterioration and unit prices are in current dollars. Both are subject to increase over time. With regard to construction costs specifically, an increase of 3 percent per year is recommended to account for inflation. Actual costs will depend on a number of factors, including the bidding environment and owner-provided constraints. Please also keep in mind that the COVID-19 pandemic has made construction pricing and scheduling less predictable, and its influence is not accounted for in this cost estimate.

These cost estimates assume that all of the work recommended for each phase (near-term and long-term) will be performed during one construction project each (i.e., one large project to address the near-term items and one large project to address the long-term items). It is possible, and may be preferable to the owner, to perform the repairs in smaller work areas and over multiple years, or in a prioritized manner, in the event that funding is limited, or parking spaces are not available. While smaller work areas occupy fewer parking spaces, an increase in both the duration and overall cost for the repair project should be anticipated. Similarly, cost efficiencies may be realized if all the recommended repairs are performed within one large near-term project.

8.2 Expected Maintenance Costs

This parking structure is nearly 55 years old. Given the exposure to moisture and deicing salts, concrete distress related to corrosion of the embedded reinforcement should be expected throughout the life of the parking structure. In particular, loose concrete removal, periodic sealant and expansion joint seal replacement, and penetrating sealer and traffic bearing membrane repairs should be anticipated. Regular repairs and maintenance can decrease the rate of deterioration and increase the longevity of the parking structure.

Therefore, WJE recommends that an annual budget be established for such repairs and maintenance. In addition, a significant concrete repair and waterproofing project should be anticipated every 5-10 years for the remaining life of the parking structure. Maintenance and repair costs of parking structures increase exponentially over time due to exposure to aggressive environments. Maintenance of the concrete and waterproofing components of this garage should be expected. For this 200,000 square foot parking structure, we recommend a budget of approximately $500,000-$600,000 every 5 years, increasing as the structure ages.

9.0 CLOSING

WJE performed an assessment of the North Old Woodward parking structure in Birmingham, Michigan, including a visual survey and materials testing. Based on the findings, WJE provided repair and maintenance recommendations, and presented our opinion of the probable repair costs for budgeting purposes. Drawings and specifications should be prepared for the recommended repairs by a licensed professional engineer familiar with parking structure repairs. At your request, WJE can provide a proposal to prepare construction documents to implement the recommended repairs.

We appreciate the opportunity to be of continued service to The City of Birmingham.

FIGURES


Figure 1. Unsound (delaminated) concrete on a slab underside.


Figure 2. Spalled concrete with exposed corroded reinforcing bars.

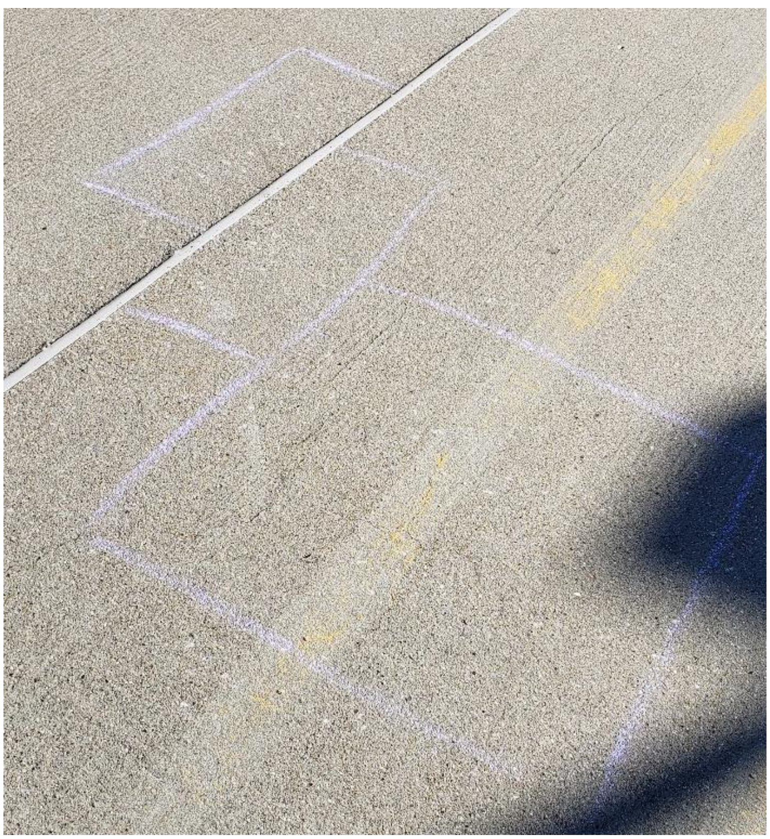
Figure 3. Unsound (delaminated) concrete outlined in chalk. Note that there are no visible indications of concrete deterioration.

Figure 4. Partial overview of Level 5 with multiple locations of unsound concrete outlined in chalk.

Figure 5. West elevation during slab edge repairs. Note the multiple cracks with efflorescence. A few locations are indicated by arrows. This photo was taken during the installation of the cable barrier system.

Figure 6. Previous concrete repair and unsound concrete (indicated by the arrow) in line with a crack with efflorescence.

Figure 7. Radial cracks extending from a column.

Figure 8. Example of crazing cracks on the top surface of a concrete slab.


Figure 9. A crack propagating beyond an existing sealed crack. Note a location adhesion failure indicated by the arrow.

Figure 10.Unsound (delaminated) previous repair.

Figure 11. Unsound (delaminated) previous repair with unsound concrete extending beyond the previous repair area.

Figure 12. Unsound and cracked previous repairs at the interior slab edges.

Figure 13. Unsound concrete at column corners.

Figure 14. Cracked previous repair.

Figure 15. Spall at an exterior wall joint with an exposed corroded reinforcing bar indicated by the arrow.

Figure 16. Corrosion stains and exposed corroded reinforcing bars at areas of low concrete cover.


Figure 17. Cracking with efflorescence at the wall corner above the top ramp.


Figure 18. Ponding water on roof slab at a concrete curb.


Figure 19. Ponding water at the ramp leading to Level 5.


Figure 20. Active water leak at a previous repair area.

Figure 21. Area of membrane scrapes at a concrete curb on Level 5.

Figure 22. Joint seal adhesive failure in a topping slab joint.

Figure 23. Corroding steel lintel and cracked brick masonry at ends (indicated by the arrow).

Figure 24. Spalled brick masonry at exterior of a stair tower.

Figure 25. Cracked and spalled concrete panel at entrance door.

Figure 26. Cohesive and adhesive failure of cove seal.

Figure 27. Deteriorated CMU blocks at the base of the stair.

Figure 28. Loose cover plate with gap.

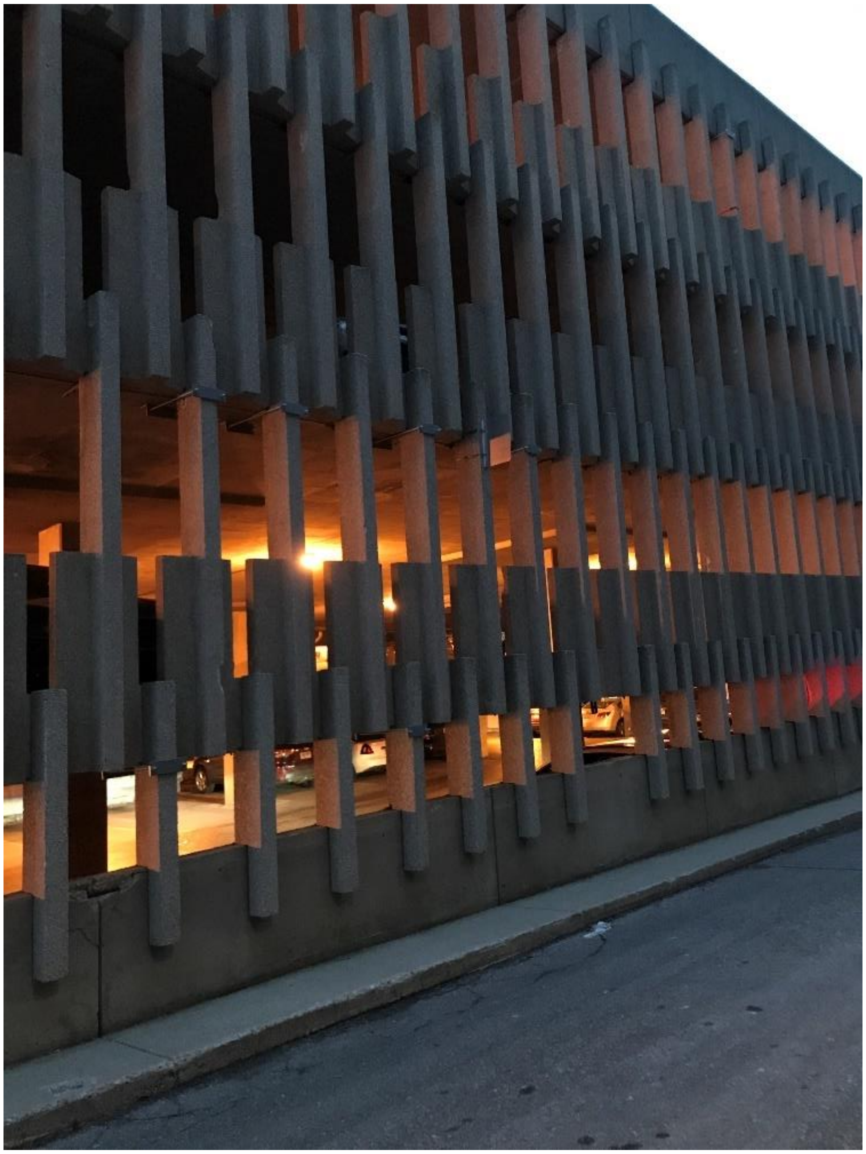

Figure 29. South elevation. Note the dim light on Level 4. Photo taken prior to the removal of the facade panels.

APPENDIX A. MATERIALS TESTING REPORT

City of Birmingham
Parking Garage Structural Assessment Program

North Old Woodward Parking Structure

333 North Old Woodward Avenue
Birmingham, MI 48009

Karla Salahshour
Senior Associate, Petrographer

LABORATORY REPORT

March 31, 2021
WJE No. 2019.6318.0

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CONTENTS

INTRODUCTION

Wiss, Janney, Elstner Associates, Inc. (WJE) completed laboratory testing on four concrete cores extracted from the North Old Woodward Parking Structure located at 333 North Old Woodward Avenue in Birmingham, Michigan. The North Old Woodward parking structure was constructed in 1966 and has five levels of parking with a centralized ramp. The structural system on the supported levels consists of reinforced concrete flat slabs supported on columns with drop panels. Level 1 is a reinforced concrete slab on ground, and Level 5 is uncovered rooftop parking. Laboratory testing was completed on concrete cores that were extracted from the elevated concrete flat slabs to characterize the material. The laboratory testing was completed as part of a larger investigation of the parking structure being performed by WJE's Detroit, Michigan office. The findings from this laboratory report will be used to assist in the repair recommendations for the parking structure.

SAMPLING

Four concrete cores were extracted throughout the parking structure and sent to WJE's Cleveland, Ohio laboratory for material testing. A summary of the core extraction locations is provided in Table 1. Photographs of the cores are provided in Figure 1. The cores were extracted vertically through the full thickness of the concrete floor slabs, and they ranged in length from 14-1/4 to 15-3/4 inches. The cores contained the original structural slab and a topping. The tops of the cores represent the exposed, wearing surface of the slab. The bottoms of all five cores are formed surfaces. Embedded steel reinforcement was intersected by the cores, and a summary of the reinforcement is provided in Table 2.

Laboratory testing was performed on all four cores. A petrographic examination was requested on only Cores 3 and 4 to characterize the concrete. Chloride ion content, water absorption, and carbonation tests were conducted on all four concrete cores. A summary of the testing performed is provided in Table 1.

Table 1. Summary of North Old Woodward Parking Structure Concrete Cores

Core ID	Core Extraction Location	Location Description	Testing Performed			
			Petrographic Examination	Chloride Ion Content	Water Absorption	Carbonation
1	Level 2	Drive lane in front of first ramp		X	X	X
2	Level 3	Interior parking stall		X	X	X
3	Level 4	Interior parking stall, location of typical cracking	X	X	X	X
4	Level 5 (roof)	Drive lane, near drain, near multiple unsound areas and previous repairs	X	X	X	X

MATERIALS TESTING

Petrographic Examination

Methodology

Cursory examinations of the as-received core samples and saw-cut cross-sectional surfaces prepared for other laboratory testing were performed on all of the cores. A petrographic examination involving more in-depth studies of the material was conducted on Cores 3 and 4 as part of the materials testing program. The petrographic studies were conducted in accordance with the procedures described in ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete*. Microscope examination and various tests conducted during the petrographic examination are designed to elicit specific information about the composition and condition of the concrete. The observations are interpreted to derive conclusions about quality, performance, and probable cause of various types of distress.

A 3/4-inch thick slab was cut along the longitudinal axis from the middle of Cores 3 and 4 using a water-cooled, continuous-rim, diamond saw blade. The saw-cut surfaces of the slabs were then lapped using discs of progressively finer abrasives to achieve a fine, matte finish suitable for examination with a stereomicroscope. Lapping exposes textural features such that the edges of air voids, cracks, and aggregate constituents can be more easily identified. A lapped cross-section of each core is shown in Figure 2. Fresh fracture surfaces were also prepared to study the physical characteristics of the concrete. Lapped and fracture surfaces were examined at magnifications up to 90X using a stereomicroscope. A thin section was prepared from near the exterior surface of the cores to further assess paste and aggregate characteristics. The thin section was examined at magnifications ranging from 50X to 630X using a petrographic (polarized-light) microscope.

Unit weight was measured for representative portions of the topping and concrete in each core according to Section 9, Unit Weight and Loss of Free Water, of ASTM C1084, *Standard Test Method for Portland-Cement Content of Hardened Hydraulic-Cement Concrete*. The results are provided in Table 2.

Findings

Topping

Each of the cores contained a topping concrete that comprised 2-1/4 to 6 inches of the top portion of the core. The topping concrete is compositionally similar in Cores 1, 2, and 3 and dissimilar to Core 4 based on a visual inspection of the saw-cut surfaces.

In Cores 1, 2, and 3, the topping contains crushed limestone coarse aggregate and blended calcareous and siliceous fine aggregate in an air-entrained paste. The coarse aggregate is angular to sub-angular in shape and tan, gray, and brown in color. The maximum sized coarse aggregate is 1/2 inch. Aggregates are uniformly distributed and well graded. Veneers of dark gray paste were frequently observed around the coarse aggregates. Chert particles contained within the fine aggregate occasionally contain a darkened, reaction rim around the perimeter of the particles. One chert fine aggregate was encased in discolored, stained paste (Figure 3). The discoloration of the paste and rims around particles are features characteristic of alkali-silica reaction (ASR), but no distress was associated with these particles. Polymeric fibers were observed in Cores 1, 2, and 3 based on laboratory-induced fracture surfaces used for carbonation testing. The paste is dark gray in color and can be scratched in Core 3. The paste contains residual portland cement particles and lumps of silica fume. Several silica fume lumps were internally cracked (Figure 4). A trace amount of fly ash spheres were identified but may represent a contaminant rather than a purposeful addition. Cement-sized limestone particles were also observed in the paste. The paste is air-entrained with small, spherical voids. The air content was estimated to be 6 to 8 percent with infrequent secondary deposits observed in air voids. Some voids were concentrated in chains and clusters, which, in combination with the dark-colored paste veneers around coarse aggregates, indicates the concrete may have been retempered. Microcracks throughout the paste were observed in the topping paste in Core 3 thin section.

The topping in Core 4 is also comprised of crushed limestone coarse aggregate and blended fine aggregate in a dark gray, air-entrained paste. The coarse aggregate has a maximum size of 3/8 inch and is composed of dark gray, angular to sub-angular particles. Chert particles within the fine aggregate also contained darkened rims around their perimeters but again, no distress was associated with these particles. However, alkali-silica gel was identified lining voids throughout the topping (Figure 5). No cracks were associated with these voids and the deposition of the gel. The paste is hard and cannot be scratched in Core 4. Residual portland cement particles were identified in thin section, but no silica fume lumps or fly ash were observed. The paste is marginally air-entrained, estimated to be 3 to 5 percent, consisting of primarily larger, spherical voids.

The topping concrete had been applied to an irregular profile of the substrate concrete (Figure 6). The amplitude of the surface profile was approximately 1/8 inch in Core 4 and 3/8 inch in Core 3. No primer or bonding agent-type of material was visible. The topping concrete had complete initial contact with the substrate concrete and remained bonded during coring and sample preparation. No obvious bruising was observed in Core 3, but horizontal voids and separations and fractures within coarse aggregates were observed immediately below the bonding surface in Core 4 (Figure 6).

Concrete Substrate

The concrete substrate in all of the cores appears compositionally similar based on a visual inspection of the saw-cut surfaces. The cores contain blended river gravel coarse aggregate and siliceous sand fine aggregate in air-entrained portland cement paste.

The coarse aggregate is composed of siliceous and calcareous natural gravel coarse aggregate. The particles are rounded in shape, multi-colored, uniformly distributed, and well graded. The maximum size particle is 1 inch. The fine aggregate consists of siliceous aggregates. A minor amount of aggregates, primarily chert particles within the fine aggregate and cherty limestone in the coarse aggregate, contain a darkened rim around their perimeter but are associated with minor, localized distress (Figure 7).

The paste in the body of Core 3 is medium gray in color and dark gray in Core 4, ranging from moderately hard to hard paste hardness. Residual portland cement particles were observed in thin section. Textural features observed microscopically are consistent with a moderate to moderately low water-to-cementitious materials ratio. The paste is air-entrained to a low level, and the total volume of air was estimated to be 2 to 4 percent in Core 3 and 4 to 6 percent in Core 4. Clustering of air voids was observed in Core 3. No secondary deposits were observed within the air voids.

Distress

Cracks were oriented perpendicular to the top surface in the topping in Core 3. One of the cracks is 3-inches long, discontinuous along its in length, and passes around aggregates along the first 1-1/2 inches of the crack (Figure 8). This is consistent with an early-age crack. Another crack was 1/4 inch long and oriented perpendicular to the top surface.

Core 3 also contained a spall along the bottom surface that intersected the plane of reinforcement. This spall was likely coring induced, as no other cracking or incipient spalls are present along the bottom of either Core 3 or Core 4.

Top Surface

The top of Core 3 is slightly irregular in profile due to minor preferential erosion of the paste (Figure 9). As a result, fine aggregates are partially exposed on the surface of the core, and the exposed paste is medium gray in color. The top surface of Core 4 is more significantly eroded with exposure of both coarse and fine aggregates (Figure 9). The exposed paste is light gray in color and readily absorptive.

Chloride Ion Content

Methodology

The water-soluble chloride ion contents were determined for the four cores at multiple depths. These depths were selected near the top surface of the topping (1/4 to 3/4 inch from the top) to determine if deicing salts, either applied directly to the slabs or carried in by vehicular traffic over time, penetrated into the concrete slab. The next depth (1-1/2 to 2 inches from the top) is located near the top level of mild reinforcing steel within the topping. A depth near the bottom of the slab (depth varies due to slight differences in core lengths), within the substrate concrete, was selected to determine if chlorides from spray from beneath the slab or sub-base conditions have penetrated into the concrete.

The water-soluble chloride analysis was performed essentially according to ASTM C1218, *Standard Test Method for Water-Soluble Chloride in Mortar and Concrete*. The results are provided in Table 2.

Findings

Studies have shown that chloride contents above approximately 0.03 percent by mass of concrete, depending on the cement content, can promote corrosion of embedded uncoated steel in non-carbonated normal weight concrete in the presence of sufficient moisture and oxygen. Levels below this threshold may accelerate corrosion in carbonated concrete. The chloride ion contents measured for the two top-most depths, except for Core 1, exceeded this threshold. The chloride ion contents measured at the surface were significantly elevated over the chloride ion content at the second depth, indicating a decrease in chloride with depth from the surface. This gradients suggests an external source of chloride, such as from deicing salts on the slab surface as would be expected. The chloride ion contents along the bottom of the cores is below this threshold, although the content measured for Core 2 is nearing the threshold.

Water Absorption

Methodology

During the laboratory testing, an assessment of the absorptivity of the top surface was requested to aid in the determination of a repair design for the parking structure. During this testing, water drops were applied to the as-received surface of each of the cores, and the shape and absorption of the water drops were recorded. Water drops were also applied at several locations on a laboratory-prepared fresh fracture surface of each core oriented perpendicular to the top surface. The absorptivity of each of the water drops was recorded with depth from the top surface. Results are provided in Table 2.

Findings

Water drops applied to the surface of Cores 3 and 4 spread on the surface, although their absorption rate into the paste of the topping varied considerably (Figure 10). The water was slowly absorbed for Core 3 and rapidly absorbed for Core 4. The absorption differences may be related to the curing, water-to-cement ratio, and/or dirt present on the surface of the cores. Interestingly, the paste on a fracture surface near the top surface of these three cores exhibited hydrophobic properties (i.e. water drops were not absorbed), to a maximum depth of 3/8 inch. The hydrophobic properties near the surface of these two cores suggests the penetration of a penetrating-type sealer that may have been applied to the surface. However, the differences in absorption on the top surface suggest such a material may have deteriorated from the surface over time.

In Cores 1 and 2, water drops applied to the surface loosely beaded and did not absorb (Figure 10). Loosely beaded refers to the water which remains in cohesive drop, rather than spreading on the surface, but not specifically a spherical shape. A beaded water drop would be expected for a surface that had been treated with a sealer-like material (that remains on the surface). The paste to a depth of 1/4 inch from the surface in Core 1 (and up to 1-3/8 inch deep along a crack in Core 2) exhibits hydrophobic properties, also consistent with penetration of a sealer.

Carbonation Depth

Methodology

One half of each of the four cores was fractured longitudinally in the laboratory for the carbonation studies. The fracture surface was blown free of debris using compressed air and treated with phenolphthalein indicator solution. The indicator solution will turn non-carbonated paste purple; carbonated paste will remain unchanged. Paste that exhibits a light purple color is judged to be partially carbonated. Carbonated paste loses its natural passivation of the embedded, uncoated reinforcing steel due to the reduction in pH of the paste. In the presence of moisture and oxygen, the steel is susceptible to corrosion. The depth of paste carbonation from the top and bottom surfaces are provided in Table 2.

Findings

The maximum depth of complete paste carbonation from the top surface of the cores, within the topping, is 1/8 inch. Paste that is partially carbonated was measured to a maximum depth of 1/2 inch for Core 3.

The depths of carbonation from the top surface, both fully and partially carbonated depths, have not yet reached the depth of reinforcing steel in the topping, measured to be 3-3/4 inch in Core 2.

The maximum depth of complete paste carbonation from the bottom surface was 1/4 inch. A depth of 5/16 inch of partially carbonated paste was measured in Core 2. This depth of carbonation has not yet reached the depth of intersected reinforcement, which has a clear cover distance of 1 inch in Core 3 and 7/8 inch in Core 4.

Table 2. Summary of Material Testing

Core ID	Topping		Concrete Slab		Presence of Rebar	Chloride		Water Absorption Description[1]	Carbonation	
	Thickness (inch)	Unit Weight (pcf)	Thickness (inch)	Unit Weight (pcf)		Depth from Top Surface (inch)	Water-Soluble Chloride (% by mass of sample)		From Top Surface (inch)	From Bottom Surface (inch)
1	3-1/8	143	12-5/8	156	1/2″ dia. at 5-3/4″ and 5/8″ dia. at 14″	$^{1}/_{4}$ - $^{3}/_{4}$	0.336	Top - water drop loosely beaded, not absorptive FF - hydrophobic to 1/4 inch	0 to 1/8	0 to 1/4
						1 $^{1}/_{2}$ - 2	0.013			
						15 - 15 $^{1}/_{2}$	0.008			
2	6	145	8-3/4	151	1/2″ dia. at 3-3/4″	$^{1}/_{4}$ - $^{3}/_{4}$	0.108	Top - water drop loosely beaded, not absorbed FF - hydrophobic to 1-3/8 inch (along crack)	1/8	5/16 (partial)
						1 $^{1}/_{2}$ - 2	0.073			
						14 - 14 $^{1}/_{2}$	0.017			
3	5-3/4	142	8-1/2	161	1″ dia. at 12″	$^{1}/_{4}$ - $^{3}/_{4}$	0.318	Top - water spread, slowly absorbed FF - hydrophobic to 1/4 inch	1/16 (full) 1/2 (partial)	1/4
						1 $^{1}/_{2}$ - 2	0.065			
						13 $^{1}/_{2}$ - 14	0.010			
4	2-1/4	--	13-1/4	151	5/8″ dia. at 14″	$^{1}/_{4}$ - $^{3}/_{4}$	0.097	Top - water spread, rapidly absorbed FF - hydrophobic to 3/8 inch	0	3/16 to 1/4
						1 $^{1}/_{2}$ - 2	0.043			
						14 $^{3}/_{4}$ - 15 $^{1}/_{4}$	0.007			

[1] FF = fresh fracture surface prepared in the laboratory to which water was applied

DISCUSSION

General Condition

The concrete cores examined during the studies are comprised of a topping concrete that had been placed over a prepared concrete substrate. Two different topping concrete mixtures were identified in the examined cores, whereas the concrete substrate was judged to be compositionally similar in all four cores.

The top surfaces of the cores represent eroded surfaces of varying degrees that would be expected for an exterior horizontal concrete element. Water drops retained a cohesive bead and were not absorbed on the surface of Cores 1 and 2 but spread and were absorbed (although at different rates) on the surface of Cores 3 and 4. The paste in all four cores was observed to exhibit hydrophobic properties with depth from the surface, indicating it is likely that a penetrating material such as a sealer may have been applied at some point. The minimal depth of paste carbonation from the top surface may also suggest a material is present inhibiting the penetration of atmospheric carbon dioxide. The difference in absorption on the top surface indicates that the sealer has deteriorated at some locations of the parking structure.

The topping concrete represented a crushed limestone coarse aggregate and blended fine aggregate in an air-entrained paste. One of the mixes contained portland cement and silica fume (with a trace amount of fly ash not judged to be a purposeful addition), and the other contained straight portland cement. The only significant distress observed in the cores was confined to the topping concrete. Two cracks were observed oriented perpendicular to the top surface, one of which was judged to be an early-age crack. Cracks allow for the penetration of moisture into the body of the concrete. However, a crack present in Core 2 (which was not examined petrographically during the studies) was noted to have hydrophobic properties to a depth of at least 1-3/8-inch along the crack face. This finding indicates that this crack was present prior to, and therefore coated with, the suspected application of a sealer-like material. While moisture can enter the crack space (if wide enough) and result in ice wedging distress, at this time, the moisture is unlikely able to saturate the paste along the crack face.

Alkali-silica gel was noted associated with primarily chert fine aggregates in the topping mixes. While distress exterior to these particles was judged to be minor, the gel formation indicates the potential for ASR and deleterious expansion. ASR is a reaction between alkalis provided by the cementitious paste and reactive forms of silica typically provided by the aggregates in the presence of sufficient moisture. Limiting the amount of moisture in the concrete can help mitigate the reaction. Localized ASR has been observed in previous projects associated with silica fume lumps. While lumps were commonly detected in one of the topping mixes, they were not observed to be associated with ASR.

The concrete substrate is in good overall condition due to the lack of distress observed in the examined cores. The concrete is composed of blended river gravel coarse aggregate and siliceous sand fine aggregate in an air-entrained, portland cement paste. No significant distress was observed within either of the two petrographically examined cores. While chert fine aggregates known to be potentially susceptible to ASR were identified, no features due to significant ASR-related distress were observed.

Corrosion Potential

The depth of paste carbonation was measured from both the top and bottom surfaces of the cores. The maximum depth of paste carbonation from the top surface was measured to be 1/8 inch for fully carbonated paste and 1/2 inch for partially carbonated paste. This depth of paste carbonation has not yet reached the depths of reinforcement intersected by the cores. Significantly elevated chloride ions were measured at the tops of all of the cores. The chloride ion significantly decreased with depth from the surface, although the chloride ion contents measured for Cores 2, 3, and 4 were still in excess of the threshold known to promote corrosion of embedded reinforcement in the presence of sufficient moisture and oxygen.

Complete paste carbonation was measured from the bottom surface to a maximum depth of 1/4 inch. Partial paste carbonation was measured to 5/16 inch in one core. This depth of carbonation has not yet reached the depth of intersected reinforcement; clear cover was measured to be 7/8 inch and 1 inch for two of the cores. The chloride content at the bottom of the cores was below the estimated threshold of corrosion initiation.

Repair Considerations

Repairs to the parking structure should consider the significantly elevated chloride ion content near the surface of the cores. The significance of elevated chloride ion content includes increased likelihood of ASR[1]; increased number of freeze-thaw cycles; and the potential for corrosion of embedded reinforcement. It is likely that the ingress of additional chlorides to greater depths has been halted by the presence of hydrophobic paste at or just below the top surface of the cores possibly due to the use of a penetrating-type sealer which restricted bulk moisture infiltration. Any reapplication of a similar penetrating sealer, which may be considered to reduce moisture infiltration, would require penetration into the cementitious paste, and as such, some level of surface preparation may be required at areas throughout the parking structure.

As previously noted, water is required for both freeze-thaw (requiring critical saturation of the concrete) and ASR deterioration mechanisms. Preventing or prohibiting bulk moisture from entering the concrete will extend the service life of the parking structure.

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[1] Chiu, Charles Yicheng, "The effects of chloride-based deicing chemicals on degradation of portland cement mortars with alkali reactive aggregate" (2016). Purdue University. *Open Access Dissertations*. 916. <https://docs.lib.purdue.edu/open_access_dissertations/916>.

FIGURES

Figure 1. The as-received appearance of the tops (upper), bottoms (middle), and sides (lower) of Cores 1 through 4 are pictured. In the upper and middle images, cores are pictured from left to right, and in the lower image, cores are pictured from top to bottom.

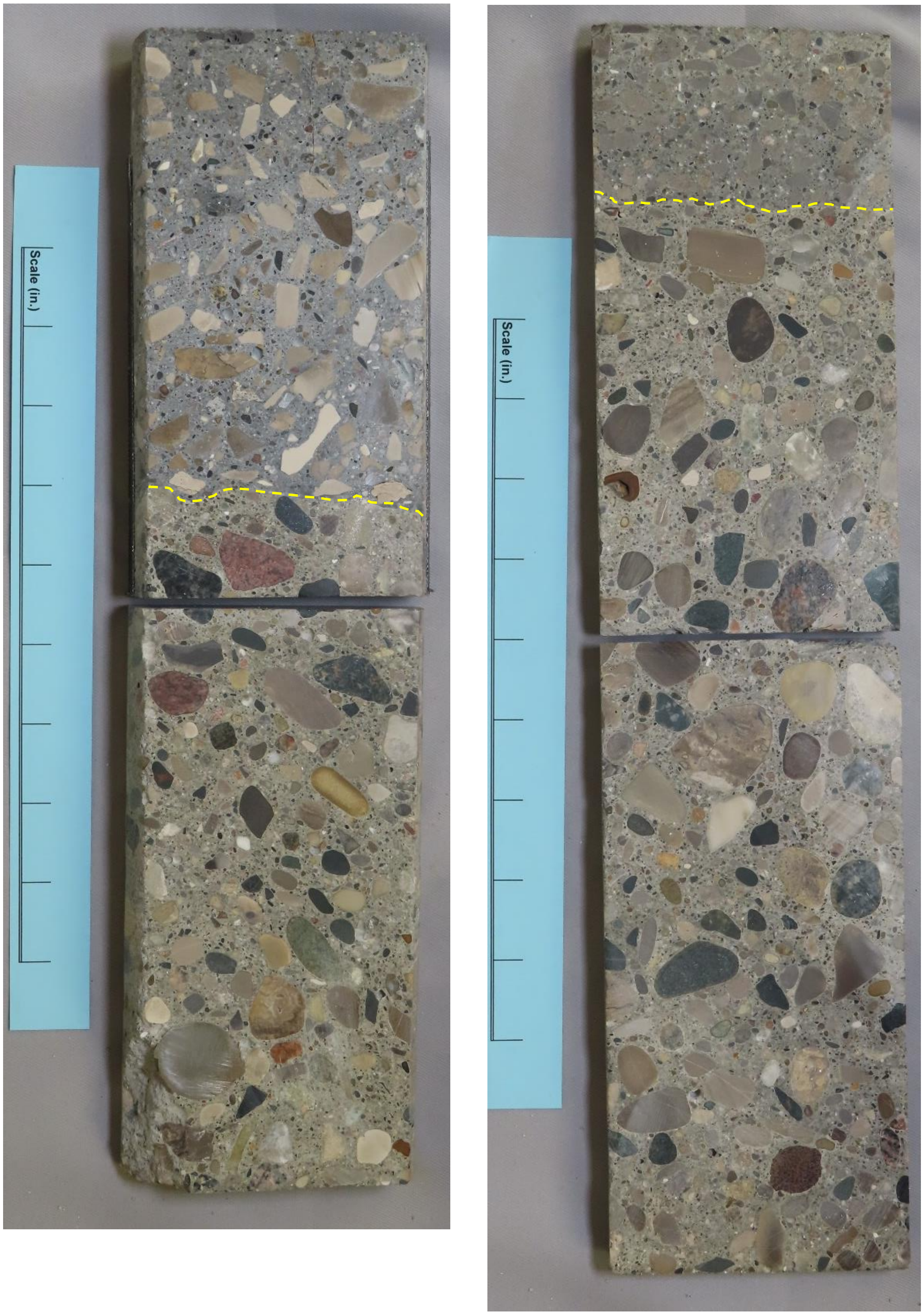


Figure 2. Lapped surface of Cores 3 (left) and 4 (right). The interface between the topping and the concrete is outlined with a dashed line.

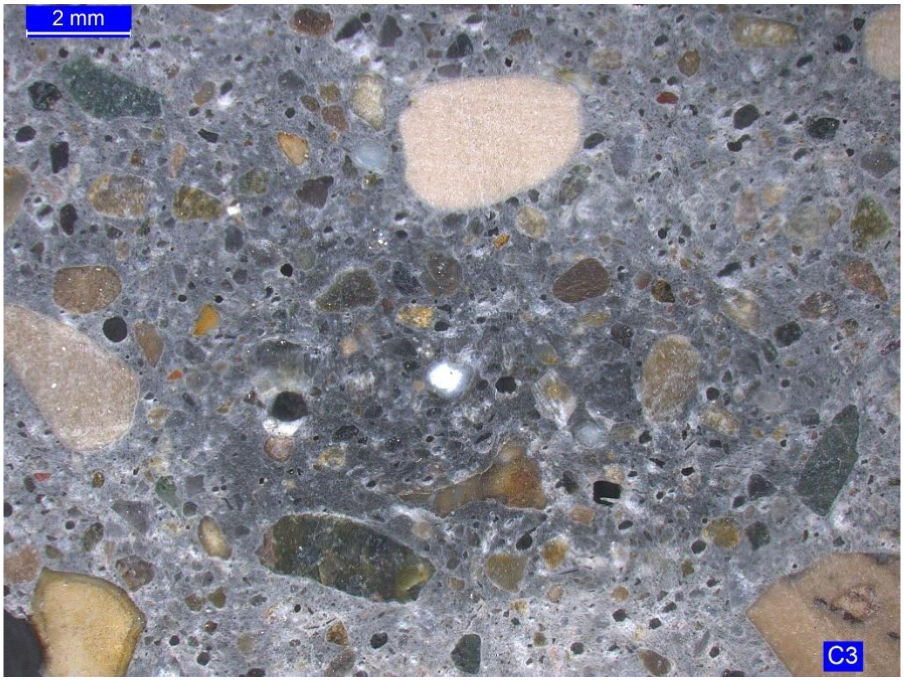


Figure 3. A reactive chert fine aggregate in the topping in Core 3 surrounded by discolored, darkened paste.

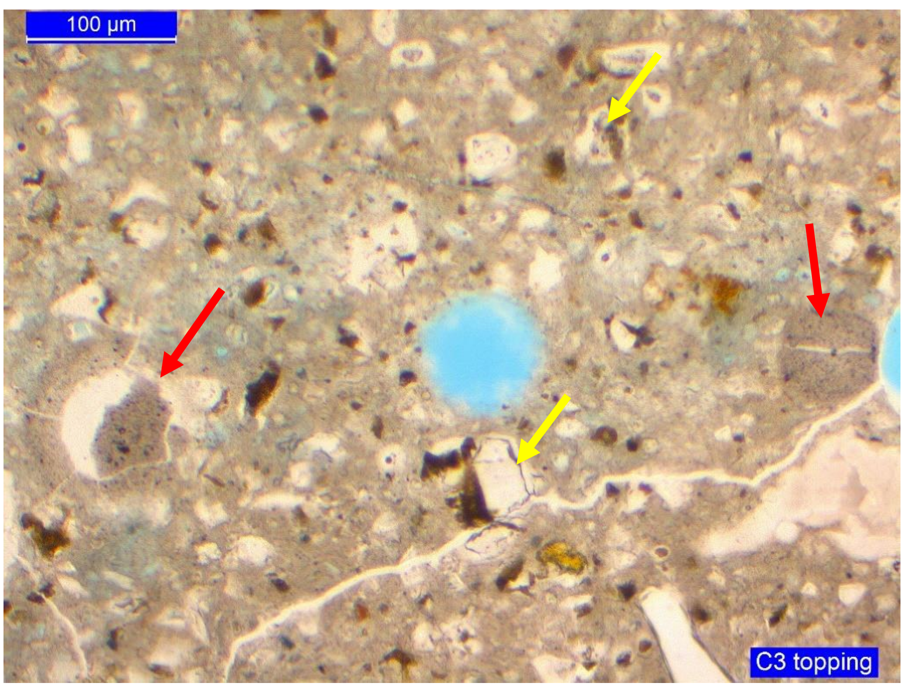


Figure 4. Silica fume lumps (red arrows) and portland cement particles (yellow arrows) in the topping for Core 3. Silica fume lumps are internally cracked.

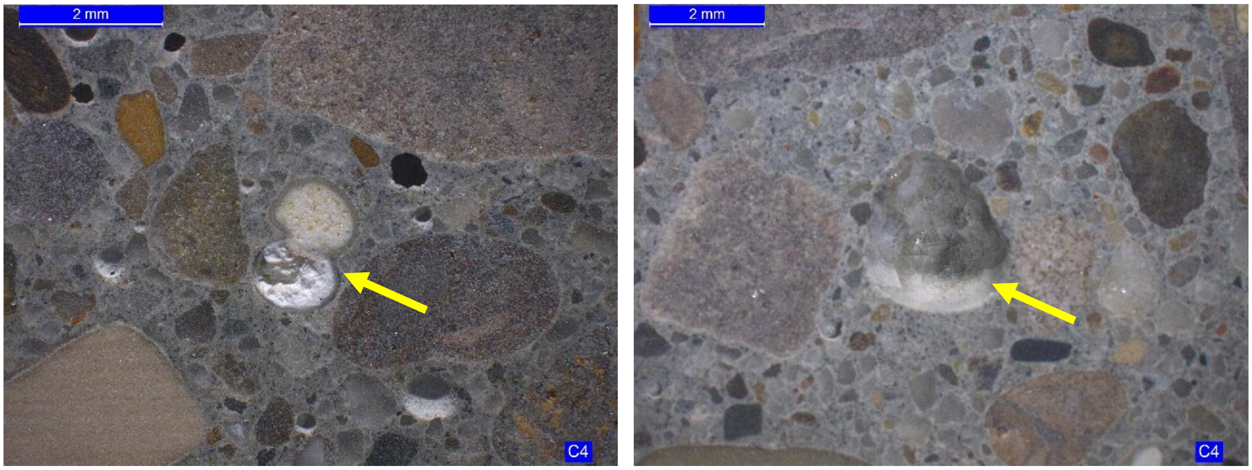


Figure 5. Alkali-silica gel in voids (arrows) in the topping in Core 4.

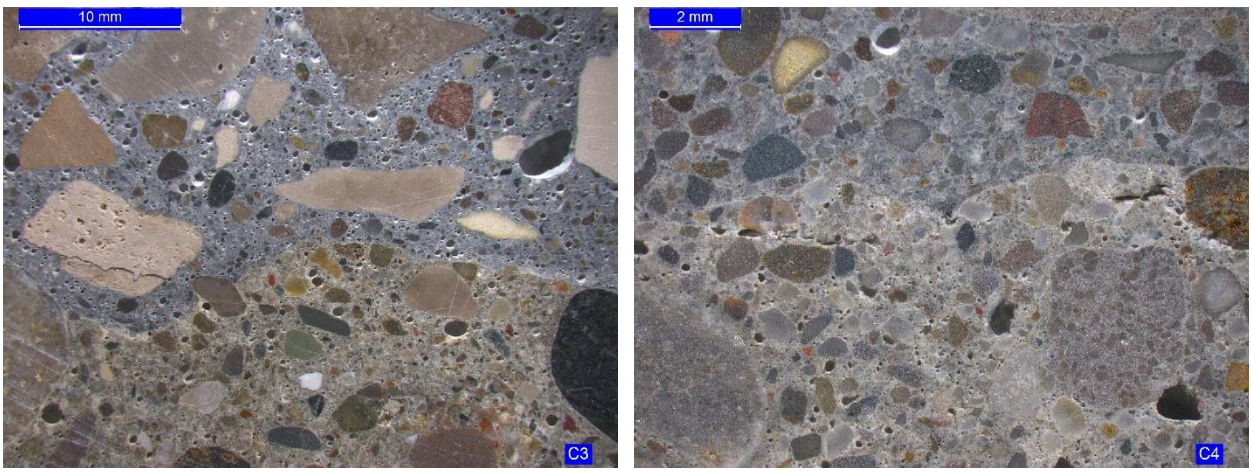


Figure 6. The irregular profile of the substrate concrete is pictured in Core 3 (left) and Core 4 (right) at different magnifications. Voids beneath the interface are pictured in Core 4.

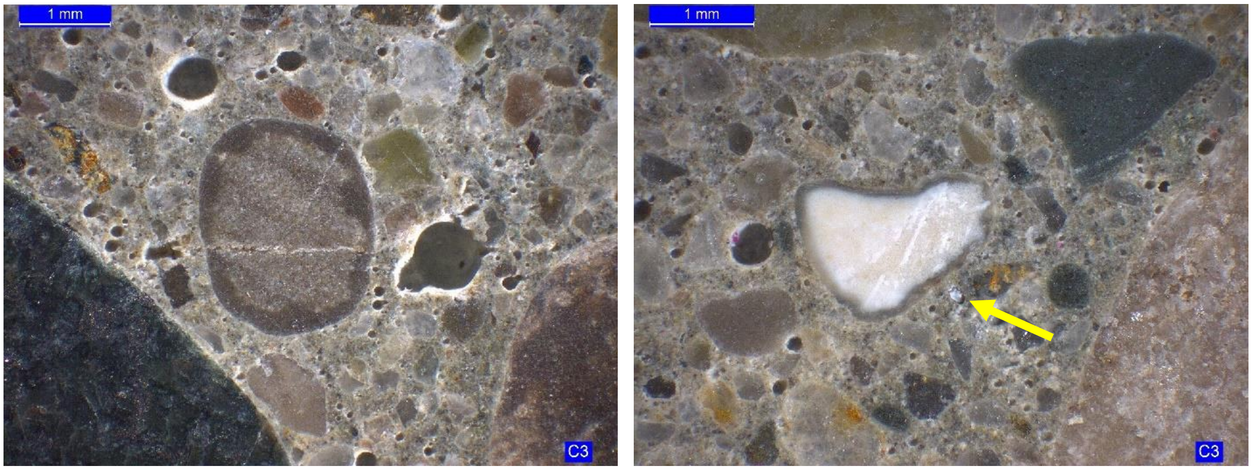


Figure 7. A crack extends through a cherty limestone particle in the concrete in Core 3 (left). Alkali-silica gel is deposited within a void (arrow) adjacent a rimmed chert particle in the concrete in Core 3 (right).

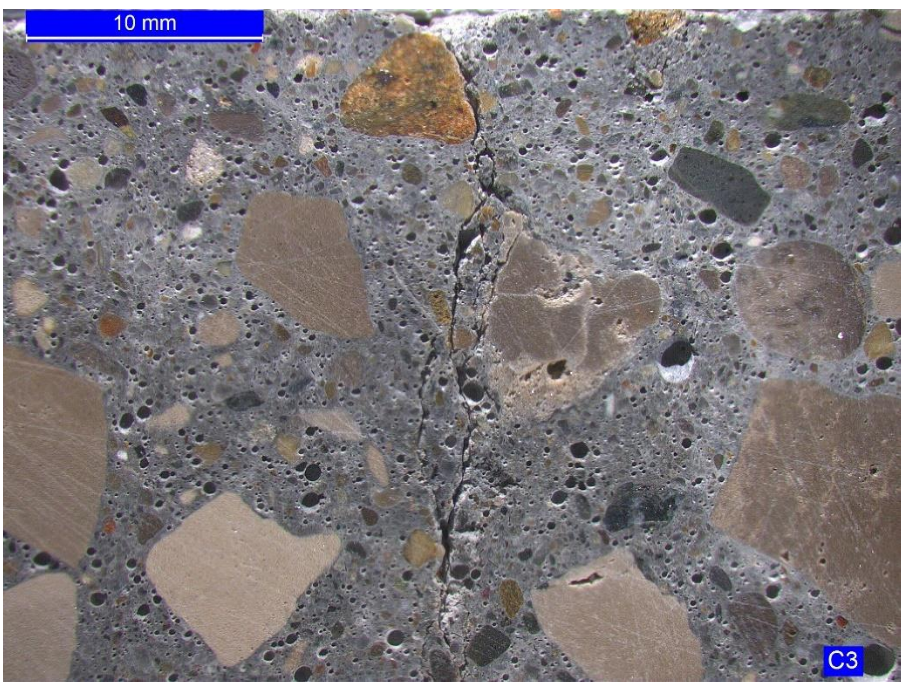


Figure 8. A crack oriented perpendicular to the top surface of the topping in Core 3.

Figure 9. The eroded top surfaces of the toppings in Core 3 (left) and Core 4 (right). A greater number of aggregates are exposed on the surface of Core 4. Note that the topping is compositionally dissimilar in Core 4 compared to Cores 1, 2, and 3.

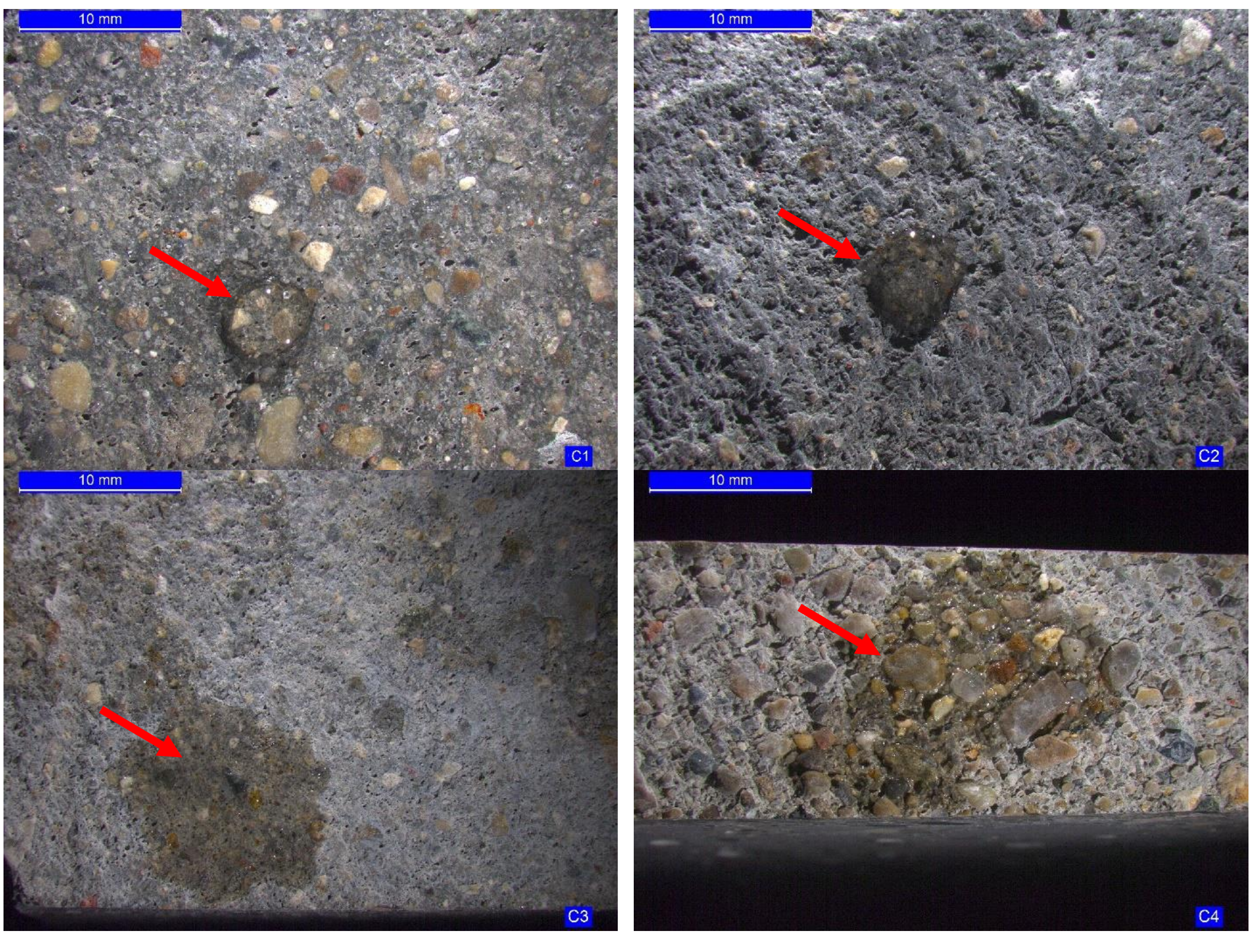


Figure 10. Water drops (arrows) applied to the surface are pictured for all five cores. Water drops beaded on Cores 1 and 2 and spread on Cores 3 and 4.

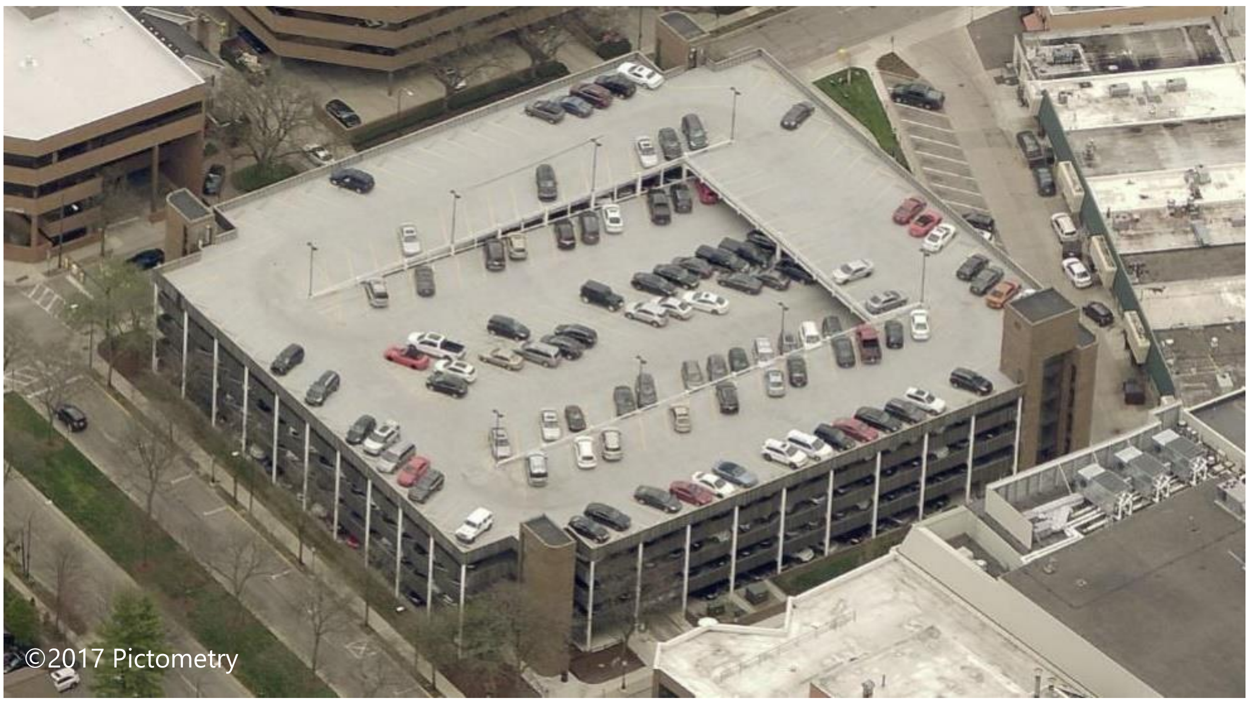
APPENDIX B. OPINION OF PROBABLE COSTS

Immediate Recommendations (within 1 year)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost***
Loose concrete removal	2	work day	$ 1,000	$ 2,000
Replace damaged/missing drain covers	24	each	$ 350	$ 8,400
Reset displaced and loose stair tower metal cover plates	10	each	$ 200	$ 2,000
Immediate Recommendations Subtotal				**$ 12,400**
Near-Term Repair Recommendations (within 1 to 2 years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost***
Concrete				
Partial-depth topside slab concrete repairs	5,000	SF	$ 45	$ 225,000
Partial-depth underside slab concrete repairs	4,500	SF	$ 100	$ 450,000
Waterproofing and Drainage Improvements				
Rout and seal cracks and joints in slab	25,000	LF	$ 6	$ 150,000
Traffic bearing membrane on Level 5	41,000	SF	$ 5	$ 205,000
Inspect and clean drain lines	1	each	$ 15,000	$ 15,000
Masonry Repairs				
Replace concrete masonry units at stair towers	50	SF	$ 20	$ 1,000
Subtotal				**$ 1,046,000**
General Conditions, Overhead and Profit (15%)				$ 156,900
Project Contingency (15%)				$ 156,900
Engineering/Testing/Construction Period Services (10%)				$ 104,600
Near-Term Recommendations Total				**$ 1,464,400**
Long-Term Repair Recommendations (within 3 to 5 years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost***
Concrete				
Partial-depth topside slab concrete repairs	5,000	SF	$ 45	$ 225,000
Partial-depth underside slab concrete repairs	4,500	SF	$ 100	$ 450,000
Concrete column repairs	150	SF	$ 110	$ 16,500
Concrete wall repairs	60	SF	$ 100	$ 6,000
Waterproofing Improvements				
Traffic bearing membrane on Level 2, 3, and 4	123,000	SF	$ 5	$ 615,000
Replace sealant at cove seal joints	1,000	LF	$ 6	$ 6,000
Masonry Repairs				
Localized repointing of clay masonry veneer	100	SF	$ 20	$ 2,000
Replacement of clay brick masonry units	60	EA	$ 15	$ 900
Steel lintel clean and coat	40	LF	$ 350	$ 14,000
Subtotal				**$ 1,335,400**
General Conditions, Overhead and Profit (15%)				$ 200,310
Project Contingency (15%)				$ 200,310
Engineering/Testing/Construction Period Services (10%)				$ 133,540
Long-Term Recommendations Total				**$ 1,869,560**
Grand Total				**$ 3,346,360**
*Prices based on current (2021) dollars and do not inlcude increases for inflation (recommended 3 percent per year)				

City of Birmingham
Parking Garage Structural Assessment Program

Park Street Parking Structure

333 Park Street
Birmingham, MI 48009



FINAL REPORT
April 30, 2021
WJE No. 2019.6318.0

PREPARED FOR:
Mr. Scott Grewe
Operations Commander - Birmingham Police Department
City of Birmingham
151 Martin Street
Birmingham, Michigan 48009

PREPARED BY:
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City of Birmingham
Parking Garage Structural Assessment Program

Peabody Parking Structure

222 Peabody Street
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Matthew E. Lewis, PE Project Manager and Senior Associate	Sarah V. Rush, PE Senior Associate	Meredith C. Crouch Project Associate

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CONTENTS

1.0 INTRODUCTION

As requested, Wiss, Janney, Elstner Associated, Inc. (WJE) has completed limited condition assessments of the North Old Woodward, Park Street, Peabody and Chester parking structures. These assessments were performed with the intent to determine the current and future infrastructure needs in support of a capital improvement plan; the intention of the plan is to extend the useful life of the structures and to maintain the structural integrity to ensure the structure can support the code-prescribed loadings. This report summarizes our observations at the Peabody Parking Structure, located at 222 Peabody Street in Birmingham, Michigan, and provides recommendations for your consideration.

2.0 STRUCTURE DESCRIPTION

The Peabody parking structure was constructed during the mid-1980s and has eight levels of parking. A Lower Level is located below grade and consists of a reinforced concrete slab on ground. The garage vehicle entrance is located on Level 1 on the east side of the building. Level 7 and a portion of Level 6 are uncovered rooftop parking. The double-threaded helix structure is rectangular in plan with a truncated corner at the southeast and approximate overall dimensions of 200 feet by 115 feet, for a total area of about 170,000 square feet of floor space between all levels.

The structural system at the supported levels generally consists of a one-way post-tensioned (PT) slab supported by PT beams and conventionally reinforced concrete columns. The PT tendons consist of single 7-wire strands in plastic sheathing with bonded mild reinforcement. The structural slab tendons span in the north-south direction with two intermediate anchorage points at construction joints in each bay. Temperature tendons span perpendicular to the structural tendons in the east-west direction. The PT beam tendons are draped and continuous between aligned bays. The conventional mild reinforcement within the PT slab is epoxy coated; plain reinforcing bars are located elsewhere within the concrete structure. Large concrete washes (sloped curbs) are present at the slab edges, which were cast monolithically with the slab.

The exterior wall assembly consists of clay brick masonry veneer with concrete masonry (CMU) back-up. These partial height walls are approximately 3 feet tall and serve as the vehicle barrier system for the garage perimeter. The brick is supported by shelf angles that are anchored to the slab edges, while the reinforced CMU bears on top of the slab. The vehicle barrier system at interior column lines consists of post-tensioned cables. Stair towers with CMU walls, brick veneer, concrete stairs, and storefront window assemblies are present at the northeast and southwest corners of the structure. Expansion joints are present between the deck and stair towers and between the slab on ground and first elevated level. Mechanical and storage spaces are located within the Lower Level, and a vaulted sidewalk plenum space is present at the vehicle entrance. A sealant joint is present between the vaulted sidewalk and the elevated slab at the vehicle entrance.

2.1 Document Review and Background

WJE reviewed relevant sheets of the original construction drawings, dated August 15, 1983 and authored by Christopher Azancy & Associates, as part of our assessment. Pertinent information is discussed within the observation sections below. However, one item of note is that the uniform design live loads used in the design of the deck are based on older code requirements and are higher than the current code-required loads [2015 *Michigan Building Code* (MBC)].

The deck slabs were designed to support a live load of 50 pounds per square foot (psf), with a roof live load of 80 psf. For comparison, the MBC requires 40 psf live load if this same garage was constructed new. Based on our site visit observations, several past restoration projects have occurred at the building; however, documentation related to these efforts was not provided to WJE for review.

3.0 FIELD ASSESSMENT

WJE visited the site multiple times in January 2020 to perform visual inspections of the accessible and exposed portions of the structure and the facade. WJE returned to the site in May 2020 to perform a delamination survey at representative locations. WJE returned to the site on multiple occasions throughout February 2021 to perform non-destructive evaluation surveys, review inspection openings, extract concrete cores for materials testing, and complete additional assessment efforts. The mechanical and storage spaces were not accessible and have been excluded from our assessment, except for the vaulted sidewalk plenum space below the vehicle entrance.

WJE's scope included a limited sounding survey of the supported levels in accordance with *ASTM D4580 - Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding.* For this survey, areas of delamination were identified using the chain-drag method, localized hammer sounding, and use of a delamination wheel at select underside locations. In areas of sound concrete, these methods produce a clear, ringing sound, and when a delamination is encountered, a hollow, drum-like sound is produced. Between 25 and 50 percent of the total area for each floor was surveyed, including all construction joints where intermediate PT anchorages are located. Sounding of the underside of the slab with a delamination wheel was primarily limited to locations of previous repair and visible indications of potential concrete deterioration (e.g. at visible cracks, spalls). A summary of pertinent observations follows.

3.1 Structural Floor Slabs

Beyond the PT slab reinforcement, bonded mild reinforcing bars are typically located at a 16-inch spacing on center near the top of the slabs over beams and the bottom of the slabs at midspan. Mild reinforcement also runs through the construction joints near the top and bottom of the slab at the same typical 16-inch spacing, as well as at the drains, slab edges, and tendon anchorages. A conventionally reinforced slab is present at the vaulted sidewalk plenum space at the vehicle entrance. The elevated floor slabs were generally in serviceable condition with localized areas of distress that are largely concentrated at the upper levels and vehicle entry level. Notable conditions and deterioration are described below.

1. Localized areas of spalled and unsound concrete were identified throughout the elevated slabs during the visual and delamination surveys. In general, less than 5 percent of the areas surveyed were unsound.
   a. Delaminations on the topside of the slab are primarily concentrated at the construction joints and over the beams, where mild reinforcement is nearer the top of the slab, which typically results in greater exposure to higher levels of chloride ions from deicing salts that penetrate the slab over time, affecting reinforcing steel with lower concrete cover (Figure 1). These delaminated areas are generally small, between 1 to 4 square feet each.

b. Previous partial-depth repairs at the topside of the slab are at similar locations (Figure 2). Less than 10 percent of the previous repairs at the construction joints or over beamlines were found to be unsound (Figure 3, Figure 4).

c. Small isolated areas of spalled concrete and exposed corroded mild reinforcement are present on the underside of the elevated slabs. Spalls are typically located near the construction joints for the upper levels and at midspan between beams where mild reinforcing steel is near the top surface (Figure 5, Figure 6). Where spalls occur at construction joints, PT tendons and anchorages are not exposed.

d. Previous partial-depth concrete repairs on the underside of the slab are present at similar locations. Multiple repairs are visible at three of the Level 1 and Level 2 construction joints (Figure 7). No previous partial-depth repairs on the slab underside were found to be unsound; however, isolated cracking, efflorescence, and corrosion staining were observed in some regions.

2. Cracks are present along the length of beams in the topside of the slab at approximately 10 locations, which have typically been routed and sealed (Figure 8).
3. Short, isolated cracks are present in the slabs adjacent to some column corners (Figure 9).
4. Several short, isolated cracks are present in the topside of the upper level slabs and entry level, particularly near the drains. These cracks were previously routed and sealed. The sealant in these cracks has adhesively failed at a few locations. The cracks generally did not visibly propagate beyond the prior sealant repairs.
5. Isolated areas of concrete pitting are present on the top surface of the slab, particularly within the vehicle entrance lanes.
6. A few isolated PT tendon repairs are present in the supported slab, primarily on Level 1 (Figure 10). These previous repairs are in serviceable condition.
7. A few grease stains are visible near construction joints on the upper levels (Figure 11).
8. Moisture staining, corrosion staining, and efflorescence are typically present along the full length of the upper level and entry level construction joints on the undersides of the slabs (Figure 6).
9. Isolated grout pockets for the slab PT tendons have debonded, though water staining or corrosion staining was not observed.

3.1.1 PT Inspection Openings

Based on the slab observations noted above, locations were chosen for inspection openings to assess the condition of the embedded PT tendons. WJE retained a local concrete restoration contractor, Pullman SST, to create and repair the inspection openings specified by WJE. Refer to Figure 12 for the locations of the tendons.

1. **Exposed Location 1:** One tendon and corroded mild reinforcement were exposed at an existing spall near a drain. The tendon was holding tension and the sheathing was intact. A minimal amount of grease was present between the tendon's individual wires, though the tendon exhibited no visible corrosion (Figure 13).

2. **Inspection Opening 1:** One tendon was exposed on the underside of the slab in-line with adjacent areas of delaminated concrete on the top of the slab over the beamlines. The tendon was holding tension, and the sheathing was intact. A minimal amount of grease was present between the tendon's individual wires and the tendon exhibited no visible corrosion (Figure 14).
3. **Inspection Opening 2:** In an unsound repair area over a beamline, conventional reinforcement with low concrete cover was exposed. No PT tendons were present in the inspection opening. The exposed reinforcement consisted of a closed stirrup and one straight bar that extended along the length of the beam. The exposed steel is in serviceable condition and is consistent with the typical beam section detail in the construction drawings (Figure 15).
4. **Inspection Openings 3 & 4:** Two tendons and one conventional steel reinforcing bar were exposed in each inspection opening location, which were located in-line with grease stains near a construction joint. The tendons were both holding tension and the sheathing was intact. A minimal amount of grease was present at both tendons, and the tendons exhibited no visible corrosion (Figure 16 and Figure 17).

3.2 Waterproofing and Drainage Components

The expansion joints in the deck typically consist of pre-molded expansion joint seals. A traffic-bearing membrane is located over the Lower Level mechanical and storage spaces at the transitions between the slab-on-grade and elevated slabs.

1. The expansion joint seals throughout the garage are typically failed (Figure 18).
2. The traffic-bearing membrane is significantly worn, exposing failed regions of the waterproofing base coat (Figure 18).
3. Construction joint sealants typically exhibit localized regions of adhesive failure, particularly at the upper levels. The sealant joint at the vehicle entry is also adhesively failed (Figure 19).
4. Perimeter cove sealants are present at Level 6 and Level 7, which typically exhibit cohesive and/or adhesive failure (Figure 20). Isolated areas of water staining were observed where perimeter cove seals are not present on other elevated levels (Figure 9).
5. Roof drains were typically clogged during WJE's site visits, though the remaining drainage system appears in serviceable condition.
6. Roof drain outlets for the stair tower discharge directly over failed expansion joint seals or over the edge of the masonry facade.

3.3 Beams and Columns

The beams are typically post-tensioned, though the beams at the slab edge near the stair towers are conventionally reinforced. The columns are also conventionally reinforced. These elements are generally in serviceable condition with minor, isolated areas of distress. The following items were observed.

1. The beams at the slab edge near the southwest stairwell typically exhibit deterioration, including cracking, spalling, water staining, and efflorescence. This condition typically correlated with failed expansion joint seals in the slab above (Figure 21).

2. A few beams at the roof level contain vertically-oriented cracks near the beam ends, which were previously repaired with epoxy injection (Figure 22). No evidence of moisture exposure or continued distress was observed.
3. Isolated columns contain cracks that propagate from the beam-column intersections (Figure 23, Figure 24). Some cracks have been previously repaired with epoxy injection. Corrosion staining was generally not observed.
4. Two columns on the southwest end of the Lower Level exhibit significantly cracked and spalled concrete near the PT beam anchorage zone (Figure 25). Previous concrete repairs have failed, and a maximum crack width of 3/16 inch was noted. Loose concrete was removed during our assessment; the exposed steel reinforcing bars exhibit surface corrosion.
5. Approximately 50 percent of the interior columns have repairs at their bases, which are typically cracked or spalled, especially near the drains. Standing water was present at a few column bases near clogged drains (Figure 26). One column contains a large concrete repair that extends above its base that is unsound (Figure 27).
6. Isolated grout pockets within the columns for the vehicle barrier cables have debonded (Figure 26). Water staining and efflorescence are present at some locations, though the concrete is generally sound.

3.4 Perimeter and Foundation Walls

The CMU perimeter walls are approximately 3 feet in height and bear on the exterior slab edges. These walls serve as vehicle barriers and provide backup for the brick masonry veneer. The exposed CMU surfaces were coated in 2020, as discussed in Section 4.0 below. Concrete cast-in-place foundation walls extend between columns at the Lower Level. In general, the perimeter and foundation walls are in serviceable condition with isolated areas of water-related distress throughout. The following was observed:

1. Cracks are present in the CMU walls at the upper levels near locations of horizontal reinforcement shown in the structural drawings (Figure 28). Corrosion staining was not observed.
2. Isolated cracks are present at the railing anchorages within the CMU walls at the roof level.
3. Localized repairs are present at the base of the CMU walls, which are typically sound.
4. The foundation walls exhibit isolated cracks and spalls, efflorescence, and water staining, particularly at the east wall below the failed sealant joint at the vehicle entrance (Figure 29). Where previous partial-depth concrete repairs are present, the repairs contain crazing cracking and were generally unsound.

3.5 Brick Masonry Facade

The brick masonry facade is supported by shelf angles at the exterior slab edges. The columns are also clad in brick on three sides from Level 1 to Level 3. The cavity spaces between the brick cladding and the concrete columns are open and exposed to the deck interior. Previous repairs exist throughout the facade, including localized areas of repointed mortar, replacement of brick masonry, and sealant replacement. External bolted connections that extend through the veneer are present in isolated regions and may also have been added as part of a previous repair effort. The clay brick masonry facade and previous repairs are generally in serviceable condition with minor distress throughout. The following pertinent conditions were observed:

1. The shelf angles are generally in serviceable condition with minor corrosion staining visible in isolated locations, although areas of significant corrosion and section loss were observed at the shelf angles near grade (Figure 30).
2. The brick veneer wall ties are generally in serviceable condition within the exposed cavity wall, though some locations exhibit minor surface corrosion. However, the tie spacing varies with localized areas of the column cladding containing few to no visible wall ties.
3. Isolated vertical cracks and step cracks are present within the brick cladding at columns, primarily on the west facade. The cracks extend from the intersection between the brick veneer and concrete structural system or shelf angles (Figure 31 through Figure 34). These cracks have typically been previously sealed, but have since propagated or reoccurred adjacent to the sealed locations, and isolated brick units are spalled. In some locations near grade, the brick is laterally offset at shelf angle locations, and the shelf angles have been concealed in joint sealant. Lack of wall ties were not associated with the observed distress in these regions.
4. Vertical cracking and isolated spalls are located below a roof level downspout (Figure 35, Figure 36).
5. Localized brick units are spalled or cracked, including at locations of vertical discontinuities within the cladding and spalled areas due to impact damage (Figure 37 through Figure 41).
6. Isolated areas of efflorescence and crazing cracks in the brick surface are present near the location of the structural slabs, at vertical discontinuities within the cladding, or near roof level downspouts.
7. The vertical joint sealant between the concrete foundation walls and brick column cladding at the Lower Level is adhesively failed and localized brick units are cracked and spalled (Figure 42).
8. The vertical joint sealant between the CMU and brick facade is typically adhesively and cohesively failed with isolated regions of brick masonry distress, such as spalls or cracks (Figure 43). The vertical mortar joints between the brick facade and stairwell walls are typically debonded (Figure 44). Near the southwest stairwell ramp, a vertical joint between brick masonry is also debonded (Figure 45).
9. Isolated areas of brick masonry are spalled at the base of the wall with mortar erosion and efflorescence in or near the stairwells at multiple levels and isolated site walls (Figure 46).

3.6 Stair Towers and Miscellaneous

WJE noted the following miscellaneous conditions elsewhere in the garage, including the stairwells and slab on grade:

1. At the stairwell storefront window framing, the coating is typically chipped or worn, and the perimeter sealant and glazing is typically adhesively failed (Figure 47). A roof level door threshold is also deformed (Figure 48).
2. Handrail posts are embedded into the concrete stairs. The edges of the stair slabs contain cracks and spalls (Figure 49). The handrails and posts typically exhibit corrosion and paint failure, with isolated areas exhibiting significant section loss (Figure 50, Figure 51).
3. Crazing cracking is present at the Level 7 slab in the northeast stairwell (Figure 52).

4. Isolated cracks, spalls, and efflorescence are present on the underside of localized concrete stair flights, including localized regions of previous partial-depth concrete repairs (Figure 53, Figure 54).
5. The vehicle barrier cables and splice couplers along the interior column lines generally exhibit signs of surface corrosion (Figure 55).
6. Isolated areas of the slab-on-ground are cracked. Some areas have been previously sealed and are in serviceable condition.

4.0 REPAIRS COMPLETED TO DATE

In an effort to take advantage of reduced occupancy during the COVID-19 pandemic, the City of Birmingham approved a limited scope of repairs on May 18, 2020 to be performed by DRV Contractors, LLC. As of the issuance of this report, the following repairs have been performed or are ongoing:

- Removal of loose concrete on the underside of slabs throughout the garage
- Washing and coating of the CMU perimeter walls
- Splices installed to re-tension loose vehicle barrier cables

5.0 MATERIALS TESTING

Four concrete cores were extracted from various locations in the structure and sent to WJE's Cleveland laboratory for materials testing. The lab studies included petrographic examination, water-soluble chloride analysis, and carbonation depth measurements. A summary of the findings is presented in this report section. See ***Appendix A*** for more testing information, figures, and discussion.

Table 1. Core Locations

ID	Core Location	Location Description
C1	Level 1 North End	In drive lane at point of entry, within about 15 feet of the drain and near the stair tower.
C2	Level 6 South End	In drive lane, within about 15 feet of the drain and near the stair tower.
C3	Level 4 West Bay	In parking stall.
C4	Level 5 East Bay	In drive lane.

The concrete slab materials are generally in serviceable condition. The concrete mix consists of blended river gravel, coarse aggregate and siliceous fine aggregate in an air-entrained, portland cement paste. Air-entrainment improves the concrete's freeze-thaw durability. No distress was observed in the body of the microscopically examined core (Core 3).

WJE visually observed surface erosion in localized areas of the concrete surface. The erosion is confined to the top surface only and does not correlate with microcracking or other indications of distress in samples with this surface condition. Otherwise, WJE did not find indications of secondary distress as a result of external factors (e.g. chlorides, moisture, freeze-thaw damage, etc.). This indicates that the deck does not appear to have experienced sustained long-term exposure to moisture over the course of its service life thus far. It is important to maintain the waterproofing components within the deck to further protect the concrete from progressive distress.

5.1 Water Absorption

Water drop testing was performed to test the hydrophobicity (water repellency) of the top surface. Refer to Table 2 of ***Appendix A*** for a summary of the test results for each core. Water beaded and was not absorbed into the concrete surfaces of Cores 1 and 2, which is an indication that an effective penetrating sealer is likely present. These cores pertain to the entry and roof levels.

Water spread on the concrete surface of Core 3 but was not absorbed, suggesting that a penetrating sealer may have been applied but has somewhat deteriorated along the surface. Water spread on the concrete surface and was absorbed by the paste of Core 4, though hydrophobic properties were observed to a depth of 3/8 inch. These observations suggest that a sealer-like material may have penetrated into the top of the concrete in the area from which the core was extracted but has since completely deteriorated along the surface. The increased resistance to water absorption at the entry and roof levels (Cores 1 and 2) compared to the intermediate levels suggests that a penetrating sealer may have been reapplied on these levels.

5.2 Carbonation Testing

The high pH of uncarbonated concrete provides protective passivation of the embedded steel reinforcement. Carbonation is a chemical process that occurs in the cement paste of the concrete and lowers the pH of the concrete. The depth of carbonation increases over time and is accelerated at cracks or joints. When the carbonation front reaches the depth of reinforcing steel, the steel becomes more susceptible to corrosion because the passivation layer from the high pH of the concrete is no longer present. The depth of the carbonation for each core is shown in Table 2 of ***Appendix A***.

The depth of full carbonation from the top surface has reached up to 3/8 inch at Cores 1, 2, and 4. The depth of full carbonation from the bottom surface has reached up to 3/4 inch at Core 1. The depth of carbonation is less than the depth of the typical embedded reinforcing steel; thus, the increased potential for corrosion due to carbonated concrete is not a concern at this time. However, embedded steel elements in areas of low cover would be expected to experience and increased potential due to carbonated concrete at these depths, which may result in deterioration of the surrounding concrete.

5.3 Water-Soluble Chloride Testing

The purpose of the chloride analysis was to determine the current chloride content at various depths of the slab. The results are contained within Table 2 of ***Appendix A***.

The water-soluble chloride content by weight of concrete at the typical depth of reinforcing steel was found to be negligible at Cores 1, 3, and 4. However, the chloride content at the depth of reinforcement at Core 2, taken from the roof level, is at a level that can promote corrosion of embedded steel. Elevated chloride contents were also measured near the top surface at Cores 1, 2 and 3. High levels of chloride contents near the slab surface are indicative of externally applied chloride sources, such as the application of deicing salts. The test findings also suggest that a chloride-containing admixture was not used in the concrete mixture during the garage’s construction. Localized areas of areas of greater chloride contamination may occur at cracks and joints. With continued use of chloride-containing deicing salts, the chloride concentration and depth would be expected to increase if the apparent existing penetrating sealer is not maintained.

6.0 DISCUSSION

Overall, the parking structure is in serviceable condition with localized areas of deterioration. Waterproofing and concrete repairs are recommended in the near future to maintain the condition of the parking structure.

6.1 Concrete - General

Concrete slabs within parking structures in Michigan are susceptible to deterioration due to their exposure to moisture, deicing salts, and temperature changes (i.e., cyclic freezing and thawing, thermal expansion and contraction, etc.). The primary causes of concrete deterioration in concrete parking structures is corrosion, typically due to chloride contamination and carbonation as both conditions can promote corrosion of embedded steel reinforcement. Because steel corrosion product occupies a larger volume than the native steel, it is common for distress in the form of cracks, delaminations, or spalls to develop when the embedded steel corrodes and expands, placing expansive forces on the surrounding concrete.

Post-tensioned structures efficiently combine steel, which is strong in tension, and concrete, which is strong in compression, to utilize the full cross section of a structural element at all points along its length. Compared to conventionally reinforced concrete, post-tensioned concrete typically offers greater durability, particularly due to its ability to minimize cracking and to protect the tendons from corrosion. The benefit of post-tensioned concrete over conventionally reinforced concrete depends heavily on adequate protection of the tendons from moisture. Locations that are most susceptible to moisture exposure include tendon anchor points, where the sheathing or anchor may not be protected, and construction joints or concrete repairs, where the tendon sheathing is made discontinuous for stressing or possibly damaged during repair, respectively. These locations of discontinuous sheathing at cracks or joints can allow water to directly reach the tendons. Deterioration of PT tendons, particularly corrosion leading to section loss, can result in failure of that tendon. If an unbonded tendon becomes de-tensioned for any reason, that tendon no longer carries load at any point along its length.

6.2 Structural Floor Slabs

The elevated concrete slabs are generally in sound condition except for a few areas of localized distress concentrated near beam lines and construction joints, often within previous repair areas. These areas are attributed to mild reinforcing steel near the top slab surface and areas of prolonged moisture exposure, such as concrete near drains and failed control or expansion joint seals. Distress at construction joints correlates with areas of higher exposure to moisture, de-icing salts, and high traffic, with distress concentrated mostly at the roof levels and entry levels, while distress over the beams is typically concentrated at Level 4.

Based on our observations, the isolated spalls and delaminated areas in the slab associated with lower cover of bonded mild reinforcement did not appear to correspond with corroded PT tendons. Although tendon damage was not observed where exposed in the inspection openings (discussed in further detail below), distress over the beams and construction joints exposes the slab assembly to moisture and chlorides, which can lead to accelerated deterioration of the PT anchors and tendons. Thus, repair of the isolated concrete distress and improved water management detailing is recommended.

Partial-depth concrete repairs are generally anticipated. If PT anchors are exposed during concrete removal at construction joints, they should be assessed by an engineer prior to completing the repairs; do not chip in front of PT anchorages.

PT sheathing repairs are anticipated wherever PT tendons are exposed within a concrete repair area. Installation of a traffic-bearing membrane may be considered over concrete repair areas to help protect the mild reinforcement with lower concrete cover and mitigate premature failure of the repairs. Installation of a traffic-bearing membrane and joint sealant along the construction joints is also recommended to protect the mild reinforcement spanning across the joints and the PT anchorages.

Isolated cracks that are present within the top slab surface and extend along the beam lines are infrequent within individual floor slabs and do not correlate with similar cracking on the underside of the slab at midspan. These cracks are attributed to restrained shrinkage and potential low concrete cover to the bonded mild reinforcing, and do not constitute a structural concern. Isolated miscellaneous cracks elsewhere in the slab, typically concentrated near the drains, are attributed to similar causes of distress. All cracks in the elevated slabs should be routed and sealed on the top slab surface to mitigate water penetration, and failed sealant materials should be replaced.

6.2.1 Slab Post-Tensioning

Of the six PT tendons inspected, all were in good condition with no surface corrosion observed on the exposed wires, despite the general lack of grease within the sheathing. A lack of grease within the sheathing is not a direct indication of PT tendon deterioration, but tendons with little grease may be more susceptible to corrosion should water enter the tendon sheathing. Isolated debonded grout pockets and grease stains do not constitute a structural concern, based on the findings of our inspection openings and a lack of corrosion staining.

Very few previous slab PT tendon repairs were observed, and all were in serviceable condition. Installation of a traffic-bearing membrane over these isolated PT repair areas is recommended for improved durability. Although the exposed tendons and previous PT repair areas are in serviceable condition, due to the limited nature of the inspection openings and to account for concealed areas of distress, we recommend budgetary cost estimates assume some PT repairs are required during concrete repair efforts.

6.3 Waterproofing and Drainage Components

In WJE's experience, the long-term durability of parking structures subjected to chloride-laden moisture from de-icing salts is extended when moisture is well managed throughout and prevented from absorbing into the structure.

6.3.1 Expansion Joint Seals and Joint Sealant

The failed pre-molded expansion joint seals and construction joint sealants are allowing water to penetrate the deck assembly and deteriorate the structural elements below. Deterioration at these joints is of particular importance due to the presence of PT anchorages in these regions; thus, maintaining these elements is a cost-effective method to mitigate the need for more costly future repairs. All expansion joint seals and construction joints are recommended for replacement.

Similarly, cove sealant between the slab edge and perimeter walls or columns is recommended for installation or replacement in areas that exhibit sign of aging or water staining.

6.3.3 Traffic Coatings and Penetrating Sealers

Traffic-bearing membrane systems are the most common waterproofing system used on parking garages to extend the life of the structure. A membrane provides an impermeable barrier on the surface of the structure and prevents moisture from entering the structure. Additionally, a membrane reduces the corrosion rate of the structure by reducing the amount of moisture in the concrete. They typically consist of multi-layer polyurethane or epoxy coating with integral aggregate broadcast for slip resistance. The bottom layer of the system provides the waterproofing, and the top layers serve as the skid resistant and wearing surface. The typical service life for a new traffic-bearing membrane in low traffic areas can easily exceed 10 years. In high traffic areas and in areas with significant turning, maintenance of a traffic-bearing membrane to address wear can be necessary in less than 5 years. Silane sealers, which in WJE's experience have proved to be an effective penetrating sealer for concrete, are also common. However, silane sealer does not have the capability to bridge cracks. Silane sealers become less effective over time and are generally reapplied at regular intervals varying between 5 and 7 years.

The existing traffic coating over the mechanical and storage areas exhibits deterioration attributed to natural aging, wear from automobile traffic, and/or deferred maintenance. Since the observed deterioration includes regions of damage within the base coat, the existing membrane requires removal and replacement in order to maintain the waterproofing system over the occupied spaces. For improved durability, a traffic-bearing membrane is recommended at the vehicle entrance lanes over the vaulted sidewalk space due to the elevated moisture and chloride exposure.

An effective penetrating sealer appears to be present at the roof and entry levels. A penetrating sealer was also likely applied to other levels of the garage in the past but was either not maintained or is exhibiting signs of deterioration. Based on the expected service life of effective penetrating sealers and lack of water repellency on the other levels, in conjunction with the use of deicing salts and elevated surface chloride levels at multiple levels, an additional sealer application at all elevated levels should be anticipated to maintain the existing waterproofing system. When reapplying a concrete sealer, surface preparation is required to ensure adequate penetration of the concrete sealer into the concrete.

The slab-on-ground areas do not require membrane or sealers. Installation of new coatings or sealer materials should occur in conjunction with localized concrete and sealant repairs in areas of cracked and unsound concrete.

6.3.2 Drainage Systems

The clogged roof drains observed at the roof level result in isolated regions of standing water, which create a condition of extended exposure of the concrete structure to chloride-contaminated water and potential cyclic freeze-thaw deterioration if the concrete was not properly air-entrained. Furthermore, standing water in the winter months will freeze, which may cause slippery walking surfaces or fractured drainpipes. Drains should be periodically cleaned. For improved durability, a traffic-bearing membrane is recommended at all drains due to their elevated moisture and chloride exposure and to bridge isolated cracks in these regions.

The downspouts for the stair tower roofs should be modified to discharge away from the expansion joints. Where the southwest stair downspout discharges over the edge of the facade, the downspout or gutter configuration should be modified to discharge at the deck level, or the downspout should be extended to discharge at grade in order to mitigate further masonry distress.

6.4 Beams and Columns

The PT beams are generally in good condition throughout the garage. The conventionally reinforced beams at the slab edge near the southwest stair tower are typically distressed due water exposure from the failed expansion joint seals above. Partial-depth concrete repairs are recommended at these distressed regions once the water management systems have been addressed.

Concrete distress at the two columns on the southwest end of the Lower Level is primarily attributed to corrosion of the embedded reinforcing steel and poor previous repairs; however, further investigation and analysis are recommended as part of a design and repair effort due to the nearby PT beam anchorage zones, large crack widths and profile, and apparent lack of mild reinforcement in the region. Repairs are anticipated to include removal of the adjacent brick masonry veneer, shoring of the structural elements above, and partial-depth concrete repairs. The columns should be monitored or shored until repairs can be conducted.

Cracking at the beam-column intersections is attributed to the applied post-tensioning forces to the structure and restraint, and does not present an imminent structural concern. When post-tensioning forces are applied to the tendons, the concrete slab and beams contract slightly, which can lead to cracking where this movement is restrained. The observed distress may also be a result of tendons that were tensioned early in the life of the concrete. The previous epoxy injection repairs generally appear to be in serviceable condition, and the cracks have not propagated beyond the past repairs. However, in isolated regions where cracks exhibit ongoing moisture penetration, additional epoxy injection, sealant, or waterproofing coating repairs are recommended to mitigate moisture exposure and subsequent deterioration of the concrete elements. Partial-depth concrete repairs maybe be required in some regions.

Concrete deterioration was noted at the interior columns near areas of ponded water, failed previous repairs at the column bases, and isolated debonded grout pockets at vehicle barriers cables. These columns are located along the drainage path due to the transverse deck slope, resulting in increased moisture and chloride exposure. Partial-depth concrete repairs are recommended in these distressed regions, in conjunction with the discussed water management measures.

6.5 Perimeter and Foundation Walls

The CMU perimeter walls have recently been repaired and coated to reduce deterioration associated with exposure and moisture. Isolated cracks not sealed as part of the coating work; it is recommended that they be sealed to mitigate moisture penetration within the masonry.

The east concrete foundation wall at the Lower Level exhibits moisture-related deterioration associated with the failed sealant joint above. Partial-depth concrete repairs are recommended in regions of unsound concrete in conjunction with replacement of the sealant joint above.

6.6 Brick Masonry Facade

The localized cracks, debonded mortar joints, and failed joint sealants throughout the garage are attributed to relative movement of dissimilar materials, discontinuous support, and restraint. Recommended repairs include replacing cracked brick units, repointing debonded or cracked mortar, sealant replacement, and installation of supplemental lateral ties. In addition, at the vertically cracked regions of brick veneer, installation of soft joints (sealant with backer or compressible filler) is recommended to accommodate further movement. At the vertical discontinuities in the brick cladding and at shelf angles supporting the brick veneer at the columns, improved water management details should be considered to mitigate further masonry distress.

The observed masonry distress at grade and stairwells is largely attributed to the exposure of deicing salts applied on the adjacent sidewalks and moisture. Corroded shelf angles near grade are recommended for repair or replacement. The remaining shelf angles are in serviceable condition and are not anticipated for repair.

6.7 Stair Towers and Miscellaneous

The stairwell storefront window assemblies exhibit distress due to prolonged exposure and deferred maintenance. Recommended repairs include replacing perimeter sealant and glazing, repainting the window frames with appropriate surface preparation and an appropriate coating material, and replacing the deformed roof level door threshold to mitigate trip hazards. The storefront assemblies may alternatively be considered for replacement (in-kind) in areas of significant distress.

The stairwell handrails and posts are recommended for cleaning and repainting, including removal of surface corrosion where present. Isolated steel post repairs are anticipated as part of this work due to the extent of section loss. Localized concrete repairs are recommended at the stair towers at deteriorated handrail post embedment locations and at spalled and unsound concrete at the underside of the stairs. The crazing cracks observed at the stair landing surface are attributed to shrinkage and a surface parge coat and are not anticipated for repair at this time.

The isolated corroded surfaces of the vehicle barrier cables and splice components are recommended to be cleaned and coated as part of the overall maintenance plan for the deck. Isolated cracks within the slab on ground do not exhibit signs of settlement or displacement. Previously sealed cracks are generally in serviceable condition. These regions should be sealed/resealed as part of the overall maintenance plan for the deck.

7.0 RECOMMENDATIONS

Based on our observations and our experience with similar parking garages, WJE offers the following categorized recommendations for your consideration.

7.1 Immediate Recommendations

Further investigation of the concrete distress within the two Lower Level columns on the southwest end of the garage is recommended within 1 year. The columns should be monitored (every 3 months) or shored until repairs can be conducted.

7.2 Near-Term Repair Recommendations

WJE recommends that the following repair items be completed in the near future (within the next 1 to 2 years). These recommendations are intended to minimize water infiltration, address concrete distress in regions of waterproofing repairs, and extend the service life of the parking structure.

1. Concrete Structure Repairs
    a. Localized repairs (both partial and full depth) of unsound concrete at the elevated slabs.
    b. Isolated slab post-tensioned tendon and anchor repairs.
2. Waterproofing Component Repairs
    a. Replace construction joint sealant. *
    b. Rout and seal isolated cracks and replace failed sealant at previous cracks in the elevated slabs.
    c. Replace expansion joint seals. *
    d. Install traffic-bearing membrane at construction joints, occupied areas, and vehicle entrance lanes.
    e. Apply concrete slab penetrating sealer at elevated levels.
    f. Inspect and clean drain lines. *

These repairs may be phased if needed to accommodate occupancy, schedule, or budgetary concerns. The highest priority repair items are indicated with an asterisk (*).

7.3 Long-Term Repair Recommendations

WJE recommends that the following repairs be completed within the next 3 to 5 years. These recommendations are intended to address structural deterioration, as well as the observed distress within the facade, stairs, vehicle barriers, and slab on grade.

1. Concrete Structure Repairs
    a. Localized repairs (both partial and full depth) of unsound concrete at the elevated slabs, edge beams, columns, and foundation walls.
    b. Isolated repairs at cracked interior columns at roof level.
2. Waterproofing Component Repairs
    a. Install traffic-bearing membrane at drains and isolated concrete repair areas.
    b. Replace cove sealant at roof level. Install isolated cove sealant at other elevated levels in locations exhibiting moisture staining.
    c. Modify existing stair tower roof downspouts for improved drainage.
3. Facade Repairs, Stair Tower, and Miscellaneous Repairs
    a. Replace isolated spalled or cracked brick units and repoint eroded or debonded mortar. Install soft joints (backer rod and sealant) at isolated regions to accommodate thermal movement.
    b. Improve water management details at vertical discontinuities in brick cladding and shelf angles of column cladding.
    c. Replace or repair corroded shelf angles near grade.
    d. Replace failed vertical joint sealant.

e. Install supplemental wall ties in select regions.

f. Clean isolated areas of efflorescence.

g. Repair stair tower storefront window assemblies.

h. Localized concrete repairs (both partial and full depth) of unsound concrete at stairs.

i. Clean and paint corroded steel stair handrail elements, repair steel posts as required.

7.4 Maintenance Recommendations

WJE recommends that the following maintenance items be completed on a regular basis, or as indicated.

1. Utilize snowplows with shoes, rubber tips, or small skis to prevent damage to the traffic-bearing membrane and perform the plowing in a manner that minimizes impacts. Do not store plowed snow on the supported levels.
2. Assess the traffic-bearing membrane on an annual basis in the spring to identify and repair de-bonded areas and scrapes related to snow plowing operations from the previous years.
3. Periodically assess the penetrating sealers and reapply as needed.
4. Remove accumulated debris and clean floor drains on a bi-annual basis.
5. Each spring, power wash and clean the deck surfaces to remove debris and the accumulation of deicing salts.
6. Periodically inspect overhead concrete surfaces and remove loose or unsound concrete.
7. Periodically assess and repair vehicle barrier cables.
8. Periodically assess and perform concrete repairs, as needed.

8.0 OPINION OF PROBABLE COSTS

8.1 Repair Project Cost

As shown in Appendix B, the probable construction cost to address the immediate and near-term repair recommendations (within the next 1 to 2 years) is on the order of $510,000. In addition, the probable cost to implement the remaining long-term recommendations (within the next 3 to 5 years) is approximately $870,000. This estimate includes a 15 percent contingency and a 10 percent budget for engineering, testing, and inspection. Based on experience with similar repair projects, WJE believes it is prudent to include a contingency to accommodate unforeseen conditions that are encountered during repair construction.

The majority of the unit costs contained in the construction cost estimate are based on costs for similar work on previous concrete repair projects located in the Midwest region. Repair quantities are based on the current level of deterioration and unit prices are in current dollars. Both are subject to increase over time. With regard to construction costs specifically, an increase of 3 percent per year is recommended to account for inflation. Actual costs will depend on a number of factors, including the bidding environment and owner-provided constraints. Please also keep in mind that the COVID-19 pandemic has made construction pricing and scheduling less predictable, and its influence is not accounted for in this cost estimate.

These cost estimates assume that all of the work recommended for each phase (near-term and long-term) will be performed during one construction project each (i.e., one large project to address the near-term items and one large project to address the long-term items). It is possible, and may be preferable to the owner, to perform the repairs in smaller work areas and over multiple years, or in a prioritized manner, in the event that funding is limited or parking spaces are not available. While smaller work areas occupy fewer parking spaces, an increase in both the duration and overall cost for the repair project should be anticipated. Similarly, cost efficiencies may be realized if all the recommended repairs are performed within one large near-term project.

8.2 Expected Maintenance Costs

This parking structure is nearly 40 years old. Given the exposure to moisture and deicing salts, concrete distress related to corrosion of the embedded reinforcement should be expected throughout the life of the parking structure. In particular, loose concrete removal, periodic sealant and expansion joint seal replacement, and penetrating sealer and traffic-bearing membrane repairs should be anticipated. Regular repairs and maintenance can decrease the rate of deterioration and increase the longevity of the parking structure.

Therefore, WJE recommends that an annual budget be established for such repairs and maintenance. In addition, a significant concrete repair and waterproofing project should be anticipated every 5-10 years for the remaining life of the parking structure. Maintenance and repair costs of parking structures increase exponentially over time due to exposure to aggressive environments. Maintenance of the concrete and waterproofing components of this garage should be expected. For this 170,000 square foot parking structure, we recommend a budget of approximately $200,000-$250,000 every 5 years, increasing as the structure ages.

9.0 CLOSING

WJE performed an assessment of the Peabody parking structure in Birmingham, Michigan, including a visual survey, investigative openings of the post-tensioned system, and materials testing. Based on the findings, WJE provided repair and maintenance recommendations and presented our opinion of the probable repair costs for budgeting purposes. At your request, and under separate authorization, WJE can prepare construction documents to implement the recommended repairs.

WJE appreciates the opportunity to be of continued service to The City of Birmingham. If you have questions, please feel free to contact us.

FIGURES

Figure 1. Area of unsound concrete along construction joint.

Figure 2. Previous partial-depth concrete repairs at topside of Level 7 construction joint.

Figure 3. Unsound previous repairs along the top of a beamline.

Figure 4. Unsound previous repairs along the top of a beamline.


Figure 5. Exposed, corroded mild reinforcement at spall in underside of slab between beams.


Figure 6. Exposed, corroded mild reinforcement within spall at construction joint. Note efflouresce and water staining along joint.

Figure 7. Previous partial-depth concrete repairs at Level 1 construction joint, aligned with mild reinforcing bars near bottom of slab surface.

Figure 8. Crack in slab at along beam line.

Figure 9. Isolated water staining and cracking in the slab near a perimeter column.

Figure 10. Previous PT tendon repair at Level 1.

Figure 11 . Grease stains at underside of elevated slab at construction joint.

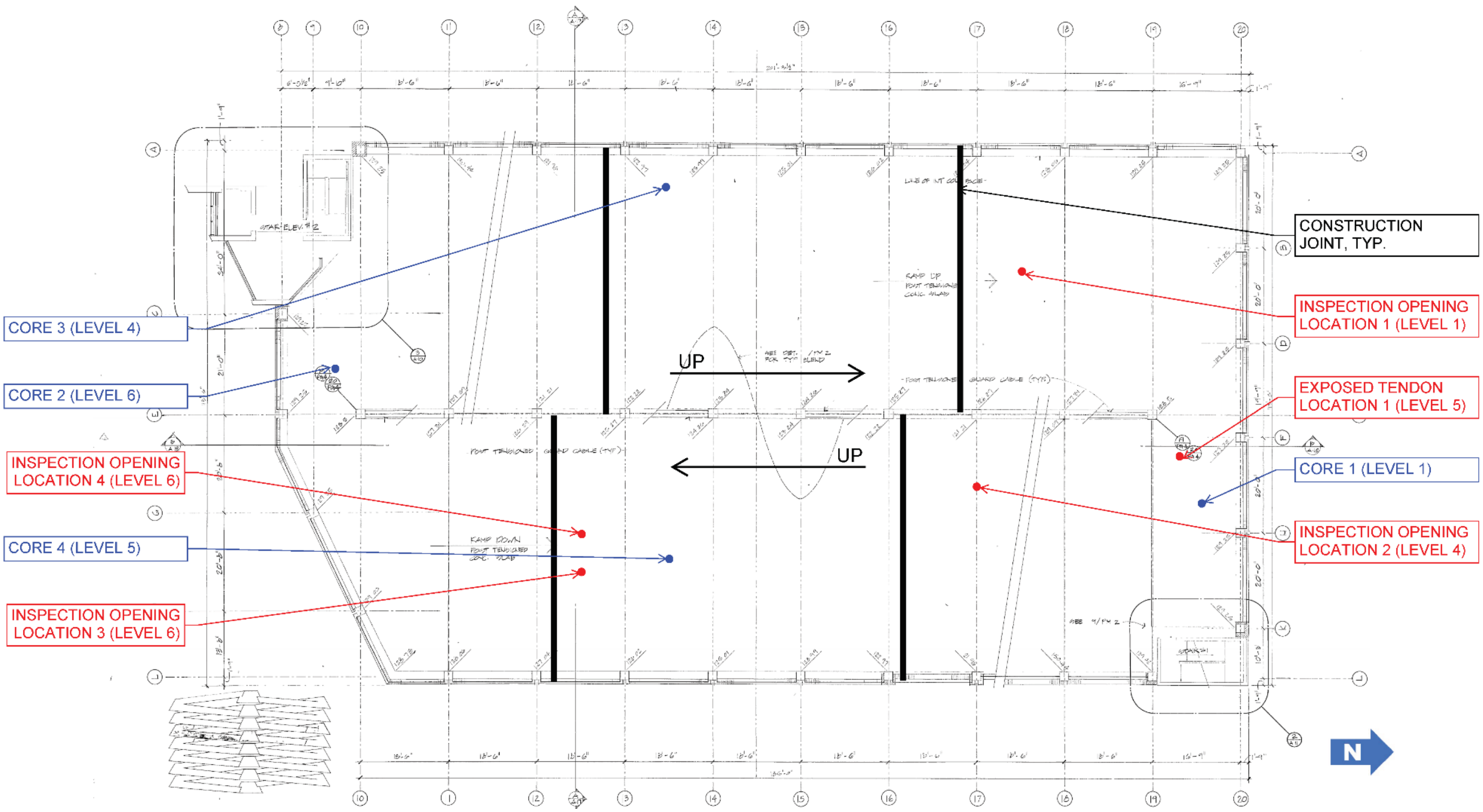

Figure 12. Inspection Opening Locations

Figure 13. Exposed Tendon Location 1

Figure 14. Inspection Opening Location 1


Figure 15. Inspection Opening Location 2


Figure 16. Inspection Opening Location 3

Figure 17. Inspection Opening Location 4

Figure 18. Failed expansion joint seal at the transition between the slab on grade and the elevated PT structural slab.

Figure 19. Joint sealant at vehicle entry between vaulted sidewalk region (plenum) and garage interior.

Figure 20. Water staining at perimeter joint between slab and wall intersection.

Figure 21. Deterioration at edge beam near southwest stairwell.

Figure 22. Previous epoxy injection repair at isolated crack at beam end.

Figure 23. Cracking and water staining at a beam-column intersection.

Figure 24. Cracking at the beam-column intersection.


Figure 25. Cracking with column on the Lower Level.


Figure 26. Debonded grout pockets, water staining, and efflorescence within column at vehicle barrier cable anchors. Note unsound concrete patch repair and standing water at base.


Figure 27. Deterioration within a large concrete patch repair at an interior column.


Figure 28. Cracking at CMU facade wall prior to recent coating of wall surface.

Figure 29. Concrete spall and water staining at foundation wall.

Figure 30. Steel corrosion and section loss of shelf angle near grade.

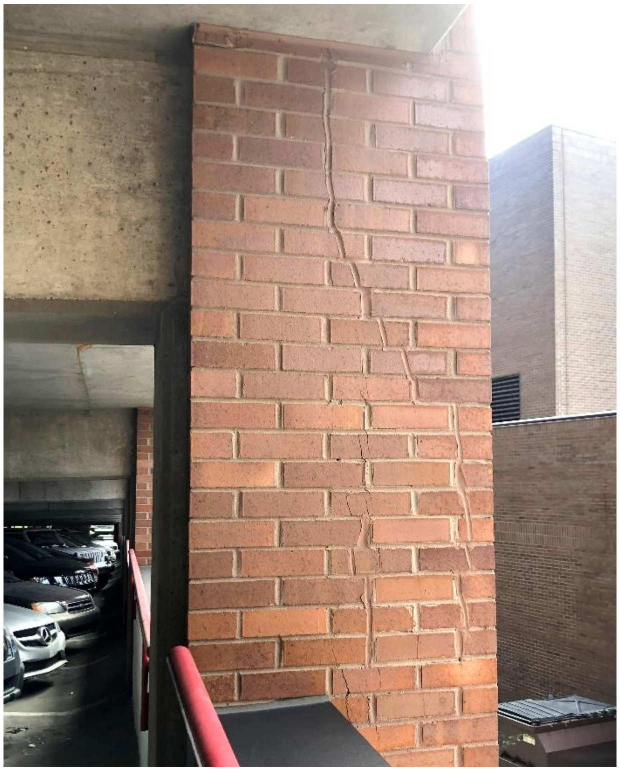
Figure 31. Vertical cracking in brick veneer at column cap.

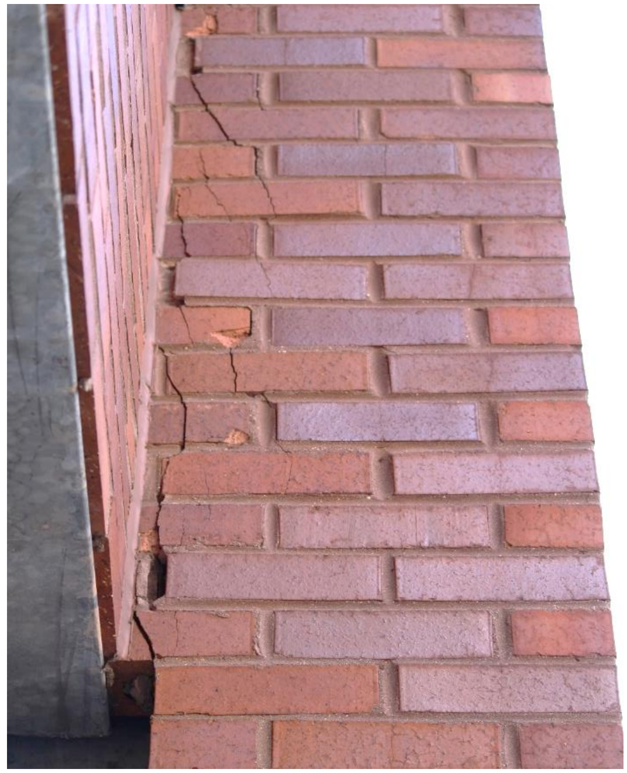
Figure 32. Vertical cracking in brick veneer at column cap.


Figure 33. Vertical cracking in brick veneer at column cap.


Figure 34. Close-up view of cracking in previous photo.

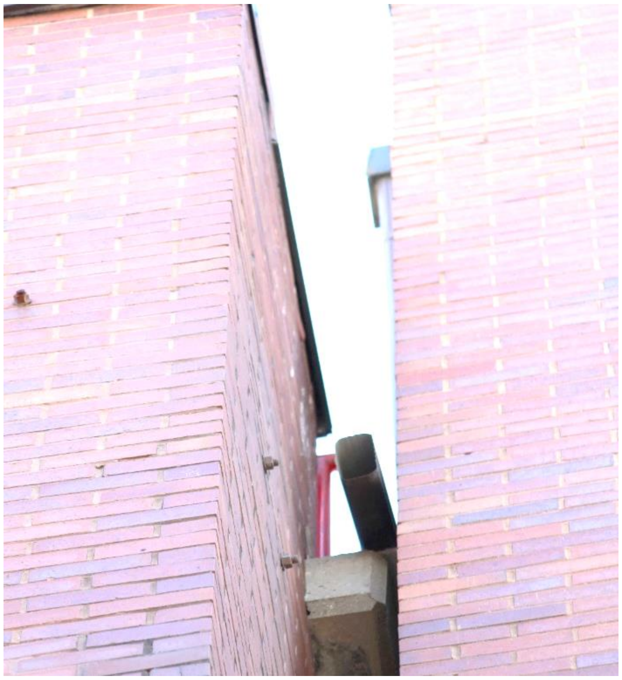
Figure 35. Stair tower roof downspout discharge.

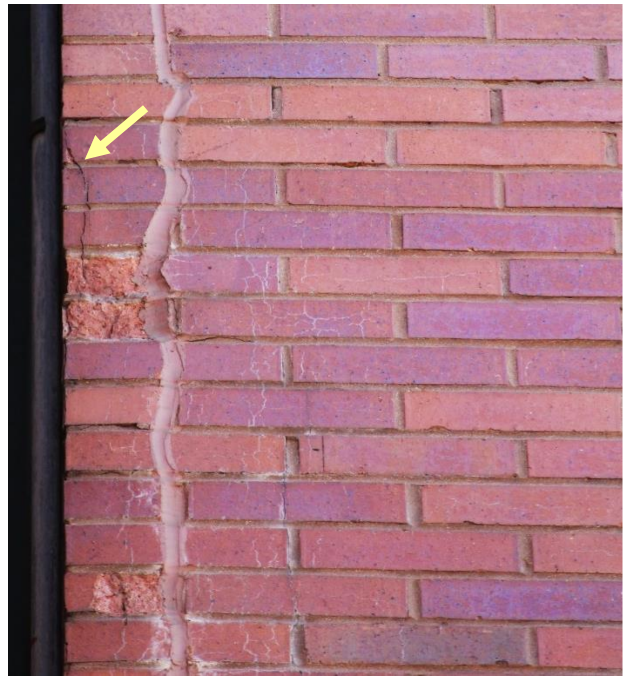
Figure 36. Distress within brick veneer below downspout.


Figure 37. Vertical crack in veneer.


Figure 38. Cracking in brick veneer.

Figure 39. Brick distress at vertical discontinuity. Note shrinkage cracks around slab grout pockets.

Figure 40. Brick distress at vertical discontinuity.

Figure 41. Debonded mortar at vertical discontinuity within brick cladding.

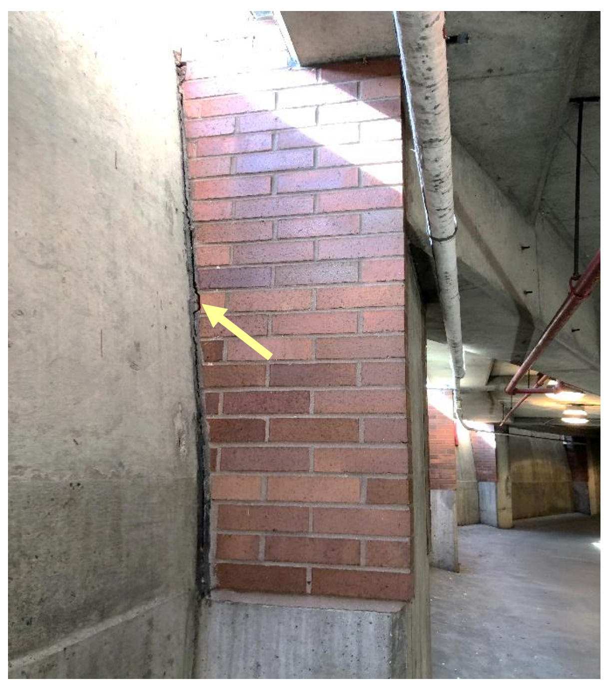

Figure 42. Failed sealant and mortar at Lower Level columns with isolated cracked brick units.


Figure 43. Cohesively failed sealant between CMU wall and brick veneer.


Figure 44. Typical debonded mortar joint between facade and stairwell.


Figure 45. Debonded mortar joint at southwest stairwell entry ramp.

Figure 46. Mortar erosion, efflorescence, and spalled brick units near base of wall at stairwell.

Figure 47. Deteriorated coating, perimeter sealant, and glazing at stairwell storefront.

Figure 48. Deformed door threshold at stairwell entry.

Figure 49. Spall at corroded stairwell handrail embed at landing.


Figure 50. Chipped paint and corrosion at stairwell handrail.


Figure 51. Corrosion at base of stairwell railing post.

Figure 52. Crazing cracking at Level 7 stairwell landing surface.

Figure 53. Cracking and efflorescence at underside of stair flight.

Figure 54. Delamination of previous concrete repair at underside of stair flight.


Figure 55. Corrosion of vehicle barriers and splice couplers.

APPENDIX A. MATERIALS TESTING REPORT

WJE

City of Birmingham
Parking Garage Structural Assessment Program

Peabody Parking Structure

222 Peabody Street
Birmingham, MI 48009

Karla Salahshour

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Senior Associate, Petrographer

LABORATORY REPORT

March 29, 2021

WJE No. 2019.6318.0

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CONTENTS

INTRODUCTION

Wiss, Janney, Elstner Associates, Inc. (WJE) completed laboratory testing on four concrete cores extracted from the Peabody Parking Structure located at 222 Peabody Street in Birmingham, Michigan. The Peabody parking structure was constructed during the mid-1980s and has seven levels of parking. The lower level is a reinforced concrete slab on ground, and Level 7 is uncovered rooftop parking. The structural system on the supported levels consists of a one-way post-tensioned (PT) slab supported by post-tensioned beams, and conventionally reinforced concrete columns. Laboratory testing was completed on concrete cores that were extracted from the elevated PT concrete slabs to characterize the material. The laboratory testing was completed as part of a larger investigation of the parking structure being performed by WJE's Detroit, Michigan office. The findings from this laboratory report will be used to assist in the repair recommendations for the parking structure.

SAMPLING

Four concrete cores were extracted throughout the parking structure and sent to WJE's Cleveland, Ohio laboratory for material testing. A summary of the core extraction locations is provided in Table 1. Photographs of the cores are provided in Figure 1. The cores were extracted vertically through the full thickness of the concrete floor slabs, and they ranged in length from 5-1/2 to 6-3/4 inches. The tops of the cores represent the exposed, wearing surface of the slab. The bottoms of all five cores are formed surfaces. No steel reinforcement was intersected by the cores, but the reinforcement is reportedly epoxy-coated.

Laboratory testing was performed on all four cores. A petrographic examination was requested on only Core 3 to characterize the concrete. Chloride ion content, water absorption, and carbonation tests were conducted on all four concrete cores. A summary of the testing performed is provided in Table 1.

Table 1. Summary of Peabody Parking Structure Concrete Cores

Core ID	Core Extraction Location	Location Description	Testing Performed			
			Petrographic Examination	Chloride Ion Content	Water Absorption	Carbonation
1	Level 1	Drive lane, near drain and stair in high traffic area at entrance		X	X	X
2	Level 6	Drive lane, near drain and stair at roof level		X	X	X
3	Level 4	Outside of drive lane in area of no visible distress	X	X	X	X
4	Level 5	Drive lane, in area of no visible distress		X	X	X

MATERIALS TESTING

Petrographic Examination

Methodology

Cursory examinations of the as-received core samples and saw-cut cross-sectional surfaces prepared for other laboratory testing were performed on all of the cores. A petrographic examination involving a more detailed examination of the material was conducted on Core 3 as part of the materials testing program. The petrographic study was conducted in accordance with the procedures described in ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete*. Microscope examination and various tests conducted during the petrographic examination are designed to elicit specific information about the composition and condition of the concrete. The observations are interpreted to derive conclusions about quality, performance, and probable cause of various types of distress.

A 3/4-inch thick slab was cut along the longitudinal axis from the middle of Core 3 using a water-cooled, continuous-rim, diamond saw blade. The saw-cut surfaces of the slab were then lapped using discs of progressively finer abrasives to achieve a fine, matte finish suitable for examination with a stereomicroscope. Lapping exposes textural features such that the edges of air voids, cracks, and aggregate constituents can be more easily identified. A lapped cross-section of the core is shown in Figure 2. Fresh fracture surfaces were also prepared to study the physical characteristics of the concrete. Lapped and fracture surfaces were examined at magnifications up to 90X using a stereomicroscope. A thin section was prepared from near the exterior surface of the core to further assess paste and aggregate characteristics. The thin section was examined at magnifications ranging from 50X to 630X using a petrographic (polarized-light) microscope.

Unit weight was measured for representative portions of each core according to Section 9, Unit Weight and Loss of Free Water, of ASTM C1084, *Standard Test Method for Portland-Cement Content of Hardened Hydraulic-Cement Concrete*. The results are provided in Table 2.

Findings

The concrete substrate in all of the cores appears compositionally similar based on a visual inspection of the saw-cut surfaces. Small areas that appear to be a different concrete mix was observed within Core 2 along the core perimeter, but this core was not selected for petrographic examination for further commentary. The cores contain blended siliceous and calcareous river gravel coarse aggregate and siliceous sand fine aggregate in an air-entrained, portland cement paste.

The coarse aggregate is composed of siliceous and calcareous natural gravel coarse aggregate. The particles are rounded in shape, multi-colored, uniformly distributed, and well graded. The maximum size particle is 1 inch. The fine aggregate consists of siliceous aggregates. A minor amount of aggregates, primarily chert particles within the fine aggregate, contain a darkened rim around their perimeter (Figure 3). These rims can be a naturally occurring feature in the aggregates prior to their incorporation into the concrete, but they can also be a result of alkali-silica reaction (ASR). Discontinuous rims around chert particles to adjacent voids were observed, suggesting some of the rims are a result of ASR. However, none of these particles were associated with distress, such as cracking.

The paste in the body of Core 3 is medium to dark gray in color. The top 1/8 inch is darker gray than in the body of the core. The paste was moderately hard and was not scratched using a copper probe. Residual portland cement particles were observed in thin section (Figure 4). No supplementary cementitious materials, such as fly ash or slag cement, were observed. Textural features observed microscopically are consistent with a moderately low water-to-cementitious materials ratio. The paste is air-entrained, and voids were observed as both small, spherical entrained air voids and irregularly-shaped, entrapped air voids (Figure 5). Some of the irregularly-shaped voids near the surface of the core and beneath coarse aggregate particles represent bleed water voids. The total volume of air was estimated to be 5 to 7 percent in Core 3. No secondary deposits were observed within the air voids.

The top surface of the core is medium gray in color. The surface profile was irregular due to minor preferential erosion of the paste resulting in fine aggregates being partially exposed on the surface (Figure 6). No distress was observed along the surface or in the near-surface region. The bottom of the core is a formed surface, and small bugholes were present. No distress was observed along the bottom surface of the core.

No significant cracking or other distress was observed in the examined core.

Chloride Ion Content

Methodology

The water-soluble chloride ion contents were determined for the four cores at multiple depths. The depths were selected near the top surface of the topping (1/4 to 3/4 inch from the top) to determine if deicing salts, either applied directly to the slabs or carried in by vehicular traffic over time, penetrated into the concrete slab. The next depth (1-1/2 to 2 inches from the top) is located near the top level of mild reinforcing, epoxy-coated steel. A mid-depth (3 to 3-1/2 inches) slice was selected to serve as a baseline for the concrete. A depth near the bottom of the slab (depth varies due to slight differences in core lengths) was selected to determine if chlorides from spray from beneath the slab or sub-base conditions have penetrated into the concrete.

The water-soluble chloride analysis was performed essentially according to ASTM C1218, *Standard Test Method for Water-Soluble Chloride in Mortar and Concrete*. The results are provided in Table 2.

Findings

Studies performed by WJE have shown that chloride contents above approximately 0.06 percent by mass of concrete, depending on the cement content, can promote corrosion of embedded epoxy-coated steel in non-carbonated normal weight concrete in the presence of sufficient moisture and oxygen[1]. Levels below this threshold may accelerate corrosion in carbonated concrete. The chloride ion contents measured for the top surface in Cores 1, 2, and 3 are significantly in excess of this threshold. The chloride ion content measured for the next depth from the surface in Core 2 is at this threshold value. The chloride

[1] Lawler, John S.; Kurth, Jonah C.; Garrett, Stephen M.; Krauss, Paul D. (2021), "Statistical Distributions for Chloride Thresholds of Reinforcing Bars," *ACI Materials Journal*, v. 118 n. 2, p. 13-20

ion content measured at the surface of Core 4 is below the threshold. The chloride ion contents measured at the surface were significantly elevated over the chloride ion content at the second depth, indicating a decrease in chloride with depth from the surface. This gradients suggests an external source of chloride, such as from deicing salts on the slab surface as would be expected. The chloride ion contents along the bottom of the cores is below the threshold, although slightly elevated compared to the body of the cores (except for Core 2) and may suggest a source of chloride, albeit very minimal, from the underside of the slabs.

Water Absorption

Methodology

During the laboratory testing, an assessment of the absorptivity of the top surface was requested to aid in the determination of a repair design for the parking structure. During this testing, water drops were applied to the as-received surface of each of the cores, and the shape and absorption of the water drop were recorded. Water drops were also applied at several locations on a laboratory-prepared fresh fracture surface of each core oriented perpendicular to the top surface. The absorptivity of each of the water drops was recorded with depth from the top surface. Results are provided in Table 2.

Findings

Water drops applied to the surfaces of Cores 1 and 2 were tightly beaded, meaning they retained their spherical shape and did not spread across the surface, and were not absorbed into the surface paste (Figure 7). The paste on a fracture surface near the top surface of these two cores exhibited hydrophobic properties (i.e. water drops were not absorbed) to a maximum depth of 1/4 inch. These observations suggest the penetration of a sealer-like material that still imparts hydrophobic properties on the surface of the cores.

The water drops applied to the surface of Core 3 beaded, although not as tightly as in Cores 1 and 2, eventually lost their surface tension and spread across the surface but were not absorbed into the surface paste. The paste on the fracture surface was hydrophobic to a depth of 1/8 inch. (This depth corresponds to the depth of darker gray paste observed along the top surface of Core 3 microscopically.) These observations may indicate that a penetrating sealer may have been applied in this area of the parking structure but has somewhat deteriorated along the surface and did not penetrate to as great of depths as in Cores 1 and 2.

Water drops applied to the surface of Core 4 spread and were absorbed by the paste. The paste on the fracture surface was hydrophobic to a depth of 3/8 inch. These observations suggest that a sealer-like material may have penetrated into the top of the concrete in the area from which the core was extracted but has since completely deteriorated along the surface. Interestingly, the top of Core 4 exhibits the most severe erosion exposing a greater number of aggregates compared to the other cores.

Carbonation Depth

Methodology

One half of each of the four cores was fractured longitudinally in the laboratory for the carbonation studies. The fracture surface was blown free of debris using compressed air and treated with phenolphthalein indicator solution. The indicator solution will turn non-carbonated paste purple; carbonated paste will remain unchanged. Paste that exhibits a light purple color is judged to be partially carbonated. Carbonated paste loses its natural passivation of the embedded, uncoated reinforcing steel due to the reduction in pH of the paste. In the presence of moisture and oxygen, the steel is susceptible to corrosion. The depth of paste carbonation from the top and bottom surfaces are provided in Table 2.

Findings

The maximum depth of paste carbonation from the top surface of the cores was 1/4 inch. The depths of carbonation from the top surface, both fully and partially carbonated depths, have not yet reached the depth of reinforcing steel, assumed to be at least 1 inch but assessed separately during Detroit's investigation of the parking structure.

The maximum depth of complete paste carbonation from the bottom surface was 3/4 inch. While the minimum depth of cover for the PT strands in the slabs was not reported to the laboratory, the depth of carbonation from the bottom surface may be nearing the reinforcement.

Table 2. Summary of Material Testing

Core ID	Core Length (inch)	Unit Weight (pcf)	Chloride: Depth from Top Surface (inch)	Chloride: Water-Soluble Chloride (% by mass of sample)	Water Absorption Description[1]	Carbonation: From Top Surface (inch)	Carbonation: From Bottom Surface (inch)
1	5-1/2	144	$^{1}/_{4}$ - $^{3}/_{4}$	0.152	Top - water drop tightly beaded, not absorbed FF - hydrophobic to 1/4 inch	1/4 to 3/8	3/4
			1 $^{1}/_{2}$ - 2	0.007			
			3 - 3 $^{1}/_{2}$	<0.003			
			5 $^{1}/_{4}$ - 5 $^{3}/_{4}$	0.007			
2	5-1/2	148	$^{1}/_{4}$ - $^{3}/_{4}$	0.206	Top - water drop tightly beaded, not absorbed FF - hydrophobic to 1/4 inch	3/8	1/4 (partial)
			1 $^{1}/_{2}$ - 2	0.062			
			3 - 3 $^{1}/_{2}$	<0.003			
			4 $^{1}/_{2}$ - 5	<0.003			
3	6-3/4	148	$^{1}/_{4}$ - $^{3}/_{4}$	0.214	Top - water drop loosely beaded then spread, not absorbed FF - hydrophobic to 1/8 inch	1/4	1/2 (partial)
			1 $^{1}/_{2}$ - 2	<0.003			
			3 - 3 $^{1}/_{2}$	<0.003			
			4 $^{3}/_{4}$ - 5 $^{1}/_{4}$	0.008			
4	6-1/8	153	$^{1}/_{4}$ - $^{3}/_{4}$	0.024	Top - water spread and was absorbed FF - hydrophobic to 3/8 inch	3/8	1/4 to 3/8 (partial)
			1 $^{1}/_{2}$ - 2	0.004			
			3 - 3 $^{1}/_{2}$	<0.003			
			4 $^{3}/_{4}$ - 5 $^{1}/_{4}$	0.008			

[1] FF = fresh fracture surface prepared in the laboratory to which water was applied

FIGURES

Figure 1. The as-received appearance of the tops (upper), bottoms (middle), and sides (lower) of Cores 1 through 4 are pictured, from the left to right, respectively. An area of dissimilar mortar in Core 2 is identified with an arrow.

Figure 2. Lapped surface of Core 3.

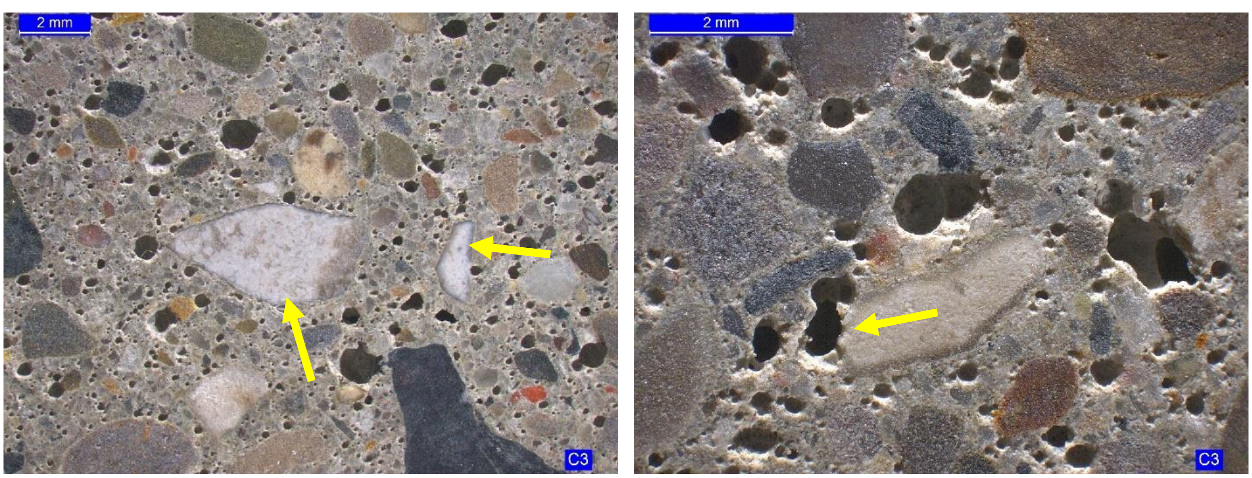

Figure 3. Darkened rims (arrows) are pictured around several chert particles within Core 3. The rim is discontinuous adjacent the air void in the right image, suggesting its formation after being incorporated into the concrete, such as from ASR. No distress was associated with any of these particles.

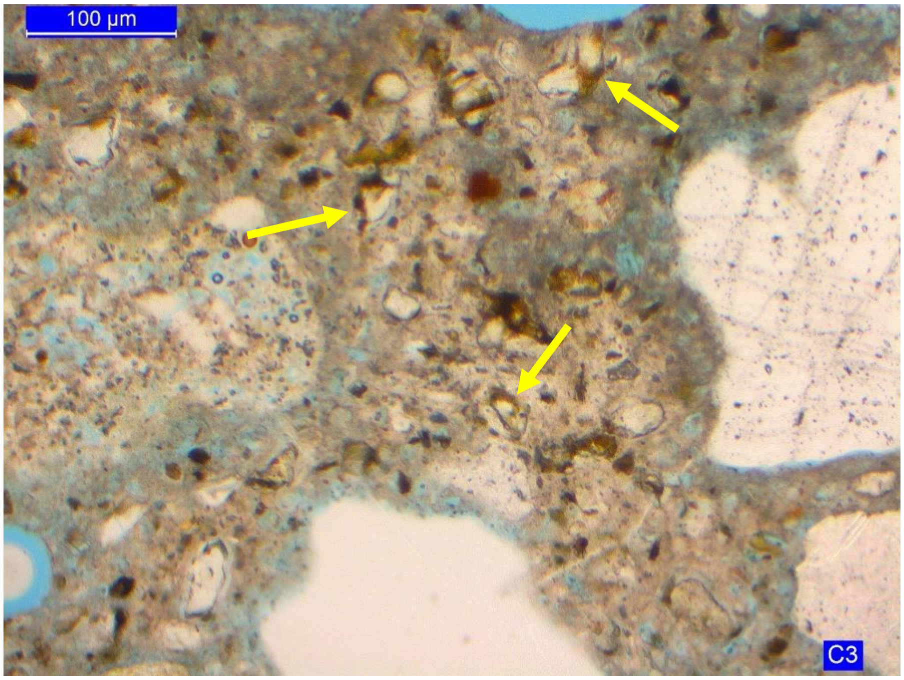

Figure 4. Portland cement particles (yellow arrows) are pictured in Core 3.

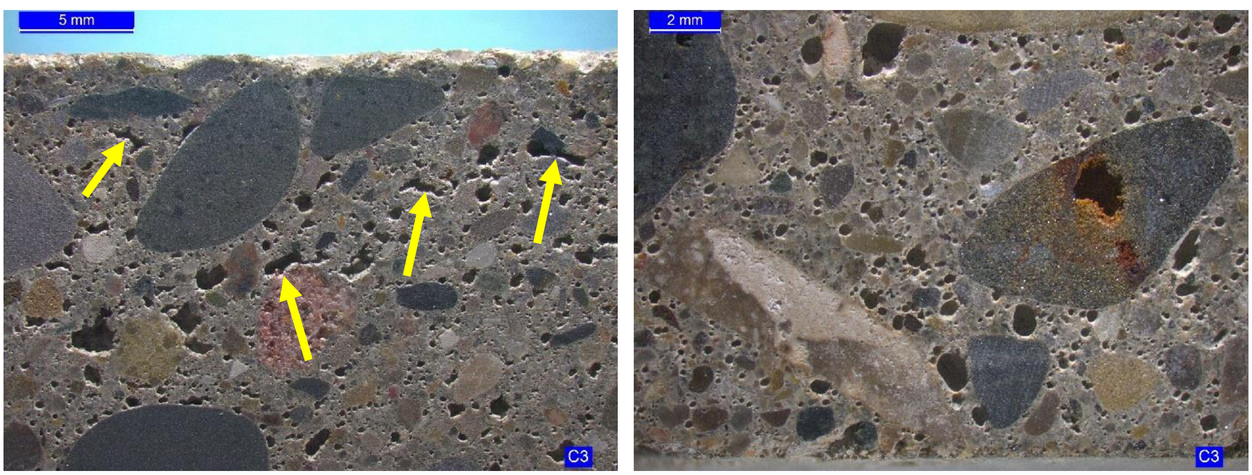

Figure 5. The air void system is pictured for Core 3 near the top (left) and the bottom (right). The irregularly-shaped voids (arrows) near the surface of Core 3 represent bleed water channels.

Figure 6. The top eroded surface of Core 3 with partially exposed fine aggregates.

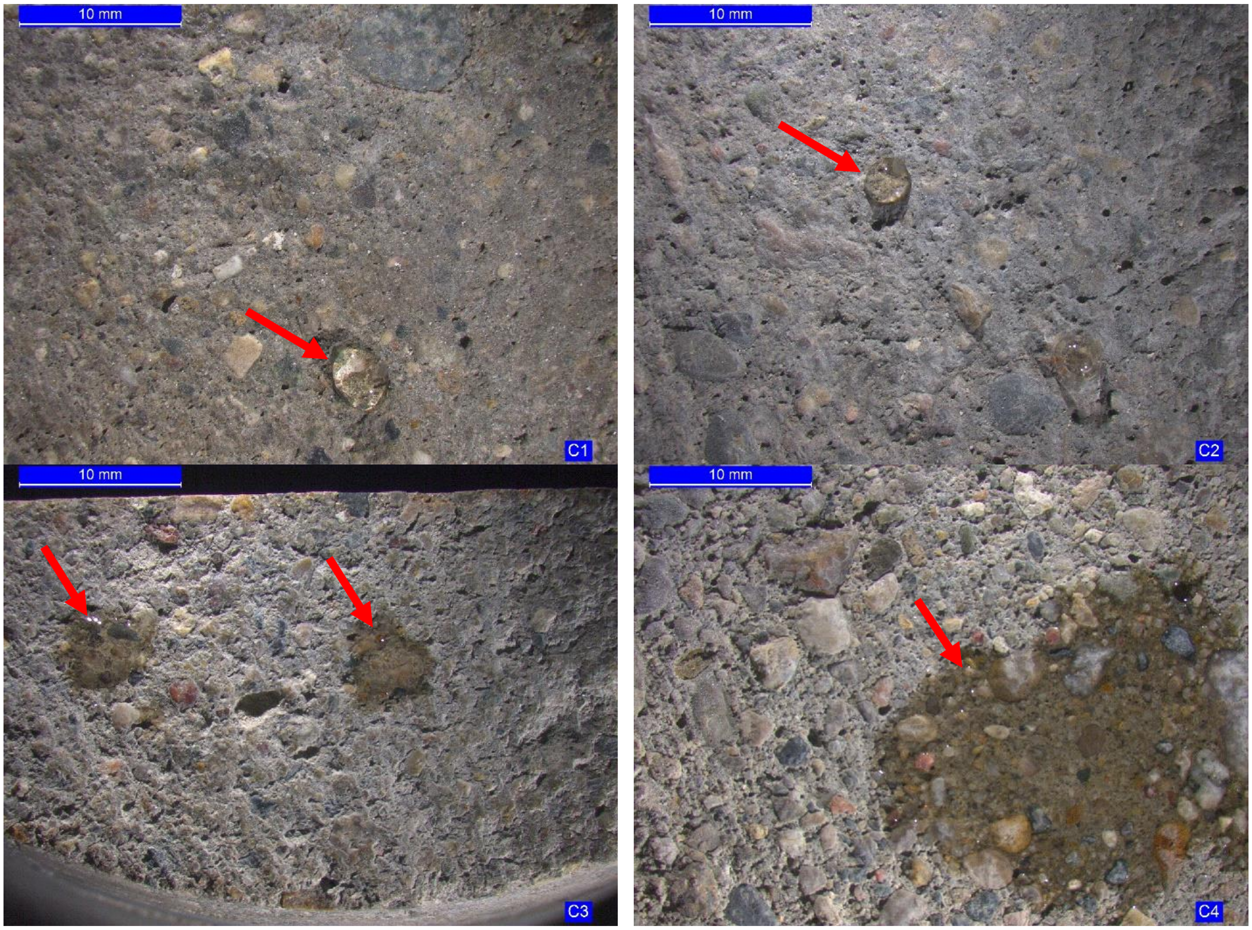

Figure 7. Water drops (arrows) applied to the surface are pictured for all five cores. Water drops beaded on Cores 1 and 2, beaded but then spread on Core 3, and spread immediately on Core 4.

APPENDIX B. OPINION OF PROBABLE COSTS

Immediate Recommendations (within 1 Year)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Investigation and repair of two Lower Level columns ǂ	1	LS	$ 50,000	$ 50,000
Immediate Recommendations Total				**$ 50,000**
Near-Term Recommendations (within 1 to 2 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Localized concrete repairs in slabs, full depth	300	SF	$ 80	$ 24,000
Localized concrete repairs in slabs, partial depth topside	500	SF	$ 45	$ 22,500
P/T slab tendon splice and materials - allowance	1	LS	$ 50,000	$ 50,000
Replace construction joint sealant*	1,500	LF	$ 6	$ 9,000
Rout and seal cracks in elevated slabs and replace failed sealant at isolated cracks	500	LF	$ 6	$ 3,000
Replace expansion joint seals*	150	LF	$ 125	$ 18,750
Install traffic bearing membrane at construction joints, occupied areas, and vehicle entrance lanes	25,000	SF	$ 5	$ 125,000
Apply concrete sealer at all elevated levels	147,500	SF	$ 0.40	$ 59,000
Inspect and clean drain lines*	1	LS	$ 15,000	$ 15,000
Subtotal				**$ 326,250**
General Conditions, Overhead and Profit (15%)				$ 48,938
Project Contingency (15%)				$ 48,938
Engineering/Testing/Construction Period Services (10%)				$ 32,625
Near-Term Recommendations Total				**$ 456,750**
Long-Term Recommendations (within 3 to 5 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost**
Concrete Structure Repairs				
Localized concrete repairs in slabs, full depth	150	SF	$ 80	$ 12,000
Localized concrete repairs in slabs, partial depth	100	SF	$ 45	$ 4,500
P/T slab tendon splice and materials - allowance	1	LS	$ 25,000	$ 25,000
Partial depth concrete repairs at beams, columns, foundation walls, and stairs and isolated crack repairs at beam-column intersections	1,000	SF	$ 90	$ 90,000
Waterproofing Repairs				
Install traffic bearing membrane at drains and concrete repairs	2,500	SF	$ 8	$ 20,000
Replace cove sealant at roof level, install cove sealant at other isolated locations	2,500	LF	$ 6	$ 15,000
Modify stair tower roof downspouts	1	LS	$ 2,500	$ 2,500
Facade, Stair Tower, and Miscellaneous Repairs				
Repair brick masonry cladding - allowance	1	LS	$ 250,000	$ 250,000
Repair stairwell storefront assemblies	1	LS	$ 50,000	$ 50,000
Stairwell handrail repairs, clean and paint metal surfaces	1	LS	$ 150,000	$ 150,000
Subtotal				**$ 619,000**
General Conditions, Overhead and Profit (15%)				$ 92,850
Project Contingency (15%)				$ 92,850
Engineering/Testing/Construction Period Services (10%)				$ 61,900
Total				**$ 866,600**
* Highest priority of near-term repair items.				
** Prices based on current (2021) dollars, and do not include increases for inflation (recommended 3% per year).				
ǂ Pending further analysis during repair design phase; includes engineering, shoring, and masonry allowances.				

City of Birmingham
Parking Garage Structural Assessment Program

Park Street Parking Structure

333 Park Street
Birmingham, MI 48009

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FINAL REPORT

April 30, 2021

WJE No. 2019.6318.0

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CONTENTS

1.0 INTRODUCTION

As requested, Wiss, Janney, Elstner Associates, Inc. (WJE) completed limited condition assessments of the North Old Woodward, Park Street, Peabody and Chester parking structures. These assessments were performed with the intent to determine the current and future infrastructure needs in support of a capital improvement plan; the intention of the plan is to extend the useful life of the structures and maintain structural integrity to ensure the structure can support the code-prescribed loadings. This report summarizes our observations at the Park Street Parking Structure, located at 222 Park Street in Birmingham, Michigan, and provides recommendations for your consideration.

2.0 STRUCTURE DESCRIPTION

The Park Street parking structure was constructed during the mid-1970s and has five levels of parking with a centralized ramp. Level 1 and a portion of the ramp from Level 1 to Level 2 are a reinforced concrete slab on ground, and Level 5 is uncovered rooftop parking. The four-bay, side-by-side structure is rectangular in plan, with approximate overall dimensions of 250 by 225 feet, for a total area of about 270,000 square feet between all levels. The north and south ends of the structure are unsloped, while the remaining bays of the garage are sloped to serve as circulation ramps.

The structure consists of 5-1/2-inch-thick, one-way, post-tensioned (PT) concrete slabs supported by steel beams, girders, and columns. The PT tendons consist of single 7-wire strands in plastic sheathing. The structural tendons run in the north-south direction, and are spaced at approximately 36 inches on center. The temperature and shrinkage tendons run in the east-west direction and are spaced at approximately 54 inches on center. One construction joint exists in each bay, located about 40 feet upslope from the bay's midpoint. The majority of the slabs are coated with a traffic-bearing membrane. The exterior perimeter columns are steel pipes, and the interior columns, beams, and girders are steel wide flange shapes. Steel tube diagonal bracing members, intended to provide lateral stability in the north-south direction, span between four pairs of columns at the ends of both interior column grid lines. Steel moment frames at the north and south ends of the interior column grid provide lateral stability in the east-west direction.

Steel channels that span between the interior columns run along the lengths of the interior slab edges at an approximate height of two feet above the slab, to act as vehicle barriers between ramps. Intermediate steel wide flange posts that run the full height of the structure are located between interior columns and provide lateral support to the vehicle barriers at their midspans. These intermediate posts are embedded in concrete foundations at their base, and are supported by bolted connections to angles at each of the slab edges above.

The facade consists of corrugated metal panels supported by light gauge steel vertical posts anchored to the top surface of the concrete slabs. Struts along the bottom edges of the panels connect to the steel edge beams below to provide lateral support to the facade panels. Stair towers with concrete masonry (CMU) walls, brick veneer cladding, and steel stairs are present at each of the four garage corners, with expansion joints separating the towers from the remaining structure.

2.1 Document Review and Background

WJE reviewed relevant sheets of the original construction drawings, dated July 31, 1973 and authored by Jickling & Lyman Architects, Inc., as part of our assessment. Based on our site visit observations, several past restoration projects have occurred at the building, which are described in detail in Section 4.0; however, documentation related to these efforts was not provided to WJE for review.

3.0 FIELD ASSESSMENT

WJE visited the site on several occasions in January 2020 to perform a visual assessment of the accessible and exposed portions of the structure and facade. WJE returned to the site in May 2020 to perform a delamination survey at representative locations. WJE also returned to the site on multiple occasions throughout February 2021 to perform non-destructive evaluation measures, review inspection openings, extract concrete cores for materials testing, and complete additional assessment efforts.

WJE's scope included a limited sounding survey of the supported levels in accordance with *ASTM D4580 - Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding.* For this survey, areas of delamination were identified using the chain-drag method, localized hammer sounding, and use of a delamination wheel at select underside locations. In areas of sound concrete, these methods produce a ringing sound, and when a delamination is encountered, a hollow, drum-like sound is produced. Between approximately 25 and 50 percent of the total area for each floor was surveyed, including all construction joints where intermediate PT anchorages are located. Sounding of the underside of the slab with a delamination wheel was primarily limited to locations of previous repair and visible indications of potential concrete deterioration (e.g. at visible cracks, spalls, etc.). A summary of pertinent observations follows.

3.1 Structural Components

3.1.1 Post-Tensioning Tendons

The structural post-tensioning (PT) tendons are grouped in pairs and span in the north-south direction, with one intermediate anchorage point at a construction joint in each bay. Temperature tendons span perpendicular to the structural tendons. In general, the PT tendons and previous PT tendon repairs are in serviceable condition with localized distress, as described below.

Visual Observations

1. At least 5 spalls that each expose one or more PT tendons exist at the underside of the slabs throughout the elevated levels. These spalls are located at an edge of a slab where temperature tendon anchors are located, near construction joints where structural tendon intermediate anchors are located, and near slab midspans in areas of low concrete cover (Figure 1, Figure 2**Error! Reference source not found.**, Figure).
2. Isolated PT tendon repairs are visible at each elevated slab and are generally sound. These repairs are often located over beams and intermediate anchorage points (Figure 4). Several repairs were noted to be unsound at the north bay of Level 4, but generally less than 25 percent of the PT repairs are unsound (Figure 5).

3. Several PT anchor repairs are visible throughout all elevated slabs at the construction joints where structural tendon anchors are present. and at the slab edges where temperature tendon anchors are present, as well as at a few isolated locations elsewhere in the garage. Typically, the anchor repairs are sound, but progressive deterioration is present at many edge of slab repairs (Figure 6).
4. Evenly spaced cracks perpendicular to two construction joints (and in line with structural tendons) exist at Level 3 at approximately 36 inch spacing (Figure 7). The cracks have been routed and sealed, but the membrane is worn over the joint. The concrete is sound at the topside near these cracks, but several failed PT anchors are visible at the underside along this joint.
5. Narrow cracks in line with temperature tendons are visible at the underside of the slab and exist primarily at the unsloped areas of Levels 3 and 4. In a few cases, these cracks extend through previous repairs with isolated unsound concrete near these cracks (Figure 8).
6. Grease stains on the underside of the slabs are visible in about 5 to 10 locations per level on average, with the most grease stains at Level 3. These stains are generally present near PT anchor failures, failed tendons, or low points between the beams (Figure 9).

Inspection Openings

Based on our visual observations, locations were chosen for inspection openings to evaluate the condition of the concealed PT tendons. A local concrete restoration contractor, Pullman SST, created and repaired the inspection openings specified by WJE. Refer to Figure 10 for the locations of these tendons.

1. Eleven tendons exposed at existing spalls were inspected on the underside of the floor slabs.
    a. **Exposed Location 1:** Two tendons are exposed at a location of low concrete cover. Both tendons were holding tension. The sheathing was intact, with grease filling the sheathing voids (Figure 11).
    b. **Exposed Location 2:** Two tendons are exposed at an area of spalled concrete. Both tendons were holding tension. One tendon had a tear in its sheathing with congealed grease that fell from the opening when probed with a screwdriver. Both tendons exhibit corrosion and minimal grease that was congealed (Figure 12).
    c. **Exposed Location 3:** One tendon is exposed at a location of spalled concrete within a previous repair area; a drain is near this location on the roof level above. The tendon is not holding tension (has failed). The strand wires exhibit significant corrosion with complete section loss of most wires. The sheathing is torn and deteriorated along the length of the exposed portion of the tendon, and grease is not present (Figure 13).
    d. **Exposed Location 4**: Four tendon intermediate anchors are exposed at a construction joint (Figure 14). All four anchors are heavily corroded, and the plastic end encapsulation is broken and/or cracked. Corrosion is visible within the encapsulated end system (Figure 15). The tendons were not tested for tension due to safety concerns related to the risk of destabilizing the anchorage.
    e. **Exposed Location 5:** Two temperature tendon end anchors at an interior slab edge are visibly failed. The anchors are located behind a corroded reinforcing steel bar perpendicular to the strands; the bar is deflected, and one of the anchors is partially dislodged at the bar. The sheathing of these tendons was missing near the anchors with duct tape partially applied, likely during a previous repair effort (Figure 16).

The tendons were not tested for tension due to safety concerns related to the risk of further destabilizing the anchorage. Additionally, the tendons and end anchors exhibit surface corrosion.

2. Six inspection openings were created throughout the elevated levels of the structure.
   a. **Inspection Opening 1**: One tendon was exposed at a location of a delaminated previous repair on the topside of the Level 3 slab. This opening exposed a stressing coupler with low concrete cover within the concrete repair area. The tendon sheathing and stressing coupler were intact with no visible distress (Figure 17). A second tendon within the opening appeared to be threaded through a section of hose within a collar, likely HDPE, covered in fabric. This tendon also did not exhibit any signs of distress.
   b. **Inspection Opening 2**: This opening was created directly in line with a grease stain, and the opening exposed two tendons. Both tendons were holding tension, and the sheathing was intact. The grease on both tendons had corrosion byproduct, indicative of corrosion somewhere along the length of the tendon, and the grease at one tendon was congealed (Figure 18).
   c. **Inspection Opening 3**: This opening was created directly in line with a previous repair and grease stain, and the opening exposed two tendons. Both tendons were holding tension. The sheathing was intact, with grease filling the sheathing voids, and the tendon exhibited no visible corrosion (Figure 19).
   d. **Inspection Opening 4**: This opening was created adjacent to Inspection Opening 3 for comparison, not in line with a PT repair area or grease stain, and the opening exposed 2 tendons. Both tendons were holding tension. The sheathing was intact. The grease was congealed and has corrosion byproduct, though no section loss of the tendon due to corrosion was observed (Figure 20).
   e. **Inspection Opening 5**: This opening was created directly in line with a grease stain, and the opening exposed two tendons. Both tendons were holding tension. The sheathing was intact, with grease filling the sheathing voids, and the tendon exhibited no visible corrosion (Figure 21).
   f. **Inspection Opening 6**: This opening was created adjacent to Inspection Opening 5 for comparison, not in line with a grease stain, and exposed 2 tendons. Both tendons were holding tension. The sheathing was intact, with grease filling the sheathing voids, and the tendon exhibited no visible corrosion (Figure 22).

3.1.2 Structural Floor Slabs

Though the post-tension tendons comprise the majority of the reinforcing steel within the structural floor slabs, mild reinforcing bars are also present, such as near PT anchorage zones and near drains. Embedded steel conduit is also present from the original deck construction but has since been abandoned. Cold joints that run in the north-south direction are located within the interior ramp of each elevated level. The condition of the concrete and mild reinforcing of slabs, beyond the PT-related distress, was generally serviceable, with localized areas of distress concentrated at the upper levels. Notable conditions and deterioration are described below.

1. A few previous concrete repairs are visible at the topside of the elevated slabs and are generally located near beamlines and drains (Figure 23, Figure 24). These repair areas are typically covered by the traffic-bearing membrane.

2. Several concrete repairs are visible on the underside of the elevated slabs. Many of these repairs exhibit signs of deterioration, including narrow cracking with signs of moisture infiltration, corrosion staining, and efflorescence. (Figure 25, Figure 26).
3. Localized areas of spalled and unsound concrete were identified throughout the elevated slabs during the delamination survey. In general, less than 5-10 percent of the areas surveyed were unsound.
    a. Delaminations in the topside of the slab are primarily concentrated over and near the beams, where the PT tendons are near the top of the slab and have lower concrete cover (Figure 27). Unsound areas of concrete at the topside of the slab were also concentrated at the north or south ends of the garage, where the floor slab is not sloped and drains are present (Figure 28). Approximately 25 percent of the previous concrete repair areas at the topside of the slab were noted to be unsound. Localized areas of spalled and incipient spalled concrete exist at the top surface of the concrete slabs throughout the garage and are concentrated at Level 5 and over and near the beams (Figure 29). These spalls are approximately 1 square foot each and were observed at up to about 15 locations at the roof level, with fewer spalls present at the remaining elevated levels.
    b. Spalls or areas of unsound concrete are present on the underside of the elevated concrete slabs and are typically concentrated at the upper levels. (Figure 30). Common locations for underside spalls and other associated distress conditions are adjacent to the exterior ends of the beams in the unsloped bays, and along the slab edges at areas of corroded mild reinforcement or embedded steel conduit (Figure 31, Figure 32). Some of the previously repaired areas contain regions of unsound material, particularly at the trowel-applied repairs rather than at the formed repairs.
4. A few previously sealed cracks at the topside of the slab exist at all levels, some of which are within areas of previous repair or near construction joints or drains (Figure 33, Figure 34). The majority of cracks visible at the top side of the slab are covered by the traffic-bearing membrane and do not exhibit visible deterioration, but failed sealant is present at a few cracks (Figure 35).
5. Longitudinal cracks exist at the underside of the elevated slabs, primarily at expected locations of embedded conduit or mild steel within the unsloped north and south bays. Most of these cracks have previously been injected and patched; the crack repairs have typically failed and exhibit moisture staining (Figure 36).

3.1.3 Primary Structural Steel Elements

The primary structural steel framing consists of sloped beams supported by columns at the interior of the garage, with girders supporting beams at the north and south ends of the structure. The original construction drawings specify that select beams have plates welded to their bottom flanges. WJE verified the presence of these plates with the as-built conditions and found that, in addition to the beams specified in the drawings, plates had also been added to the beams supporting Level 5. Based on the quality of the welds, WJE assumes this modification was made during the original deck construction. Moment connections are present along grid lines 3 and 12 of the original construction drawings between interior columns. Steel tube diagonal bracing between four pairs of interior columns provides lateral support in the north-south direction. The steel members typically exhibit varying levels of corrosion, but are generally in serviceable condition with localized regions of distress.

1. About half of the interior columns at the slab on ground have missing or loose nuts at their base plates (Figure 37, Figure 38). One of the bolts is missing and appears to never have been installed at the northeast interior column (Figure 39).
2. Most of the exterior pipe columns are embedded into the slab on ground and exhibit minor section loss (Figure 40, Figure 41). Where exposed, the base plate of the exterior column typically exhibits failed coating and corrosion (Figure 42).
3. The interior columns are embedded into the elevated slabs are and typically corroded but do not exhibit significant section loss. (Figure 43, Figure 44).
4. The column cap plates and top flanges of steel beams in contact with the floor slabs typically exhibit minor to moderate corrosion staining (Figure 45). Significant section loss was not readily visible.
5. Within the moment frame connections, the column stiffener plates typically exhibit severe section loss while the bottom flange plates are often deformed due to the buildup of corrosion byproduct between the bottom flange plate and the beam (Figure 46, Figure 47, Figure 48). The moment connections near the diagonal bracing members and slab edges are typically more severely deteriorated than the moment connections near the center of the garage with no nearby openings in the slab.
6. Elements of other framing connections, such as shear connections or diagonal bracing connections, are typically corroded, but do not exhibit significant section loss or deformation (Figure 49, Figure 50).
7. The steel beams are generally visibly deflected under dead loads only (self-weight). Steel beams supporting the exterior bays were deflected as much as 3-1/2" at midspan without cars above, while midspan deflections were closer to 2" at the beams supporting the interior bays (Figure 51). These beams nominally span 57 feet. Deflection measurements were recorded at select locations using a laser level.
8. All structural steel framing elements have recently been recoated. The corrosion buildup within the connections described above has typically been coated over (Figure 52).

3.1.4 Secondary Structural Steel Elements

The secondary structural steel elements as defined herein by WJE consist of the steel channel vehicle barriers, the intermediate posts between columns, and the wide flange beams along the edges of the slabs. The vehicle barriers are supported at either end by bolted connections to the column flanges, while the intermediate posts provide lateral support to the vehicle barriers at midspan via a plate that is welded to the vehicle barrier and bolted to the post. The intermediate posts are embedded into a concrete foundation at the slab on ground and are bolted to angles at each slab edge along their length. Sheet metal flashing covers the exposed concrete slab edge and steel angles. Steel wide flange beams that span between columns run along the edges of the slabs just below the edge of slab angles. These beams are bolted to column flanges at either end and support the slab edge on the north and south ends of the deck. Steel wide flange beams also span in the north-south direction between columns along the central column line of the interior ramp.

1. A steel wide flange beam supporting Level 4 that spans in the north-south direction between columns along the central column line of the interior ramp is laterally deformed (Figure 53).

2. The intermediate posts exhibit moderate to severe and, in a few cases, nearly complete section loss at their bases at the slab on ground (Figure 54, Figure 55). One post that has section loss at its base is also deformed due to a potential vehicle impact at Level 1 (Figure 56).
3. Where diagonal bracing passes through the intermediate posts, the posts have been spliced with welded plate connections. These plates are typically deformed and the intermediate posts above and below the splice are not in line with one another (Figure 57, Figure 58). The flange plates are corroded in some cases or are missing bolts (Figure 59, Figure 60).
4. The guardrail channels are supported by bolted connections to steel plates, which are welded to the intermediate post flanges. These connections are typically corroded with significant buildup, up to 3/4" thick, of corrosion byproduct between the plate and the post flange (Figure 61). In a few cases, particularly at the upper levels, this buildup has deformed the channel laterally (Figure 62). A few of these connections are missing bolts (Figure 63).
5. About five guardrails are deformed, likely due to a vehicle impact (Figure 64).

3.2 Waterproofing Components

The primary waterproofing components consist of the existing traffic-bearing membrane and sealant at joints and cracks. Traffic-bearing membrane is present at all elevated slabs but is limited to the washes and along the beamlines at portions of Levels 2 through 4. All construction joints are sealed and covered by a traffic-bearing membrane. Expansion joints are present at the entrance thresholds between the stair towers and parking deck.

1. The traffic-bearing membrane is in poor condition throughout the roof level, with missing or failed areas concentrated near the drains or previous repairs (Figure 65).
2. The traffic-bearing membrane is worn or failed (debonded) throughout most other elevated levels, particularly at high-traffic areas, such as at the turn-arounds (Figure 66).
3. Most cracks that were previously sealed are covered by the traffic-bearing membrane and appear in serviceable condition (Figure 67).
4. The expansion joint seals between the elevated slabs and the stair tower entries are typically failed cohesively, resulting in water infiltration to the level below (Figure 68).
5. Directly below the expansion joints, the coating at the CMU stair tower walls is often failed and the steel stair framing is typically corroded (Figure 69).
6. A roof drain outlet for the stair tower discharges directly above a failed expansion joint seal at the northwest stairwell of Level 5, which allows water infiltration to the level below (Figure 70).
7. Minor areas of ponded water, typically under 10 square feet, were observed, primarily near the drains and at the unsloped north and south bays of the structure (Figure 71).
8. Clogged drains were present throughout the garage, especially at the roof level and at Level 4 (Figure 72).
9. De-icing salts were observed to be applied at all drive lanes at all levels during WJE's 2021 site visits.

3.3 Facade

The facade consists of corrugated metal panels supported by light gauge steel posts that are anchored through the concrete slab to the top flanges of the edge beams below. At the base of the corrugated metal panels, steel tie-backs welded to the bottom flanges of the edge beams provide lateral support to the facade. The facade panels are generally in serviceable condition.

1. The majority of the facade panels are tilted inward throughout the parking garage at all elevations (Figure 73).
2. Approximately 50 to 75 percent of the welds between the steel tie-backs to the bottom flanges of the edge beams at the base of the corrugated metal panels are failed. A few of the tie-backs have screw anchors, perhaps from a previous repair effort, which have also failed. Gaps are present between the failed tie-backs and the bottom flanges (Figure 73, Figure 74). The tie-backs are generally in better condition at the west facade than at the remaining facades, but still exhibit widespread failure.
3. There are several instances of minor vehicle impact damage to the facade panels and posts throughout each level (Figure 75, Figure 76). The nearby slab where the panels are anchored typically does not exhibit distress related to the impact.
4. Some posts had a different profile and base connection type than the typical condition. At these locations, base plates and anchors were not present or exposed (Figure 77). Most of the facade panel anchors bolts were corroded. (Figure 78).
5. An opening exists between facade panels at the roof level that is about 6 inches wide. The nearby facade panel and post are deformed, potentially due to a vehicle impact (Figure 79).
6. A localized section of the facade has been removed and replaced with plywood at the roof level near the northeast stair tower.

3.4 Stair Towers and Miscellaneous

The stair towers are generally in serviceable condition with a few typical instances of distress. WJE also noted the following miscellaneous conditions elsewhere in the garage:

1. Cracks in the concrete masonry typically exist at locations where the supports for the stairs embedded into the masonry walls (Figure 80). Moisture-related distress, including corrosion of steel stair components and failed CMU coating is typically visible at these locations.
2. A few of the brick headers over entries to the stair towers are cracked along their length (Figure 81).
3. Isolated areas of the slab-on-ground are cracked. Some areas have been previously sealed and are in serviceable condition.

4.0 REPAIRS COMPLETED TO DATE

In an effort to take advantage of reduced occupancy during the COVID-19 pandemic, the City of Birmingham approved a limited scope of repairs on May 18, 2020 to be performed by DRV Contractors, LLC. As of the issuance of this report, the following repairs have been performed:

- Removal of loose concrete on the underside of slabs throughout the garage
- Localized concrete repair at the topside of the roof level slab

- Parking curb replacement and repair at the roof level
- Cleaning and painting of structural steel members throughout the garage

5.0 MATERIALS TESTING

Five concrete cores were extracted from various locations in the structure and sent to WJE's Cleveland laboratory for materials testing. A summary of the core locations is provided in Table 1 below. The lab studies included petrographic examination, water-soluble chloride analysis, water absorption testing, and carbonation depth measurements. One concrete mix was observed within all cores extracted from the slabs. A summary of the findings is presented in this report section. See ***Appendix A*** for more testing information and figures.

Table 1. Core Locations

ID	Core Location	Location Description
C1	Level 3 East Bay	In parking stall outside of drive lane. No membrane installation.
C2	Level 3 Central Ramp	In parking stall outside of drive lane. Traffic-bearing membrane present.
C3	Level 2 Northwest Bay	In drive lane, within about 30 feet of the drain and near the stair tower. Near the transition between the elevated slab and slab on grade. In region of failed/delaminated traffic-bearing membrane, though no membrane remained intact at this core.
C4	Level 5 (Roof) Northwest Bay	Drive lane, within about 30 feet of the drain and near the stair tower. Worn membrane nearby.
C5	Level 5 (Roof) Northwest Bay	Drive lane, within about 30 feet of the drain and near the stair tower. Worn membrane nearby.

The concrete slab materials are generally in serviceable condition. The concrete mix consists of limestone, sand, portland cement, and fly ash. The concrete is air entrained, which improves the concrete's freeze-thaw durability. Some of the aggregates contain chert, which is a reactive aggregate that can cause distress within the concrete. However, in the samples WJE collected, only internal microcracking was observed within the aggregates themselves, with minimal indications of external distress within the surrounding concrete materials. This presence of fly ash within the mix may be contributing to the lack of external distress associated with these chert aggregates.

WJE visually observed surface erosion in some of the exposed concrete surfaces. However, when analyzed in the lab, we did not find microcracking or other indications of significant distress in samples with this surface condition. WJE found few indications of secondary distress as a result of external factors (e.g. chlorides, moisture, freeze-thaw damage, etc.). This indicates that, although the existing traffic-bearing membranes have failed and isolated drains were clogged with surrounding ponded water, the deck does not appear to have experienced sustained long-term moisture ingress over the course of its service life thus far, or evidence of concrete deterioration mechanisms that can be promoted by the presence of moisture. It is important to repair and maintain the damaged waterproofing components within the deck to further protect the concrete from progressive distress.

5.1 Water Absorption

Water drop testing was performed to test the hydrophobicity (water repellency) of the top surface. Refer to Table 2 of ***Appendix A*** for a summary of the test results for each core.

At Core 2, where the traffic-bearing membrane contained a visually intact topcoat, water was not able to penetrate the membrane. Further, the concrete contained hydrophobic properties to a depth of 1/4 inch, indicating a penetrating sealer was likely applied prior to the installation of the membrane. At Cores 4 and 5 where the membrane was worn, the concrete was able to absorb the applied moisture, indicating the performance of the membrane is poor where the topcoat is compromised. At Cores 1 and 3, where no coating was present or the existing coating was visibly failed, the concrete was not absorptive but did not readily bead on the surface; therefore, the lack of water absorption is attributed to the accumulation of dirt and debris that has clogged the concrete pores and/or a previous application of a penetrating sealer.

5.2 Carbonation Testing

The high pH of uncarbonated concrete provides protective passivation of the embedded steel reinforcement. Carbonation is a chemical process that occurs in the cement paste of the concrete due to the penetration and reaction with atmospheric carbon dioxide and lowers the pH of the concrete. The depth of carbonation increases over time and is accelerated at cracks or joints. When the carbonation front reaches the depth of reinforcing steel, the steel becomes more susceptible to corrosion because the passivation layer from the high pH of the concrete is no longer present. The depth of the carbonation for each core is shown in Table 2 of ***Appendix A***.

At Core 1 and Core 3, where the slab surface was uncoated or where the existing membrane had failed, the depth of carbonation on the top surface varied from 1/8 inch to 1/2 inch. At Core 2 where the existing membrane was sound, some carbonation was observed, indicating the coating was installed later in the deck's service life. At Cores 4 and 5, which were extracted from the roof level in areas of worn membrane, the extent of carbonation was negligible, indicating that a coating or sealer has likely been present and well maintained on the roof surface throughout the deck's service life. The depth of carbonation measured at Core 1 and Core 3 is less than the depth to the embedded reinforcing steel, thus the increased potential for corrosion due to carbonated concrete is not a concern at the top surface of the slab *at this time*. Anchorage of the secondary steel elements, such as the facade panel posts or interior vehicle barrier posts, would be expected to experience an increased potential for corrosion due to carbonated concrete at these depths, which may lead to cracking and spalling of the surrounding concrete. These findings highlight the importance of maintaining steel coatings, sealant, and membranes through the deck.

The depth of carbonation on the underside of all five cores varied from 3/4 inches to 1 inch. The extent of carbonation is attributed to the age of the structure, the lack of a protective sealer or coating on the underside of the deck (in comparison to the top surface), and the natural exposure of the underside of the slab to carbon dioxide from vehicles within the garage. At these depths, the embedded reinforcing steel and anchorage elements near the bottom of the slab have an increased potential for corrosion due to the carbonated concrete. Thus, it is important to mitigate the exposure of the concrete to chlorides and moisture by maintaining the waterproofing and drainage elements on the top slab surface.

5.3 Water-Soluble Chloride Testing

The purpose of the chloride analysis was to determine the current chloride ion content at various depths of the slab. The results are contained within Table 2 of ***Appendix A***.

The water-soluble chloride content by weight of concrete at the typical depth of reinforcing steel was found to be elevated at Cores 1, 2, and 3. Similarly, near the slab surface, high levels of chloride contents were found at Cores 1, 2, 3, and 4, with very high surface values found within Cores 1 and 3, where a membrane had not been applied or was visibly failed. Lower surface chloride levels were found at Core 5, which is attributed to a maintained membrane or sealer in this area over the life of the structure. Localized areas of greater chloride contamination may occur at cracks. With continued use of chloride-containing deicing salts, the chloride concentration and depth would be expected to increase. Based on these findings, and in conjunction with the carbonation test results and our observations, a traffic-bearing membrane is strongly recommended throughout the deck, as well as continued maintenance of the sealant, expansion joint seals, and drainage components in the deck.

6.0 DISCUSSION

Overall, the parking structure is in serviceable condition with localized areas of deterioration. However, the deterioration has progressed such that repairs, especially waterproofing repairs, are warranted in the near future to maintain the condition of the parking structure.

6.1 Concrete - General

Concrete slabs within parking structures in Michigan are susceptible to deterioration due to their exposure to moisture, deicing salts, and temperature changes (i.e., cyclic freezing and thawing, thermal expansion and contraction, etc.). The primary causes of concrete deterioration in concrete parking structures is corrosion, typically due to chloride contamination and carbonation, as both conditions can promote corrosion of embedded steel reinforcement. Because steel corrosion product occupies a larger volume than the native steel, it is common for distress, in the form of cracks, delaminations, or spalls, to develop when the embedded steel corrodes and expands, placing expansive forces on the surrounding concrete.

The condition of the concrete slab varies, particularly between levels. This difference in condition between levels correlates with exposure to moisture, with distress concentrated mostly at the roof level. Spalls in the slab are most common at the roof level, though spalls or delaminated areas are also concentrated near drains or beamlines at all levels, which is likely related to moisture exposure and low concrete cover of embedded reinforcement. The other typical locations of cracked or spalled concrete are along the slab edges, where mild reinforcement with minimal cover are present.

This distress increases the risk of water infiltration into the PT tendon sheathing at the anchorage points. Repairs are recommended to address the observed concrete distress. Full-depth concrete repairs are recommended in most locations of concrete distress that are not associated with the PT tendons, while partial depth repairs may be performed where small delaminated areas are present at the topside of the slab with no associated distress at the underside of the slab.

The longitudinal cracks viewed from the underside of the elevated slabs typically correlate with expected locations of embedded steel conduit or mild reinforcement and are attributed to the corrosion of these elements. Most of the previous crack repairs, consisting of injecting and patching the cracks at the underside of the slab, have failed. Active water infiltration is typically present at these cracks. It is not known at this time if these cracks extend through the full depth of the elevated slabs since the topside is typically concealed by the existing traffic-bearing membrane. Recommended repairs include routing and sealing the cracks noted to be full depth. Otherwise, installation of a traffic-bearing membrane or repairs to the existing membrane throughout the garage will reduce moisture exposure to the corroded embedded steel conduits and reinforcement. Refer to the *Waterproofing Components* section below for more information regarding membrane repair and replacement. Isolated concrete repairs may be necessary in a few locations.

About 75 percent of the previous concrete repair areas not associated with PT tendon repairs are sound, but many sound concrete repairs exhibit isolated cracking and efflorescence within the repair area. Progressive deterioration of these concrete repairs can lead to delaminations or failure of the repairs. These observed distress conditions are primarily associated with water infiltration. Installation of a traffic-bearing membrane, or repair/replacement of the existing membrane, is recommended to reduce the rate of deterioration of the previous sound repairs. Of the repair areas that were found to be unsound, a majority were within trowel-applied patch repairs. WJE has found that the form-and-pour repair technique results in more durable repairs than trowel-applied repairs. Therefore, we encourage using this technique over trowel-applied repairs in the future. Where concrete repairs occur in areas of embedded steel conduit, which has since been abandoned, the conduit materials should be removed from the repair areas to mitigate further distress.

6.2 Post-Tensioned Structures

Post-tensioned structures efficiently combine steel, which is strong in tension, and concrete, which is strong in compression, to utilize the full cross section of a structural element at all points along its length. Compared to conventionally reinforced concrete, post-tensioned concrete typically offers greater durability, particularly due to its ability to minimize cracking and to protect the tendons from corrosion. The benefit of post-tensioned concrete over conventionally reinforced concrete depends heavily on adequate protection of the tendons from moisture. Locations that are most susceptible to moisture exposure include tendon anchor points, where the sheathing or anchor may not be protected, and construction joints or concrete repairs, where the tendon sheathing is made discontinuous for stressing or possibly damaged during repair, respectively. These locations of discontinuous sheathing at cracks or joints can allow water to directly reach the tendons. Deterioration of PT tendons, particularly corrosion leading to section loss, can result in failure of that tendon. If an unbonded tendon becomes de-tensioned for any reason, that tendon no longer carries load at any point along its length.

PT tendons are the primary reinforcement of the elevated slabs at the Park Street Garage, and loss of the tendons will result in a reduction of load-carrying capacity of the structural slabs. Potential sources of water into the PT system include the delaminated areas of concrete over the beamlines which correlate with the high points in the tendon drape, construction joints, stressing pockets at the slab edges, and contamination from the original construction.

Of the 22 PT tendons inspected, 5 structural PT tendons and 2 temperature PT tendons were de-tensioned or had failed anchorages. These tendons are recommended for repair and re-stressing. Six PT tendons were tensioned, but generally exhibited some level of surface corrosion with reduced grease content and frequent grease stains along the tendon length. A lack of grease within the sheathing, combined with the presence of corrosion, indicate that water is infiltrating the tendon sheathing, which can ultimately cause section loss and de-tensioning as the tendon deteriorates.

The remaining 9 PT tendons were generally found near grease stains, but in areas with no visible distress in the concrete nearby or along the length of the tendon. Grease was filling the sheathing voids of these tendons and little to no corrosion was observed. PT repairs should be performed at failed tendons or anchorages and at tendons exhibiting section loss due to corrosion. Near-term waterproofing improvements, which are described below, are recommended to reduce the rate of corrosion of the tensioned tendons in good condition or exhibiting only minor corrosion.

Based on our findings, the spalls and isolated delaminated areas in the topside of the slab over the beamlines generally correlate with low cover of PT tendons or PT tendon repair components and were not directly related to the presence of PT damage or deterioration. Although tendon damage was not observed, the loss of concrete cover exposes the tendon to moisture, chlorides, and potential sheathing damage, accelerating that tendon’s rate of deterioration. At minimum, sheathing and concrete repairs should be performed at these locations, but a few tendon repairs should be expected to address this condition.

Cracks, spalls, and incipient spalls at the slab edges were generally found to be caused by corrosion of the mild reinforcement and low concrete cover. At the spalled regions, mild-to-moderately corroded tendons and anchors were exposed as a result of the extent of concrete distress and moisture and chloride exposure, and 2 PT tendon anchors were found to have failed. Failed anchorages at the two isolated temperature and shrinkage tendons should be de-tensioned in a controlled manner and do not require restressing. Loose, delaminated concrete cover materials should be removed at the remaining locations without chipping in front of the PT anchorages, and should be repaired. The water management elements above (i.e. sealant or membranes) should be repaired or maintained to mitigate further distress.

Previous PT repair efforts include restressing tendons, anchorage repairs, and sheathing repairs. Most of the PT repairs are sound, though a few repair areas exhibit progressive deterioration of the surrounding concrete. A few previous PT repairs exhibit isolated shrinkage cracking and efflorescence, but are sound and generally do not exhibit active water infiltration. Cracks that appear to align with temperature PT tendons exist through previous PT repairs in the unsloped north and south bays, but these repair areas are also typically sound. Most of the previous PT repair areas are not anticipated to require near-term repair, though waterproofing repairs are recommended to protect the existing repair areas.

6.3 Structural Steel Members

6.3.1 Primary Structural Steel Elements

In general, the primary steel framing members are in serviceable condition; however, some locations of beam deflection and severe corrosion at the framing connections are present throughout the parking garage.

The moment frames that provide lateral support in the east-west direction depend on the ability of the top and bottom flange plates at beam-to-column connections to develop rigid connections that resist rotation of the members and lateral deflection of the frame. Column stiffener plates in line with the top and bottom flange plates provide additional stiffness to the localized section of the column loaded by the moment connection. The corrosion byproduct built up between the bottom flange plates and the beam's bearing ends has compromised the weld between these two elements in most cases and its ability to transfer load to the column. Cleaning the corrosion buildup may allow the bottom flange plates to return to their original non-deflected position and perform as designed, but where failed welds or severe section loss of the flange plates are present, these areas require replacement of the flange plates and welded connections, rather than repair. The severe and nearly full section loss at many of the column stiffener plates may significantly reduce the columns' capacity to carry the localized lateral load. Most of the stiffener plates will need to be replaced due to the severity of the section loss observed. Framing connections outside the moment frames primarily carry gravity loads and are generally in good condition, with few visible indications of corrosion.

Multiple phases of cleaning and coating the structural steel have been performed during previous repair efforts. The significant corrosion buildup noted at the moment connections and at the vehicle barrier connections is typically coated, indicating that the connections were not adequately cleaned prior to installation of the coating. The uneven or scaled surface along the length of other structural members is not currently a structural issue, but the significant corrosion buildup at the connections may reduce the capacity of the connecting elements. Future cleaning and coating efforts should include thorough removal of corrosion buildup at connections to prevent premature failure of the coating and further deterioration of the connections. Repair or replacement of some connections or connecting elements may be required as part of this effort.

The observed corrosion at the bases of the exterior pipe columns is associated with coating failure and lack of a pedestal, sloped grade, or other form of protection from water at the column-to-slab interface. Where concrete sidewalk distress is also present at the exterior columns, removing the affected concrete is recommended to assess the presence and condition of the concealed column base plates and anchorage. In such cases, cleaning and coating of the base plate, anchors, and corroded section of column and concrete repairs to the nearby slab, when affected, are recommended. The corrosion at the bases of the interior wide flange columns where embedded into the elevated slabs is mild and should be monitored for progression and cleaned and painted as needed.

The interior wide flange columns have base plates at the slab on ground, with grout pads between the base plates and the slab. The base plates and anchors of the interior columns and the bottom section of the columns exhibit mild-to-moderate corrosion with no significant section loss. The bases of all interior columns at the slab on ground should be cleaned and coated to mitigate further corrosion. The loose or missing nuts and anchors are more structurally significant and should be tightened or replaced.

The vertical deflections measured at select beams are up to twice the expected deflections for this structure under dead load, based on WJE's preliminary structural analysis. However, distress within the PT slab and steel framing elements associated with the deflected beams is not readily visible, and the beam capacities are adequate under design loads. As such, the measured beam deflections are considered to be a serviceability issue, are not a structural concern, and do not require corrective action.

6.3.2 Secondary Structural Steel Elements

The intermediate posts between columns serve as lateral bracing for the interior vehicle barriers. These posts are bolted to angles along the edges of each elevated slab and are embedded at their bases into a foundation below the slab on grade. The post bases are exposed to de-icing salts and ponded water without a sloped grade surface or pedestal to shed water or provide concrete cover. This condition has led to significant section loss at most post locations, including up to 80-90 percent section loss at a few of the posts. These bases require repair. Several options for repair are available, as conceptually described below, though the final repair concept should include improved water management details for improved durability.

Concrete pedestals anchored to the existing foundations can be installed to raise the bearing point of the steel posts, protecting the steel from direct exposure to ponded water and de-icing salts. Alternatively, the existing foundations could also be partially demolished and supplemental steel reinforcing plates can be used to splice across the area of corrosion. Water management improvements for this option may include encapsulating the base of the repaired posts with a concrete pedestal, sloping the slab on grade away from the pedestals, and installation of a cove sealant or traffic-bearing membrane at the intersection between the pedestal and slab. Selection of a repair option for the intermediate posts will depend on further analysis and design by an engineer, a cost comparison exercise, and understanding the impacts to the deck operations during the repair work.

The condition of the post connections to the edge of slab angles is not well understood, since a rigid sheet metal fascia is present at the slab edges, concealing a majority of these connections. Minor repairs should be anticipated to address surface corrosion and isolated concrete distress.

The connections from the vehicle barriers to the intermediate posts often exhibit significant corrosion buildup, leading to deformation of the vehicle barriers and deformation at the splices of the intermediate posts at locations of diagonal bracing. Other causes for the observed deformation of the spliced intermediate posts may include large construction tolerances, vehicle impact, and previous repairs. Cleaning and coating the exposed steel surfaces, with selective replacement of the significantly deteriorated vehicle barrier-to-post connections and splice flange plates, is recommended.

6.4 Waterproofing Components

Traffic-bearing membrane systems are the most common waterproofing system used on parking structures to extend the life of the structure and the time between repair efforts. A membrane provides an impermeable barrier on the surface of the structure and prevents moisture from entering the structure. Additionally, a membrane reduces the corrosion rate of the structure by reducing the amount of moisture in the concrete. They typically consist of multi-layer polyurethane or epoxy coating with integral aggregate broadcast for slip resistance. The bottom layer of the system provides the waterproofing, and the top layers serve as the skid resistant and wearing surface. The typical service life for a new traffic-bearing membrane in low traffic areas can easily exceed 10 years. In high traffic areas and in areas with significant turning, maintenance of a traffic-bearing membrane to address wear can be necessary in less than 5 years.

A traffic-bearing membrane has already been installed over the majority of the garage, covering some levels entirely, while coating only the beamlines and washes at other levels. The bottom layer of the membrane is typically failed (delaminated) or worn at Levels 2 and 5 and is recommended for full removal and replacement. At Levels 3 and 4, the top layer of the membrane is generally intact, but worn, exposing isolated areas of damage within the bottom layer. Membrane removal and replacement is recommended based on the extent and distribution of the observed distress; however, repair of the existing membrane is possible in regions where the base layer is intact. Based on the chloride levels measured in the concrete, membrane installation is recommended in areas where no membrane has previously been applied.

Repair or replacement of the traffic-bearing membrane may expose the condition of the construction joints and the sealed cracks in the slab that are currently concealed. Based on the visible deterioration at the underside of the slab, some of these joints and cracks are likely allowing water infiltration into the slab. The sealant at the construction joints should be replaced, while the condition of the sealant at the cracks should be assessed during work associated with the traffic-bearing membrane. Sealant in poor condition at cracks should be removed and replaced, while cracks not previously sealed should be routed and sealed prior to membrane installation. Failed expansion joint seals between the stairwells and the garage structure allow water infiltration to the areas below, which has led to coating failure and corrosion of steel embedded in the masonry walls and deterioration of the concrete slab edge. Replacement of the expansion joint seals is recommended.

6.5 Facade

Several facade posts are missing base plates or anchors to the concrete slab, which may reduce the structural integrity of the facade and its capability to withstand vehicle impact or wind loads. Corrosion staining present at the anchor bolt connections indicate deterioration of the anchors that can lead to cracks and spalls of the associated concrete. The metal facade panels also illustrate permanent deformation as a result of vehicle impact in several locations. Missing or boarded up sections of facade panels are also present at the roof level. Missing facade panels, base plates, and bolts should be replaced in kind. The corroded anchors and the nearby concrete should be monitored for further corrosion and deterioration.

Most of the welds at the steel tie-backs that provide lateral support to the facade panels are failed, leading to the observed inward tilt of most of the facade panels. The inward tilt of the panels correlates with lateral loading from wind, rather than a vehicle impact. The loss of support from the steel tie-backs causes the bolts through the slab edges to carry a greater portion of the lateral load. The observed cracks and spalls at the slab edges, largely due to corrosion of the embedded mild steel reinforcement, are likely influenced by these increased loading conditions. The cracks and spalls also reduce the bolts' capacity to resist rotation. Rewelding or replacing the tie-backs is recommended, in conjunction with the recommended concrete and waterproofing work in these area.

6.6 Stair Towers and Miscellaneous

The cracks in the CMU walls at the stair towers are typically located where the supports for the stairs are embedded into the masonry walls. Corrosion and failed coating at the CMU walls indicate that distress in this area correlates with moisture exposure. Such distress is typically most severe near the entries, where the failed expansion joint seals between the parking deck and the stair tower doors are located.

Replacement of the expansion joint seals will reduce further moisture-related distress at the stair tower walls and steel stair supports. The corroded steel stair elements should be cleaned to prevent further cracking of the CMU walls. The coating at the interior of the CMU stair tower walls is currently preventing water from escaping the wall assembly, as evidenced by the coating failure when exposed to moisture. Further or repeated coating failure is likely if replacement of the expansion joint seals is not performed to reduce moisture exposure.

The cracks in the brick headers over the interior entries to the southwest stair tower are likely caused by corroded reinforcement through the brick units. The headers are currently in serviceable conditions but should be replaced or repaired with a steel lintel.

Cracks exist throughout the slab on ground and some have previously been sealed. Signs of settlement that correlate with these cracks are not visible. The slab on ground is generally level across these cracks and does not require repair at this time.

7.0 RECOMMENDATIONS

Based on our observations and our experience with similar parking garages, WJE offers the following categorized recommendations for your consideration.

7.1 Near-Term Repair Recommendations

WJE recommends that the following repair items be completed in the near future (within the next 1 to 2 years). These recommendations are intended to address significant structural deterioration, minimize water infiltration, and extend the service life of the parking structure.

1. Structural Steel Repairs
   a. Repair or replace the deteriorated moment connection elements and column stiffener plates. *
2. Concrete Structure Repairs
   a. Localized repairs (both partial and full depth) of unsound concrete at the elevated slabs.
   b. Isolated slab post-tensioned tendon and anchor repairs.
3. Waterproofing Repairs
   a. Sealant replacement at the construction joints at all floors. *
   b. Rout and seal isolated cracks and replace failed sealant in the elevated slabs.
   c. Traffic-bearing membrane repairs/replacement and installation at all elevated slabs.
   d. Replace all expansion joint seals located at stair towers entries from elevated slabs.
   e. Inspect and clean drain lines. *
   f. Modify roof drain outlet to discharge away from expansion joint.

These repairs may be phased if needed to accommodate occupancy, schedule, or budgetary concerns. The highest priority repair items are indicated with an asterisk (*).

7.2 Long-Term Repair Recommendations

WJE recommends that the following repairs be completed within the next 3 to 5 years. These recommendations are intended to address structural deterioration, as well as the observed distress within the secondary structural system, vehicle barriers, facade, stairs, and slab on grade.

1. Structural Steel Repairs
   a. Clean and coat or replace all vehicle barrier to intermediate post connections.
   b. Repair the bases of the intermediate posts.
   c. Repair or replace column base plates and anchors as necessary at the interior columns.
   d. Repair the bases of the exterior pipe columns.
   e. Clean and coat beam to column shear connections and diagonal bracing connections as necessary.
   f. Replace vehicle barriers impacted by vehicles.
2. Concrete Structure Repairs
   a. Localized repairs (both partial and full depth) of unsound concrete at the elevated slabs as needed.
3. Facade Repairs
   a. Replacement of facade panels and posts at the roof level that exhibit signs of vehicle impact.
   b. Installation of new anchor bolts at facade post anchorages where missing or significantly corroded.
   c. Repair or replace steel tie-backs at the bottom of the facade panels.
4. Miscellaneous Repairs
   a. Repair or replace cracked brick headers at the southwest stair tower entries.
   b. Repair cracked and spalled CMU at stair tower walls.
   c. Clean and paint corroded steel stair framing and handrail elements.

7.3 Maintenance Recommendations

WJE recommends that the following maintenance items be completed on a regular basis, or as indicated.

1. Utilize snowplows with shoes, rubber tips, or small skis to prevent damage to the traffic-bearing membrane and perform the plowing in a manner that minimizes impacts. Do not store plowed snow on the supported levels.
2. Avoid *excessive* de-icing salt applications.
3. Assess the traffic-bearing membrane on an <u>annual basis</u> in the spring to identify and repair de-bonded areas and scrapes related to snow plowing operations from the previous years.
4. Remove accumulated debris and clean floor drains on a bi-annual basis.
5. Each spring, power wash and clean the deck surfaces to remove debris and the accumulation of deicing salts.
6. Periodically inspect overhead concrete surfaces and remove loose or unsound concrete.
7. Periodically assess the coating materials on the steel members and repair damaged areas as needed.
8. Periodically assess and perform concrete repairs, as needed.

8.0 OPINION OF PROBABLE COSTS

8.1 Repair Project Cost

As shown in **Appendix B** below, the probable construction cost to address the near-term repair recommendations (within the next 1 to 2 years) is on the order of $2.9 million dollars. In addition, the probable cost to implement the remaining long-term recommendations (within the next 3 to 5 years) is approximately $770,000. This estimate includes a 15 percent contingency and a 10 percent budget for engineering, testing, and inspection. Based on our experience with similar repair projects, WJE believes it is prudent to include a contingency to accommodate unforeseen conditions that are encountered during repair construction.

The majority of the unit costs contained in the construction cost estimate are based on costs for similar work on previous concrete repair projects located in the Midwest region. Repair quantities are based on the current level of deterioration and unit prices are in current dollars. Both are subject to increase over time. With regard to construction costs specifically, an increase of 3 percent per year is recommended to account for inflation. Actual costs will depend on a number of factors, including the bidding environment and owner-provided constraints. Please also keep in mind that COVID has made construction pricing and scheduling less predictable, and its influence is not accounted for in this cost estimate.

These cost estimates assume that all of the work recommended for each phase (near-term and long-term) will be performed during one construction project each (i.e., one large project to address the near-term items and one large project to address the long-term items). It is possible, and may be preferable to the owner, to perform the repairs in smaller work areas and over multiple years, or in a prioritized manner, in the event that funding is limited, or parking spaces are not available. While smaller work areas occupy fewer parking spaces, an increase in both the duration and overall cost for the repair project should be anticipated. Similarly, cost efficiencies may be realized if all the recommended repairs are performed within large one near-term project.

8.2 Expected Maintenance Costs

This parking structure is nearly 50 years old. Given the exposure to moisture and deicing salts, concrete distress related to corrosion of the embedded reinforcement should be expected throughout the life of the parking structure. In particular, loose concrete removal and periodic sealant replacement and membrane repairs should be anticipated. Regular repairs and maintenance can decrease the rate of deterioration and increase the longevity of the parking structure. Therefore, WJE recommends that an annual budget be established for such repairs and maintenance. *In addition, a significant concrete repair and waterproofing project should be anticipated every 5-10 years for the remaining life of the parking structure.*

Maintenance and repair costs of parking structures increase exponentially over time due to exposure to aggressive environments. Maintenance of the structural steel, concrete, and waterproofing components of this garage should be expected. For this 270,000 square foot parking structure, we recommend a budget of approximately $400,000-$600,000 every 5 years, increasing as the structure ages. For comparison, we estimate the cost to replace this deck with a new deck of a similar size and capacity would be between $32 to $37 million, including demolition costs for the existing structure.

Maintenance costs for the new deck would be estimated at approximately $20,000-25,000 annually depending on the type of construction. Maintaining the existing deck is currently less expensive and is recommended over replacing the structure, since the garage is in serviceable condition and has been relatively well-maintained, based on the materials testing results and our assessment findings.

9.0 CLOSING

WJE performed an assessment of the Park Street parking structure in Birmingham, Michigan, including a visual survey, preliminary structural analysis, investigative openings of the post-tensioned system, and materials testing. Based on the findings WJE provided repair and maintenance recommendations and presented our opinion of the probable repair costs for budgeting purposes. At your request, and under separate authorization, WJE can prepare construction documents to implement the recommended repairs.

We appreciate the opportunity to be of continued service to The City of Birmingham. If you have any questions, please feel free to contact us.

FIGURES

Figure 1. Spall at slab edge exposing PT temperature tendons

Figure 2. Spall at construction joint exposing PT tendon anchorages and an embedded conduit


Figure 3. Spall at a previous repair exposing PT tendon


Figure 4. PT repair area

Figure 5. PT intermediate anchorage repairs at a construction joint, as viewed from the underside of Level 4


Figure 6. Progressive deterioration at slab edge repair. Note that mild steel and temperature and shrinkage PT anchorages are in this region.


Figure 7. Evenly spaced previous crack repairs at Level 3 construction joint, in line with PT tendons

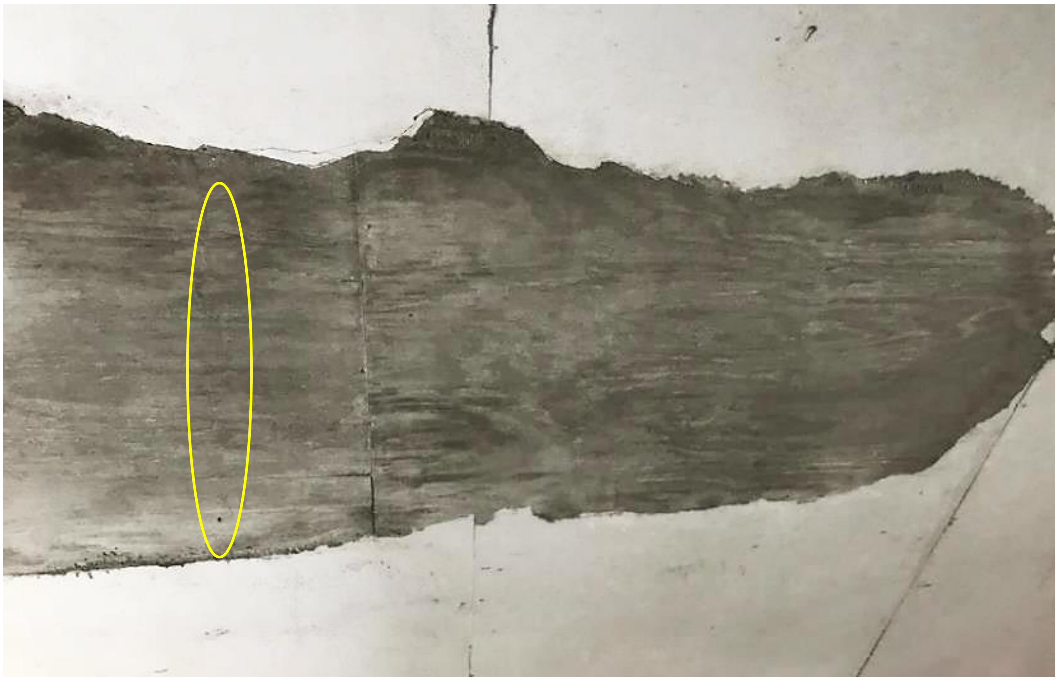

Figure 8. Narrow crack in line with temperature tendons within sound PT repair

Figure 9. Grease stains near construction joint

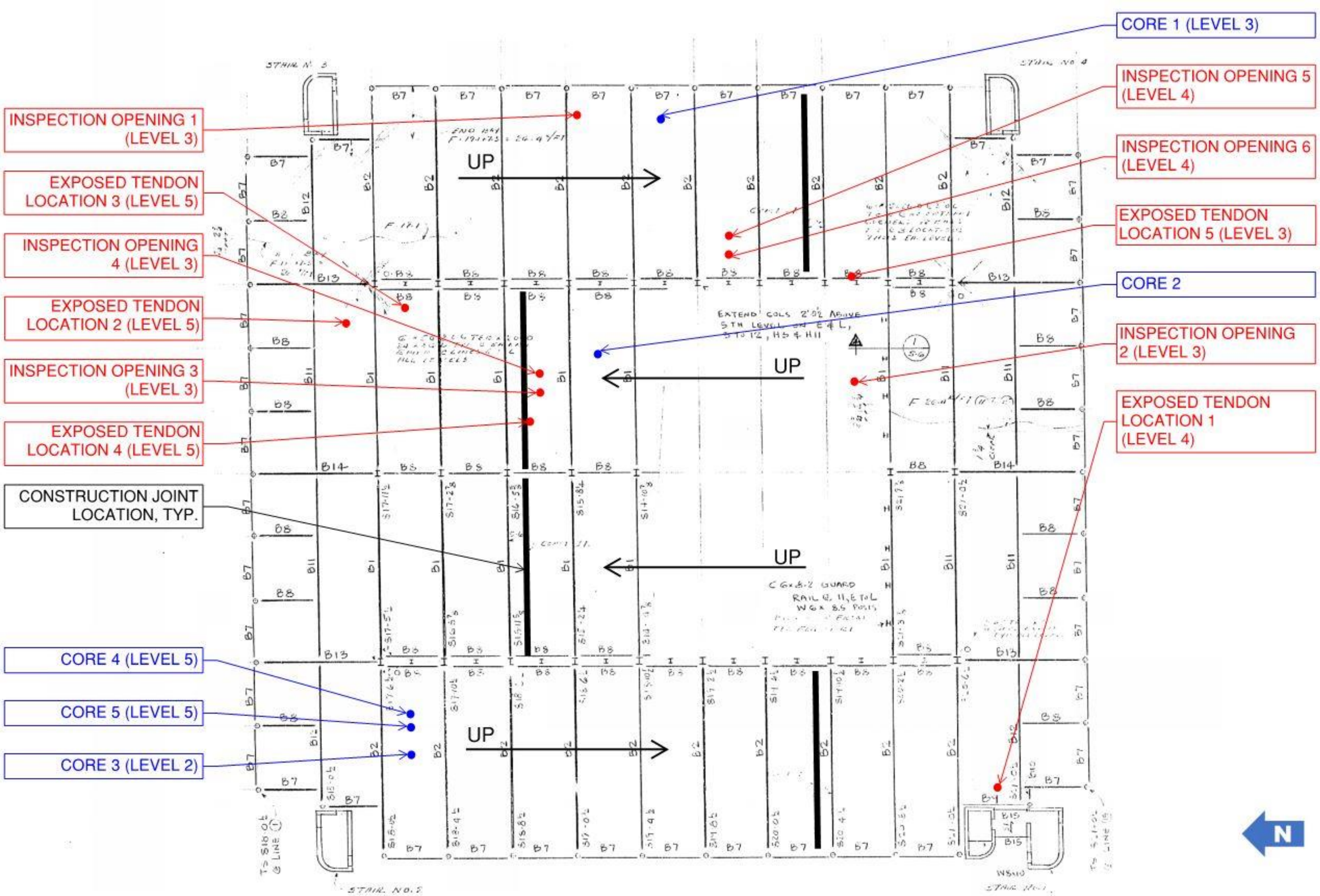

Figure 10. Inspection Opening Locations


Figure 11. Exposed Tendon Location 1


Figure 12. Exposed Tendon Location 2

Figure 13. Exposed Tendon Location 3

Figure 14. Exposed Tendon Location 4. Conduit indicated by arrow.

Figure 15. Exposed Tendon Location 4.

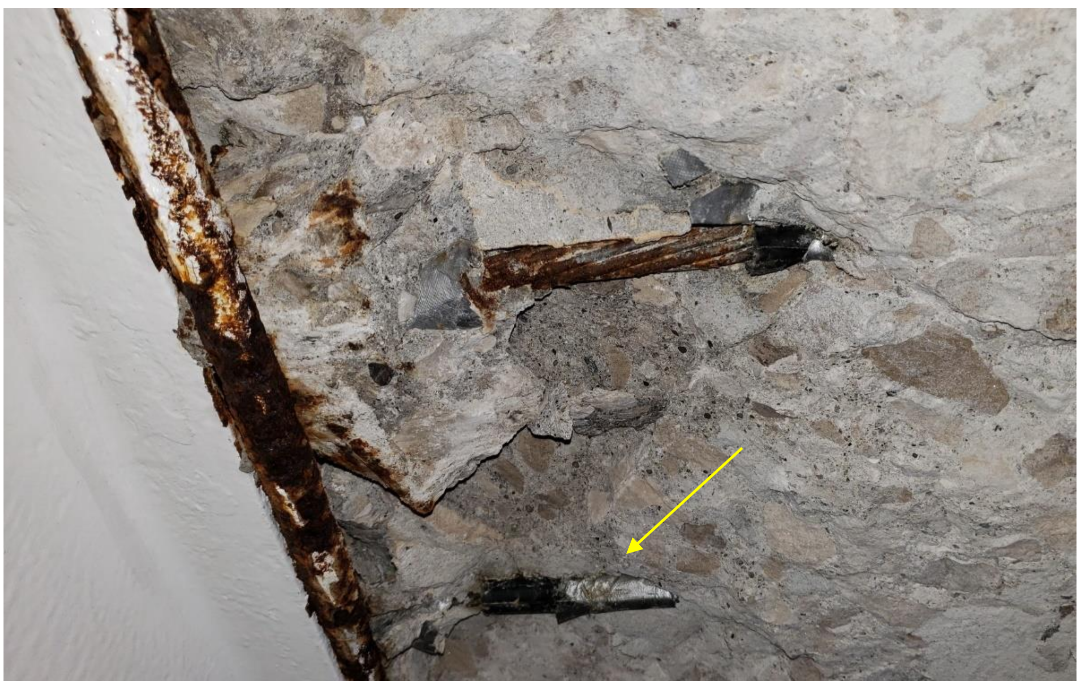

Figure 16. Exposed Tendon Location 5. Arrow indicates location of duct tape sheathing repair.


Figure 17. Inspection Opening 1. PT tendon coupler indicated by arrow.


Figure 18. Inspection Opening 2

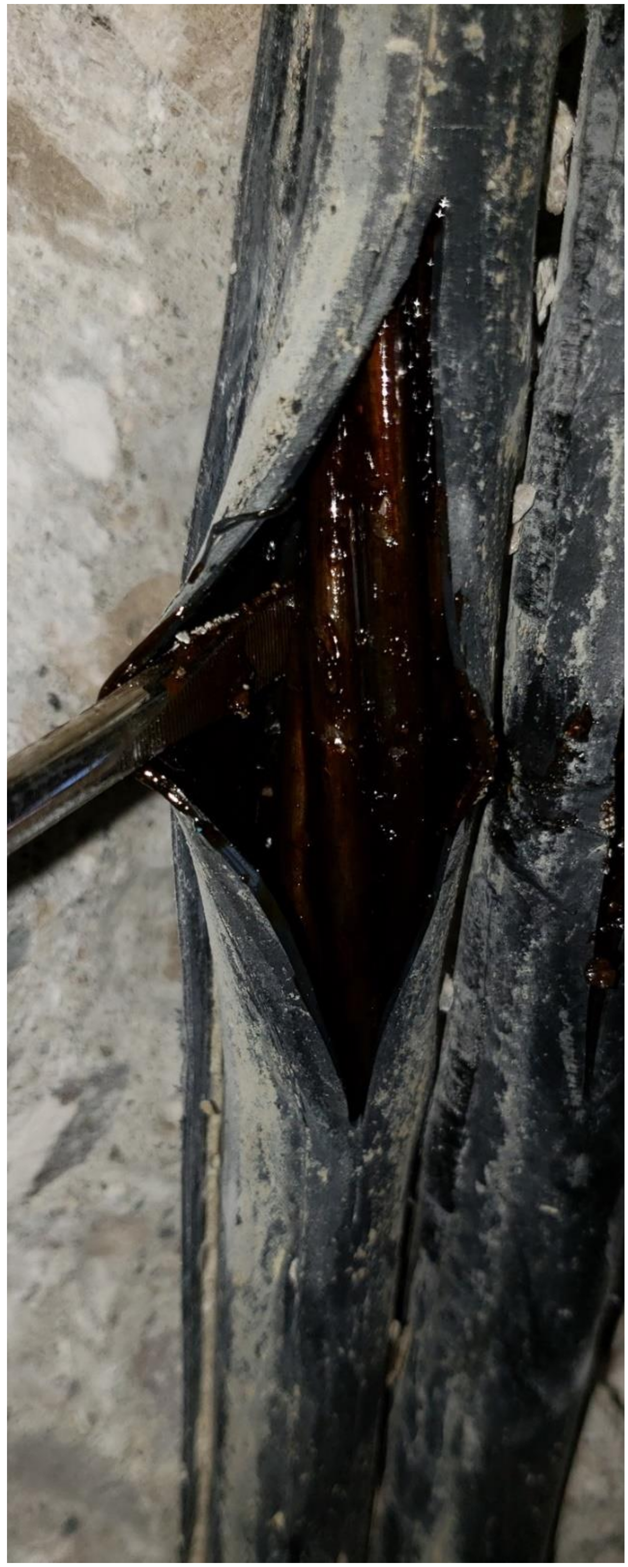
Figure 19. Inspection Opening 3

Figure 20. Inspection Opening 4

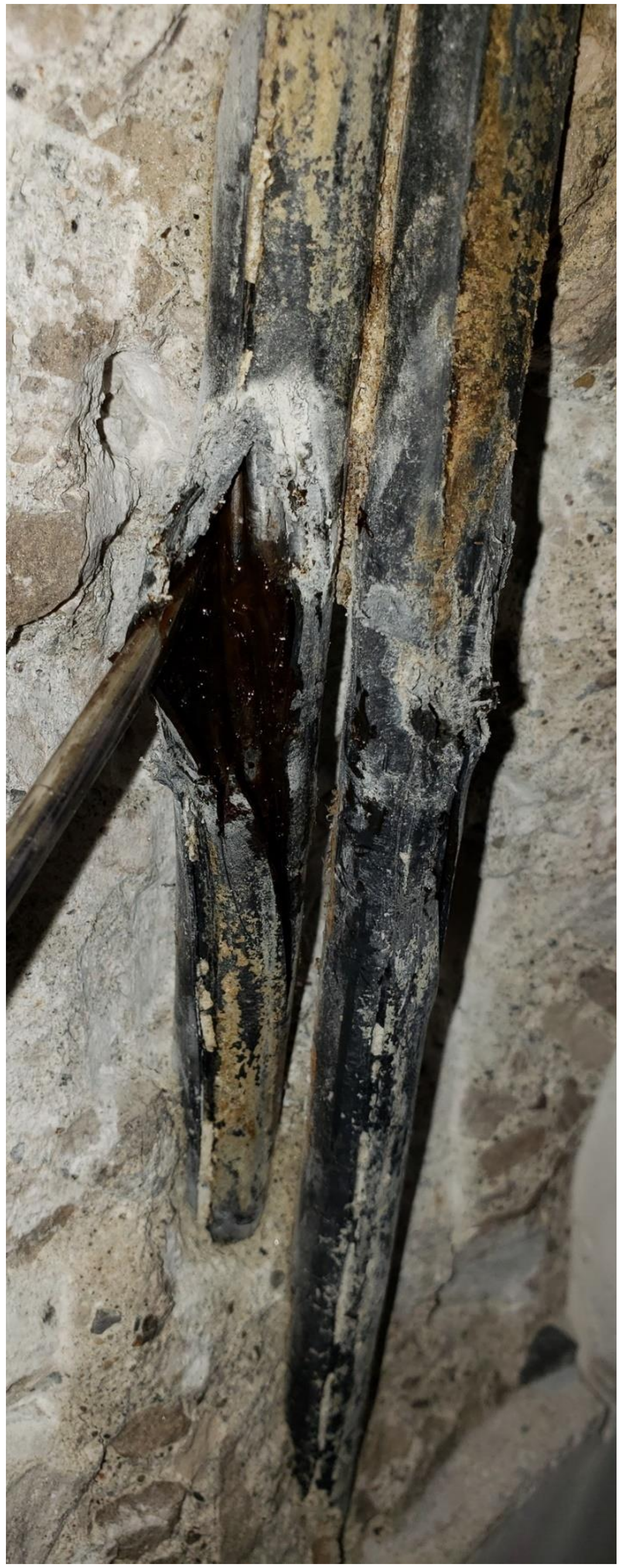

Figure 21. Inspection Opening 5

Figure 22. Inspection Opening 6

Figure 23. Previous repair at construction joint

Figure 24. Previous repair at roof drain

Figure 25. Previous repair exhibiting cracking, efflorescence, and corrosion staining

Figure 26. Previous repair exhibiting cracking and efflorescence


Figure 27. Delaminations near the beamlines marked in chalk


Figure 28. Delamination near drain within ponded water (indicated with arrow)


Figure 29. Spall at topside of roof level slab

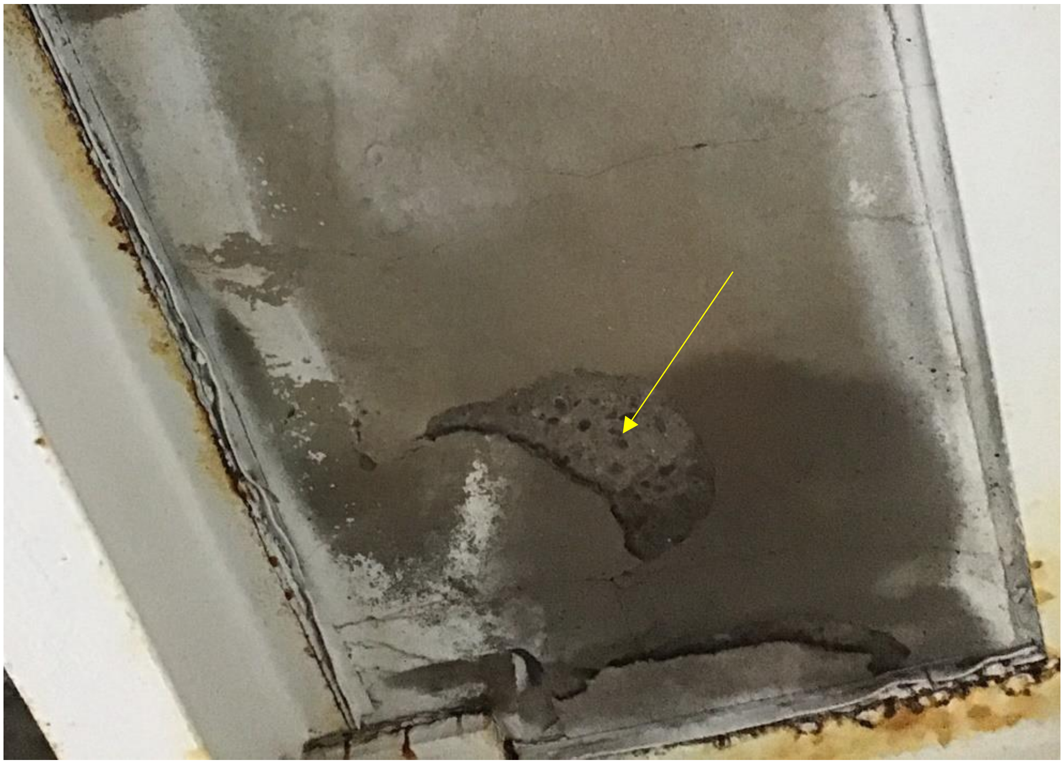
Figure 30. Spall at underside of slab


Figure 31. Cracks and incipient spalls adjacent to the exterior end of beam in unsloped bay

Figure 32. Cracked concrete at slab edge


Figure 33. Previously sealed cracks


Figure 34. Previously sealed cracks


Figure 35. Failed sealant at crack

Figure 36. Moisture staining at previous repaired crack at conduit.

Figure 37. Loose nut at base plate

Figure 38. Missing nut at base plate

Figure 39. Missing bolt at base plate

Figure 40. Corrosion at base of exterior pipe column


Figure 41. Concrete sidewalk distress at corroded exterior pipe column base


Figure 42. Corroded base plate at exterior pipe column


Figure 43. Corrosion where interior column contacts slab edge

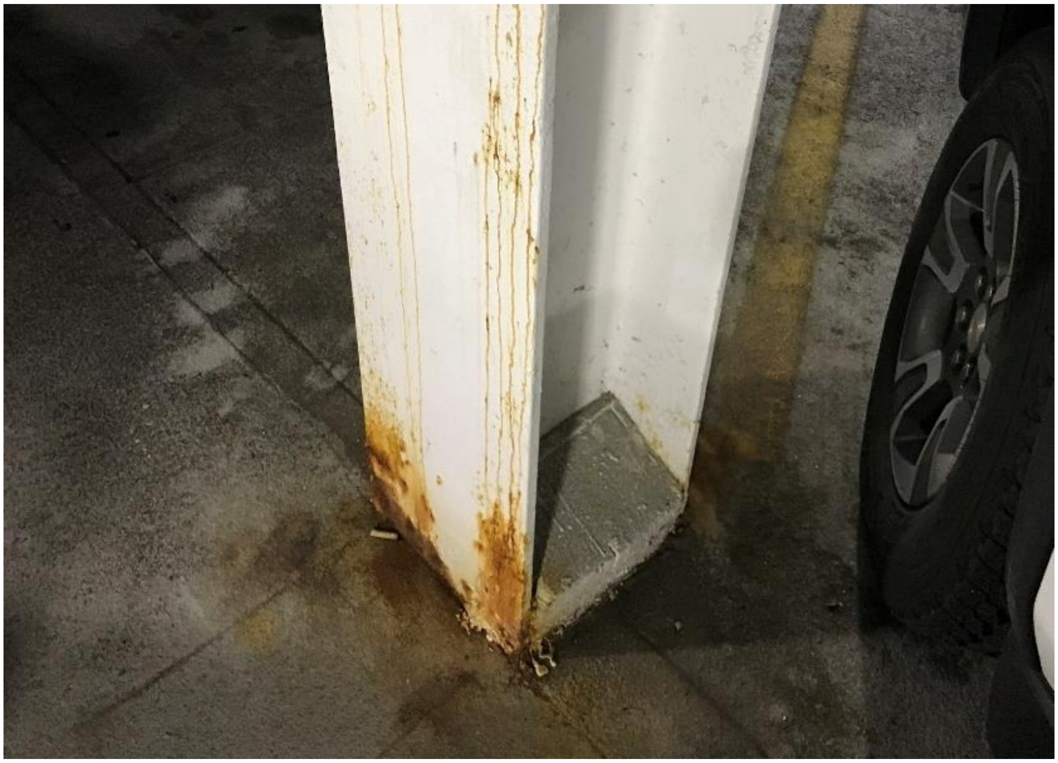

Figure 44. Corrosion at interior column base at elevated slab

Figure 45. Corroded top of column and beam

Figure 46. Section loss at column stiffener plate

Figure 47. Corrosion byproduct buildup within moment connection

Figure 48. Nearly full section loss of top column stiffener plate

Figure 49. Corroded connection at exterior column

Figure 50. Corroded diagonal bracing connection

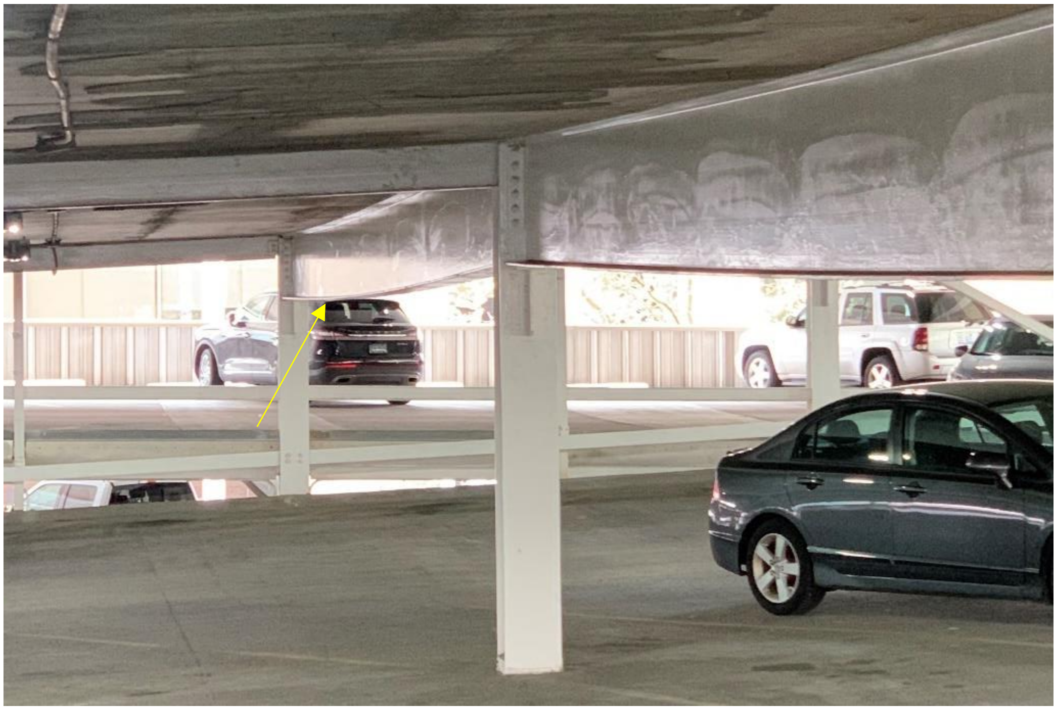

Figure 51. Deflected steel beams

Figure 52. Uneven surface below coating at steel member


Figure 53. Laterally deformed steel beam


Figure 54. Severe section loss at intermediate post


Figure 55. Severe section loss at intermediate post

Figure 56. Vehicle impact deformation at intermediate post

Figure 57. Deformed intermediate post splice

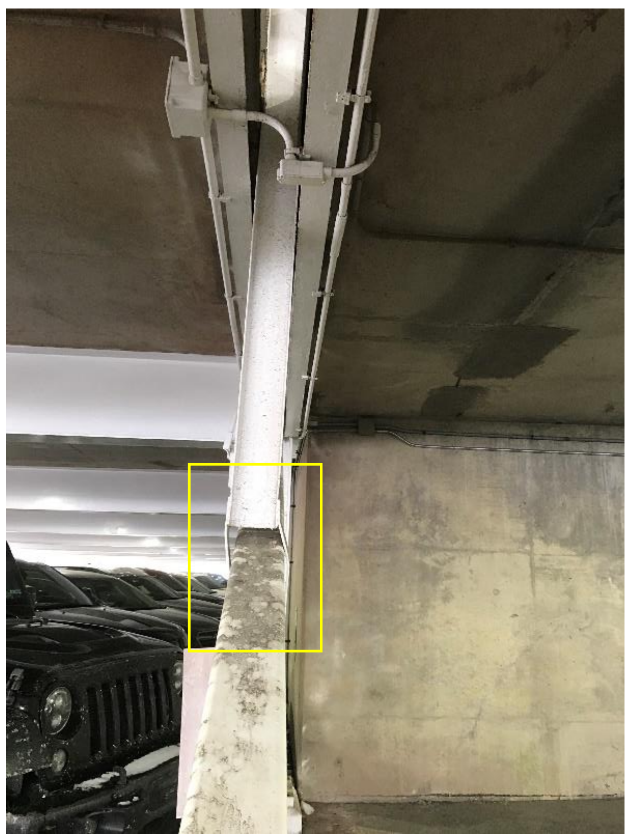

Figure 58. Deformed intermediate post splice

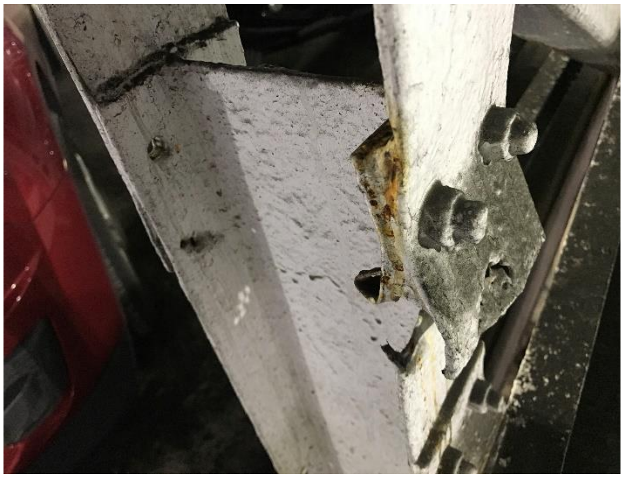

Figure 59. Corroded flange plate at intermediate post splice

Figure 60. Missing bolt at flange plate at intermediate post splice

Figure 61. Corrosion byproduct buildup at channel to post connection

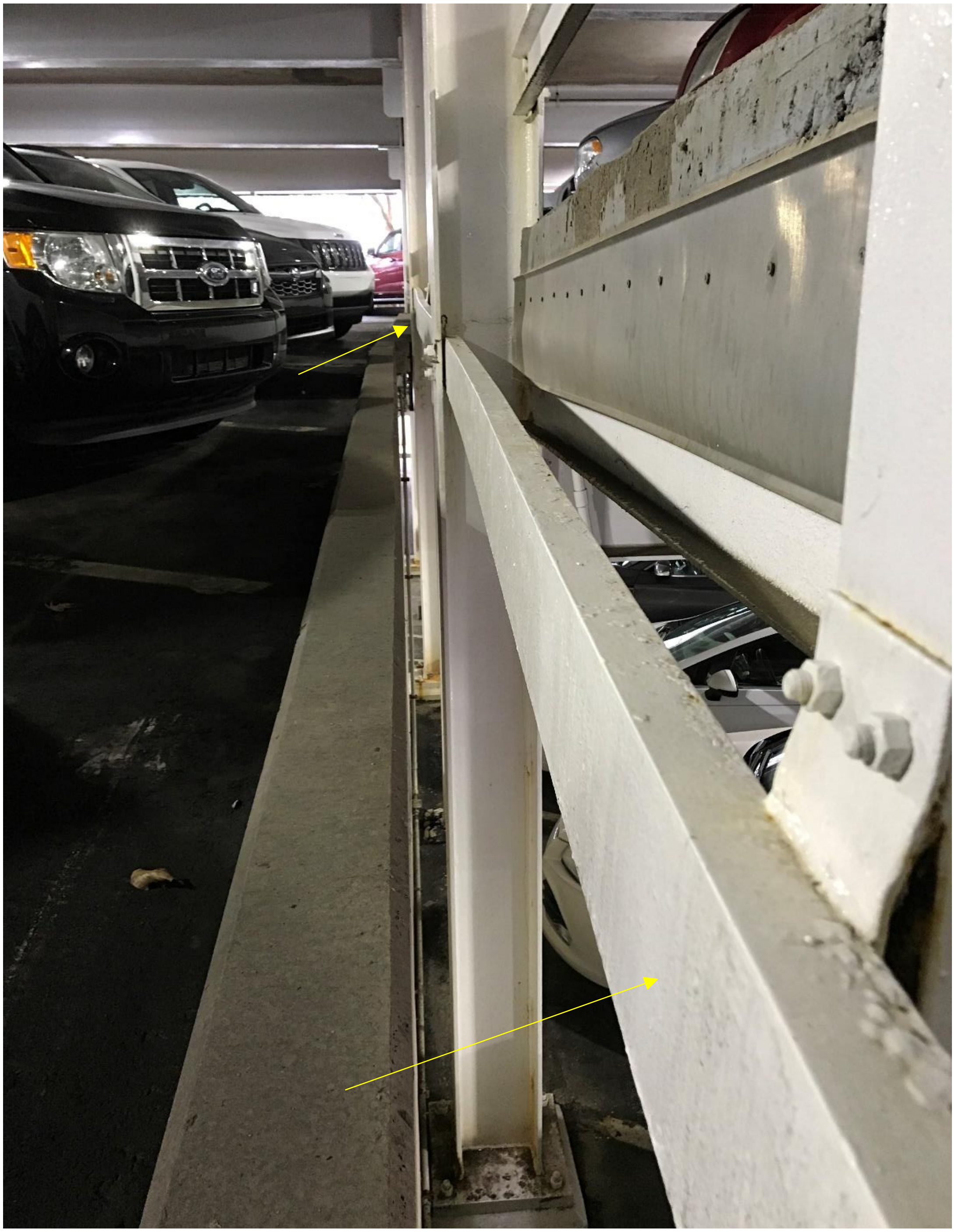

Figure 62. Lateral deformation of channels

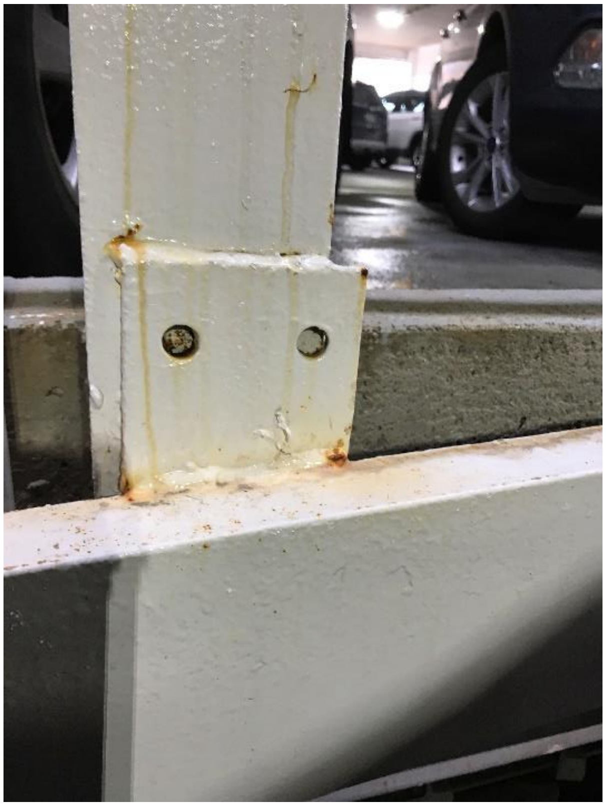
Figure 63. Missing bolts at channel to post connection, but welds present

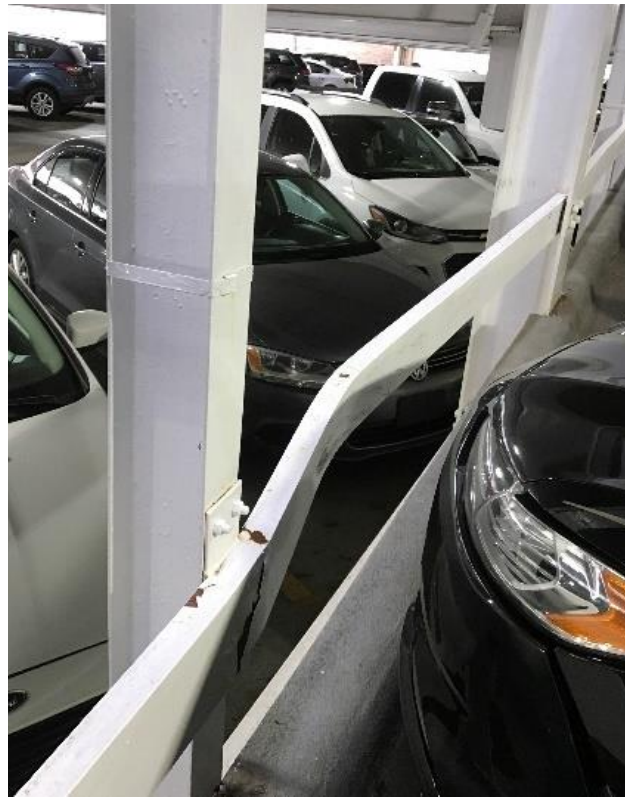
Figure 64. Vehicle impact at guardrail


Figure 65. Traffic-bearing membrane failure at previous concrete repair


Figure 66. Worn and failed traffic-bearing membrane at Level 2 turnaround


Figure 67. Sealed crack covered by traffic-bearing membrane

Figure 68. Failed stairwell expansion joint

Figure 69. Failed coating at CMU stair tower wall below expansion joint seal

Figure 70. Roof drain outlet above expansion joint

Figure 71. Standing water at unsloped bay of roof level

Figure 72. Clogged drain and standing water

Figure 73. Facade panels tilted inward and failed tie-backs.

Figure 74. Failed welds and gaps at tie-backs

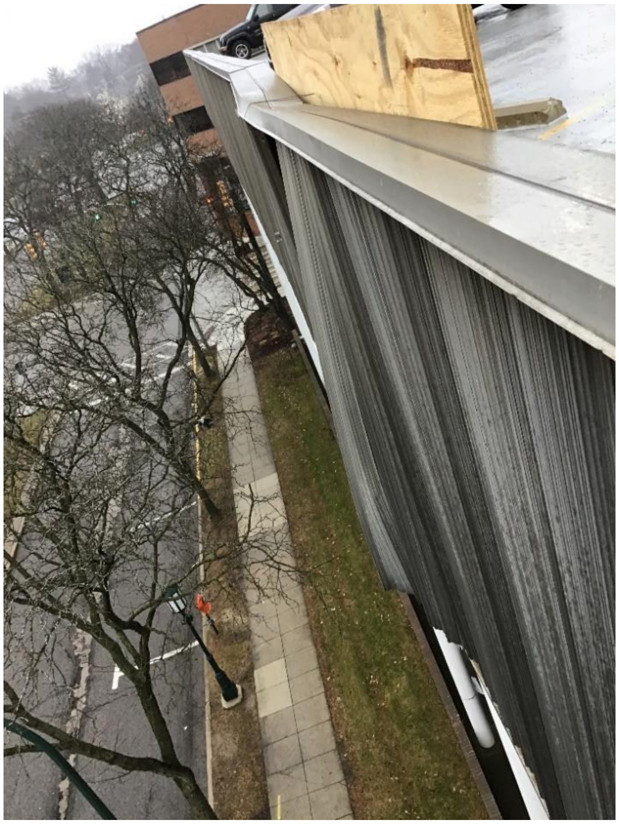

Figure 75. Vehicle impact at facade panel

Figure 76. Vehicle impact at facade panel posts


Figure 77. Missing base plates and anchors


Figure 78. Corroded and missing anchors at facade panels


Figure 79. Opening in facade panels at roof level

Figure 80. Cracks in CMU stair tower walls near corroded steel

Figure 81. Cracked brick masonry header

APPENDIX A. MATERIALS TESTING REPORT

City of Birmingham
Parking Garage Structural Assessment Program

Park Street Parking Structure

222 Park Street
Birmingham, MI 48009

Karla Salahshour
Senior Associate, Petrographer

LABORATORY REPORT

March 26, 2021
WJE No. 2019.6318.0

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CONTENTS

INTRODUCTION

Wiss, Janney, Elstner Associates, Inc. (WJE) completed laboratory testing on five concrete cores extracted from the Park Street Parking Structure located at 222 Park Street in Birmingham, Michigan. The Park Street structure was constructed during the mid-1970's and has five levels of parking with a centralized ramp. The structure consists of 5-1/2-inch-thick one-way post-tensioned (PT) concrete slabs supported by steel beams, girders, and columns. Laboratory testing was completed on concrete cores that were extracted from the PT concrete slabs to characterize the material. The laboratory testing was completed as part of a larger investigation of the parking structure being performed by WJE's Detroit, Michigan office. The findings from this laboratory report will be used to assist in the repair recommendations for the parking structure.

SAMPLING

Five concrete cores were extracted throughout the parking structure and sent to WJE's Cleveland, Ohio laboratory for material testing. A summary of the core extraction locations is provided in Table 1. Photographs of the cores are provided in Figure 1. The cores were extracted vertically through the full thickness of the PT slab, and they ranged in length from 5 to 6-3/4 inches. The tops of the cores represent the exposed, wearing surface of the slab. The tops of Cores 2, 4, and 5 contained a traffic membrane on the concrete slab. The tops of Cores 1 and 3 represent exposed, eroded concrete. The bottoms of all five cores are formed surfaces. No reinforcement was intersected by the cores.

Laboratory testing was performed on all five cores. A petrographic examination was only requested on Core 1 to characterize the concrete. Chloride ion content, water absorption, and carbonation tests were conducted on all five concrete cores. A summary of the testing performed is provided in Table 1.

Table 1. Summary of Park Street Parking Structure Concrete Cores

Core ID	Core Extraction Location	Location Description	Testing Performed			
			Petrographic Examination	Chloride Ion Content	Water Absorption	Carbonation
1	Level 3, East Bay	In parking stall outside of drive lane, no membrane installation	X	X	X	X
2	Level 3, Central Ramp	In parking stall outside of drive lane, traffic bearing membrane		X	X	X
3	Level 2, Northwest Bay	In drive lane, within about 30 feet of the drain and near the stair tower, near the transition between the elevated slab and slab on grade		X	X	X
4	Level 5 (Roof), Northwest Bay	Drive lane, within about 30 feet of the drain and near the stair tower, worn membrane nearby		X	X	X
5	Level 5 (Roof), Northwest Bay	Drive lane, within about 30 feet of the drain and near the stair tower, worn membrane nearby		X	X	X

MATERIALS TESTING

Petrographic Examination

Methodology

Cursory examinations of the as-received core samples and saw-cut cross-sectional surfaces prepared for other laboratory testing were performed on all of the cores. A petrographic examination involving a more detailed examination of the material was conducted on Core 1 as part of the materials testing program. The petrographic studies were conducted in accordance with the procedures described in ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete*. Microscope examination and various tests conducted during the petrographic examination are designed to elicit specific information about the composition and condition of the concrete. The observations are interpreted to derive conclusions about quality, performance, and probable cause of various types of distress.

A 3/4-inch thick slab was cut along the longitudinal axis from the middle of Core 1 using a water-cooled, continuous-rim, diamond saw blade. The saw-cut surfaces of the slab were then lapped using discs of progressively finer abrasives to achieve a fine, matte finish suitable for examination with a stereomicroscope. Lapping exposes textural features such that the edges of air voids, cracks, and aggregate constituents can be more easily identified. A lapped cross-section of the core is shown in Figure 2. Fresh fracture surfaces were also prepared to study the physical characteristics of the concrete. Lapped and fracture surfaces were examined at magnifications up to 90X using a stereomicroscope. A thin section was prepared from near the exterior surface of the core to further assess paste and aggregate characteristics. The thin section was examined at magnifications ranging from 50X to 630X using a petrographic (polarized-light) microscope.

Unit weight was measured for representative portions of each core according to Section 9, Unit Weight and Loss of Free Water, of ASTM C1084, *Standard Test Method for Portland-Cement Content of Hardened Hydraulic-Cement Concrete*. The results are provided in Table 2.

Findings

The concrete in all of the cores appears compositionally similar based on a visual inspection of the saw-cut surfaces. The petrographic examination was conducted, however, only on Core 1. The cores represent a crushed limestone coarse aggregate and blended fine aggregate in a portland cement-fly ash, air-entrained paste.

The coarse aggregate is composed of limestone, dolomite, and cherty limestone particles. Particles are tan, brown, and gray in color, angular to sub-rounded in shape, and frequently porous. The coarse aggregates have a maximum size of 3/4 inch and are uniformly distributed and well graded. The fine aggregate consists of a blend of calcareous and siliceous aggregates. A minor amount of coarse aggregates are elongate in shape, and their long dimensions are oriented parallel to the top and within the near-surface of the core. Veneers of dark gray paste adhered to portions of coarse aggregates were frequently observed. This feature in combination with air void clusters suggests the concrete was retempered. A minor amount of fine aggregates, primarily chert particles, contain a darkened rim around their perimeter (Figure 3). While some of these rims may have formed naturally prior to their incorporation in the concrete, some may be a “reaction rim” as a result of alkali-silica reaction (ASR). A concentration of chert particles with these reaction rims at a depth of 1-1/4 inches contain internal cracks and occasionally internal disruption. Distress external to these particles was frequently observed. Deposits with optical properties consistent with alkali-silica gel were observed within a crack at a depth of 1-1/4 inches (Figure 4). Microcracks extended from a chert fine aggregate particle adjacent a gel-lined void (Figure 5).

The paste in the body of Core 1 is medium gray in color. Discoloration was observed near the top surface that corresponds with the depth of paste carbonation, discussed in a subsequent section. The paste is soft and can be scratched using a copper probe in the body of the core. In thin section, residual portland cement, fly ash, and limestone fines were observed in the paste (Figure 6). Textural features observed microscopically are consistent with a moderate water-to-cementitious materials ratio. The paste is air-entrained, and the total volume of air was estimated to be 6 to 8 percent (Figure 7). Clustering of air voids and a slight increase in air content near the bottom of Core 1 was observed. No secondary deposits were observed within the air voids.

The top of Core 1 is slightly irregular in profile due to minor preferential erosion of the paste (Figure 8). As a result, fine aggregates are partially exposed on the surface of the core. A few of the exposed aggregates contain internal fractures (Figure 9). No distress was observed within the near-surface surface beneath the eroded surface of Core 1 (Figure 10). Coarse and fine aggregates are exposed on the surface of Core 3, and the surface may have been scarified (Figure 8). Coating membranes are present on the tops of Cores 2, 4, and 5. The coating is cracked, deteriorated, and worn on the surface of Cores 4 and 5. The coating is more intact on the surface of Core 2, although cracks were observed in localized areas microscopically.

Chloride Ion Content

Methodology

The water-soluble chloride ion contents were determined for the five cores at multiple depths. These depths were selected near the top surface (1/4 to 3/4 inch from the top) to determine if deicing salts, either applied directly to the slabs or carried in by vehicular traffic over time, penetrated into the concrete slab. The next depth (1-1/2 to 2 inches from the top) is located near the top level of mild reinforcing steel. A mid-depth (3 to 3-1/2 inches) slice was selected to serve as a baseline for the concrete. A depth near the bottom of the slab (depth varies due to slight differences in core lengths) was selected to determine if chlorides from spray from beneath the slab or sub-base conditions have penetrated into the concrete. These slices of concrete were saw-cut from one-half of each of the cores to be used for the testing.

The water-soluble chloride analysis was performed essentially according to ASTM C1218, *Standard Test Method for Water-Soluble Chloride in Mortar and Concrete*. The results are provided in Table 2.

Findings

Studies have shown that chloride contents above 0.03 percent by mass of concrete, depending on the cement content, can promote corrosion of embedded uncoated steel in non-carbonated normal weight concrete in the presence of sufficient moisture and oxygen. Levels below this threshold may accelerate corrosion in carbonated concrete. The chloride ion contents measured for the two top-most depths, except for Cores 4 and 5, greatly exceed this threshold. The chloride ion contents along the bottom of the cores is below this threshold except for Core 2.

While chloride-containing admixtures were commonly used in the 1970s (the reported era of construction for this parking structure), the low chloride levels measured near the middle of the cores, except for Core 2, suggest a chloride-containing admixture was not used during original construction for the concrete slabs. Rather, the elevated chloride levels are the result of penetration of chlorides from an external source.

All of the cores contained an elevated chloride ion content for the top-most depth analyzed. This observation coupled with a general decrease in chloride ion contents from the top surface suggests penetration of chloride-based deicing salts over the life of the parking structure, as would be expected in a northern climate.

The increase in chloride ion content along the bottom of the five cores indicates a slight exposure and penetration of chloride-laden spray from the parking deck below into the underside of the elevated slabs.

Water Absorption

Methodology

During the laboratory testing, an assessment of the absorptivity of the top surface was requested to aid in the determination of a repair design for the parking structure. During this testing, water drops were applied to the as-received surface of each of the cores, and the shape and absorption of the water drop were recorded. Water drops were also applied at several locations on a laboratory-prepared fresh fracture surface of each core oriented perpendicular to the top surface, prepared for one of the core halves of each core. The absorptivity of each of the water drops was recorded with depth from the top surface. Results are provided in Table 2.

Findings

The water drops applied to the surfaces of Cores 1 and 3 spread but were not absorbed into the concrete (Figure 11). Water drops were applied to the coating on the surface of Core 2, and the drops were not absorbed. Water drops applied to the deteriorated coating on the surface of Cores 4 and 5 were absorbed.

On the fracture surfaces, water drops were absorbed near the surface and with depth from the surface for Cores 3, 4, and 5. The concrete was hydrophobic (i.e. water drops were not absorbed) to a depth of 1/8 inch in Core 1 and to a depth of 1/4 inch in Core 2.

Water drops that bead and are not absorbed by the concrete paste may indicate the presence of a sealer that had been applied and penetrated into the concrete. The lack of absorption noted on the surfaces of Cores 1 and 3 may indicate something is present on the surface prohibiting the absorption of the water, such as a sealer but could also be dirt and debris infilling the concrete pores. The hydrophobicity of the concrete to a depth of 1/8 inch from the top of Core 1 suggests the penetration of a sealer-like material that may have worn, at least slightly, from the surface. The hydrophobic concrete measured to a depth of 1/4 inch in Core 2 (beneath the coating) also suggests a sealer may have been applied prior to the application of the coating, but this feature was not noted beneath the coating in Cores 4 and 5.

Carbonation Depth

Methodology

One half of each of the five cores was fractured longitudinally in the laboratory for the carbonation studies. The fracture surface was blown free of debris using compressed air and treated with phenolphthalein indicator solution. The indicator solution will turn non-carbonated paste purple; carbonated paste will remain unchanged. Paste that exhibits a light purple color is judged to be partially carbonated. Carbonated paste loses its natural passivation of the embedded, uncoated reinforcing steel due to the reduction in pH of the paste. In the presence of moisture and oxygen, the steel is susceptible to corrosion. The depth of paste carbonation from the top and bottom surfaces are provided in Table 2.

Findings

The maximum depth of carbonation from the top surface of the concrete slab was 1/2 inch. Minimal carbonation was measured beneath the membrane in Core 4, but up to 1/4 inch of carbonation was noted beneath the relatively intact membrane on Core 2. The presence of the carbonation may indicate the concrete slab was exposed for a period of time prior to the installation of the traffic membrane. The depth of carbonation from the top surface has not yet reached the assumed depth of mild reinforcing steel (at least 1-inch deep).

The maximum depth of carbonation from the bottom surface was 1 inch. While the depth of cover for the PT strands along the length of the slabs was not reported to the laboratory, the depth of carbonation from the bottom surface may be nearing the reinforcement, the location and condition of which is being investigated by WJE's Detroit office.

Table 2. Summary of Material Testing

Core ID	Core Length (inch)	Unit Weight (pcf)	Chloride: Depth from Top Surface (inch)	Chloride: Water-Soluble Chloride (% by mass of sample)	Water Absorption Description[1]	Carbonation: From Top Surface (inch)	Carbonation: From Bottom Surface (inch)
1	5-1/2	141	1/4 - 3/4	0.632	Top - water spread, not absorbed	1/4 to 1/2	7/8
			1 1/2 - 2	0.262			
			3 - 3 1/2	0.011			
			4 3/4 - 5 1/4	0.013	FF - hydrophobic to 1/8 inch		
2	5-1/2	143	1/4 - 3/4	0.276	Top - coating not absorptive	1/4 (below the coating)	1
			1 1/2 - 2	0.049			
			3 - 3 1/2	0.031			
			4 1/2 - 5	0.046	FF - not hydrophobic		
3	6-3/4	144	1/4 - 3/4	0.610	Top - water spread, not absorbed	1/8	3/4 (partial)
			1 1/2 - 2	0.205			
			3 - 3 1/2	0.010			
			6 - 6 1/2	0.012	FF - hydrophobic to 1/4 inch		
4	5	144	1/4 - 3/4	0.228	Top - coating is absorptive	0 to 1/32 (below the coating)	3/4
			1 1/2 - 2	0.008			
			3 - 3 1/2	0.006			
			4 1/4 - 4 3/4	0.014	FF - not hydrophobic		
5	5-1/2	144	1/4 - 3/4	0.039	Top - coating is absorptive	at least 3/8 (below the coating)	3/4
			1 1/2 - 2	0.006			
			3 - 3 1/2	0.006			
			4 1/4 - 4 3/4	0.013	FF - not hydrophobic		

[1] FF = fresh fracture surface prepared in the laboratory to which water was applied

DISCUSSION

General Condition

The concrete examined as part of the studies is in good overall condition. Portions of the parking structure have been coated with a membrane, but portions of the slabs are left uncoated or the membrane has since debonded. The concrete that was left exposed (uncoated), exhibits erosion of the top surface resulting in the exposure of fine and coarse aggregates. Some of these exposed aggregates contain microcracks that likely formed due to repeated exposure to vehicular traffic. No significant distress was observed beneath the eroded surface or in the body of the cores. For the areas of the parking structure that were coated, wide variability exists in the condition of the membrane. The membrane was severely deteriorated in two of the three cores containing a membrane, and in the third core, the coating was generally intact but contained localized deterioration. The concrete in the body of the cores contains a minor amount of microcracking associated with ASR of a low volume of chert that is primarily contained within the fine aggregate. The majority of the ASR-related distress was confined to the near-surface region in Core 1 in an area with elevated chlorides. Previous studies have suggested that commonly used deicers, including chloride-based deicers, can accelerate ASR[1]. Significant distress was not observed in the concrete cores, likely, in part, due to the presence of air-entrainment which results in freeze-thaw durability of the concrete. The lack of secondary deposits in voids suggests significant bulk moisture has not migrated through the deck, which is also likely contributing to the overall good condition of the concrete as moisture is required for several deterioration mechanisms, such as freeze-thaw and ASR. Fly ash, which was observed within the paste, can also mitigate ASR, depending on the volume and composition of the fly ash.

Reaction rims, internal microcracks and disruption, external cracking and alkali-silica gel deposition are all features of ASR. A trace amount of particles contained external distress, and gel appeared to have been harmlessly deposited within cracks and void space (i.e. air voids). Distress due to ASR is judged to be very minor. Given the age of the parking structure, future significant distress due to ASR is unlikely. However, any repairs that may be performed that alter the moisture content, relative humidity, or other features pertaining to the ambient conditions of the concrete may alter the rate of reaction. Additional laboratory testing to determine the potential for future distress can be performed if requested.

Carbonation and Chlorides

The depth of paste carbonation was measured from both the top and bottom surfaces of the cores. The presence of carbonated paste beneath the coating in Core 2 suggests the concrete had been exposed for some time prior to the installation of the membrane. The maximum depth of paste carbonation from the top of the concrete was 1/2 inch. The maximum depth of carbonation from the bottom of the cores was 1 inch. This depth of carbonation may be nearing the depth of the post-tensioning tendons, and the presence of carbonation should be considered during the repair design. Couple the presence of

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[1] Chiu, Charles Yicheng, "The effects of chloride-based deicing chemicals on degradation of portland cement mortars with alkali reactive aggregate" (2016). Purdue University. *Open Access Dissertations*. 916. <htps://docs.lib.purdue.edu/open_access_dissertations/916>.

carbonated paste with elevated chlorides from the bottom in Core 2 and the condition of the tendons should be evaluated.

Water-soluble chloride ion content was measured at several depths throughout the thickness of the slabs. The chloride levels typically decrease toward the middle of the slab, indicating a chloride gradient from the top and bottom surfaces. This gradient is consistent with an external source of chlorides, such as from deicing salts commonly used on parking structures, rather than an internal source of chlorides. During the era of construction of the parking structure, chloride-containing admixtures were commonly used. However, the low chloride levels near the middle of the cores suggests such admixtures were not used for this structure. The chloride levels measured at and near the tops of Cores 1, 2, and 3 are significantly elevated and would be expected to promote corrosion of embedded steel in the presence of moisture and oxygen. A slight increase in chloride ion content exists along the bottom of the cores suggesting an introduction of chloride from the underside of the slabs, likely from spray from vehicular traffic.

Repair Considerations

Repairs to the parking structure should consider the significantly elevated chloride ion content near the surface of the cores, and the slightly elevated chloride values near the bottom of the cores. The significance of elevated chloride ion content includes: increased likelihood of ASR associated with elevated chloride ion content; increased number of freeze-thaw cycles; and the potential for corrosion of embedded reinforcement. Carbonation of the paste was also observed from both the top and bottom of the cores, and carbonated paste results in a loss of the natural passivation protecting embedded steel reinforcement from corrosion.

The paste along the surface of Cores 1 and 3 is not absorptive and exhibits hydrophobicity to a maximum depth of 1/4 inch beneath the surface on the fracture surfaces. These findings may indicate the presence of a sealer, such as a silane or siloxane-based penetrating material, that had been applied to the uncoated areas of the slab at some point. Paste that is not absorptive would inhibit adherence of a membrane sealer and ingress of penetrating sealers, if they are being considered as part of the proposed repair scheme, and surface preparation would be required prior to their application.

As previously noted, water is required (although different amounts) for both freeze-thaw and ASR deterioration mechanisms. Preventing or prohibiting bulk moisture from entering the concrete (both from above and below) will extend the service life of the parking structure.

FIGURES

Figure 1. The as-received appearance of the tops (upper left), bottoms (upper right), and sides (lower) of Cores 1 through 5 are pictured. In the lower image, Cores 1 through 5 are pictured from the left to right, respectively.

Figure 2. Lapped surface of Core 1.

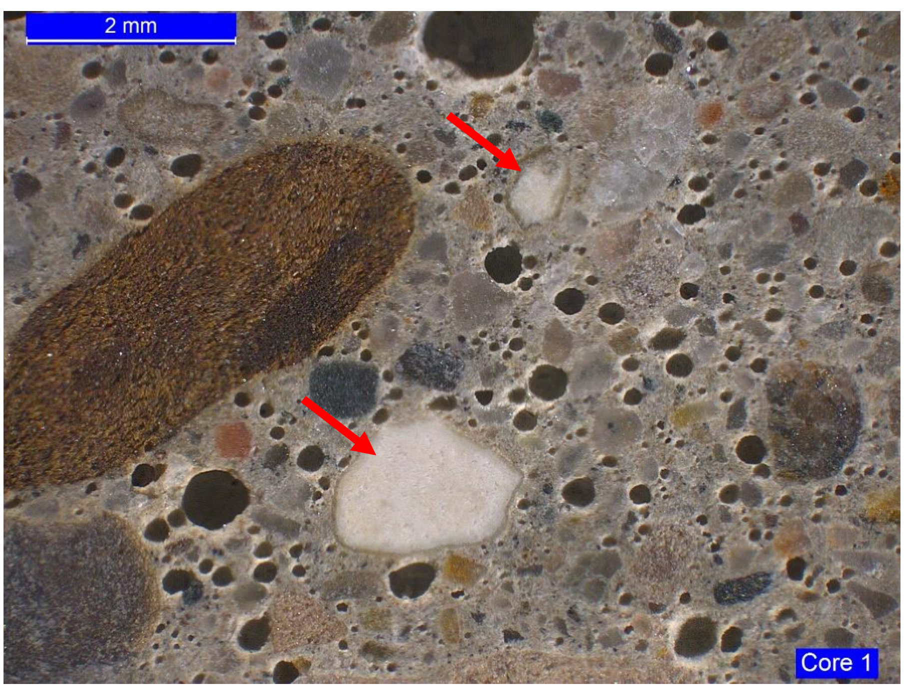

Figure 3. Two fine aggregate particles with a darkened reaction rim (arrows).

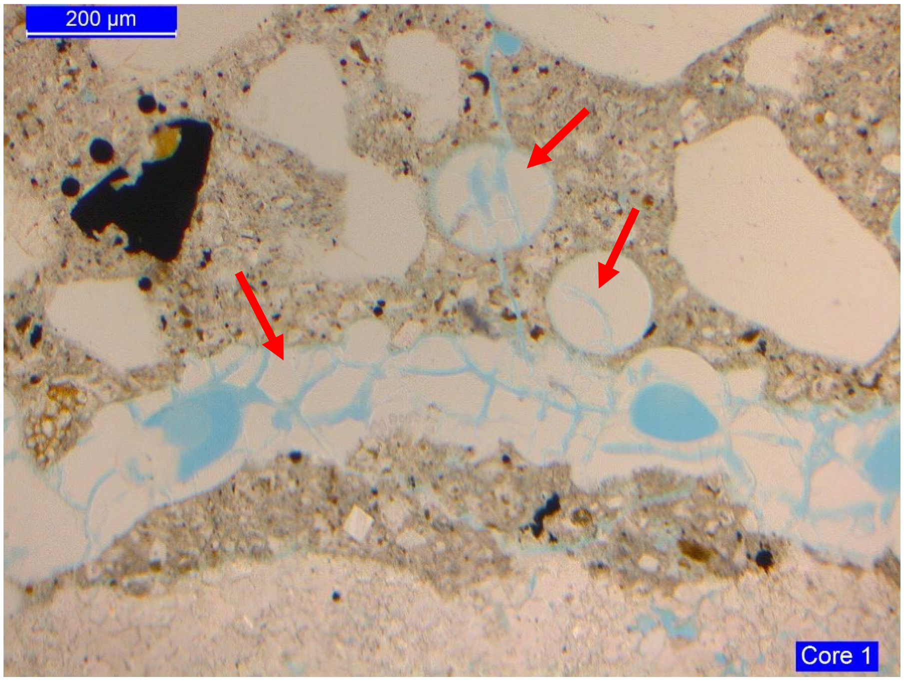

Figure 4. Alkali-silica gel infilling a crack and voids (arrows) in Core 1.

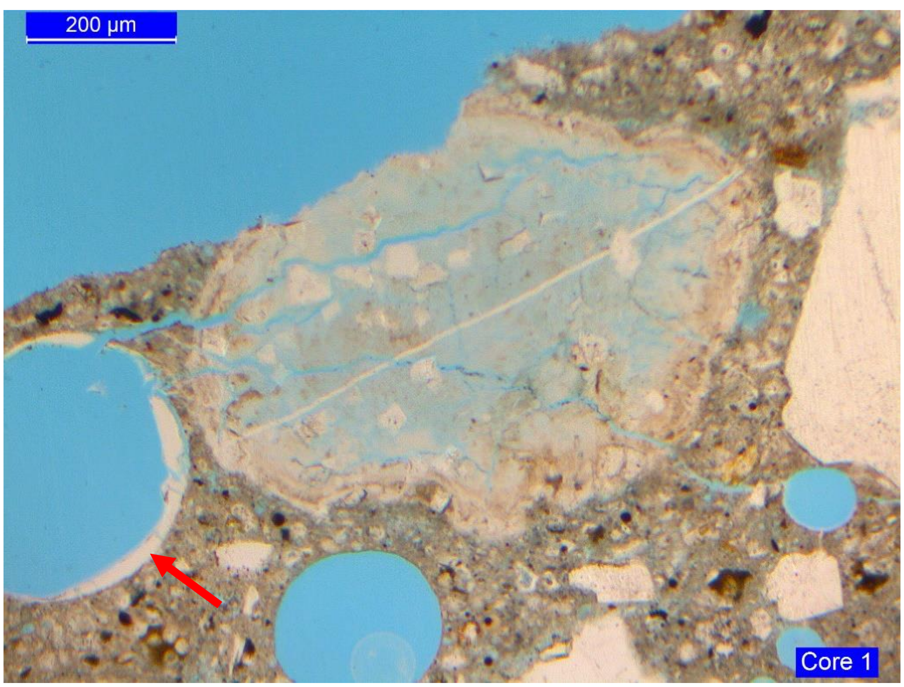

Figure 5. Microcracks extending from a chert particle into adjacent voids, one of which contains alkali-silica gel (arrow).

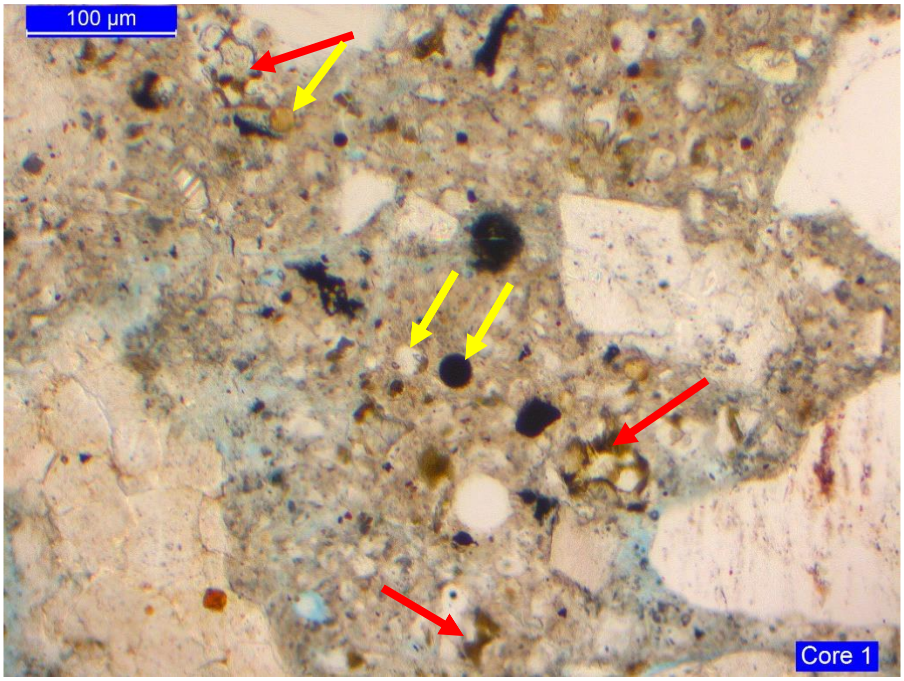

Figure 6. Residual portland cement (red arrows) and fly ash (yellow arrows) in the paste in Core 1.

Figure 7. Air void system in Core 1. Entrained air voids appear primarily as small, black circular areas due to the use of low-angle light illumination.

Figure 8. The eroded top surfaces of Core 1 (left) and Core 3 (right).

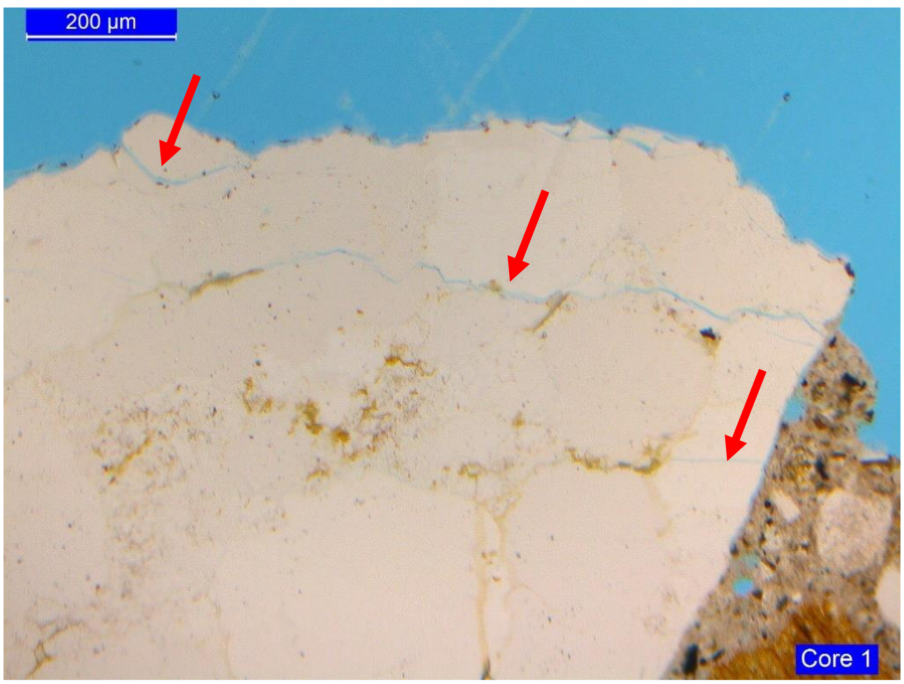

Figure 9. Cracks (arrows) within a protruding aggregate along the top of Core 1.

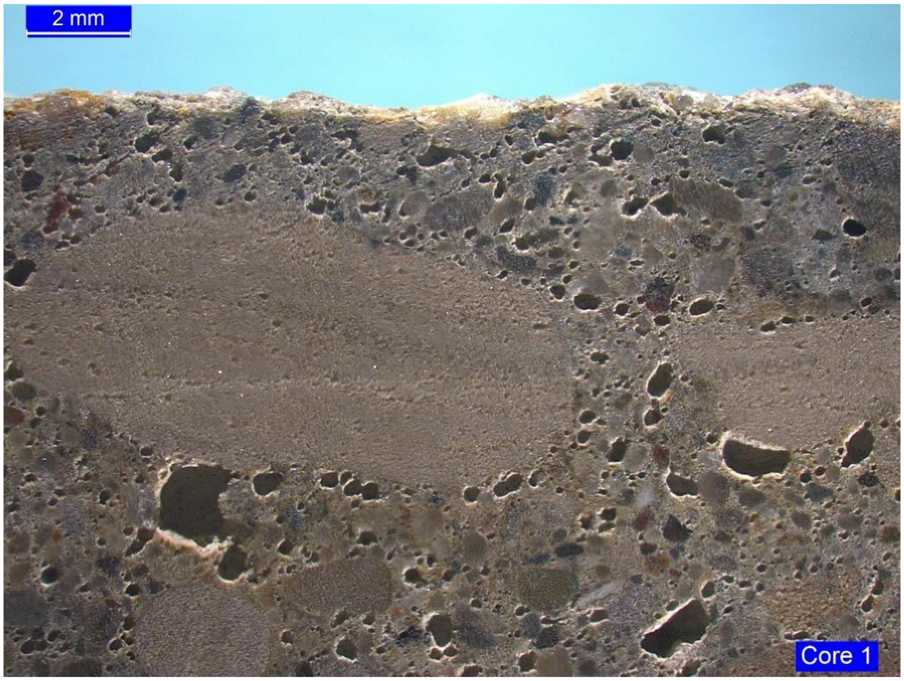

Figure 10. Near-surface region of Core 1 is pictured with no distress.

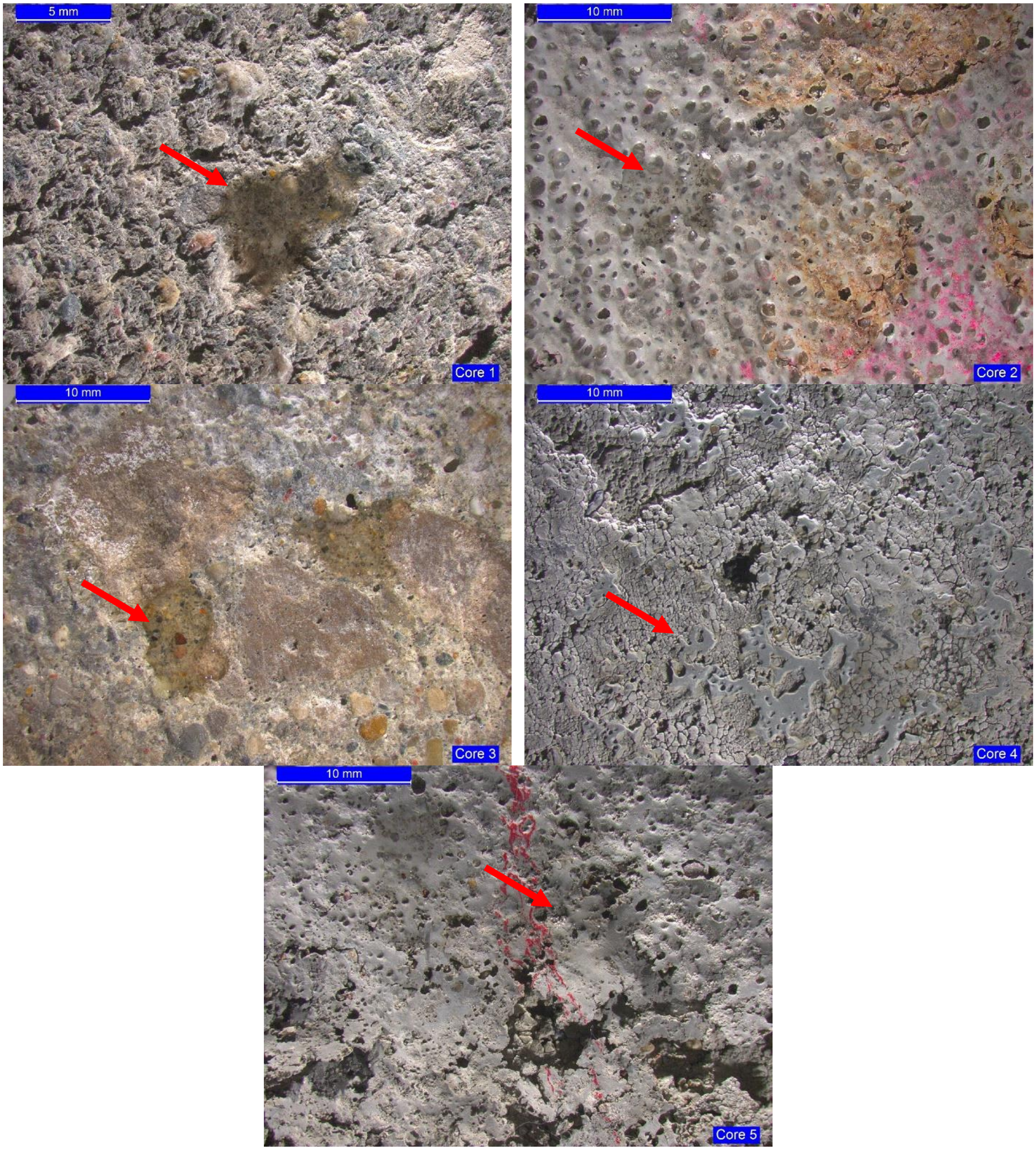

Figure 11. Water drops (arrows) applied to the surface are pictured for all five cores. Cores 2, 4, and 5 contain a coating membrane on the surface of the concrete. The water was absorbed on Cores 4 and 5 but not in Cores 1, 2, and 3.

APPENDIX B. OPINION OF PROBABLE COSTS

OPINION OF PROBABLE COSTS

Near-Term Recommendations (within 1 to 2 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Replace construction joint sealant*	900	LF	$ 6	$ 5,400
Repair column stiffener and moment connection plates*	24	EA	$ 1,000	$ 24,000
Inspect and clean drain lines*	1	LS	$ 15,000	$ 15,000
Traffic bearing membrane - complete replacement or new installation	142,000	SF	$ 4	$ 568,000
Traffic bearing membrane - add'l top coat only	72,000	SF	$ 2.50	$ 180,000
Rout and seal cracks in elevated slabs	1,500	LF	$ 6	$ 9,000
Replace expansion joint seals at stair towers	100	LF	$ 125	$ 12,500
Localized concrete repairs in slab, partial depth topside	2,500	SF	$ 45	$ 112,500
Localized concrete repairs in slab, full depth	11,000	SF	$ 80	$ 880,000
P/T slab tendon and anchor repair - allowance, approx. 50 repairs	1	LS	$ 250,000	$ 250,000
			Subtotal	**$ 2,056,400**
			General Conditions, Overhead and Profit (15%)	$ 308,460
			Project Contingency (15%)	$ 308,460
			Engineering/Testing/Construction Period Services (10%)	$ 205,640
			Near-Term Recommendations Total	**$ 2,878,960**
Long-Term Recommendations (within 3-5 Years)				
Item Description	**Est. Qty.**	**Units**	**Unit cost**	**Est. Cost****
Concrete Structure Repairs				
Localized concrete repairs in slab, partial depth topside	100	SF	$ 45	$ 4,500
Localized concrete repairs in slab, full depth	150	SF	$ 80	$ 12,000
Structural Steel Repairs				
Repair column base plates and/or anchorages	10	EA	$ 250	$ 2,500
Repair exterior pipe column bases	44	EA	$ 750	$ 33,000
Repair beam-to-column shear connections	70	EA	$ 500	$ 35,000
Repair intermediate vehicle barrier post connections, properly clean and paint	115	EA	$ 250	$ 28,750
Replace intermediate vehicle barrier post connections, in-kind	50	EA	$ 500	$ 25,000
Repair intermediate vehicle barrier post bases ǂ	18	EA	$ 10,000	$ 180,000
Replace vehicle barriers, in kind	5	EA	$ 400	$ 2,000
Facade Repairs				
Replace facade panels and posts impacted by vehicles	1	LS	$ 5,000	$ 5,000
Replace missing anchors at facade base plates	1	LS	$ 2,500	$ 2,500
Reattach facade panel tie-backs ǂ	1	LS	$ 200,000	$ 200,000
Miscellaneous				
Repair stair landings, tread/risers, CMU walls, and brick headers	1	LS	$ 20,000	$ 20,000
			Subtotal	**$ 550,250**
			General Conditions, Overhead and Profit (15%)	$ 82,538
			Project Contingency (15%)	$ 82,538
			Engineering/Testing/Construction Period Services (10%)	$ 55,025
			Long Term Recommendations Total	**$ 770,350**
			Grand Total	**$ 3,649,310**

* Highest priority of near-term repair items.

** Prices based on current (2021) dollars, and do not include increases for inflation (recommended 3% per year).

ǂ Pending further analysis during repair design phase.