EXECUTIVE SUMMARY ................................................................. ii

INTRODUCTION .............................................................................. 1
   Objectives ................................................................................. 1
   Structure Description ............................................................... 1
   Background Information ......................................................... 2

CONCLUSIONS ............................................................................... 3

RECOMMENDATIONS ................................................................. 4
   Short-term Parking Restrictions ............................................... 4
   Conceptual Long-term Repair/Replacement Options ......... 4
   Conceptual Short-term Strengthening/Repair Approaches ................................................................. 5

DISCUSSION .................................................................................. 6
   Conclusions ............................................................................. 6
   Evaluation Findings ................................................................. 6
   Structural Analysis of Original Slab Capacity .................. 7
   Conceptual Repair Alternatives ........................................... 8

SUMMARY .................................................................................... 9
   Observations .......................................................................... 9
   Ground Penetrating Radar Testing .................................... 11
   Concrete Core Samples ......................................................... 11

LIMITATIONS .............................................................................. 13

APPENDIX A - Photographs
APPENDIX B - Field Sketches
APPENDIX C - Scope of Services

TABLE OF CONTENTS
EXECUTIVE SUMMARY

At the request of Northwestern University, Walker Restoration Consultants performed a structural evaluation of the two-way reinforced concrete roof slab at the Evanston Water Utility Finished Water Reservoir in Evanston, Illinois. Our evaluation included visual review of the roof slab concrete and embedded steel reinforcing at two inspection areas from the top of the slab, visual review of extracted concrete core samples, non-destructive ground penetrating radar testing, and a limited structural analysis of the roof slab. We also peer reviewed two other reports from other consultants regarding the finished water reservoir.

The report from CTLGroup indicated the roof slab was undergoing freeze/thaw deterioration which induced horizontal cracking in the slab that reduced the capacity of the slab but the slab could remain in use for passenger vehicles with yearly evaluations. CTL’s repair recommendation was to completely demolish and replace the roof slab and recommended a construction budget of approximately $4 million. The report from CDM Smith was a peer review of the CTL report and concurred with their recommendations and gave a repair budget of $3.8 to $6.7 million.

We found the top of the finished water reservoir roof slab to exhibit significant freeze/thaw deterioration resulting in near disintegration of the first 1 ½ inches to 2 inches of concrete in both of the top of slab inspection areas. In addition, we observed locations of localized section loss at the top steel reinforcing due to corrosion. Finally, in the extracted cores, we observed that the cores came out in a number of pieces indicating that there is horizontal, surface parallel cracking in the remaining portion of the slab. All of these conditions can reduce the structural capacity of the roof slab, particularly in the negative moment region over the columns and drop panels.

Walker then performed a limited structural analysis of the concrete roof slab to understand the approximate original design load capacity based on the configuration and reinforcing steel shown on the original drawings (assuming everything is in good condition). The results of that analysis indicate that the original design had a live load capacity significantly higher than the current code required live load capacity to support passenger vehicles.

Walker then attempted to perform a limited structural analysis of the concrete roof slab trying to take into account the deterioration described above. However, we found that there is no reliable way to specifically quantify the reduction in capacity due to the horizontal, surface parallel cracking. Based on our structural analysis of the original slab capacity and the observed deterioration, it is our opinion that the roof slab can safely support the current loadings (passenger vehicles only) in the current condition. Since the deterioration is ongoing, we recommend monitoring the condition of the slab. In addition to a yearly condition survey of the slab, we recommend implementing a simple technique of slab monitoring by monitoring the slab deflection in the center of at least two bays from the top using surveying techniques at four month intervals. This will allow us to determine if the slab is deflecting. Our opinion that the slab can remain in service for another year is due to the fact that two-way slabs are inherently robust, especially when designed for a significant live load as this slab was.
Although full-depth replacement of the roof slab, as recommended by the two previous consultants, is a viable and sound long-term repair option for addressing the deterioration, Walker believes there are less invasive and less costly repair approaches that could be implemented to extend the service life of the structure. These repair alternatives involve removal of only the asphalt, fill, and the deteriorated 1 ½ inches of concrete at the top of the existing slab and the installation of a new concrete slab on top of the existing slab that could be properly designed to support the old slab in place, all within the weight and depth of the existing fill. Re-using the old slab as a stay-in-place form for a new slab is a much greener solution that preserves the embodied energy of the old slab by reducing the cost and energy of the demolition, disposal and forming process. There are a few different types of new slabs that could be constructed to perform this function.

In comparison to the full-depth replacement option, the repair options described above would require significantly shorter shutdowns of the reservoir portion (some options could require reservoir shutdowns as short as 1 month), as well as lower construction costs. We believe that the option of re-supporting and keeping the existing slab in place could still give the reservoir a long service life on the order of 15 to 20+ years and cost less than previous replacement estimates. Our opinion of probable repair cost for these options could range from $2.5 to $3.5 million depending on the specifics of the final design. Walker is available to perform the schematic designs for a few different slab options to allow a more detailed cost opinion and reservoir shut-down times if desired.

Please see the attached discussion for a detailed report of our evaluation.

Daniel E. Moser, S.E., P.E.  
Principal/Restoration Department Head

March 31, 2014
INTRODUCTION


Restoration Engineer Trent Steffen and Principal/Restoration Department Head Dan Moser performed the evaluation on November 4 through November 7, 2013. The evaluation included visual review and hammer sounding of a limited portion of the top of roof slab concrete, non-destructive ground-penetrating radar testing to determine the as-built configuration of reinforcing in representative locations, and visual review and documentation of the condition of the embedded top layers of reinforcing steel at four test pit openings. The work was performed at two excavations to expose the top of the roof slab concrete approximately 10 feet x 20 feet in size, as shown in SK-1 in Appendix B.

OBJECTIVES

The objectives of the structural evaluation were as follows:

- Obtain pertinent information with regards to the condition of the top portion of the roof slab (including condition of the concrete and embedded reinforcing) to gain a better understanding of the current condition of the roof slab as a whole.
- Based on the results of the evaluation and limited structural analysis of the roof slab, determine necessary parking restrictions for the surface lot in relation to code required levels of safety.
- Develop practical strengthening approaches, if any that may be implemented in the short-term to maintain parking until repairs or replacement are completed.
- Determine potential conceptual long-term repair options that minimize the impact of repairs/replacement relative to the impact caused by full slab replacement.

STRUCTURE DESCRIPTION

The subject structure is a reinforced concrete water reservoir located on the Northwestern University Campus at the corner of Lincoln Street and North Campus Drive in Evanston, Illinois. The reservoir contains capacity for approximately 5 million gallons of treated water and supports an at grade parking lot on the roof slab providing parking for Northwestern University. According to the original drawings, the structure was built around 1934, making it approximately 79 years old.

As viewed in the plan, the structure is rectangular and measures approximately 260 feet in the north-south direction, approximately 140 feet in the east-west direction and extends approximately 22 feet below the surrounding grade. The reservoir is constructed of conventional reinforced concrete and consists of an 8 ½ inch thick two-way roof slab
supported by 20 inch diameter columns with 7 foot-4 inch square drop panels and 5 foot-0 inch diameter column capitals. The columns are located in a 21 foot square. The perimeter of the structure is enclosed by reinforced concrete walls which taper from 2 feet in thickness near the floor slab of the structure to 1 foot in thickness near the roof slab. Access to the interior of the structure is provided via four hatches, one at each corner of the structure.

BACKGROUND INFORMATION

Prior to the evaluation, Walker performed a limited peer review of a draft condition assessment report for the reservoir prepared by CTLGroup dated November 29, 2012. This assessment included visual assessment and documentation of the condition of the interior surfaces of reservoir, including the underside of roof slab, top side of floor slab, interior wall surfaces, drop panels and columns. In addition, concrete cover measurements were taken and concrete core samples were extracted for petrographic analysis and compressive strength testing.

The CTLGroup draft report identified deterioration present in the roof slab, including cracking with leaching and efflorescence, localized corrosion of embedded reinforcing steel, and limited areas of delaminated and deteriorated concrete. In addition, petrographic analysis of concrete core samples removed from the underside of the roof slab revealed the presence of freeze/thaw cracking at various depths within the cores. Based on the observed conditions, CTLGroup described the slab as being in generally poor condition and recommended imposing parking restrictions (limit access to passenger vehicles only) on the supported surface lot and replacement of the roof slab as soon as possible. CTL gave an estimated cost of $4 million for the recommended repairs.

Walker also performed a limited peer review of a peer review memorandum prepared by CDM Smith dated August 22, 2013. The CDM Smith memo concurred with the recommendation of full slab replacement and gave an estimated cost range for the recommended repairs of $3.8 to $6.7 million.
CONCLUSIONS

Based upon our visual observations, testing and analyses, we offer the following conclusions:

- The finished water reservoir roof slab exhibits significant freeze/thaw deterioration of the concrete at the top of the roof slab and locations of section loss of the embedded top steel reinforcing. Additionally, in the extracted cores, we observed that the cores came out in a number of pieces indicating that there is horizontal, surface parallel cracking in the remaining portion of the slab. These conditions can significantly reduce the structural capacity of the roof slab.

- Based on a structural analysis using the information shown on the original drawings (and generally confirmed by GPR non-destructive testing), the original roof slab construction in an undeteriorated state has an approximate live load capacity of 200 pounds per square foot when analyzed by the current building code and current ultimate strength design methodology. This is much higher than current passenger vehicle loading code requirements.

- The observed deterioration decreases the amount of the effective top steel reinforcing present to resist negative moment bending, leading to potential overstress of this region. However, a portion of the negative moments can be redistributed to positive moment regions (since there is sufficient reserve capacity to accommodate this redistribution) to make up for this deficiency, thus allowing the roof slab to safely resist the current loadings in the near term. It should be noted that only the uppermost layer of top steel (in the east-west direction) is in deteriorated concrete and requires significant negative moment redistribution. The second layer of top steel (in the north south direction) is contained in more sound concrete and does not require significant moment redistribution.
RECOMMENDATIONS

Based on our evaluation, we believe the existing finished water reservoir roof slab currently has sufficient capacity to safely support passenger vehicle loading for the next year while repairs are designed and bid out. However, the observed deterioration is ongoing and will continue to accelerate, leading to additional reduction in capacity and an unsafe condition. We recommend repairs be implemented as soon as possible (within one year) to address the deteriorated condition and restore the serviceability of the structure. Our detailed recommendations are presented below.

SHORT-TERM PARKING RESTRICTIONS

1) We recommend continuing to restrict parking on the surface lot area over the finished water reservoir to passenger vehicles only.

2) We also recommend roof slab deflection monitoring to determine if the roof slab is starting to deflect due to the deterioration. Deflection monitoring could be conducted by surveying the top of slab elevation in the center of at least two bays at a maximum four month interval.

3) If repairs or replacement are not completed within a year (By November 2014), we recommend the structure be thoroughly reevaluated before continuing to allow parking on the finished water reservoir roof slab. This would include the following:
   a. An updated survey and evaluation of the underside of the roof slab.
   b. Additional evaluation of the top of the slab at a minimum of six different areas.

CONCEPTUAL LONG-TERM REPAIR/REPLACEMENT OPTIONS

In their November 2012 report, CTLGroup recommended full replacement of the finished water reservoir roof slab to restore the structure. CTL indicated that the slab replacement cost would be approximately $4 million. CDM Smith concurred with this recommendation and gave a cost estimate range of $3.8 to $6.7 million for the repairs.

We believe that full slab replacement is a viable and sound long-term option, but also feel that less invasive and expensive repair options could be implemented to extend the life of the structure another 15 to 20+ years. These repair alternatives involve removal of only the asphalt, fill, and the deteriorated 1 ½ inches of concrete at the top of the existing slab and the installation of a new concrete slab on top of the existing slab that could be properly designed to support the old slab in place. We believe that there are a number of types of new slabs that could be properly designed to perform this function. Since the existing slab has approximately 12 inches of fill and asphalt on top, the new slab system could be slightly thinner than 12 inches and weigh less or similar to the existing 12 inches of fill.
Our opinion of probable repair cost for these options could range from $2.5 to $3.5 million depending on the specifics of the final design.

CONCEPTUAL SHORT-TERM STRENGTHENING/REPAIR APPROACHES

Due to the nature of the deterioration and current condition of the roof slab, we do not believe there are viable cost-effective measures to allow use of the surface lot beyond a year other than monitoring and performing the above described repairs.
DISCUSSION

In summary, we found the top of the finished water reservoir roof slab to be in generally poor condition. The conditions observed during the evaluation of the roof slab are discussed in the following paragraphs. In addition, the conceptual repair and replacement recommendations previously shown are discussed in further detail.

EVALUATION FINDINGS

As stated above, significant freeze/thaw deterioration was observed at the top surface of the finished water reservoir concrete roof slab over approximately 90% of the two areas excavated for visual review and testing. It is likely that the existing fill materials and lack of significant drainage slope or drain system create a situation where moisture is continually present at the top surface of the roof slab concrete. During cold temperatures, this moisture alternately freezes and thaws, subjecting the concrete to destructive expansive forces. Based on visual review of the concrete and review of core samples extracted from the test excavations, the freeze/thaw deterioration has significantly compromised the integrity of the top 1 ½ inches to 2 inches of concrete, as evidenced by numerous cracks and delaminations parallel to the roof surface, observable degradation of the cement paste, easy removal of the top concrete surface, and saturated, soft concrete experienced by the contractor while chipping test pit openings. However, once the excavation extended below the 1 ½ inches to 2 inches, the concrete appeared hard and intact based on chipping. This harder concrete envelops the second layer of top steel.

The observed top of slab concrete deterioration significantly affects the structural capacity of the slab in at least two ways. First, the top steel reinforcing running in the east-west direction is contained within the region of severely deteriorated concrete. In order for the steel reinforcing to act compositely with the concrete in resisting the imposed loads, it must be bonded in sound concrete. However, the top reinforcing steel in the north-south direction appeared to be located in relatively sound concrete and is therefore considered to be developed and effective at this time. Second, the noted widespread top of slab concrete deterioration reduces the effective thickness of the slab, which reduces the flexural capacity of the slab for positive moment.

At three of the four smaller concrete test pit openings, we also noted corrosion and section loss of the top two layers of steel reinforcing. The corrosion and section loss were more significant in Test Pits 1 and 2 in Test Excavation 1, with only minor corrosion and section loss noted on one bar in Test Pit 3 (Test Excavation 2). The observed section loss of the reinforcement also reduces the structural capacity of the roof slab, as the effective area of reinforcement is reduced.

It should also be noted that the existing deteriorated surface is significantly more permeable and subject to moisture infiltration than the original construction. This observation coupled with the fact that the original concrete is not air entrained make the roof slab susceptible to
continued freeze/thaw deterioration at an accelerated rate if the source of moisture is not cut off.

STRUCTURAL ANALYSIS OF ORIGINAL SLAB CAPACITY

A structural analysis of the existing structure was completed, based on the reinforcing shown on the existing drawings and using the material properties and loadings discussed below.

MATERIAL PROPERTIES

In order to determine the capacity of the roof slab, it is required to know the physical properties used in its construction. The original drawings by Alvord, Burdick & Howson Engineers, dated September 1933 were reviewed to gain an understanding of the original design intent and construction. These drawings do not include any reference to the design compressive strength of the concrete or to a specified yield stress for the steel reinforcing, both of which are necessary to determine the strength of a reinforced concrete member. In their 2012 report, CTLGroup tested three concrete core samples from the Finished Water Reservoir walls and floor slab and reported compressive strengths of 8,480, 5,830, and 7,760 psi for an average of 7,357 psi. Based on the unknown compressive strength of the roof slab, it was decided to use a conservative estimate of 4,000 psi, which is well below the values obtained by testing of the other elements. For the reinforcing steel, it is reasonable to assume a structure built in 1933 used “intermediate grade” reinforcing bars with a minimum specified yield strength of 40 ksi and a minimum specified tensile strength of 70 ksi (Gustav G. Erlemann, Consultant. Reinforcing Bar Specifications – 1911 through 1968. Engineered Concrete Structures, pp. 3-4).

LOADS

To determine the loading for the structure, the existing fill profile was documented at representative locations within the test excavations. Based on field measurements, the fill consists of an asphalt surface, gravel, unidentified earthen fill, and a bottom layer of sand against the concrete roof slab. Based on average unit weights of these materials, the dead load of the fill on the roof slab was calculated to be 100 pounds per square foot. In addition, the self-weight of the 8 ½ inch thick reinforced concrete slab adds approximately 107 pounds per square foot, for a total dead load of 207 pounds per square foot.

On the first sheet of the original drawings, a handwritten note states that the roof is designed for 300 pounds per square foot; however, this note was dated July 16, 56, well after the reservoir was constructed. To verify the original capacity of the slab, a structural analysis was performed as described below.

ANALYSIS RESULTS

Based on the criteria described above and analyzing the existing slab per current building code requirements and ultimate strength methodologies, the existing roof slab in an un-
deteriorated state was found to have sufficient capacity to safely support a live load of 200 pounds per square foot, assuming code allowed negative moment redistribution.

Walker also attempted to perform a limited structural analysis of the concrete roof slab trying to take into account the deterioration we observed. However, we found that there is no reliable way to specifically quantify the reduction in capacity due to the horizontal, surface parallel cracking.

CONCEPTUAL REPAIR ALTERNATIVES

Previous reports have recommended addressing the deterioration by demolishing and replacing the entire roof slab and drop panels, with estimates of cost in the range of $3.8 to $6.7 million. While this is a robust option with a long service life, it is also expensive and would shut down the facility for a significant length of time while repairs are completed. In addition, the original concrete base structure (walls, columns, and floor slab) would still be present.

We believe a less invasive repair approach could be implemented that would extend the service life of the structure an additional 15 to 20+ years. This would allow the water utility to develop replacement options or plan and construct a new reservoir in a different location.

As described in the recommendations section, this approach would involve removal of the fill, removal of the existing freeze/thaw deteriorated concrete at the level of the roof slab and the installation of a new concrete slab on top of the existing slab. The new slab would be properly designed to support the existing slab and prevent surface water infiltration into the existing slab minimizing the water saturation of the existing slab. This would address the severe freeze/thaw deterioration at the top of the existing roof slab and significantly slow further freeze/thaw deterioration of the remainder of the existing roof slab. Re-using the old slab as a stay-in-place form for a new slab is a much greener solution. This solution preserves the embodied energy of the old slab by reducing the cost and energy of the demolition, disposal and forming process.

Depending on the specifics of the new slab design, the reservoir would only need to be taken out of service for a short interval of approximately 1 month versus the ten months estimated by CDM for the full-depth replacement option. Crews could also perform limited underside repairs during the shutdown.

Our opinion of probable repair cost for the new slab on top of the existing slab ranges from $2.5 to $3.5 million depending on the specifics of the final new slab design.
SUMMARY


The evaluation included visual review and hammer sounding of the top of roof slab concrete, non-destructive ground penetrating radar testing to determine the as-built configuration of reinforcing in representative locations and visual review and documentation of the condition of the embedded top layers of reinforcing steel at small test pit openings. The work was performed at two excavations to expose the top of the roof slab concrete approximately 10 feet x 20 feet in size, as shown in SK-1 in Appendix B.

The following conditions were noted; representative photos may be found in Appendix A and sketches of the test excavations, core sample locations and small test pits may be found in Appendix B.

OBSERVATIONS

TEST EXCAVATIONS

1. Two test excavations of the surface asphalt and compacted fill on top of the roof slab were completed with the assistance of Zera Construction to allow review and testing of the roof slab.
   a. Test Excavation 1 (SK-2 in Appendix B) – 10 foot x 21 foot opening centered on Grid F, spanning from Grid 4 to Grid 5 (Photo 1). Test pits TP-1 and TP-2 and core samples C1 and C2 were located within this excavation.
   b. Test Excavation 2 (SK-3 in Appendix B) – 10 foot x 20 foot excavation directly south of Grid M and centered on Grid 6. (Photo 2). Test pits TP-3 and TP-4 and core samples C3 and C4 were located within this excavation.

2. Upon removal of the existing asphalt and fill, the exposed surface of the roof slab concrete was observed to exhibit heavy scaling, delamination and cracking over approximately 85% of the exposed area in Test Excavation 1 and 100% of the exposed area in Test Excavation 2 (Photos 1 and 2).

3. The top layer of deteriorated concrete was easily removed by air blasting with compressed air, leaving a heavily scaled surface (Photo 3).

4. During excavation of the test pits with a chipping hammer, the concrete was observed to be saturated and exhibited degradation of the cement paste, as evidenced by softness of the concrete and separation of the aggregates from the paste (Photo 4). These conditions were observed to be present from the surface to approximately 1½ inches to 2 inches of depth, depending on the test pit location.
5. The surface lot fill over the roof slab was observed to consist of the following from top to bottom:
   a. 3 inches of compacted bituminous asphalt (two 1 ½ inch lifts)
   b. 2 inches of compacted gravel (wet)
   c. 5 inches of unidentified earthen fill (moist)
   d. 2 inches of sand (wet)

**TEST PITS**

2 foot x 4 foot Test pits were excavated in the top of slab concrete to expose the top mat of reinforcing. At each test pit, the condition of the reinforcing steel and amount of concrete cover were noted, as well as the orientation and placement of the bars. All reinforcing steel exposed was observed to be 5/8 inch diameter round bars with “Havemeyer” deformation patterns. Significant observations for each test pit are noted below, detailed sketches of each test pit may be found in Appendix B.

**Test Pit 1 (Sk-4)**

1. 2 straight bars running north-south, 4 truss bars running east-west, and 4 straight bars running east-west were exposed (Photo 5).
2. Section loss was noted at a number of the bars, see SK-4 for detailed notes (Photo 6).
3. Multiple layers of cracking parallel to the surface were noted at the saw cut edges of the test pit. This cracking typically extended to the depth of the top steel layer running east-west (Photo 7).

**Test Pit 2 (Sk-5)**

4. 2 truss bars running east-west, 2 straight bars running east-west, and 5 straight bars running north-south were exposed (Photo 8).
5. Section loss was noted at a number of the bars, see SK-5 for detailed notes (Photo 9).
6. One straight bar in the east-west direction was observed to be fractured upon removal of deteriorated concrete (Photo 9).
7. Multiple layers of cracking parallel to the surface were noted at the saw cut edges of the test pit. This cracking typically extended to the depth of the top steel layer running east-west (Photo 10).

**Test Pit 3 (Sk-6)**

8. 8 bars running north-south and 3 bars running east-west were exposed (Photo 11).
9. 10% section loss was noted at a single east-west bar, see SK-6 for detailed notes. The remainder of the bars exposed were observed to be in good condition.
10. Layers of cracking parallel to the surface were noted at the saw cut edges of the test pit, similar to Test Pits 1 and 2.

**Test Pit 4 (Sk-7)**

11. 2 truss bars running east-west, 3 straight bars running east-west, 1 truss bar running north-south and 4 straight bars running north-south were exposed (Photo 12).

12. All bars exposed were observed to be in good condition, with no signs of corrosion.

13. Layers of cracking parallel to the surface were noted at the saw cut edges of the test pit, similar to Test Pits 1 and 2.

14. All test pits were repaired with a high quality concrete repair mortar at the conclusion of the investigation (Photo 15). After the repair mortar cured, new crushed stone fill was placed and the test excavation areas repaved (Photo 16).

**GROUND PENETRATING RADAR TESTING**

1. At each test excavation, GPR (ground penetrating radar) was used to measure the as-built concrete cover and configuration of the top mat of steel reinforcing (Photos 13 and 14).

2. Based on the testing, the areas tested generally conform to the specifications of the original design drawings. Concrete cover ranged from 1 ¼ inches to 2 ½ inches for top bars running east-west and from 2 inches to 3 ¾ inches for top bars running in the north-south direction.

**CONCRETE CORE SAMPLES**

Four concrete core samples were extracted for visual review to identify cracking within the depth of the slab. At each core, the first ½ inch of the core turned to rubble during the coring process due to heavy freeze/thaw deterioration of the surface concrete. Therefore, in the pictures below, the top of the core is actually about a ½ inch below the actual top of the slab.

1. Core 1 (Photo 17) was extracted from an area of freeze-thaw deterioration near Test Pit 1 (See SK-2 for exact location). The following was noted:
   
   a. The extracted core was approximately 4 inches long and 4 inches in diameter.
   
   b. Due to the deterioration in the area, the core fractured into segments during removal.
   
   c. Cracking parallel to the surface was present at depths of 1 ½ inches and 2 inches from the surface.
2. Core 2 (Photo 18) was extracted from an area of transition between freeze/thaw deterioration and a slab surface in relatively sound condition near Test Pit 2 (See SK-2 for exact location). The following was noted:
   a. The extracted core was approximately 6 inches long and 3 inches in diameter.
   b. Cracking parallel to the surface was present at depths of 1 ½ inches from the surface. A crack through the core is also present at 2 ½ inches to 4 inches from the surface and is likely the result of a fine crack from freeze/thaw damage that completely fractured during removal of the core.

3. Core 3 (Photo 19) was extracted from an area of freeze/thaw deterioration near Test Pit 3 (See SK-3 for exact location). The following was noted:
   a. The extracted core was approximately 4 ½ inches long and 3 inches in diameter.
   b. Cracking parallel to the surface was present at a depth of 2 inches from the surface. A fracture through the core at approximately 3 inches of depth can likely be attributed to a reinforcing bar at this layer.

4. Core 4 (Photo 20) was extracted from an area of freeze/thaw deterioration near Test Pit 4 (See SK-3 for exact location). The following was noted:
   a. The extracted core was approximately 5 inches long and 3 inches in diameter.
   b. Fine cracking parallel to the surface was present at a depth of 1 inch and 3 inches from the surface.
LIMITATIONS

This report contains the professional opinions of Walker Restoration Consultants based on the conditions observed as of the date of our site visit and documents available to us. This report is believed to be accurate within the limitations of the stated methods for obtaining information.

We have provided our opinion of probable costs from visual observations, limited testing, and field survey work. The opinion of probable repair costs is based on available information at the time of our evaluation and from our experience with similar projects. There is no warranty to the accuracy of such cost opinions as compared to bids or actual costs. This condition assessment and the recommendations therein are to be used with additional fiscal and technical judgment.

It should be noted that our renovation recommendations are conceptual in nature and do not represent changes to the original design intent of the structure. As a result, this report does not provide specific repair details or methods, construction contract documents, material specifications, or details to develop the construction cost from a contractor.

Based on the proposed scope of services, the evaluation was based on certain assumptions made on the existing conditions. Some of these assumptions cannot be verified without expanding the scope of services or performing more invasive procedures on the structure.

The recommended repair concepts outlined represents current available technology for similar concrete structures. This report does not provide any kind of guarantee or warranty on our findings and recommendations. Our evaluation was based on and limited to the proposed scope of work. We do not intend to suggest or imply that our appraisal has discovered or disclosed all latent conditions or has considered all possible improvement or repair concepts.

A review of the facility for compliance with the Americans with Disabilities Act (ADA) requirements was not part of the scope of this project. However, it should be noted that whenever significant repair, rehabilitation or restoration is undertaken in an existing structure, ADA design requirements may become applicable if there are currently unmet ADA requirements.

Similarly, we have not reviewed or evaluated the presence of, or the subsequent mitigation of, hazardous materials including, but not limited to, asbestos and PCB.

This report was created for the use of Northwestern University and use of this report by others is at their own risk.
APPENDIX A
PHOTOGRAPHS
Photo 1 – Overview (looking east of Test Excavation 1 centered on Grid F between Grids 4 and 5.

Photo 2 – Overview (looking east) of Test Excavation 2 to the south of Grid M centered on Grid 6.
Photo 3 – Typical heavily scaled concrete roof slab surface.

Photo 4 – Saturated top of slab concrete and degradation of cement paste.
Photo 5 – Overall view of Test Pit 1 looking east.

Photo 6 – Typical corrosion and section loss at truss bar in Test Pit 1.
Photo 7 – Typical cracking and freeze/thaw deterioration of roof slab concrete to depth of top layer of reinforcing steel in Test Pit 1.

Photo 8 – Overview of Test Pit 2 looking north.
Photo 9 – Typical corrosion and section loss of reinforcing steel in Test Pit 2. Also note bar discovered to be fractured upon demolition of concrete.

Photo 10 – Typical cracking and freeze/thaw deterioration of roof slab concrete to depth of top layer of reinforcing steel in Test Pit 2.
Photo 11 – Overall view of Test Pit 3 looking north.

Photo 12 – Overall view of Test Pit 4 looking east.
Photo 13 – Markings from ground penetrating radar (GPR) survey of roof slab reinforcing steel.

Photo 14 – Typical GPR scan showing truss bar and perpendicular mat of steel underneath truss bar.
Photo 15 – Contractor placing concrete repair mortar at test pit openings.

Photo 16 – Contractor placing new crushed stone fill at test excavations.
Photo 17 – Core C1.

C1 - 3'-3" NORTH OF GRID F, 1'-6" WEST OF GRID 5, TOP OF ROOF SLAB IN AREA OF FREEZE-THAW DETERIORATION.
FINISHED WATER RESERVOIR ROOF SLAB
APPENDIX A - PHOTOGRAPHS

WRC PROJECT NO. 31-7397.40

MARCH 2014

Photo 18 – Core C2.
C3 - 4'-0" SOUTH OF GRID M, 3'-0" EAST OF GRID G, TOP OF ROOF SLAB IN AREA OF FREEZE THAN DETERIORATION

Photo 19 – Core C3.
Photo 20 – Core C4.
APPENDIX B
FIELD SKETCHES
SCOPE OF SERVICES

Walker performed the following approved Scope of Services:

PHASE I – WATER RESERVOIR SLAB STRUCTURAL EVALUATION

1. Perform a limited peer review of the CTL and CDM Reports, as well as a review of the original structural drawings for the water reservoir roof slab.

2. Work with an experienced restoration contractor, remove asphalt and gravel from the top surface at two test pit locations to visually review the condition of the top of the slab concrete. We propose that the two test pit locations for asphalt removal be approximately 10 feet x 20 feet. We will remove two concrete cores at each test pit for visual review of slab cracking.

3. We will also use non-destructive testing consisting of GPR to locate the depth and spacing of reinforcing bars within the test pit locations. We will have the restoration contractor locally chip the concrete top of the slab in smaller areas to observe the condition, size and spacing of embedded top of slab reinforcing bars. Chipped openings will be patched back with appropriate repair concrete. Compacted stone will be replaced and new asphalt paving will be installed at the test pits. Test pit information will be documented with notes and photographs.

4. Perform limited structural calculations on the original structural capacity of the slab based on the member sizes and reinforcing steel shown on the original structural drawings.

5. Using the data gathered in the field, including reinforcing steel spacing and condition, perform additional limited structural calculations attempting to take into account observed deterioration. This analysis will be performed with significant engineering judgment as to the validity of the results.

6. Formulate conclusions and conceptual repair recommendations both for the short-term, as well as for the long-term. Our recommendations will include whether or not cars should be allowed to park on all areas of the structure in the short-term or if certain portions of the parking area should be closed to parking or strengthened if practical. Also, our long-term conceptual repair recommendations will try to determine potential options besides full slab replacement that may minimize the impact of repairs/replacement.

7. Prepare a report summarizing our findings, conclusions and conceptual repair recommendations. Our report will also include our opinion of cost for the recommended repairs.

8. Attend one meeting with Northwestern to discuss the report.