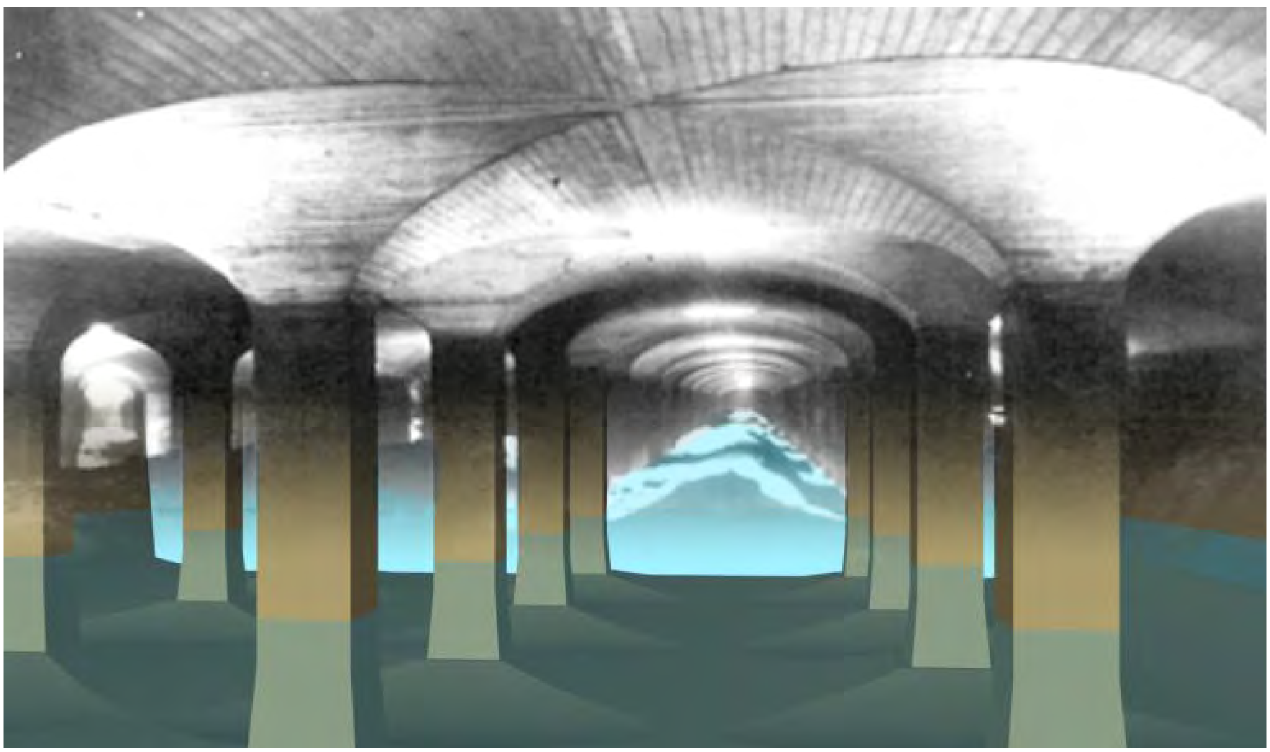
EXISTING CONDITIONS ASSESSMENT & FEASIBILITY EVALUATION

McMillan Slow Sand Filtration Plant Site
Washington, DC



April 10, 2014

prepared for
Vision McMillan Partners
EYA | Jair Lynch Development Partners | Trammell Crow Company

prepared by
Robert Silman Associates
1053 31st Street NW
Washington, DC 20007

RSA PROJECT #W2771

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EXECUTIVE SUMMARY

Robert Silman Associates (RSA) was retained to perform a structural investigation and feasibility analysis for potential restoration and/or adaptive reuse of the slow sand water filtration structurers at McMillan Reservoir. The structures were built in 1912 and consist of 20 underground filters, each consisting of multiple vaults of unreinforced concrete. The vaults are typically 14' x 14' and supported on a single column over a shallow inverted vault slab, cast on grade.

The structural investigation included a review of documents provided by the client and a structural condition assessment of representative areas of the installation. Particular attention was paid to areas documented in the most recent study, in 2000, in order to assess the general rate of deterioration. In general, the damage observed by RSA was similar to the conditions documents in 1944, 1967, and 2000; however, it was apparent that severe damage continues to propagate, and other areas at these filters are at risk of imminent collapse.

RSA performed a structural analysis and stability investigation of the slow sand filter assemblies to further explore the concerns outlined in prior investigations to develop an understanding of this historic structural system, identify the causes of the observed damage, and to determine its current and future ability to support both current and potential future loadings.

As part of this study, RSA provided a review of the "Structural/Geotechnical Engineering Evaluation of the McMillan Filter Site", (CCJM 2000). The report includes a summary of a finite element analysis concluding that the structure has a capacity for 640 psf live load. However, it was also reported that "lean concrete ceilings are unsafe for public access because of their nearly zero elasticity". RSA commented that this report does not fully address the safety concerns for occupancy within the filters, does not address the concern about ongoing settlement or spread of the existing structure, and fails to note the important role of the perimeter wall in providing thrust resistance for the unreinforced concrete vaults within the filter system. Finally, the existing foundation capacity for new load was not evaluated.

To determine the ability of the slow sand filter structure to carry existing loads, and additional loads for future development, RSA used graphic analysis. Stability investigations were performed utilizing assumed failure mechanisms based on the results of the graphic analysis and field observations. Under the idealized case of uniform distributed loading, a theoretical live load capacity of approximately 2,800 psf can be calculated; however, the footing configuration of the slow sand filter structural assembly is vulnerable to failure at low levels of superimposed loading. Calculations of foundation capacity indicated that loads cannot exceed 150 psf under the most favorable soil conditions. When considering foundation settlement, the capacity reduces to 100 psf under settlement of ¾ inch and 1-1/2 inches respectively (vertical or horizon), and becomes unstable (causing collapse) at any displacement beyond that value.

The structural assembly of the slow sand filters, how their different parts come together, and how these parts are affected by different conditions such as unbalanced loads and settlement could create conditions leading to progressive collapse or global instabilities. The initial trigger for the collapse appears to be the lateral and/or vertical movement of the end wall. The fact that no interior bays have collapsed is attributed to the amount of restraint offered by the surrounding bays and friction on the supporting columns. At entrance portals, the disruption of the structural symmetry affects the internal forces on either side adjacent to the entrance and, similar to the end bays, if enough hinges formed, this will lead to instability or collapse.

The slow sand filters are industrial structures, never intended to provide the level of safety and serviceability required for occupied use. There are unsafe conditions at the site that indicate settlement of a brittle heavy structure susceptible to sudden collapse. The concrete ceilings in the unreinforced concrete structure harbor loose material that can fall without warning. At the current pace, the entire end bay of filter No. 24 are expected to collapse within five years and it is expected that this would trigger collapse mechanisms at adjacent interior bays and other filters. Thus, RSA recommends that no activity take place on the property in its current condition.

Various potential options for the future of the site have been developed by the design team and are presented in the report: (1) do nothing; (2) repair/replace top surface for light use only; (3) repair/replace top surface for light use and interior tours; (4) repair/replace for programmed top surface and interior tours; (5) repair/replace for programmed top surface and interior programmed use; (6) repair/replace for overbuild and interior programmed use; and (7)new development at the site.

All adaptive re-use options and work at this site must be performed with great care and the knowledge of the limitations regarding loadings, sensitivity to settlement (vertical and lateral) from adjacent excavation or demolition. Where excavation or vibration generating construction activity is proposed, special measures, (including underpinning, rigid temporary retention systems, or temporary retrofit), will be necessary to protect the filters from further damage or collapse. Calculations have indicated that the existing foundations have likely failed but are not accessible for visual inspections. This consideration will become the limiting factor for even the minimal re-use. The foundations will need to be exposed and soils tested to determine the allowable bearing capacity of the sub-grade, and to establish repairs that need to be made, appropriate for the proposed restoration or re-use.

INTRODUCTION

This report represents the formal issuance of a previous report that was developed as a working draft and used throughout an investigation and masterplanning phase that has been ongoing since 2010. No investigations have been performed since 2010 and the observations and findings are now outdated. It should be noted that, since the time of the investigation, Washington, DC suffered a significant seismic event. Although earthquakes are not typically a concern for 'underground structures', the McMillan Reservoir structure includes significant portions that are above the surrounding grade elevations and are therefore vulnerable to earthquake damage. Also, a portion of the site is currently under the control of DC Water - who is performing demolition of cells 25 and 26, as well as stabilizing cell 14. RSA has no information or involvement in these efforts.

Robert Silman Associates (RSA) was retained by Vision McMillan Partners (client) to perform a structural investigation and feasibility analysis for potential restoration and/or adaptive reuse of the slow sand water filtration installation at McMillan Reservoir, east of First Street, in NW Washington DC. While the installation includes a variety of strucure types, above- and below-grade, this study is limited to the below-grade "filters", which are constructed of unreinforced concrete vaults.

The structural investigation included a review of documents provided by the client, additional documents obtained through research, visual assessments of existing conditions, and structural analyses to estimate the capacity of the unreinforced concrete vaulted structure. This analysis was used to establish correlations to observed conditions and establish the structural feasibility of adaptive reuse scenarios.

RSA reviewed the following documents:

- Topographic contours lines from 1894 and 1901.
- Original plans by the Army Corps of Engineers, dated December 13th, 1902.
- 1944 and 1967 condition assessment maps by the Army Corps of Engineers with added notes by C.C. Johnson and Malhorta [CCJM, 2000].
- Architectural & Archaeological Survey of the Eastern Portion, McMillan Water Treatment Plant, by Engineering-Science, Inc, June 1990.
- Historic Preservation Report prepared by EHT Traceries, Inc, July 2010.
- Structural/Geotechnical Engineering Evaluation of the McMillan Filter Site by C.C. Johnson and Malhotra, [CCJM, 2000].

RSA performed a structural condition assessment of representative areas of the installation, primarily between February 23 and March 7, 2012. Visual assessment consisted of walkthroughs with flashlights and included only accessible areas of the structure away from collapsed zones. Structural conditions were documented using field notations, digital photography, and measurement of representative geometry to compare to existing drawings and previous studies. Particular attention was paid to areas documented in the most recent study, by CCJM in 2000, in order to assess the general rate of deterioration. RSA performed a more detailed visual survey was performed at Filters 14 and 20 of the structure, as described herein, to document conditions and evaluate the suitability of these areas for reuse.

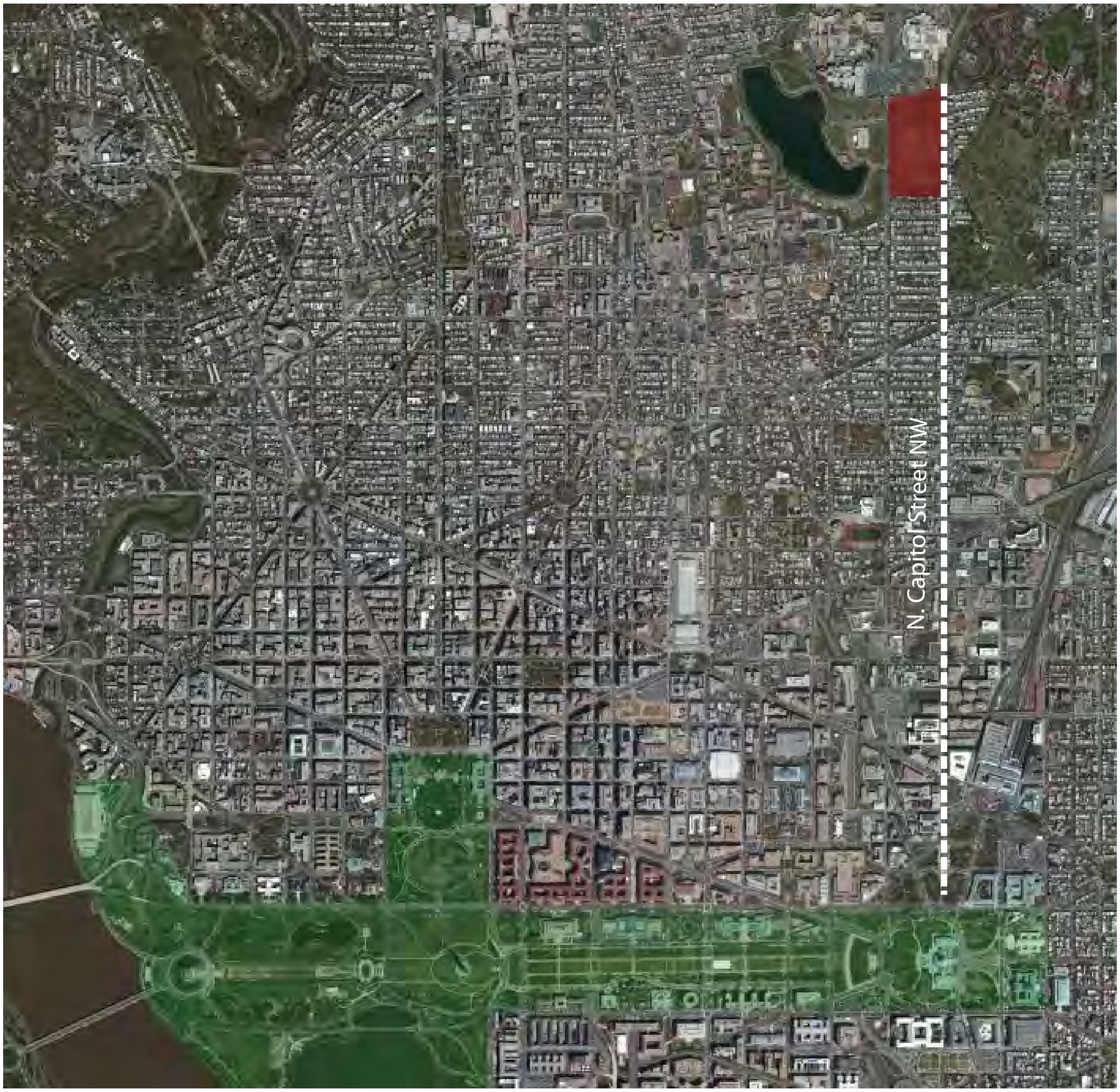


FIGURE 1: SITE LOCATION

IMAGE © BING.COM

SITE DESCRIPTION

The site (shown in Figure 1 & Figure 2) contains twenty-nine buried "filters" - concrete structures with four walls, concrete 'floors' covered in several feet of sand, exposed 'ceilings' and a roof covered in soil. Twenty filters were constructed on the east side of 1st Street N.W., on the property currently owned by the DC government and nine filters were constructed on the west side of First Street on property currently owned and operated by the Army Corps of Engineers. The latter include several filters that have been modified or demolished and are currently part of the DC Water facility.

FIGURE 2: SITE LOCATION

IMAGE © GOOGLE.COM

The east portion of the site (see Figure 2) consists of four rows of five abutting filters, with the northern and southern rows separated from the middle two by two service courts containing multiple above-grade structures that supported the sand washing and pumping operations.

FIGURE 3: LOCATION OF SLOW SAND FILTERS INVESTIGATED

Filter construction began in 1903 and the facility was in service by 1905 (Figure 4). The filters remained in service until the 1980s. Each filter is bounded by walls on four sides and accessible by a single ramped entrance portal. The plan area of a filter varies, particularly at those near Michigan Avenue. The approximated average area of the typical filter is 45,000-square-feet (150 ft. x 300 ft.). In the east-west direction, filters are as wide as 196 ft. (filter No. 10) and as a narrow as 133 ft. (filter No.14). The filters in the north-south direction have lengths ranging from 252 ft. (filter No. 10) to 336 ft. (filter No. 14).

FIGURE 4: PHOTO SHOWING TOP SIDE OF FORMWORK OF TYPICAL BAY IN FOREGROUND AND ENTRANCE RAMP BEYOND
(1902-1904), COURTESY OF THE ARCHIVES OF THE WASHINGTON AQUEDUCT

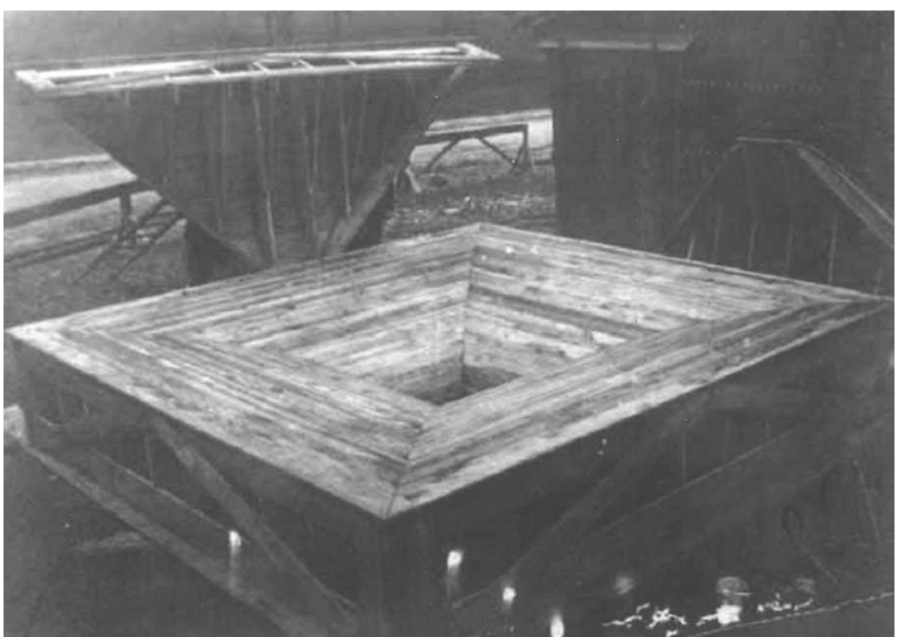


FIGURE 5: PHOTO SHOWING FORMWORK

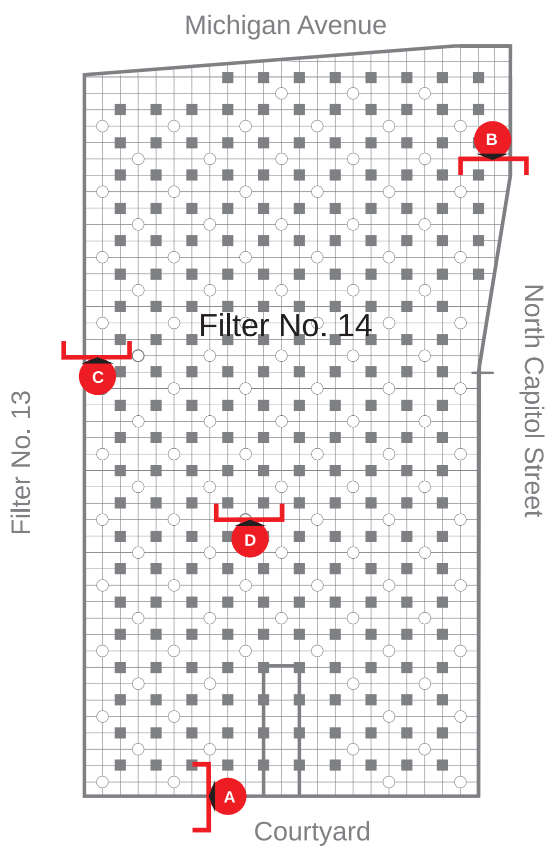
Two topographic surveys were performed prior to construction. The first survey, performed in 1894, shows a stream that cuts through the southeast corner of the area investigated in this report. A second survey, performed in 1901, shows grade elevations that are as much as 20 feet higher than those previously taken near the stream reported in 1894. The difference between the two topographic surveys is attributed to grade modifications after 1894 in preparation for construction work that began in 1903. Figure 5 shows the 1894 topographic survey and superimposed on the satellite view are the areas where the original grade was below the elevation of the underside of the footings, as well as the location of the stream. Filters located within the highlighted area depicted in Figure 4 were constructed on areas of built up fill materials according to the 1894 topographic survey.

FIGURE 5: TOPOGRAPHIC MAPS OF SITE PLAN (1894) AND SATELLITE VIEW SHOWING AREAS OF FILL AND THE ORIGINAL STREAM

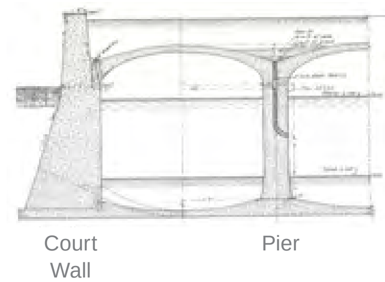
STRUCTURAL DESCRIPTION

The filters are constructed with vaulted unreinforced concrete. At the time of design and construction the use of reinforced concrete in the United States was in its infancy for building structures. The lead design engineers most likely chose unreinforced concrete for technical reasons. Steel is highly susceptible to deterioration over time, particularly when subjected to increased levels of moisture and air. Thus, for the slow sand filters, where elevated levels of moisture are constant, unreinforced concrete's combination of strength and durability was likely the main reason for the choice of building material.

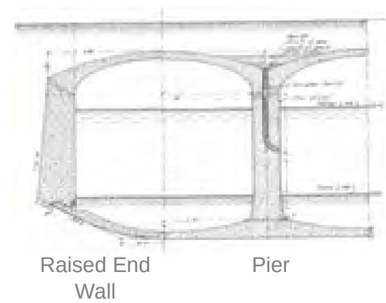
Figure 6 depicts a plan of filter No. 14 (at the NorthEast corner of the site) and typical cross sections through the filter structures. The unreinforced concrete vaults are supported on a grid of piers and perimeter walls. The pier supports are typically spaced at 14 feet in each direction and are 22"x22" in cross section. Typically, the interior height, from underside of vault apex to top of foundation slab, is approximately 12 feet. The filter floor is covered with more than two feet of sand. Each pier is founded on a shallow, inverted vault slab intended to control the flow of water and distribute loads uniformly over the soils below. The foundations were designed with two distinct configurations along the end walls, depending on whether the filter was bearing on virgin soil or backfill materials. Raised foundations, as depicted in Figure 6B, were used at end gravity walls built on deep cuts into virgin soils. On the other hand, level foundations were constructed at walls built on shallow cuts or areas of built up fill materials, as depicted in Figure 6A.



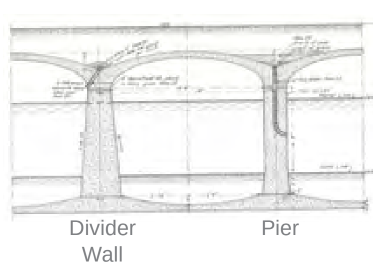
PLAN: FILTER No. 14



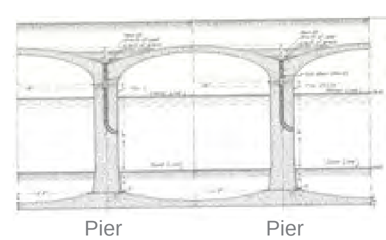
6A – Typical Section at Court Yard Wall



6B – Section at Typical Raised End Wall



6C – Typical Section at Divided Wall Between Filters



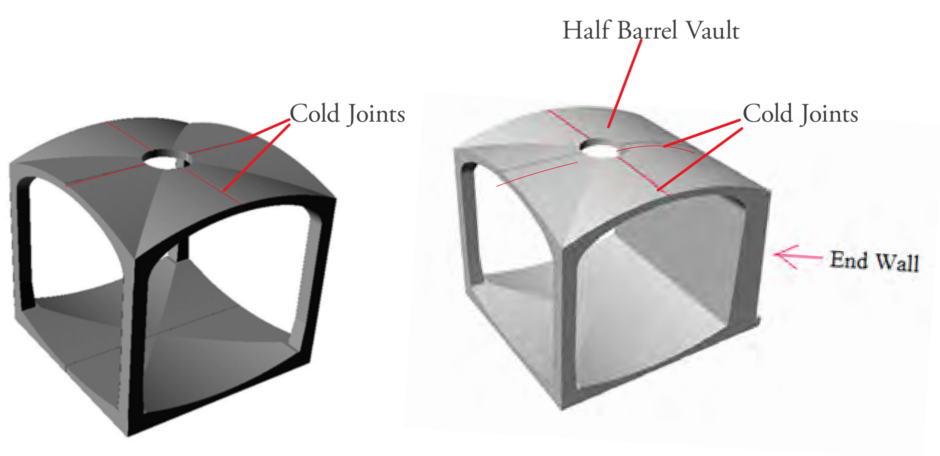
6D – Typical Section Inside Filters

FIGURE 6: CROSS SECTIONS OF SLOW SAND FILTER No. 14 WITH TYPICAL SECTIONS

The concrete vaults are constructed of two primary structural forms. The slab is constructed in segments that, when set against adjacent segments, form a series of groin vaults, as depicted in Figure 7A. The second primary structural form is a half barrel vault, depicted in Figure 7B. The half barrel vaults were constructed around the perimeter of each filter, transmitting roof loads both vertically and laterally along these perimeter walls.

Materially, concrete is strong in compression but very weak in tension. In more modern construction, steel reinforcement is generally used within the concrete matrix to carry tensile stresses that may develop from bending. The addition of steel reinforcement also creates conditions where failures occur in a controlled manner, instead of sudden failure or collapse. This allows for increased strength and versatility because of added capacity. Due to concrete's weakness in tension, the vaulted structure was a natural design choice. Under a known loading, a vaulted form can be designed to carry forces exclusively in compression. When loadings deviate from this assumption, as is inevitably the case with live loads, which are inherently variable, or transient loads, the structure will undergo bending or 'flexure' subjecting the concrete to tensile stresses. While it is not known what the top surface was designed for, it is clear that these live or transient loadings were significantly less than the self weight of the structure and supported soil.

The symmetry of the vault geometry and the structure assembly indicates that the original design of the slow sand filter relies in balancing forces between adjacent vaults that are abutting at the apex. Accordingly, under balanced load conditions the vault is in equilibrium and the design intent of keeping the unreinforced concrete from undergoing significant bending and tensile stresses is accomplished. As discussed further below, structural distress is encountered when these fundamental parameters are significantly altered, such as when the vaulted structure is subjected to excessive unbalanced loads or support movements from foundation settlement or end wall rotation.



7A – Typical Interior Bay,
Groin Vault

7B – Typical End Bay,
Half Barrel Vault

FIGURE 7: PRIMARY STRUCTURAL FORMS

The vault slabs vary in thickness, with a minimum thickness of six-inches at the apex. The top and bottom surfaces of the vault take on different trajectories along their spans. The top of the vault slab transitions from a thickness of one-foot to five-inches from the center line of the supports to the vault apex, following a parabolic shape. The geometry of the vault slab near the end walls follows a similar configuration.

Manhole openings are located at the apex of every other vault bay in each orthogonal direction. The vault slab was designed to support approximately two-feet of soil at the apex and more than three-feet of soil over the piers. Each monolithic vault slab concrete section, as depicted in Figure 8, narrows to the pier plan dimensions and is supported vertically through bearing on the pier. Cold joints at the pier to vault and pier to foundation interfaces indicate that these components were poured separately. Restraint against lateral displacement at the vault apex is provided by abutment to the adjacent vaults.

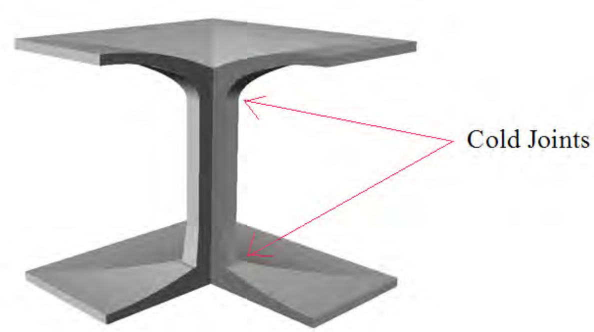


FIGURE 8: TYPICAL 14’x14’ PANEL SECTION AROUND PIERS, MONOLITHIC PANELS

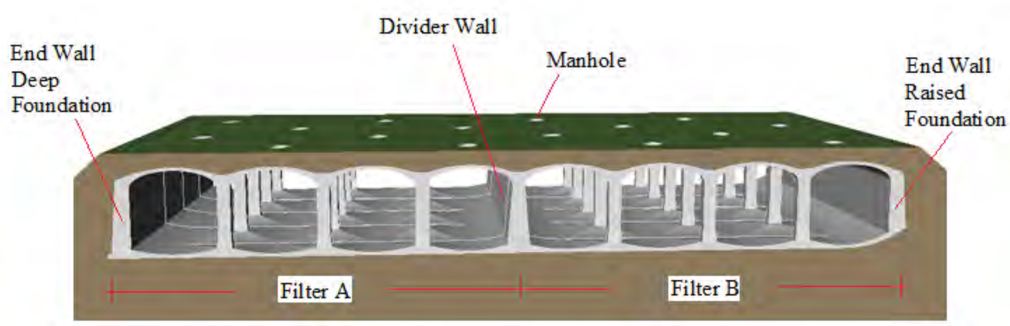


FIGURE 9: SCHEMATIC CROSS SECTION OF FILTER STRUCTURAL FORMS

STRUCTURAL CONDITION ASSESSMENT

A structural condition assessment was performed by RSA at the interior of all filters on the east portion of the site, to investigate the existing conditions and identify changes in conditions relative to previous condition assessment by others. For the majority of filters, RSA made cursory observations to develop general confirmation of conditions and their relative change with respect to the previous condition assessments in 1944, 1967 and 2000. More detailed observations within these filters focused on areas with representative damage or significant change since the latest survey (2000) to understand the ongoing mechanisms causing damage to the structure. Two filters, Nos. 14 and 20, were studied in more detail, and quantitative assessments were made noting damage conditions. RSA utilized the Army Corp of Engineers condition assessment maps, with the additional notes added by the CCJM report, as a reference [CCJM, 2000]. It appears that typed markups on these condition assessment maps are from the 2000 CCJM study and handwritten markups are from the two condition assessments performed by the Army Corp of Engineers, one in 1944 for some of the filters (Nos. 1, 6, 10, 18, 22, and 25), and a second condition assessment in 1967 performed at all slow sand filters located east of first street. New damage observed and identified by RSA in addition to the previous damage noted in 1944, 1967 and 2000, has been added to filters Nos. 14 and 20 maps and is summarized in the figures within Appendix A.

RSA also visited the facilities west of First Street, which are still owned and operated by the Army Corps of Engineers, to provide a general review of the condition of the slow sand filters at this location. A summary of our observations is included in Appendix B.

ASSESSMENT LIMITATIONS

- Safety concerns prohibited detailed assessment at several areas, particularly areas near bays that had collapsed.
- The top side of the vault slab and the filter foundation were not accessible for investigation at the time this report was written. Test pits would be required to expose areas of the buried structure for a field condition assessment.
- Soil conditions were not investigated at the time this report was written, however previous soil studies are included in a study by C.C. Johnson and Malhorta [CCJM, 2000], subsequent studies were performed by ECS, indicating similar soils conditions. [ECS, 2013]

GENERAL OBSERVATIONS

The following is a summary of the types of conditions observed during RSA's 2012 investigation. A more detailed description, with photos of each condition observed, is enclosed in Appendix C.

New crack formation and existing crack propagation at ceiling vaults and walls. These cracks typically range from hairline cracks to 3/8 in. wide cracks at filters with significant damage and from hairline to 3/16 in. wide cracks at many locations of filters with moderate damage. Damage categories for the vaults are discussed herein.

Vault slab cracks around the top of the piers. This vertical cracking emanating from the vault bearings on the piers may be caused by one or a combination of conditions including: differential settlement, and end wall movement. This condition is most common at piers near the end walls.

Cracks at perimeter and divider wall springing line. These cracks are indicative of support movement and were observed at almost all filters investigated by RSA.

Concrete spalls. This condition was observed by RSA typically near the manholes. These concrete spalls are consistent with anticipated stress concentration at the manhole openings in the vaulting.

Rotation at the intersection between the top of pier and the ceiling vault slab. This condition was typically observed near the entrance portals where forces from the barrel vaulted center bay most likely induced bending on the pier. This condition was also observed at piers located at the center of the filters away from the end walls and portal. Here, the cause may more likely be differential settlement at the foundation level.

Vertical displacements were observed at the vault joints located at the high point of the interior space where the 14'x14' monolithic panels (Figure 8) abut at the highest elevation of the vault; differential displacements in the vertical direction as great as three-inches were observed.

Horizontal separations as great as one-inch were observed at the vault joints between adjacent 14'x14' monolithic panels (Figure 8) and at end bays near the collapsed areas.

Entrance portal, ramp and adjacent bays. Significant displacements at the vault joints were typically observed at bays near the portal. Cracking and vertical displacement was observed at the underside of the portal ceiling. Rotation at piers supporting the portal barrel vault was also observed. These conditions are most likely caused by disruption of the structural symmetry of the filter at the entrance portal which affects the balance of internal forces. There is a regular pattern of damage at the entrance portal jamb walls caused by a combination of corrosion of the gate metal anchors, and the stress caused by structural rotation of the end wall and the restraint offered by the portal ceiling slab.

Areas with collapsed vault slabs and areas where collapse appears imminent. Collapsed areas were observed by RSA along the eastern bays of filters Nos. 19 and 24 adjacent to North Capitol Street. As additional bays have collapsed since the most recent survey in 2000, it was apparent that severe damage continues to propagate to other areas at these filters, particularly near the collapsed areas where collapse appears imminent. See Figure 10 for graphic depiction of collapse mechanism, and section __ for structural stability results and discussion.

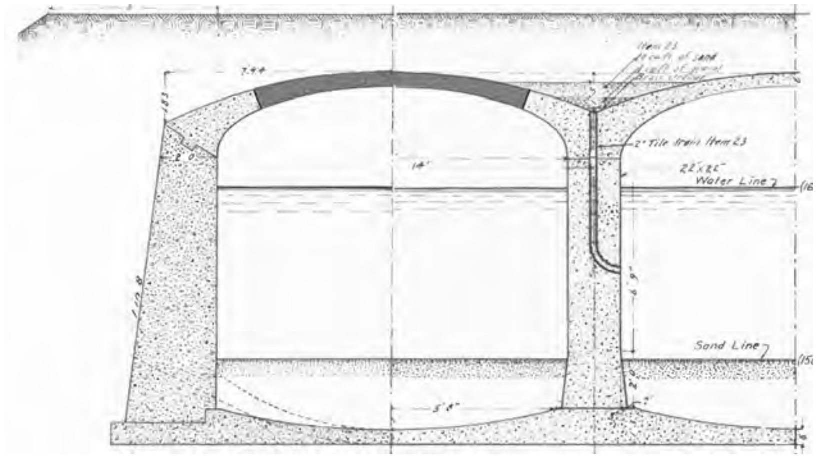


FIGURE 10A : COLLAPSE MECHANISM: INITIAL CRACKING

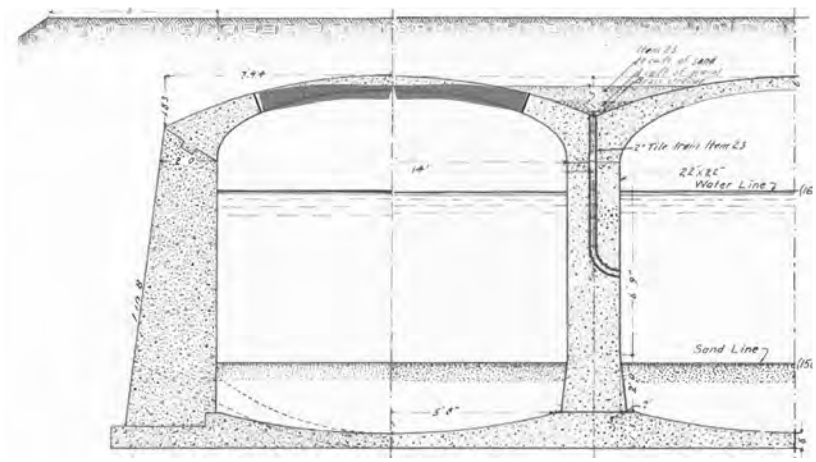


FIGURE 10B :COLLAPSE MECHANISM: INITIAL MOVEMENTS

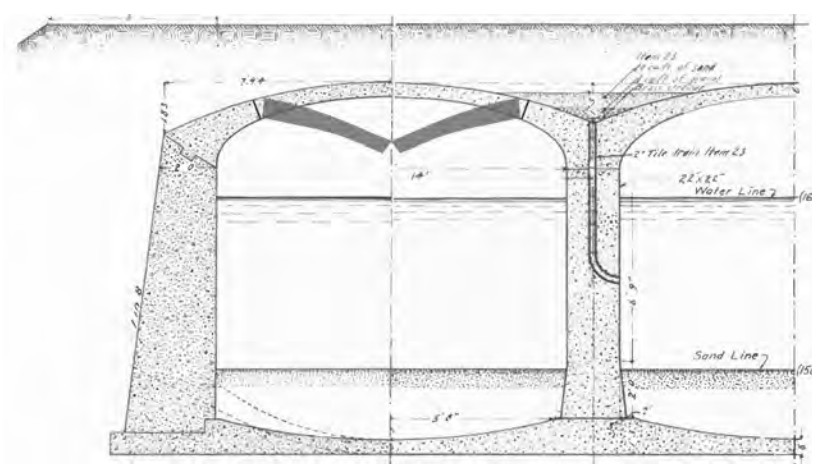


FIGURE 10C : COLLAPSE MECHANISM: COLLAPSE

PREVIOUS RETROFITS

Displacement Arrestors

Based upon available documentation, "displacement arrestors" made of thick metal blocks and thin square metal plates were installed sometime before 1967 to arrest vertical movements due to settlement or the formation of collapse mechanisms across cracks and some joints. The intent of the retrofit repair was to clamp the vault together along crack lines and along the vault joints between adjacent concrete panels to limit the potential for crack propagation and differential movement. It appears that this approach can be effective in the short term, as it does address the important need to maintain a continuous thrust line across the vaulted span. However, the approach did not address the root causes of the vault distress, which are likely related to support movements.

To arrest horizontal and vertical shifting, two types of "displacement arrestors," consisting of steel plates bolted to the ceiling slab at either side of an inter-panel joint, were also added sometime before 1967. The apparent intent of these connectors was to provide tensile resistance across the joint to limit horizontal or vertical separation. The effectiveness of this approach again appears to have been short-lived, as cracks have developed near the joints and these connectors eliminating their effectiveness. Due to the moist environment, the iron components were observed to have section loss often accompanied by localized damage to surrounding materials.

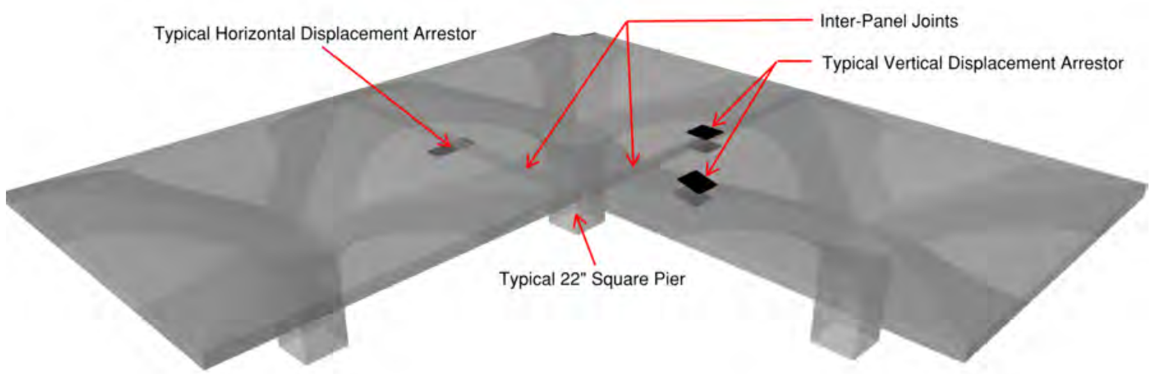


FIGURE 11: HORIZONTAL AND VERTICAL DISPLACEMENT CRACK ARRESTORS

Banding Detail

A system of metal straps, tie rods and turnbuckles were observed in a few cells at or near the entrance portals. Review of documents indicated that these components near the interface between the pier and the ceiling were part of a banding system added sometime after 1951, and were intended to provide additional thrust resistance for the groin vaults abutting the center bay. As depicted in Figure 12, the barrel vault of the center bay springs from a point higher than the adjacent groin vaults. The added tie rods appear to be intended to resolve the resulting imbalanced thrust forces.

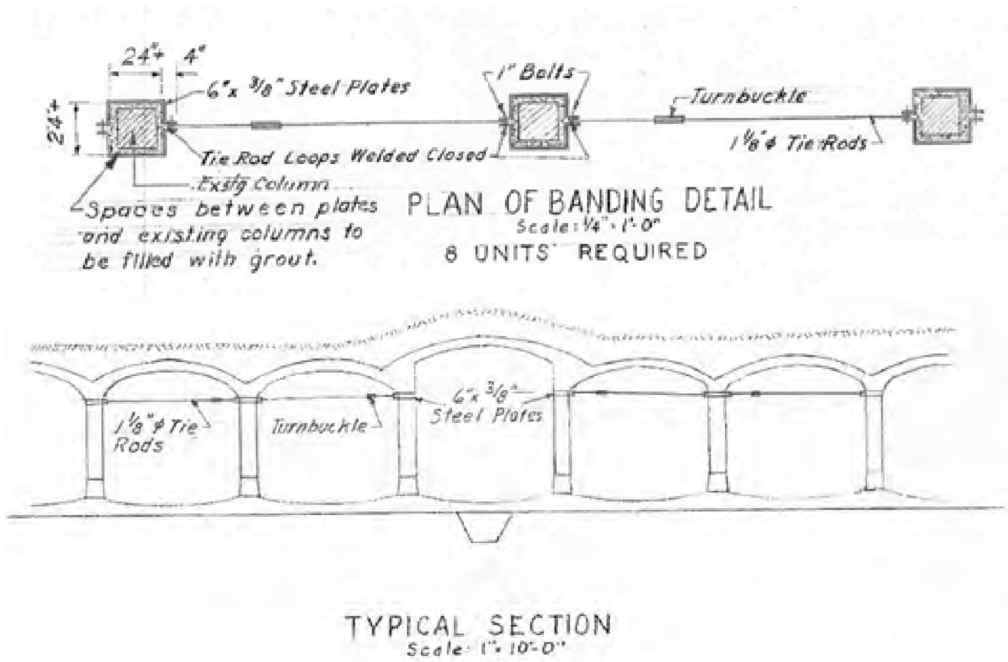


FIGURE 12: BANDING DETAIL

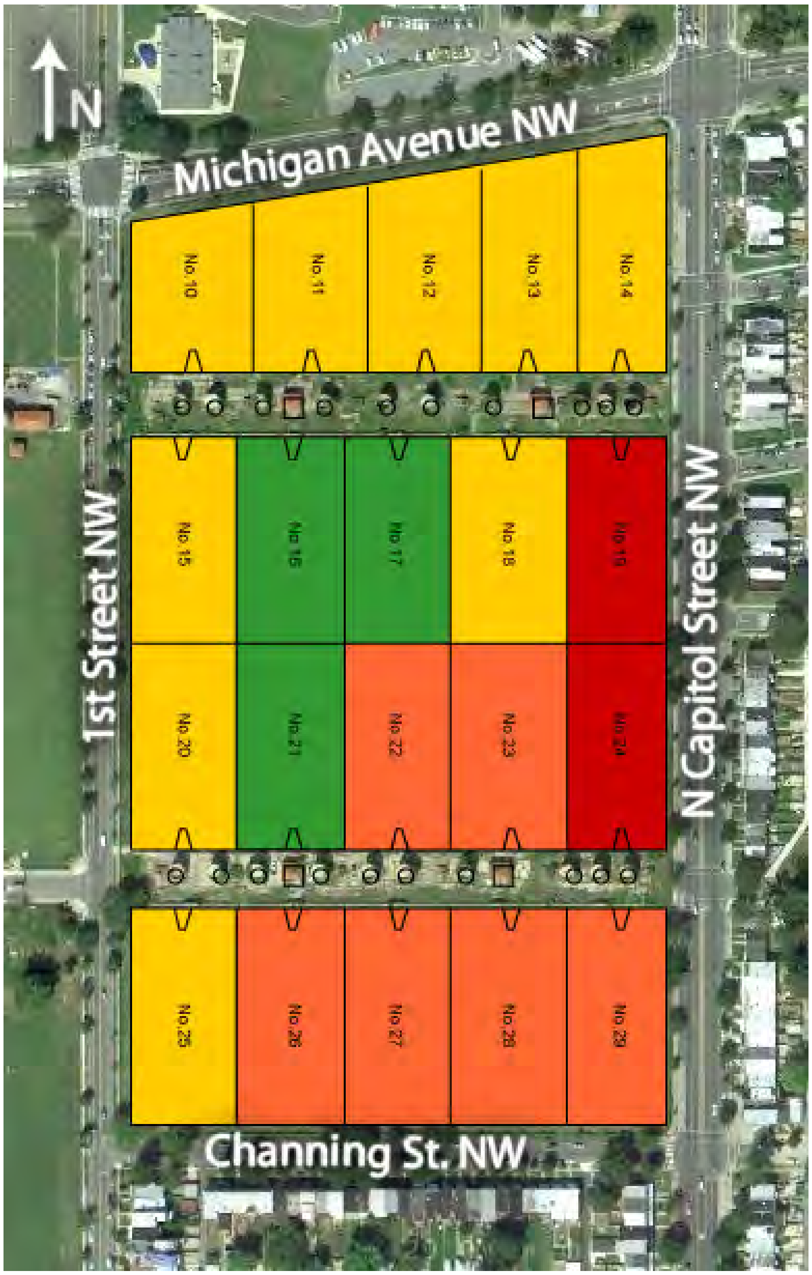
The banding system approach can be effective at balancing these forces, however it was noted at the site that piers at the entrance portal with these straps had typically cracked as depicted in Figure 13. At other filters where these restraints were not present, a rotation at the entrance ramp pier-to-vault joint was observed without any pier cracks (see Figure C-5, in the appendix). Thus, although the banding may have limited some lateral movement or rotation, the unreinforced piers did not have sufficient capacity to transfer forces at this added connection.

FIGURE 13: PORTAL ENTRANCE PIER AT SLOW SAND FILTER No. 26

DAMAGE CATEGORIES

Based on level of damage observed during the 2012 field condition assessment the slow sand filters investigated were divided into four categories:

Figure 14, below, depicts the categories of the slow sand filters investigated.



Collapsed Vaults

Severe Damage

Moderate Damage

Minor Damage

FIGURE 14: SLOW SAND FILTERS DAMAGE CATEGORIES

Collapsed Vaults

These collapsed areas are concentrated along the eastern bays adjacent to North Capitol Street at both filters. It was not safe to perform a detailed investigation at the filters in the immediate vicinity of the collapsed vaults. Adequate bracing or shoring of these severely damaged areas is not present. However, based on RSA's observations from a safe setback distance, it was apparent that severe damage continues to propagate, and other areas at these filters are at risk of imminent collapse. Collapsed areas were found in filters Nos. 19 and 24.

The collapsed area found at filter No. 24 is more extensive and has continued to progress since the 1944, 1967 and 2000 condition investigations (see Figure 16). The collapsed area found at filter No. 19 is much smaller and newer but the conditions leading to collapse also date back to the 1967 survey. More specific information about the collapsed vaults is provided in more detail in Appendix D

Severe Damage

This level of damage includes filters with items described as minor and moderate damage, but with more significant conditions such as areas where collapse appears imminent. Filters within this category are filters Nos. 22, 23, 26, 27, 28 and 29. A summary of observations of the more representative filters in the severe damage category is included in Appendix D.

Moderate Damage

In general, the damage observed by RSA was similar to the conditions documents in 1944, 1967, and 2000. Further damage, consistent with earlier observations, included additional cracks, crack lengthening, continued shifting of the vault joints, and additional concrete spalling. Filters within this category include those with cracks having widths measuring $^{1}/_{16}$ in. up to $^{3}/_{16}$ in., and typically includes moderate shifting at vault joints, concrete spalls, and rotations at the pier-to-ceiling slab interface. Filters assessed within this category are Nos. 10, 11, 12, 13, 14, 15, 18, 20, and 25. A summary of observations from the more representative filters within the moderate damage category is included in the Appendix D.

FIGURE 15: CEILING VAULT DAMAGE NEAR COLLAPSED BAY

Minor Damage

In general, damage at these filters is mainly limited to hairline cracking as previously reported in 1944, 1967 and 2000, and confirmed by RSA in 2012. Minor damage, noted on previous surveys, appears to have increased only marginally. This category includes filters Nos. 16, 17, and 21, which are all interior filters founded on firm substrate soils. A summary of observations of the more representative filters in the minor damage category is included in Appendix D.

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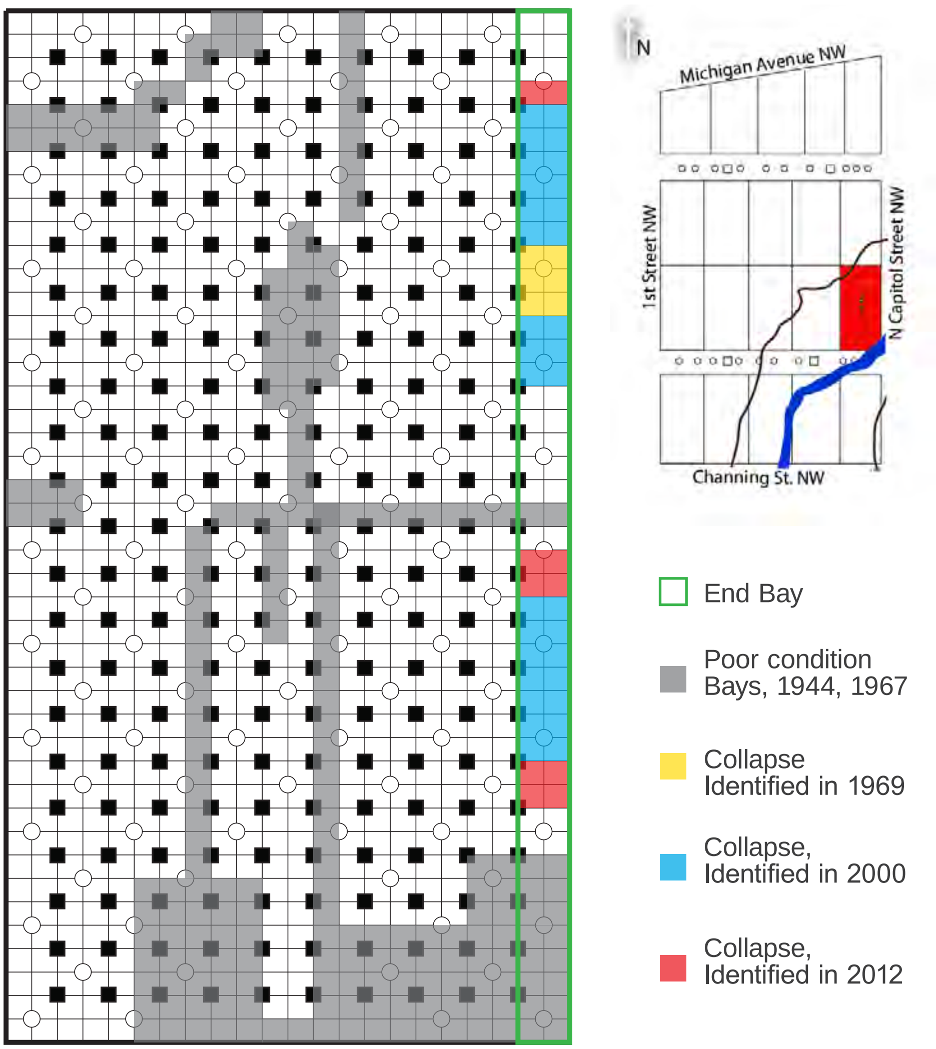


FIGURE 16: FILTER No. 24, DAMAGED AND COLLAPSED BAYS

The collapsed areas at filter No. 24 reported in the 1944, 1967 and 2000 surveys, and areas that have been identified as recent collapse by RSA, were plotted as a function of a percentage of a total end bay area. The graph (Figure 17) shows a higher rate of collapse between 1968 and 2012. Assuming a current and steady rate of collapse, extrapolating the rate of collapse indicates that sometime by year 2025 the entire eastern end bay of this filter will have collapsed as depicted in Figure 17.

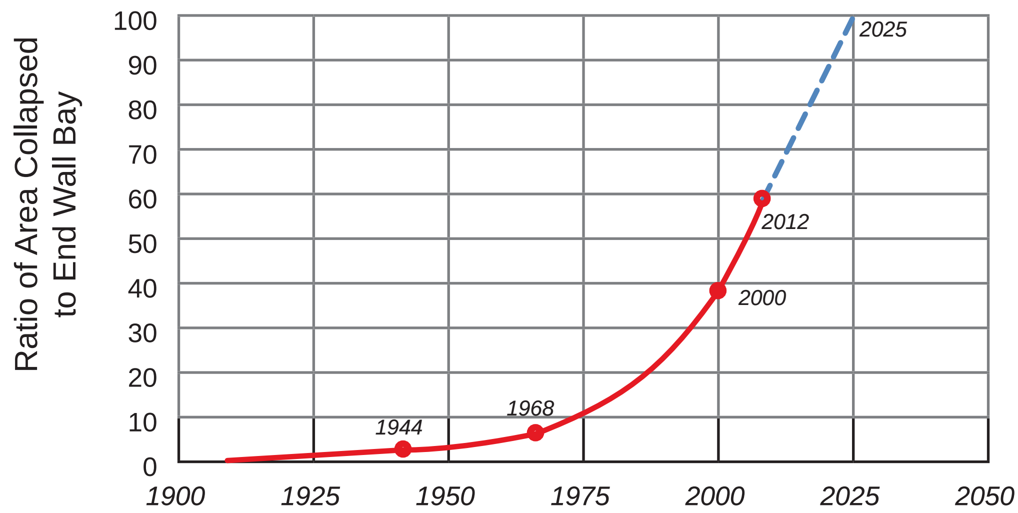


FIGURE 17: RATE OF COLLAPSED AT SLOW SAND FILTER NO. 24

As the collapsed area increases, the rate of collapse is expected to accelerate due to new mechanisms in play. It is expected that, as this happens, adjacent bays within filter No. 24 will either collapse or initiate a collapse mechanism at other areas. This is due to unstable conditions created by the missing bays. Accordingly, it is very possible that, like the increase in collapse rate after 1968, the rate at which these bays are collapsing will continue to accelerate.

OBSERVED DAMAGE

Review of existing reports and historic documentation reveals a consistent correlation between the location of the filters, their level of damage, and the 1894 topographic survey elevations. There is a clear correlation between the grade elevations recorded by the 1894 topographic survey and the level of settlement-related damage observed during the site investigation. Filters with severe deterioration and specifically areas where the ceiling vaults have collapsed were usually constructed on fill. In contrast, those filters with minor or moderate deterioration were typically constructed on areas where the original grade was at the highest elevations, so that the foundations were bearing on virgin soils as opposed to fill materials.

The southeast corner of filter No. 27 and southwest corner of filter No. 28 are located at the lowest elevation recorded in 1894. Filter No. 29 is located at a higher elevation but the east end wall of this filter is next to North Capitol Street. Filter 24 has sustained much more damage than filter Nos. 27 and 28. Filter No. 24, located at the lowest 1894 elevation of the four filters next to North Capitol Street, has sustained the most severe level of damage among all the filters investigated, with damage including the collapse of more than half of the bays framing into its east end wall (Figure 18). Filter No. 19, sharing a divider wall with filter No. 24, is also next to North Capitol Street, and it is believed that the redistribution of forces due to the severe damage sustained by filter No. 24 has accelerated the damage of filter No. 19, causing the collapse of an area near the southeast corner of this filter. On the other hand, the lowest level of damage was at filter Nos. 16, 17, and 21, which are located at the highest grade elevations reported in 1894, all of which are interior bays.

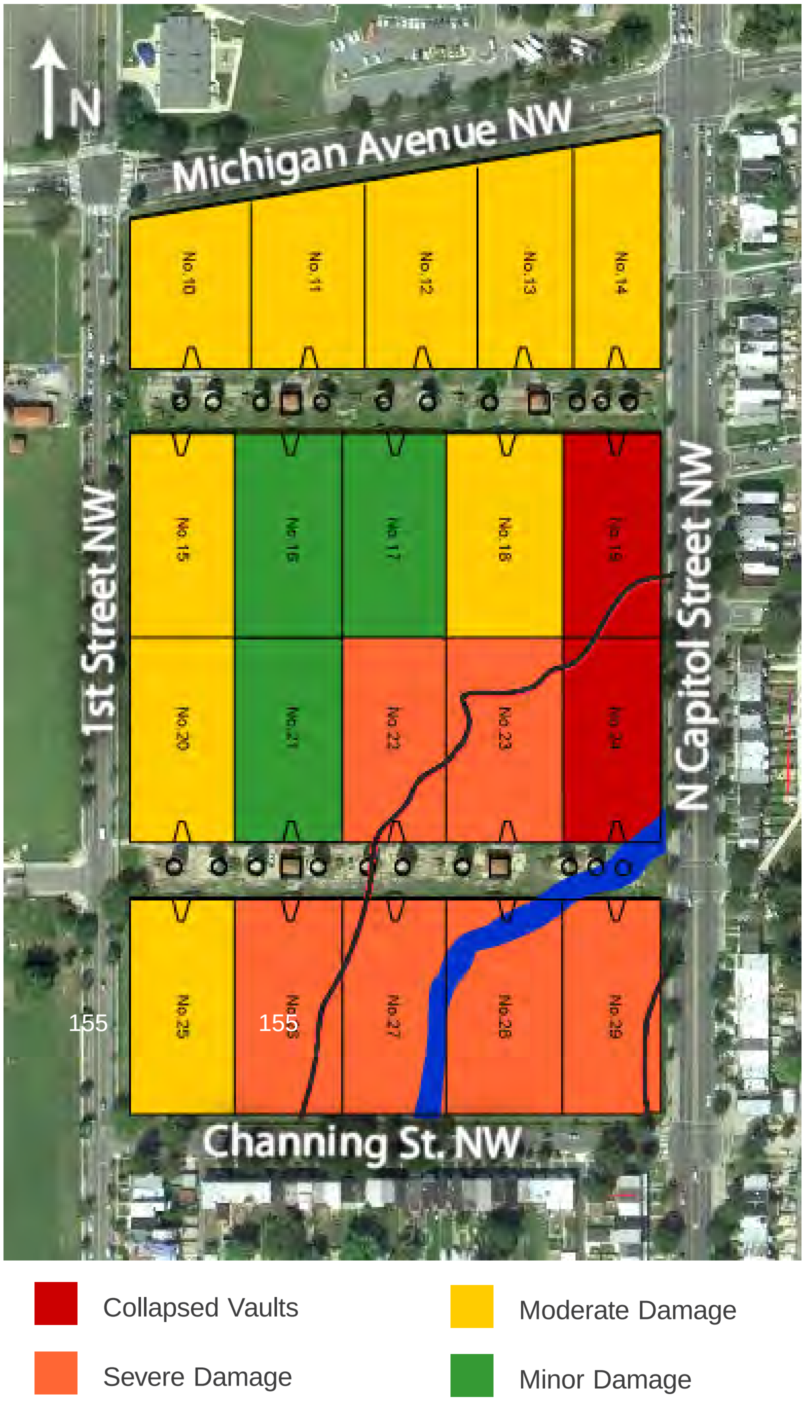


FIG. 18 - LEVEL OF DAMAGE VS. 1894 GRADE ELEVATIONS

STRUCTURAL ANALYSIS & STABILITY INVESTIGATION

RSA has performed a structural analysis and stability investigation of the slow sand filter assemblies to further explore the concerns outlined in prior investigations. Specifically to:

(1) develop an understanding of this historic structural system and the role of historic materials contributing to its performance

(2) identify the causes of the observed damage. This included the vaults, piers, walls, and foundations. The vaults were evaluated for stability under the observed conditions with lateral or vertical support displacements.

(3) determine its current and future ability to support both current and potential future loadings. Future loadings would include more demanding requirements for uniform or unbalanced gravity loads.

REVIEW OF PRIOR ANALYSIS (CCJM 2000)

As part of this study, RSA reviewed a draft of the "Structural/Geotechnical Engineering Evaluation of the McMillan Filter Site", prepared by C. C. Johnson and Malhotra, PC, Environmental Engineers and Scientists for the D. C. Department of Housing and Community Development, in August 2000 (CCJM 2000).

<u>RSA Comment</u>: Neither the final version of this document was available, nor the reference documents. These included "Existing Conditions Report", dated April 4, 1988 prepared by the Organization for Environmental Growth, Inc. and a report titled "Architectural and Archaeological Survey, Eastern Portion, McMillan Water Treatment Plant" by Engineering – Science, Inc. dated June 1990.

In June and July 2000, CCJM walked through each of the 20 filter cells to inspect the condition of concrete columns walls and ceilings and compare their findings to results reported in a 1967 Army Corps of Engineer's condition survey. General observations included damage to the ceilings (visible portions of the vaults) ranging from hairline cracks to collapse of large areas. Columns and piers were noted to be in good shape with the exception of cell number 27 where columns were severely displaced and retrofit with steel collars and rods. A test pit on the top surface of one cell confirmed cracking through the thickness of the vaults.

<u>RSA Comment</u>: The report failed to note that the columns with severe displacement were likely accompanied by significant cracking of the foundation slab which would lead to additional foundation distress due to wash out of subgrade soils under the high water flows from the active filtration system. RSA has recommended a probe of these conditions in any cells to be retained.

Observations were categorized into three types of deterioration.

- Type I/significant deterioration
- Type II/moderate deterioration
- Type III/no significant deterioration

In general, conditions had worsened and some areas of prior collapse had expanded.

The report presented results of a petrographic examination of concrete cores taken from the structure. The test results concluded that alkali silica reaction (ASR) products were observed in all cores. Results also noted that top surfaces of the concrete had been chemically altered to a shallow depth due to low level sulfate attack. Conclusions noted that failure may have been due to loads on portions of the structure weakened by low level sulphate attack and ASR.

Structural analysis was performed using finite element analysis software. The results of the analysis show that, in an ideal situation, the original structure has sufficient capacity to carry in excess of 640 PSF live load or an HS – 20–44 truck load per AAASHTO code, in addition to the 2 feet of existing soil.

> <u>RSA Comment</u>: This analysis was performed assuming an un-cracked continuum of concrete and no vertical or horizontal settlement was considered. Both of these issues render the results of this analysis inconclusive.

The report notes that under current code requirements "lean concrete ceilings are unsafe for public access because of their nearly zero elasticity". To address this issue the report recommended the addition of reinforced concrete cast in place slabs on top of the existing ceiling.

> <u>RSA Comment</u>: This approach does not fully address the safety concerns for occupancy within the filters and does not address the concern about ongoing settlement or spread of the existing structure. The interior of the structures must remain visible and accessible for inspection and repair to prevent any life safety threats from failed unreinforced concrete fragments from being buried above false ceiling. In order to reuse the existing structure, there must be a means of assuring that the movement of the structure is arrested. These means must provide structural resistance and the ability to identify any unwanted movement through monitoring.

> <u>RSA Comment</u>: The report did not include an analysis of the existing footing capacity for continued use or preservation of the cells with enhanced topside loading. RSA performed an analysis and found the footings to be a limiting factor for use on the top side of the structure. Based on this concern, RSA has recommended that the existing footings be exposed for inspection and that sub-grades be investigated further.

Perimeter wall was noted to be in poor structural condition and need to repaired or replaced in certain areas.

> <u>RSA Comment</u>: The report failed to note the important role of the perimeter wall in providing thrust resistance for the unreinforced concrete vaults within the filter system. Great care is needed if walls are to be modified and or replaced, or if there is any adjacent site activity that will cause settlement of shifting of the existing structure. Relatively small movement of the perimeter walls can lead to failure or collapse of the perimeter vaults, leading to a compromised structural capacity or progressive failure of the interior vaults.

The report includes data from cores taken from the filters showing an average compressive strength greater than 5,000 psi.

> <u>RSA Comment</u>: Data from limited field testing must be adjusted for sample size and correlated to the core sample size per latest ACI standards.

Geotechnical testing was performed and reported.

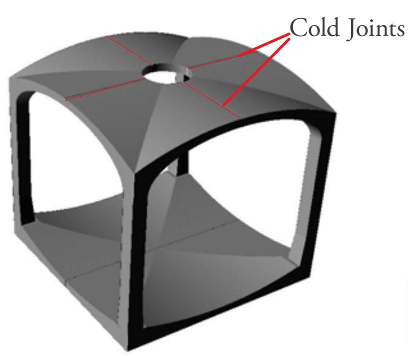
RSA Comment: The basic questions of the existing foundation capacity for new load was not evaluated. A value of 3,000 psf was made reference to in the text, so this value was used by RSA in our evaluation.

VAULT LOAD CAPACITY ANALYSIS METHOD

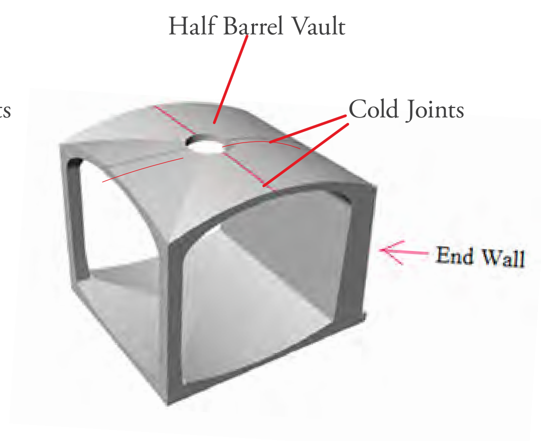
The objective of the current load capacity analysis was to determine the ability of the slow sand filter structure to carry existing loads, and additional loads for future development. The analysis considered both the end bays, located at the end wall and divider walls, and the interior bays, which are supported only by piers.

The method of analysis implemented to estimate the capacity of the exterior bay vaults is based on thrust line analysis - a means of calculating the compressive stress at any given segment of the unreinforced concrete vault and determining the location of the thrust line by graphic statics. Stability investigations were performed utilizing assumed failure mechanisms based on the results of the graphic analysis and field observations.

The end bay analysis assumes a barrel vault supported by abutments, whereas the interior bays and the inverted vault footing are modeled as a groin vault. (See Figure 19)



19A – Typical Interior Bay, Groin Vault



19B – Typical End Bay, Half Barrel Vault

FIGURE 19: PRIMARY STRUCTURAL FORMS

Typical interior bays are supported in such a way that two-way action results. When loaded, the load path of the vault takes two orthogonal directions. However, once cracks develop and a failure mechanism is triggered, the behavior is in one primary direction. Accordingly, interior bay stability investigation was based on a simplified method that assumes a failure mechanism similar to the exterior bays.

The allowable compressive strength of the unreinforced concrete was estimated to be 22.5% of the ultimate compressive strength (f'c), which was common practice at the time the slow sand filters were designed and built. With an ultimate concrete strength (f'c) estimated at 4000 psi, an allowable compressive stress (f'c) of 900 psi was used. The tensile capacity of the unreinforced concrete was assumed to be negligible for the purposes of this analysis, to be consistent with the general design approach for unreinforced concrete both historically and today. However, it should be noted that the interior vaults, due to the double curvature of the intersecting barrel vaults, will carry significant more load, before a tension crack initiates – which should be assumed to result in a sudden transfer of loads and movement as the structure cracks and settles into the mechanisms outlined herein. This transfer may cause dynamic effects, or in the worst scenario may cause instantaneous collapse. This is why unreinforced concrete is not permitted for use in overhead applications in occupied structures in current buildings codes.

UNIFORM DISTRIBUTED LOAD

Given that live loadings are typically transient and not evenly distributed, uniform loads represent an idealized scenario that results in a theoretical upper limit of potential capacity (meaning that the capacity cannot exceed this value, but may be significantly less). Preliminary analysis results demonstrate the vault configuration provides adequate capacity to sustain significant uniform live loads in addition to permanent loads. RSA employed traditional equilibrium methods to calculate the structural capacity of both the end bays and interior bays based on a thrust line calculation using graphical statics, see Appendix E.

Under the idealized case of uniform distributed loading, analysis results indicate that the end vaults can reach a theoretical live load capacity of approximately 2,800 psf. This value is much greater than typical code-required live load. However, the real capacity would need to consider unbalanced loadings, irregular geometry, foundation capacity pier settlement, and wall movements. When considering foundation capacity this number reduces to approximately . When considering foundation settlement , the capacity reduces to 100 psf under settlement of ¾ inch and 1-1/2 inches respectively (vertical or horizontal) and becomes unstable (causing collapse) at any displacement beyond that value.

Similar to the exterior bay analysis, the calculation of the horizontal thrust at interior bays translates to a compressive stress in the concrete, which is compared to the allowable stress for the material. Under the idealized case of uniform distributed loading, groin vault analysis results indicate that the interior vaults reach an allowable surface live load capacity of approximately 2,840 psf, which is almost identical to the calculated live load capacity of the end bays following the graphical statics method. This indicates that foundation capacity and unbalanced loading will be the limiting factors.

UNIFORM DISTRIBUTED LOAD

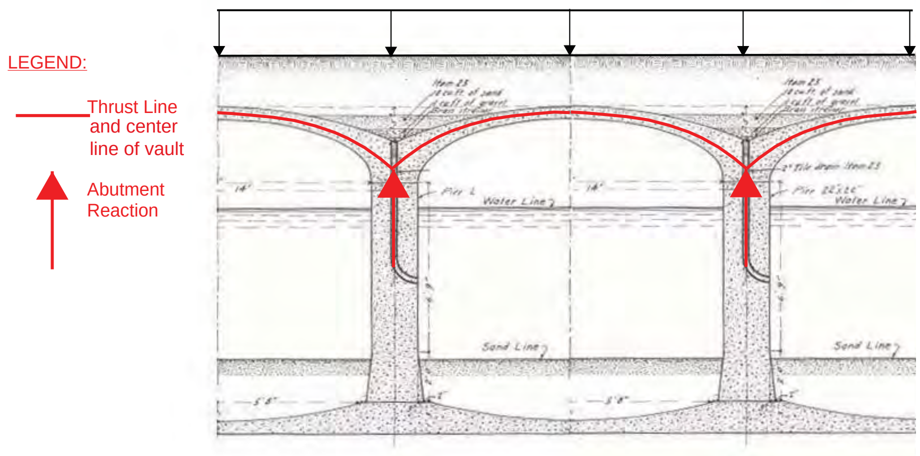


FIG. 20 - LOCATION OF THE VAULT THRUST LINE UNDER BALANCED LOADING CONDITIONS

UNBALANCED LOADS

In keeping with the variable nature of live loads, the second case considered in the slow sand filter analysis is to assume unbalanced live loads by applying a uniform live load at only half the span of the vault. The variation of the load from the base uniform assumption shifts the thrust line away from the center of the vault section (see Figure 21), resulting in increasing compression stresses and potential cracking.

Analysis results for this case indicate that the end vaults can reach an allowable surface live load capacity of approximately 900 psf, before reaching the unreinforced concrete allowable compressive strength estimated at 900 psi. This value is still greater than common code-required live loads, However, other observed conditions such as pier settlement, or wall movement have reduced capacity even more.

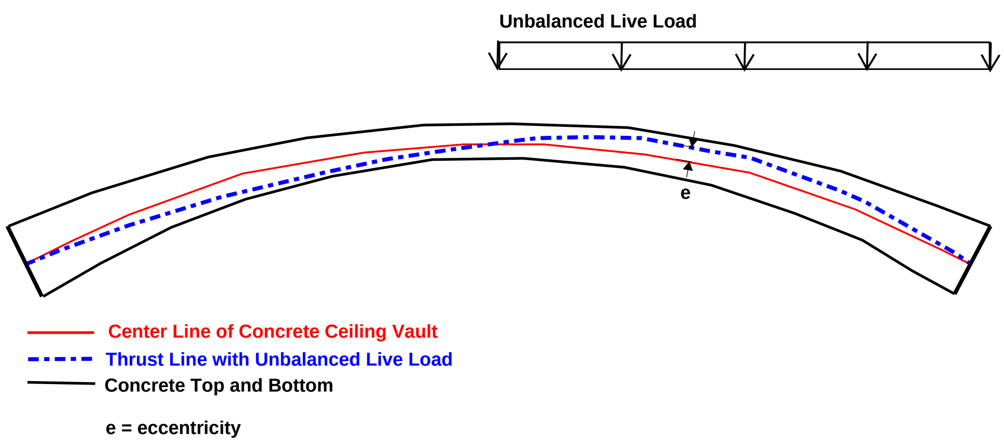


FIG. 21 - UNBALANCED LIVE LOAD ANALYSIS CASE

VAULT SUPPORT MOVEMENT

Both lateral and vertical support movement were considered in this analysis, both of which were observed on site. Lateral displacement is to be expected given the nature of the structure and its perimeter supports. The relatively shallow vaults, supporting significant vertical loads, create a horizontal thrust that must ultimately be resisted at the perimeter. The use of a gravity retaining wall to resist this thrust leaves the structure susceptible to lateral displacement. Generally, geotechnical engineers will require that a structure is designed to move approximately 1 inch if is expected to develop passive pressures against the surrounding soils when the soils are completely confined below grade. If the surrounding soils are not fully embedded below grade, as is the case as the site drops off to the south, this movement will be larger. Vertical displacement is also to be expected given the clear correlation between areas constructed on fill material and areas of collapsed vaults.

Lateral Displacement– Spreading Supports

Because of the need to ultimately resist the vault thrust at the perimeter, this analysis focused on the behavior of the end bays, modeled as barrel vaults. RSA looked at the progressive effect of spreading the supports on the end vaults, starting from 1 inch displacement of each support end and increasing incrementally. For a 1 inch horizontal spread at the supports, the apex of the ceiling vaults drops approximately 4.5 inch, the compressive stresses at the apex increases by approximately 30%, and then by 50% for a spread of 1.5 inches. Although stresses are increasing appreciably corresponding to these initial movements, the overall vault section remains in compression and the stress levels are still below the allowable compressive capacity of 900 psi. Figure 19 depicts the typical location of the thrust line for this analysis case.

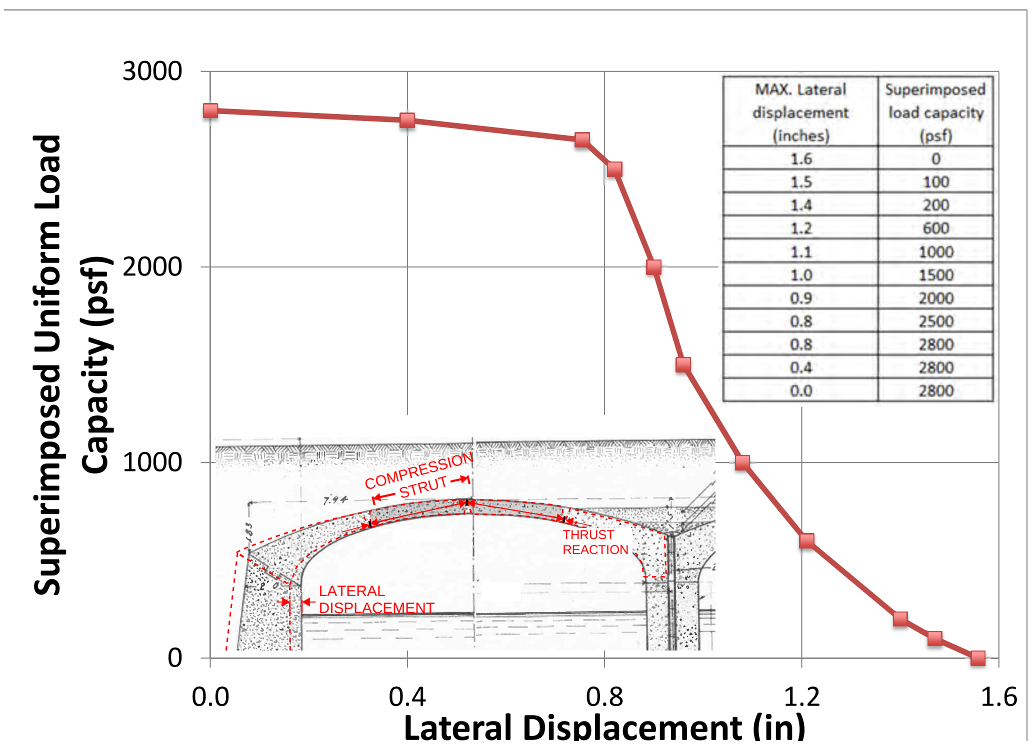


FIGURE 22: LOAD CAPACITY VERSUS LATERAL DISPLACEMENT

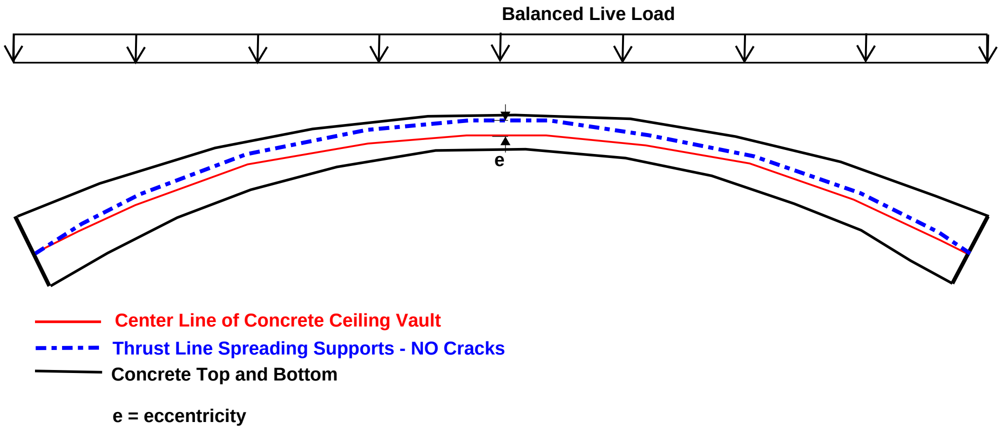


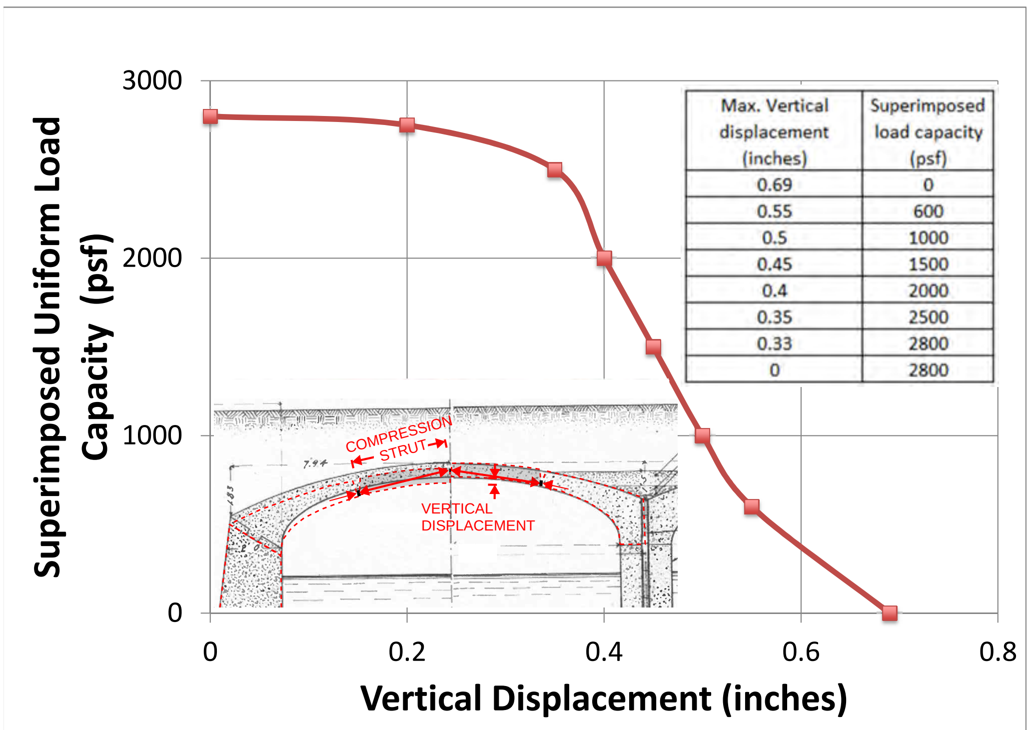
FIG. 23 - SPREADING SUPPORTS ANALYSIS CASE

As the lateral displacement of supports increases further, the apex continues to drop. As the vault becomes shallower, the horizontal thrust increases proportionally to resist the unchanging weight above being carried. Considering the geometric non-linearity of this progressive behavior, the analysis shows that prior to reaching two-inches of lateral displacement on each side (four-inch total spread), the vault will fail.

Separation at the vault apex results in the loss of restraint against lateral movement and initiates cantilever action of the vault. A cantilever is a beam that is fixed at one end and unsupported at the other and must function like a beam in bending which is always subject to tensile stresses. Therefore, given the lack of tensile capacity in the unreinforced concrete, we would expect the vault to crack along its top surface, begin to rotate and deflect further, and potentially re-establish contact with the adjacent panel until further lateral movement occurs. This mechanism formation can repeat as movement continues. In addition, the realignment of the joint, as discussed herein, can result in local stress concentrations between panels, and quickly generate further cracking and overstress. Given the inherent higher stiffness of the interior bays with respect to the end bays, thanks to the groin vault geometry, the end bay half-barrel vaults are more sensitive to horizontal displacements and more flexible when transitioning to cantilever behavior. This variation in vault stiffness, when subjected to lateral support displacement, appears to have lead to panel misalignment, local overstress, and the onset of a collapse mechanism sequence.

Vertical Displacement – Support Settlement

Similar to the analysis of lateral displacement, RSA considered the progressive effects of differential settlement between supports. Again, because of the critical role played by the end bay vaults, this analysis focuses on the barrel vault behavior. The analysis revealed a pattern of structural response very similar to that for lateral displacement. For relatively small differential settlements, the vault structure can maintain stability and continue to support loads from above. As the differential settlement continues, the distance between support points on either side of the end bay vault gets greater. Considering the geometric non-linearity of this progressive behavior, the analysis shows that prior to reaching five-inches of differential settlement, the vault will fail. This assumes that the continuity between the panels at the high point of the vault remains intact throughout the movement. Although the joint represents distinct pours between panels without material continuity, the transfer of thrust across this joint and the resulting friction, can serve to maintain this contact surface. Exceptions to this were observed, where there was measurable misalignment between the panels at the vault apex. As much as three-inches of misalignment was documented in the vicinity of collapsed vaults.



Max. Vertical displacement (inches)	Superimposed load capacity (psf)
0.69	0
0.55	600
0.5	1000
0.45	1500
0.4	2000
0.35	2500
0.33	2800
0	2800

FIGURE 24: LOAD CAPACITY VERSUS VERTICAL DISPLACEMENT

Misalignment at the vault apex may be the result of multiple influences, ranging from shifting due to lateral displacement, foundation settlement, and potential flaws from the original construction. In addition, this condition was commonly observed in the area of the entrance portals. Here, it appears that the misalignment is the result of rotations generated by the eccentricity between thrust lines of the central bay barrel vault and adjacent groin vaults. Figure 20 depicts the condition where misalignment occurs, yet the vaults continue to bear against each other. With this thrust misalignment, local bending stresses are induced at the inter-panel joint which accelerate the formation of cracks or hinges in the vault structure and reduce the effective depth of the arch mechanism.

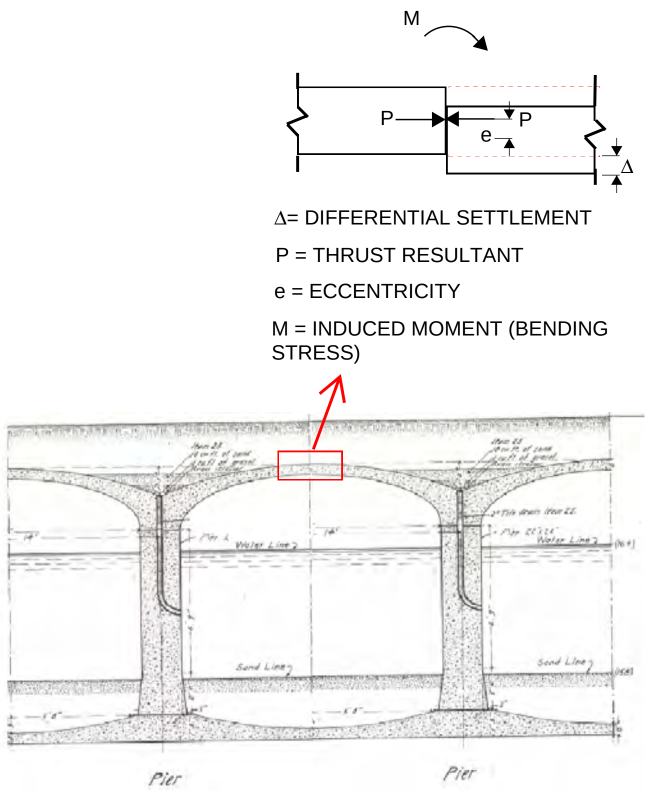


FIGURE 25: VERTICAL DISPLACEMENT AT VAULT APEX

PIER & FOUNDATION CAPACITY

The axial load capacity of a typical unreinforced concrete column, or pier, was estimated to be 347 kips (service loads) which allows for a significant topside capacity of more than 1000 psf. However, this represents a theoretical upper limit of capacity, where the pier is subjected to pure axial forces with no bending stresses, and where the realities of unbalanced loads, irregular low-bearing capacities, and perimeter wall movement represent significant concerns that can result in much smaller pier capacities.

The foundations of the slow sand filters are inverted groin vaults in unreinforced concrete, very much like the vaulted roof structure above. This footing construction system is very effective in supporting the weight of the filtration sand and filler water. However, without the presence of reinforcement, the capacity of the footing in supporting topside loads coming from the piers, in combination with the realities of unbalanced loads and pockets of soil with low bearing capacity, compromise the functioning of the foundation as an inverted vault system. Accordingly the foundation is only able to distribute loads more directly around the pier areas resulting in an effective increase in soil pressure with respect to the even pressure distribution beneath the full inverted vault foundation (Figure 26).

A precise determination of an effective spread footing area is somewhat speculative. Figure 27 shows a range of sizes for equivalent modern spread footings, assuming load distribution planes emanating from the pier at ratios of 2:1, 1:1 and 1:2. The most generous distribution assumption reaches to an equivalent footing size of 7'-4" square, which reduces bearing pressures to less than the preliminary

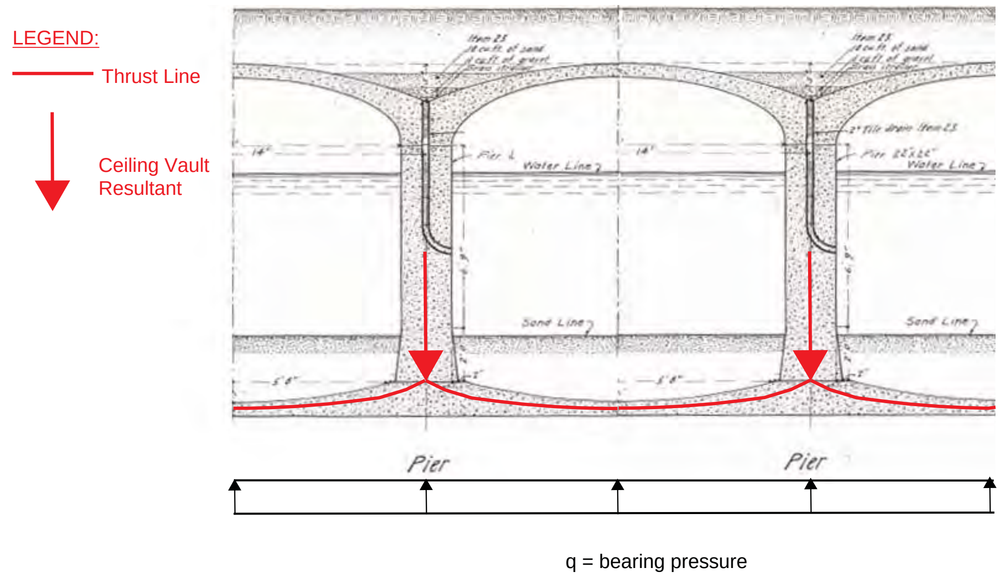


FIG. 26A - INVERTED VAULT
FOOTING BEARING PRESSURE

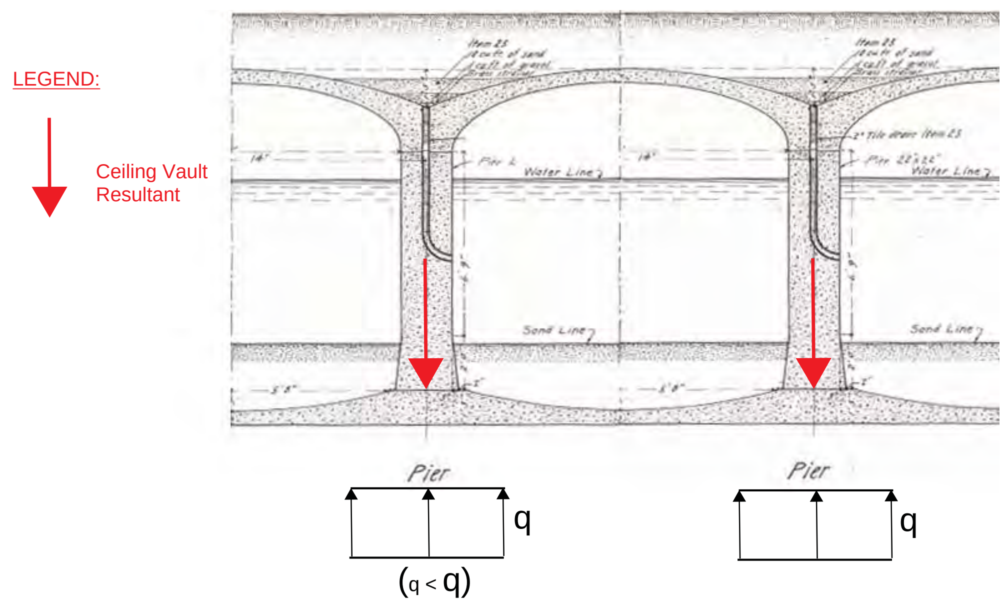


FIG. 26B - CRACKED FOOTING
CONCRETE BEARING PRESSURE

FIG. 26 - FOOTING ASSEMBLY

allowable pressures reported by the CCJM geotechnical investigation of about 3000 psf. Although these would be reasonable sizes for modern reinforced concrete spread footings, analysis results indicate that the hypothetical spread footing, in isolation from the remaining foundation inverted vault, does not have sufficient flexural capacity.

The punching shear capacity of the footing is estimated, based on an unreinforced concrete allowable compressive shear stress of 40 psi [Kidder-Nolan, 1921], at 70 kips (service loads). This punching shear capacity is less than the self-weight of the concrete vault and pier plus the estimated weight of the topsoil above the vault. This limits the footing size to a 3'-7" square area which is equivalent to the critical perimeter for punching shear. For this footing size, the bearing pressure, not including topside live loads, was estimated to be 7500 psf. The higher bearing pressure exerted to the underlying soil consequently implies significant settlements. Figure 23 shows the capacities of the estimated 3'-6" square footing as a function of allowable bearing soil pressures.

During the site condition assessment of slow sand filter No. 27, pier rotations were consistently observed through six rows of piers immediately south of the entrance portal (see Figure 29). This filter is near the stream reported by the 1894 topographic survey, where the stream ravine was filled, which indicates that the pier rotations are most likely due to settlement caused by a pocket of badly compacted fill, altering the load path of the gravity loads going through the vaults and piers. As this happened, it is likely that the foundation inverted vaults cracked.

When the foundation concrete cracks due to flexure, concrete punching shear, or settlements at pockets of badly compacted soil, in addition to reducing effective bearing area, the cracking also created an avenue for the filler water to travel through, further weakening the soil bearing capacity and consequently making the footing settle even more.

For these reasons the footing configuration of the slow sand filter structural assembly cannot be considered adequate without proper inspection of the vaults, testing of the soils and appropriate

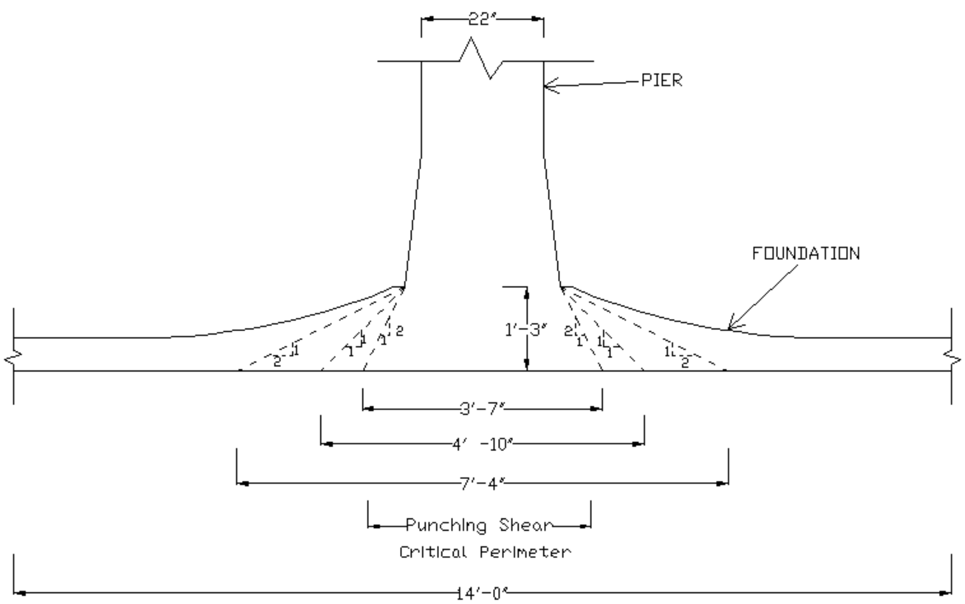
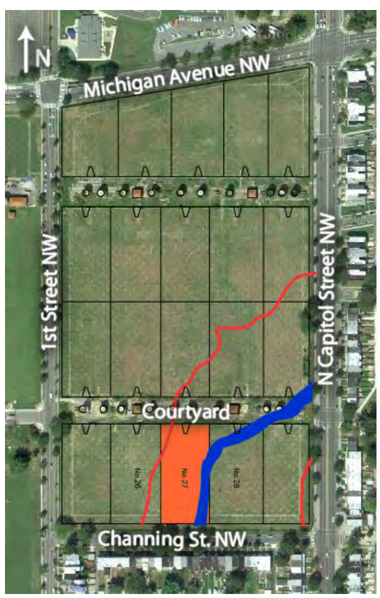


FIG. 27 - FOUNDATION SLAB: EQUIVALENT SPREAD FOOTING

Allowable Bearing Pressure (psf)	3'-6" square footing axial load bearing capacity (service) 'Kips'	topside (liveload) capacity (service) 'Psf'
3000	37	0
4000	49	0
5000	61	0
6000	74	0
7000	86	0
7500	92	0
8000	98	30
9000	110	90
10000	123	150

FIG. 28 - TOPSIDE UNIFORM LOAD CAPACITY



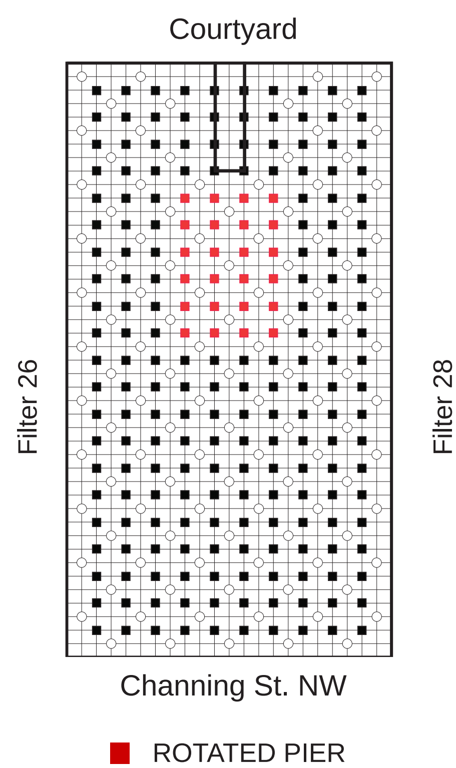


FIGURE 29: SLOW SAND FILTER NO. 27 DAMAGE

WALL CAPACITY

Field observations of damage and soil conditions highlight the susceptibility of the vault structures to support movements. To further understand the nature of such displacement, RSA performed a structural analysis of the end wall assemblies. Figure 30 depicts the general arrangement of forces that are imposed and resisted at this critical perimeter of the vaulted filters.

Different levels of passive pressures, which contribute to the stability of the gravity end walls against overturning and sliding, were considered to account for the different soil profiles at the site. Generally, the walls me the required factors of safety against sliding and overturning when assuming that they will develop passive pressures when subject to thrust pressures from the vaults. However, these pressures assume a level of movement which will impact the behavior of the vaults as discussed below.

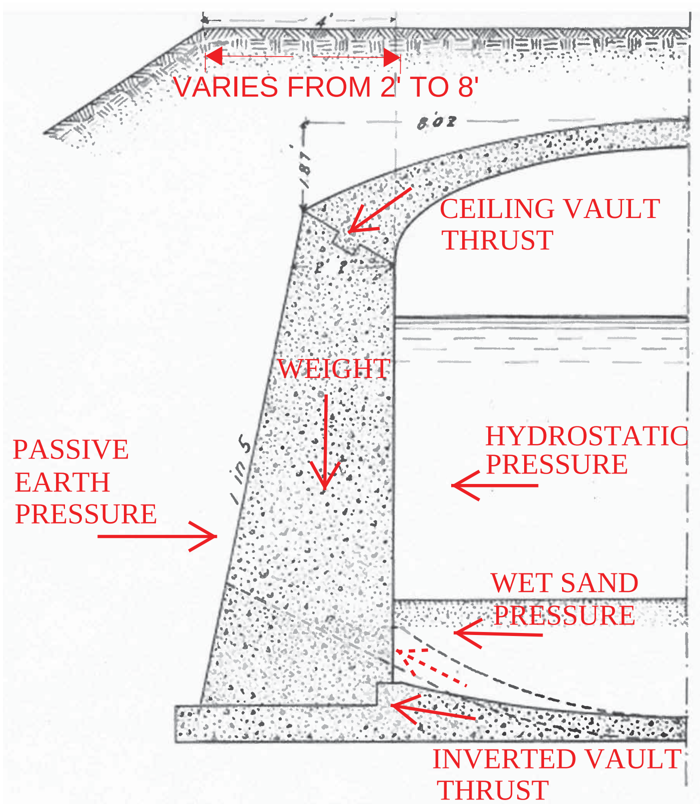


FIGURE 30: GRAVITY END WALL FREE BODY DIAGRAM

OVERALL STABILITY

The structural assembly of the slow sand filters, how their different parts come together, and how these parts are affected by different conditions such as unbalanced loads and settlement has been addressed in previous sections. This section addresses how end wall movement and unbalanced loading conditions effect of the overall stability of the vaults, and how unbalanced loading could create conditions leading to instabilities, or collapse.

Observation at the collapsed bays has made it clear that a displacement of the end bay walls preceded the conditions causing collapse. Figure 32 depicts the observed hinge formations. Similar conditions were observed around the entrance portals. Hinge formation does not necessarily mean that collapse is imminent, but the presence of hinge mechanisms indicates that the ceiling vault is in a state of reduced capacity (Figure 32), and if enough hinges are formed, they are likely to lead to instability or collapse.

The initial trigger for the collapse appears to be the lateral and/or vertical movement of the end wall. It appears that the movement is sufficient to cause separation at the vault apex ' Step 1'. This separation precludes the transfer of arch thrust, and each side begins to behave like a cantilever. Unable to carry significant tensile stresses, cracking and hinge formation ensues 'Step 2'. 'Step 3' in the diagram shows the misaligned apex joint, highlighting the potential for new cracking and localized overstress as thrust tries to re-establish a path through the joint. The failure mechanism of the typical interior bays is also similar to the mechanism depicted in Figure 27. The fact that no interior bays have collapsed is attributed to the amount of restraint offered by the surrounding bays.

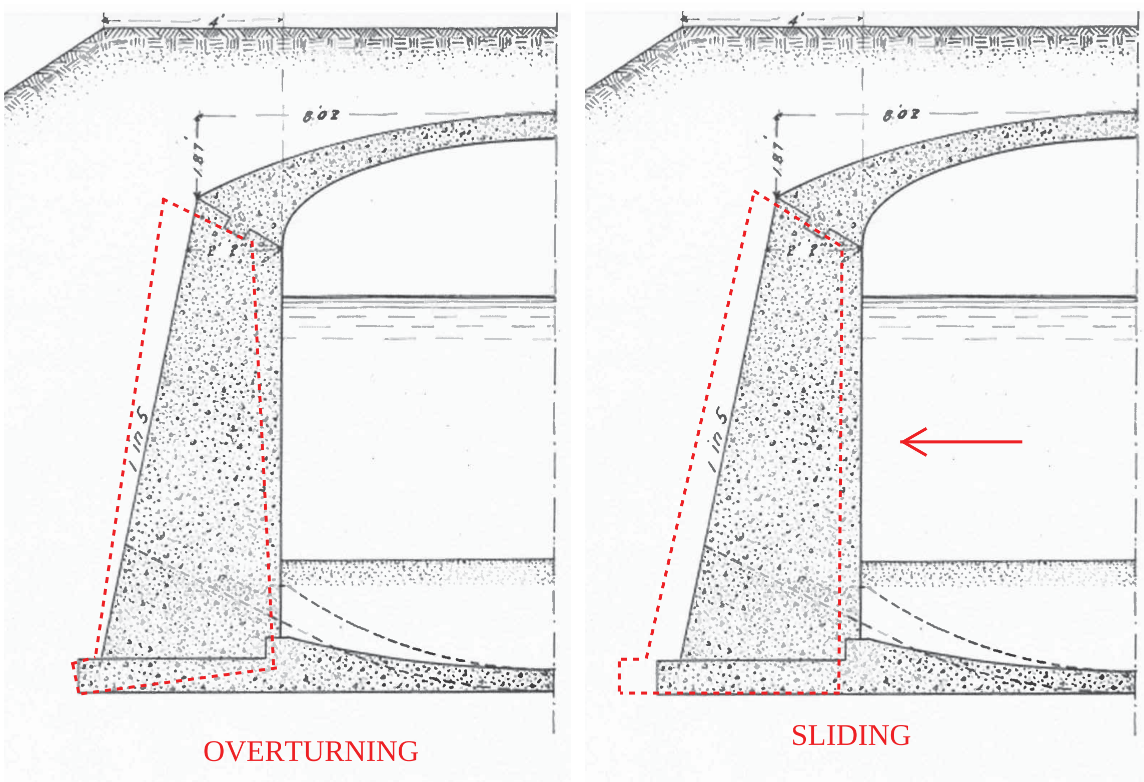


FIGURE 31: END WALL STABILITY LIMIT STATES

At the entrance portal the failure mechanism observed in several cells is attributed to the misalignment of forces due to the disruption of the structural symmetry. This affects the internal forces of interior groin vaults on either side adjacent to the entrance. The misalignment of forces creates a condition similar to that described at the end wall bays, where moments and tensile stresses are induced to the unreinforced concrete, which consequently hinges (cracks) and finds a new state of equilibrium. Similar to the end walls bays, if enough hinges formed, this will lead to instability or collapse.

The correlation of vault failures and distress to end bays and areas where filters were originally constructed on fill soils remains a strong indicator that movement of the supports may well be the most significant factor leading to vault instability. The reduced depth of the surrounding grade towards the south appears to be another significant factor contributing to the reduced effectiveness of soils in limiting both lateral and vertical displacement at the end walls.

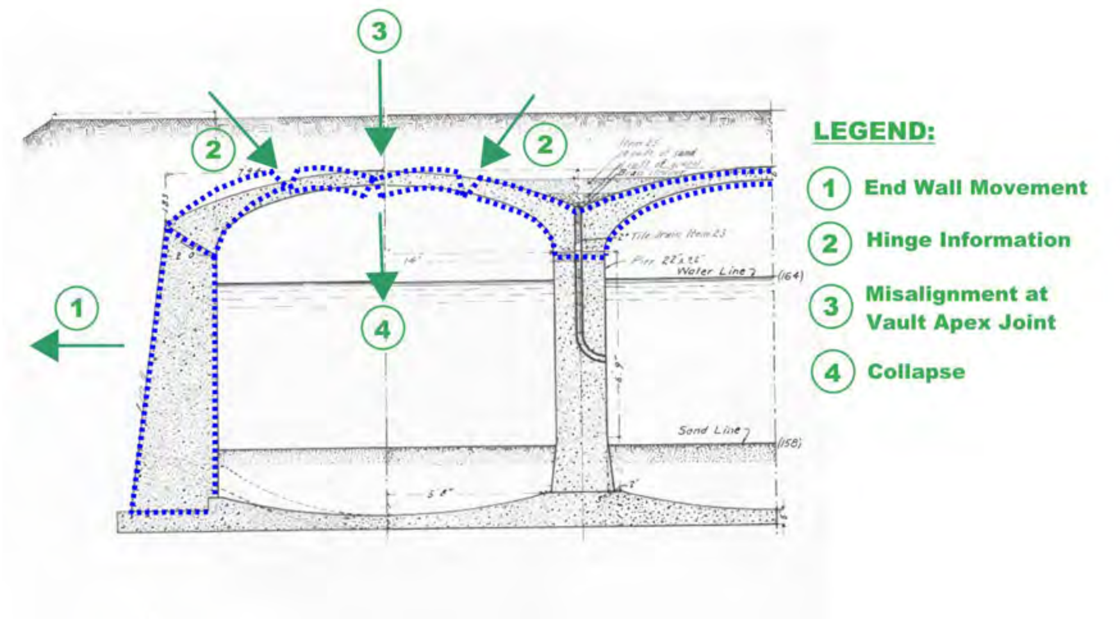


FIGURE 32: COLLAPSE MECHANISM SEQUENCE.

FEASIBILITY STUDY

The structural analysis and stability investigation performed by RSA under this study has demonstrated the limitations of the existing structure the susceptibility of the existing structures to support movement. As such, any reuse scenario will need to address three primary technical issues:

- Reinforcement for Top Surface Loading
- Foundation Evaluation and Reinforcement
- Reinforcement to Protect Against Settlement Related Damage and Instability

REUSE OPTIONS

The following treatment options have been developed by the design team. Each are discussed in the following section relative to these technical issues.

OPTION 1: DO NOTHING

In its current state, the sand filters should not be used for any use without. Conditions are rapidly deteriorating and the conditions should be considered dangerous. The rate of decay has been illustrated herein and the rate of instability and collapse is expected to continue to accelerate.

OPTION 2: TOP SURFACE LIGHT USE ONLY

In order to use the top surface for light loading – human pedestrian access only, it is critical to restore and maintain the structural integrity of the vaults. Reinforcement of the top surface will be necessary to avoid collapse of the cells and to permit loading. Reinforcement will likely involve a new slab which must incorporate drainage (Figure 33). Foundations will require evaluation and may require reinforcement. Where demolitions or excavations are proposed in adjacent areas, additional measures will be required for reinforcement to protect against settlement related damage and instability.

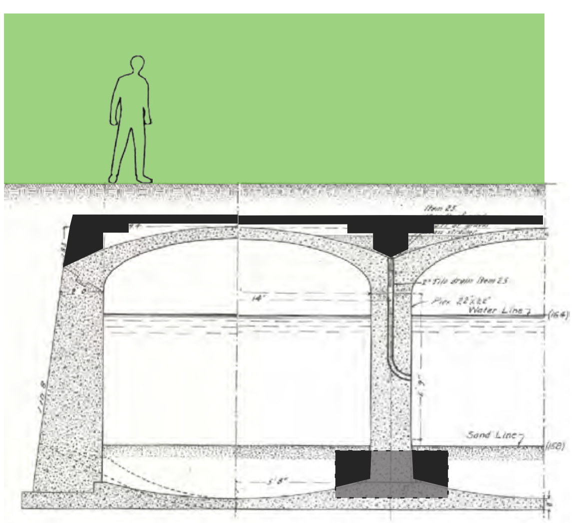


FIG. 33 - TOP SURFACE LOADING

OPTION 3: TOP SURFACE LIGHT USE/ INTERIOR TOUR

In addition to the measures listed above in Option 2, it will be necessary to investigate, repair and regularly monitor the interior of the re-used cells to assure that no hazardous conditions develop on the interior of the cells.

OPTION 4: PROGRAMMED TOP SURFACE/ INTERIOR TOUR

In addition to the measures listed in Option 3, the reinforcement of the top surface will be heavier and will require supplemental foundation reinforcement within the cells. These could be done within the depth of the sand so that they are not visible during interior tours.

OPTION 5: PROGRAMMED TOP SURFACE / INTERIOR PROGRAM

In addition to the items listed in Option 4, an interior program will require that the new slab supporting the top surface also incorporate waterproofing drainage and insulation, to provide an envelope for the historic vaults below (Figure 34). This option should be planned to account for the additional cracking and related distress that will come from volume changes as the structural concrete adjusts to a new state – without the regular exposure to ground moisture and temperature. Foundations for this scheme may be more challenging if the floor elevation is to take advantage of the depth of the structure.

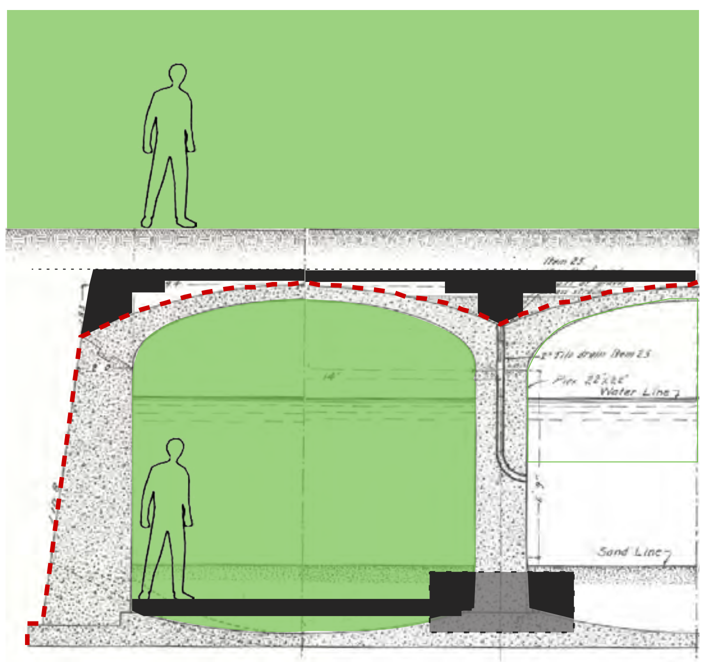


FIG. 34 - INTERIOR USE

OPTION 6: OVERBUILD/ INTERIOR PROGRAM

In addition to the items listed in Option 5, a vertical expansion (overbuild) will require new heavy foundations Vertical additions to the existing structure would most likely require a new, independent structure woven between the existing systems (Figure 35). New columns would be supported on new interior column footings. It would appear most economical at this time to maximize the column spacing to minimize the interior work. Foundations would likely involve micropiles or another system that could be drilled in to reduce the risk of disturbance of the sub soils supporting the history structure to remain. Should this strategy be pursued, consideration should be given to the impact of this new structure and foundation on the historic integrity of the interior of the existing filters.

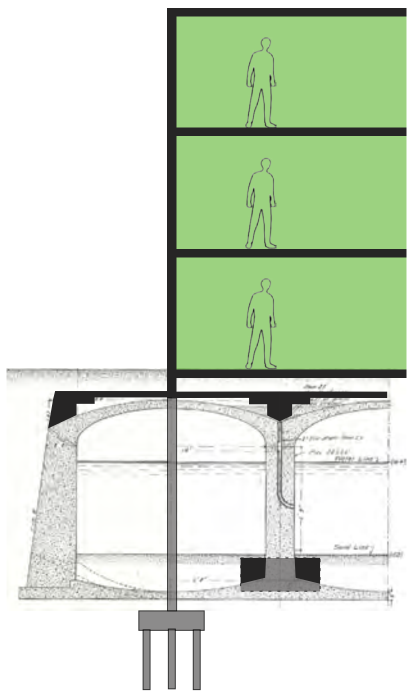


FIG. 35 - OVER-BUILD

OPTION 7: NEW DEVELOPMENT AT THE SITE

In the case that some filters are demolished to make way for new construction, the existing soils have been evaluated, and recommendations have been developed for appropriate foundations to compensate for the soils conditions that have lead to the problems with the existing historic filters. (discussed in the Structural Condition Assessment chapter of this report).

REINFORCEMENT FOR TOP SURFACE LOADINGS

In order to allow live loads at the vault roof level a new top slab is required, presumably set above the existing soils that sit on the top surface of the vault. The top slab should be designed to engage the perimeter of the structure to provide thrust resistance by way of a turned down edge that is anchored into the existing structure. Given the sensitivity of the existing structures to movement and tensile stresses, it may be appropriate to design the new slab and turned down perimeter beam as post-tensioned construction (see Figure 33) .

A new two-way slab that spans over the top of the existing soil and vault would be supported on new piers set directly above the existing piers. The new slab and piers would be designed to span over the existing vault structure, carrying new live loads directly to the line of existing pier supports. As noted above, the slab would engage the existing structure at regular intervals to restrain against thrust forces and settlements.

FOUNDATION EVALUATION AND REINFORCEMENT

The foundation system will need to be further evaluated to assure an undamaged state and sufficient bearing capacity. Areas of confirmed damage or settlement will require foundation reinforcement. Reinforcement may consist of a reinforced concrete column jacket and slab overlay that connects the column and surrounding foundation slab.

Vertical additions to the existing structure would most likely require new columns supported on new interior column footings. As noted above, foundations would likely involve micropiles or another system that could be drilled in to avoid disturbance of the sub soils supporting the history structure to remain.

PROTECTION AGAINST SETTLEMENT RELATED DAMAGE AND INSTABILITY

Strengthening, as noted previously, can likely be implemented without significant impact to the existing interior appearance. However, should that strategy be pursued, it will be necessary to investigate, repair and regularly monitor the interior of the re-used cells to assure that no hazardous conditions develop on the interior of the cells. Concrete repairs of all interior surfaces, including spall patching and crack repair will be required.

If individual sand filters are to be detached from adjacent filters and preserved, additional bracing would have to be carefully engineered and constructed sequentially along the divider walls in order to resolve unbalanced vault thrust, at both the roof and foundation levels.

FINDINGS, CONCLUSIONS & RECOMMENDATIONS

The following is a summary of key findings, conclusions and recommendations.

FINDING: UNSAFE CONDITIONS:

There are unsafe conditions at the site in several of the sand filters investigated.

- Conditions indicate settlement of a brittle heavy structure susceptible to sudden collapse.
- Open holes and areas of collapse are unmarked.
- Concrete ceilings in the unreinforced concrete structure harbor loose material that can fall without warning.

CONCLUSION:

No activity can safely take place on the propoerty in its current condition owing to the significantly unsafe conditions of the property.

RECOMMENDATION:

It is highly recommended to keep the general public from entering the facilities. Authorization to access the facilities should only be granted to individuals that are aware of the nature of the conditions described in this report and understand the risk of entering the facilities.

FINDING: FILL SOILS HAVE LEAD TO SETTLEMENT DAMAGE AND INSTABILITY:

The extent of observed damage is consistent with the fill depths estimated per grade elevations identified by the 1894 topographic survey. Based on RSA observations, it is evident that there is a correlation between areas constructed on backfill materials, the proximity to North Capitol Street and the severity of damage.

Two main patterns are apparent from the damage level classifications: The first is a correlation to the 1894 topographic survey, where filters built upon fill materials (over the lowest 1894 elevations) have tended to suffer increased damage. The second correlation is the proximity to and elevation relative to bounding streets, North Capitol Street in particular. At the south end of the site, North Capitol Street is well below the elevation of the top of the vaults. This increases the risk for lateral 'spread' of the perimeter wall of the cell.

The confluence of both factors is where the greatest damage overall is observed.

CONCLUSION:

A confluence of site related factors correlate with with locations where the greatest damage overall is observed.

RSA RECOMMENDATION:

Monitoring is recommended at the filters where bays have collapsed to verify the pace at which the end bays collapse is believed to be propagating in the years to come, and also at the filters that would be preserved. This would help determine the proper stabilization measures to be taken to prolong the life span of the slow sand filters.

Cells in the zone most clearly correlated to poor soils conditions should be the least preferred candidates for preservation.

FINDING: COMPROMIZED STABILITY:

The original design concept of the slow sand filter structure, which consists of balancing forces between adjacent vaults that are abutting at the apex, relies heavily on the stability of the end walls the capacity of the inverted groin vault foundation and the stability of the underlying soils. However, damage observed by RSA, including propagation of damage identified by previous surveys, demonstrates that the structure experienced the following:

- Vertical settlement of the foundations.
- Sliding of the foundations.
- Rotation of piers or abutments.
- Cracking and hinge formation in the vaults.
- Areas of vault collapse and imminent failure.
- At the current collapse propagation pace, the entire end bay of filter No. 24 are expected to collapse within five years and it is expected that this would trigger collapse mechanisms at adjacent interior bays and other filters.

CONCLUSION:

All work at this site must be performed with great care and the knowledge of these historic industrial structures and their sensitivity to settlement (vertical and lateral) from adjacent excavation or demolition:

RSA RECOMMENDATION:

All construction activity must incorporate:

- Monitoring to assure control of movements.
- Limitation to top side loading.
- Where excavation of vibration generating construction activity is proposed, special measures, including underpinning, rigid temporary retention systems, or temporary retrofit, will be necessary to protect the filters from further damage or collapse.
- Seismic evaluations will need to be made of any vaults to be restored or re-used, particularly in areas where the surrounding grade elevations do not provide for full restraint of the perimeter walls and/or do not permit the development of passive pressures.

FINDING: EXISTING FOUNDATIONS WILL LIMIT REUSE:

The footing configuration of the slow sand filter structural assembly is vulnerable to failure at low levels or superimposed loading. Calculations have indicated that the existing foundations have likely failed but are not accessible for visual inspections. This considerations will become the limiting factor for even the minimal re-use.

CONCLUSION:

Before adaptive reuse, the foundations will need to be exposed and soils tested.

RSA RECOMMENDATION:

The allowable bearing capacity of the sub-grade would have to be properly determined, and repairs made appropriate for the proposed restoration or re-use.

FINDING: MULTIPLE REUSE OPTIONS:

The findings and recommendations reported herein were developed and used as guidance for the master plan stage of this project. A broad range of general treatment options (pages 41-43) have been developed by the design team ranging from light use to heavy, programmed uses to new development at the site. Each are discussed in the report relative to these technical issues.

CONCLUSION:

Given the variation of conditions and opportunities across the site, a uniform approach should not be adopted.

RSA RECOMMENDATION:

A variety of approaches should be used to develop design solutions in response to the context of the site and the existing conditions.

Project design team needs to work with Structural Engineering team to determine the appropriate strategies to make best use and protect the historic resources.

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GLOSSARY/TERMINOLOGY

UNIFORM LIVE LOAD

Load from human activity resulting in uniform distribution of pressure on the surface of the structure.

UNBALANCED LOAD

Uneven distribution of loading due to the random nature of human activities.

KIPS

1,000 pounds of load.

COLD JOINT

An interface between separate pours of concrete where little or no bond may exist..

APPENDIX

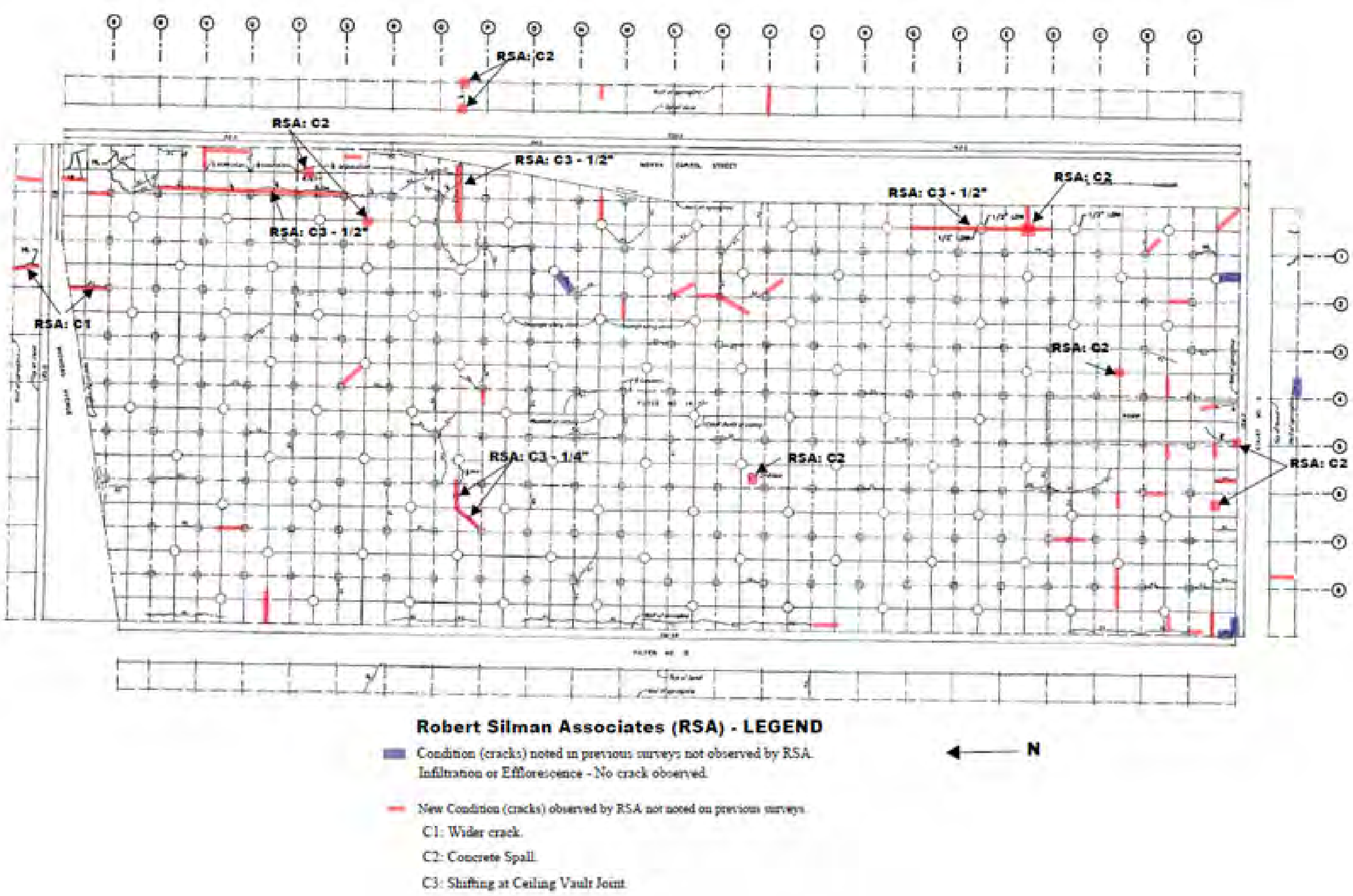
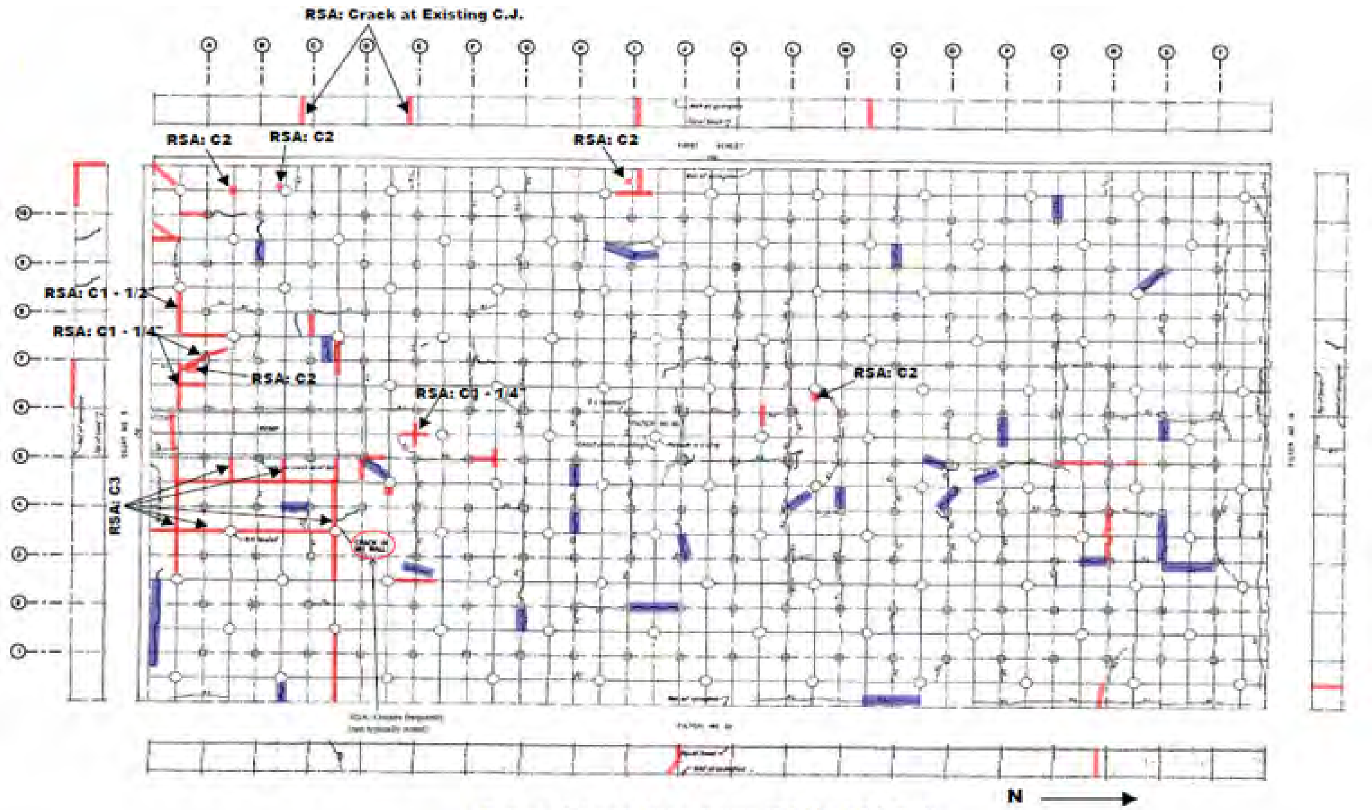


FIG. A.1 - FILTER NO. 14 RSA CONDITION ASSESSMENT FIELD NOTES



Robert Silman Associates (RSA) - LEGEND

- Condition (cracks) noted in previous surveys not observed by RSA. Infiltration or Efflorescence - No crack observed.
- New Condition (cracks) observed by RSA not noted on previous surveys.
  - C1: Wider crack.
  - C2: Concrete Spall.
  - C3: Shifting at Ceiling Vault Joint.

FIG. A.2 - FILTER NO.20 RSA CONDITION ASSESSMENT FIELD NOTES.

APPENDIX B
FIELD REPORT

SITE VISIT TO FILTERS WEST OF FIRST STREET

The following was noted:

BACKGROUND

Robert Silman Associates (RSA) visited the McMillan Reservoir facilities west of First street on May 23, 2012 from 1400h to 1630h to observe the conditions of the Slow Sand Filters. These facilities are still owned and operated by the Army Corps of Engineers (ACE) which currently has 24 people working on the site and maintains security as required by federal law after 9-11. Nine (9) of the twenty-nine (29) slow sand filters were located at the site, of which three filters were removed, two in the 1980's to make way for the Chemical building – housing the treatment process buildings that replaced the slow sand filtration process, and another filter was removed in the 1990's for another building, the Chloramine Building, to add another chemical process to the system. The slow sand filters were operational until 1984 when the new water treatment system was placed in service. RSA reviewed all drawings that ACE had on site that were relevant to the original construction. All of the drawings reviewed were already in RSA's possession.

FIG. B.1 - AERIAL VIEW OF MCMILLAN RESERVOIR (BING.COM)

FIG. B.2 - ENTRANCE TO FILTER No. 7.

FIG. B.3 - FILTER No. 7 BOARDWALK AND FILTRATION PROCESS DISPLAY.

FINDINGS

During our site investigation, RSA noted:

1. ACE staff recognizes the condition of the underground structures and does not permit any loading on the top of the Filters except for a landscape maintenance worker to cut the grass.

2. The cells are currently unused except for a few where materials have been stored in the past. These materials are planned to be removed.

3. One Filter (Filter #7) has been modified by the American Water Works Association to allow people on the water management industry to visit and see the original structures and materials that were used in the slow sand filtration operation. A 'boardwalk' was added to the end of the ramp, leading to a bay where sand collecting cleaning equipment was placed. Conduits and lighting was installed on the surface of the ceiling but was not used when we visited. A display was placed at the end of the ramp to explain the process (Figures B.2, B.3, and B.4).

4. In general the observed damage was similar to that found at the slow sand filters on the east side of First Street. There were no collapses.

5. Shifting at the top of the vaulted ceiling and concrete spalls were observed near the entrance ramp (Figure B.5) and also at the corners of the filters ceiling vaults (Figure B.6).

6. Hinge formations were observed at the end wall bays and also at interior bays (Figure B.7).

7. Rotations at the columns supporting the entrance barrel vault ceiling were observed. It was observed that some of these columns at the entrance ramp were retrofitted (Figure B.8).

FIG. B.4 - SAND COLLECTING CLEANING EQUIPMENT AT FILTER No. 7.

FIG. B.5 -SHIFTING AT THE COLD JOINT AND CONCRETE SPALLING.

FIG. B.6 - SHIFTING AT VAULTED CEILING JOINT.

FIG. B.7 - HINGE FORMATIONS ALONG THE END WALLS AND BETWEEN PIERS.

FIG. B.8 - RETROFIT TO COLUMNS AT THE ENTRANCE RAMP.

APPENDIX C

TYPES OF DAMAGE OBSERVED

The following is a brief summary of the types of conditions observed during RSA's recent investigation:

Vertical displacements were observed at the vaulted ceiling joints located at the top of the ceiling slab between adjacent concrete pours; displacements as high as 3 in. were observed in the vertical direction (Figure C.1); particularly at bays adjacent to the collapsed area which are undergoing significant damage. Based upon documentation review, vertical shifting arresters made of a thick metal block and thin square metal plates, were installed sometime before 1967. The intent of the retrofit repair is to clamp the vault together along crack lines limiting the potential for differential movement. It appears that this approach can be effective in the short term, as it does address the important need to maintain a continuous thrust line across the vaulted span. However, the approach does not address the root causes of the vault distress, which are likely related to support movements.

Horizontal separations as great as 1 in. were also observed at the vaulted ceiling joints located at the top of the ceiling slab between adjacent concrete panels. Horizontal shifting arresters made of a relatively slender metal plate bolted to the ceiling slab at either side of the joint located at the top of the ceiling were also added sometime before 1967. It was observed that these arresters are delaying the mechanism creating the horizontal separation at the joint, but are not keeping the ceiling vault from shifting horizontally as cracks had developed nearby the joints following a new load path (Figure C.2).

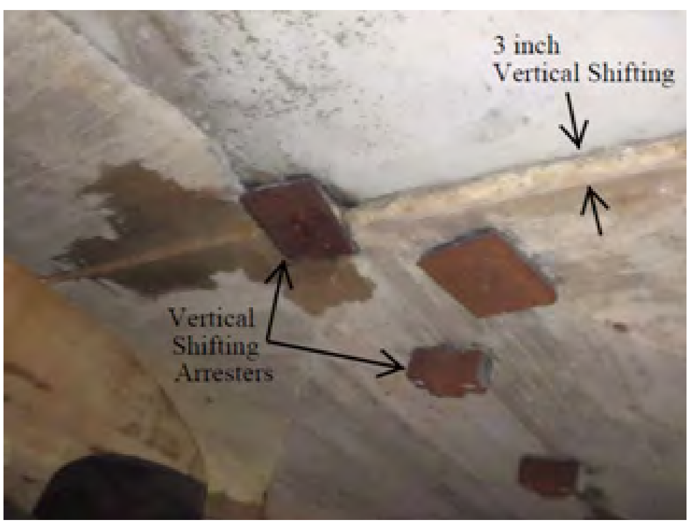


FIG. C.1 - VERTICAL DIFFERENTIAL DISPLACEMENT ARRESTORS AT INTER-PANEL JOINT.

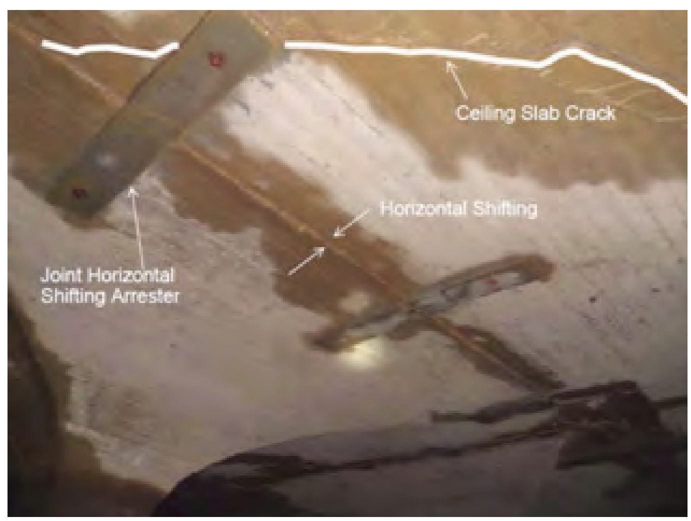


FIG. C.2- HORIZONTAL DISPLACEMENT ARRESTOR AT INTER-PANEL JOINT.

New crack formation and existing crack propagation at ceiling vault and walls. These cracks typically range from hairline cracks to 3/8 in. wide cracks at filters with significant damage and from hairline to 3/16 in. wide cracks at many locations of filters with moderate damage. It was observed that filters built below the grade elevation reported by the 1894 survey, on firm substrate, have sustained much less damage, mostly in the form of hairline cracking, than other filter built above the 1894 grade elevations, (Figure C.3).

FIG. C.3 - VERTICAL DISPLACEMENT AT CEILING SLAB JOINT AND VERTICAL DISPLACEMENT ARRESTERS.

Areas with collapsed vault slabs and areas where collapse appears imminent (Figure C.4). Collapsed areas were observed by RSA along the eastern bays adjacent to North Capitol Street of slow sand filters No. 19 and No. 24. It was apparent that severe damage continues to propagate, and other areas at these filters, particularly near the collapsed areas are at risk of imminent collapse.

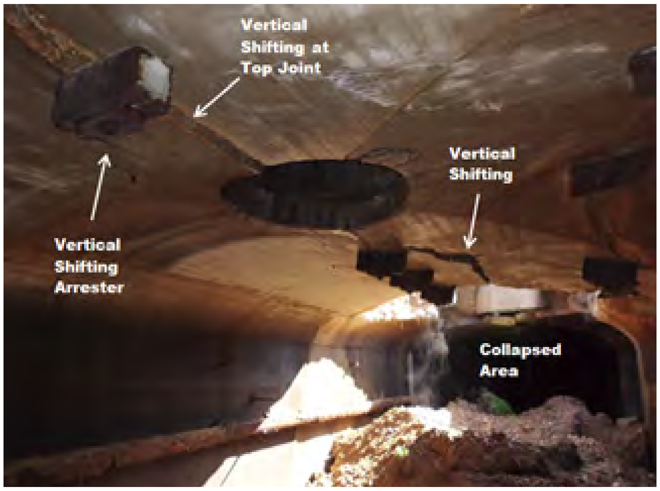


FIG. C.4 - CEILING VAULT DAMAGE NEAR COLLAPSED BAY.

Rotation at the intersection between pier top and vault slab (Figure C.5). This condition was typically observed at the portals where forces from the barrel vaulted ceiling most likely induced bending to the pier. This condition was also observed at piers located at the center of the filters away from the end walls and portal which indicates that this condition may also be caused by either unbalanced loads or differential settlement of the filter footing. The rotations observed are as large as five degrees in some instances.

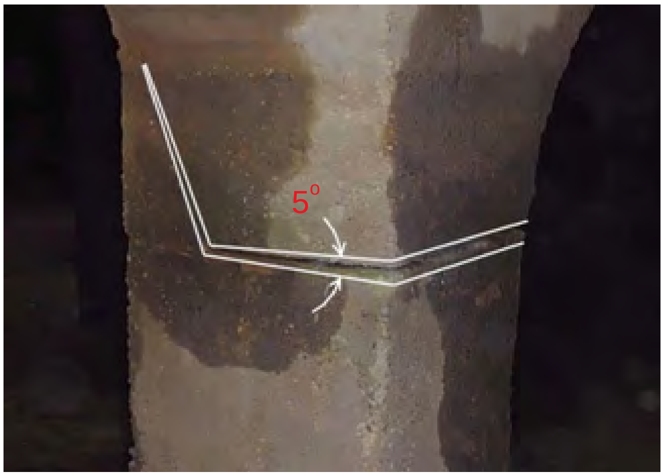


FIG. C.5 - TYPICAL ROTATION AT ENTRANCE RAMP PIER-TO-CEILING JOINT.

Vault slab cracks around the pier top (Figure C.6). This type of damage may be caused by different conditions triggering this two-way shear action, among these conditions are: differential settlement with respect to adjacent piers, and end wall movement, particularly at piers near the end walls, where this condition is most common.

FIG. C.6 - TYPICAL DAMAGE OBSERVED NEAR THE PIER.

Concrete spalls. Typical near the manholes (Figure C.7). RSA confirmed a concrete spall at the manhole near the southwest of the filter that was initially reported by CCJM. This same condition was observed by RSA at other locations. The concrete spall is consistent with anticipated high stress concentration caused by the manholes opening at the ceiling vault which creates a geometric discontinuity.

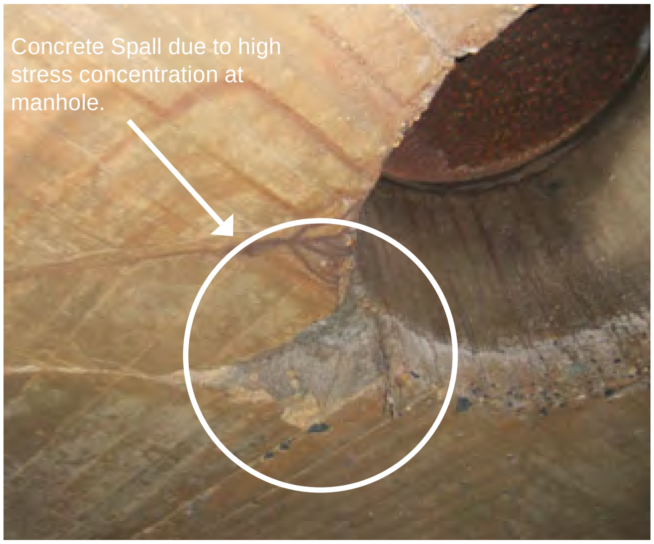


FIG. C.7 - SPALL AT UNDERSIDE OF VAULTED CEILING NEAR MANHOLE.

Cracks at the perimeter and divider walls at the springing line. These cracks are indicative of support movement and were observed at almost all filters investigated by RSA (Figure C.8).

FIG. C.8 - TYPICAL CRACK AT THE BARREL VAULT CEILING NEAR A FILTER END WALL.

Significant damage was typical at the portal entrance, ramp and adjacent bays (Figure C.9). Significant displacements at the ceiling vault joints were typically observed at bays near the portal. Cracking and also vertical displacement was observed at the underside of the portal ceiling. Rotation at the pier to portal barrel vault continuous abutment. At the filter gate, there is extensive damage caused by the gates anchor corrosion.

FIG. C.9 - TYPICAL DAMAGE AT ENTRANCE PORTAL.

APPENDIX D

DAMAGE DESCRIPTION AT REPRESENTATIVE FILTERS

MODERATE DAMAGE

Slow Sand Filter No. 13

Bays next to this filter's walls were investigated in detail by RSA. In general, the damage observed is consistent with that reported by the Army Corp of Engineer survey maps with added noted by CCJM. New cracks were identified between piers at west wall bays, one location on the north side wall, and one location on the court yard wall. Additional to the ½ inch vertical shifting at the ceiling slab joint reported by CCJM near the entrance ramp, RSA observed an additional bay with new vertical shifting of about 2 ½ inch at a ceiling slab joint also located near the entry ramp.

RSA confirmed a concrete spall at the manhole near the southwest of the filter that was initially reported by CCJM. This same condition was observed by RSA at other locations. The concrete spall is consistent with the high stress concentration caused by the manholes opening at the ceiling vault which has created a geometric discontinuity.

Slow Sand Filter No. 14

A detailed assessment of the entirety of Filter No.14 was performed, since this filter, along with Filter No.20, has been proposed to remain in the development plans. The moderate damage to Filter No. 14, described below, is similar to that observed during RSA's condition assessment investigation at Filter No. 20.

Most of the deterioration noted by the Army Corp of Engineers survey in 1967 and CCJM survey in 2000 was verified. New deterioration in addition to the documentation during the previous two surveys was mostly found near the courtyard wall and also near the perimeter filter walls. Very few sporadic new cracks were observed on the vault slab bays located away from the walls.

New damage, observed by RSA's recent investigation, accounts for a significant percentage of the total damage to date, estimated between 15%-20% of the existing damage. Most of the new damage includes cracks on the ceiling vault slab, small spalls, and shifting at the joint located at the top of the ceiling vault. The findings are summarized graphically in Appendix A.

Near the court yard walls of Filter No. 14 there was little damage reported between the 1967 and 2000 surveys but RSA's recent investigation revealed that this portion of the filter has now sustained additional significant damage in the last 12 years. Almost 50% of the total existing Filter No. 14 damage to date was observed at the bays near the courtyard wall.

Slow Sand Filter No. 18

In general the damage observed by RSA at Filter No. 18 is also consistent with that identified by the two preceding surveys. The only new damage identified is pier rotation at the entry ramp first row of piers, which appears to be leaning over from thrust forces at the portal barrel vault ceiling (Figure C.5).

SEVERE DAMAGE

Slow Sand Filter No. 22

The damage observed by RSA at this filter is consistent with the damage reported by previous surveys. This filter has sustained extensive damage near the courtyard wall, particularly at the southeast corner where the 1894 topographic survey shows an area with a grade elevation below the filter footing elevation at 155 ft. The severe damage observed by RSA is confined to this area at the southeast corner demarcated by the 155 ft grade elevation originally identified in 1894.

Slow Sand Filter No. 26

RSA performed a detailed investigation near the courtyard wall where new cracks, mostly hairline-width cracks, were found. RSA observations at other areas of this filter confirmed the damage identified by the Army Corps of Engineers in 1967, which is now more severe at the northwest corner of this filter. Similar to the damage pattern observed at Filter No. 22 (described above), the most severe damage observed at this filter is also consistent with the grade elevation identified by the 1894 topographic survey. This severe damage also follows the 155 ft. elevation contour line identified by the 1894 survey, which indicated settlement toward the southeast corner of the Filter at the line. RSA observed horizontal separation arresters at the ceiling vault joint. These horizontal arresters were made with a relatively slender metal plate bolted to the ceiling slab at either side of the joint and positioned at the apex of the ceiling. The arresters appear to be delaying the mechanism which creates the horizontal separation at the joint, but does not keep the ceiling vault from shifting horizontally at the cracks that have developed near the joints, following a new load path.

Metal straps were observed at the entry ramp piers of this filter. It appears that the intent of these straps was to defend the pier from the damaging effects of the lateral thrust coming from the portal barrel vault. Similar retrofit configurations were observed at other filters.

Slow Sand Filter No. 27

The damage observed by RSA is consistent with that identified by the preceding condition assessments. RSA identified a rotation at the intersection between piers and the ceiling vault. These piers are located at the center bays in front of the portal ramp. This is a condition that, although was observed at the portal ramp of other filters and with a more sporadic occurrence than what was observed at Filter No. 27, indicates that this filter was founded on an area where the substrate varies from firm substrate soils to pockets of soft substrate soils.

Slow Sand Filter No. 28

A detailed investigation was performed only at the divider wall bays on the east and west sides of this filter and also at the south end wall located next to Channing Street. At bays near the south end wall, RSA identified an increasing vertical displacement at the vault slab apex between grids No. 6 and No. 10, where one side of the joint dropped an additional ½ inch since the last condition assessment was performed by CCJM in 2000. New cracks were observed near the east side divider wall, particularly near the top of the pier at the face in front of the wall, the condition was observed at four locations. Also, deterioration of the vault slab joint at the apex was observed at two locations near the manholes at the east end wall. Two piers at the first row of the portal ramp appear to be leaning outwards, apparently resulting from the barrel vault ceiling thrust.

Slow Sand Filter No. 29

The south side end wall bays that are near Channing Street were thoroughly investigated. The damage reported by previous surveys was confirmed and new damage was found. New damage included vertical cracking at the wall near the southeast corner of the filter, and an area at the vault slab apex at the manhole in the middle of this bay appears to be on the verge of spalling. RSA also observed an imminent spall at a recent crack on the ceiling slab. A few cracks near the northwest corner of the filter appear to be wider than the previously reported hairline-crack width. RSA also identified a drop of about ¼ in. of one of the two vault joints at the northwest corner bay of this filter as new damage. The most substantial damage at this filter was observed at the east end wall bays, which is next to North Capitol Street.

COLLAPSED VAULTS

Slow Sand Filter No. 19

RSA observed a collapsed area near the southeast corner of the filter that extended over one bay at the end wall. RSA's investigation observed other bays near the collapsed area in poor condition with propagation of the current collapsed area to adjacent bays appearing imminent.

These bays adjacent to the collapsed area are undergoing significant damage in the form of ½ inch to three-inch. vertical displacements at the vault slab joints. Based upon documentation review, vertical shifting arresters, made of a thick metal block and thin square metal plates, were installed sometime before 1967. The intent of the retrofit repair was to clamp the vault together along crack lines limiting the potential for differential movement. It appears that this approach could be effective in the short term because it addresses the important need to maintain a continuous thrust line across the vaulted span. However, the approach does not address the root causes of the vault distress which are likely related to support movements.

Slow Sand Filter No. 24

As mentioned before, of the two filters with collapsed ceiling vaults, Filter No. 24 has the largest collapsed area, representing the most extensively and severely damaged assessed filter. Based on the topographic contour lines of 1894 survey, the portion of the collapse filter was built on top of one of the deepest landfills in the area. Only Filters No. 27 and No. 28 are built on deeper fills, but the depth of the fill combined with the proximity to North Capitol Street appears to be the major factors causing the accelerated the damage of the ceiling vault at Filter No. 24.

APPENDIX E

VAULT ANALYSIS

The method of analysis implemented to estimate the capacity of the end bay vaults is based on calculating the compressive stress at any given segment of the unreinforced concrete vault by static structural analysis and determining the location of the thrust line by a graphic method.

The concept of a thrust line or true line of resistance [Kidder-Nolan, 1921], or, the resultant compressive force passing through a vaulted structure from span to support, is used to visualize forces within the structure (Figure E.1). For a given vault structure, the thrust line will shift positions within the cross section as loadings are changed. Generally, vault forms are designed to accommodate the constant load case of self-weight and superimposed dead load (soil over vault). Accommodation for variable loads, such as unbalanced or concentrated surface loads, is made by varying the vault thickness. If the thrust line remains within the middle third of the concrete vault thickness, the vault is considered stable. The 'middle third rule' was in fact originally an elastic concept concerned with the avoidance of tension [Heyman, 1999]. On the other hand, when the thrust line extents outside of the middle third, the structure is subjected to tensile stress, cracking. As the thrust line approaches the outer surface of the vault, the compressive stresses become high and an unstable condition is rendered. Once unstable, a significant hinge forms which can allow rotation and lead to the formation of a collapse mechanism.

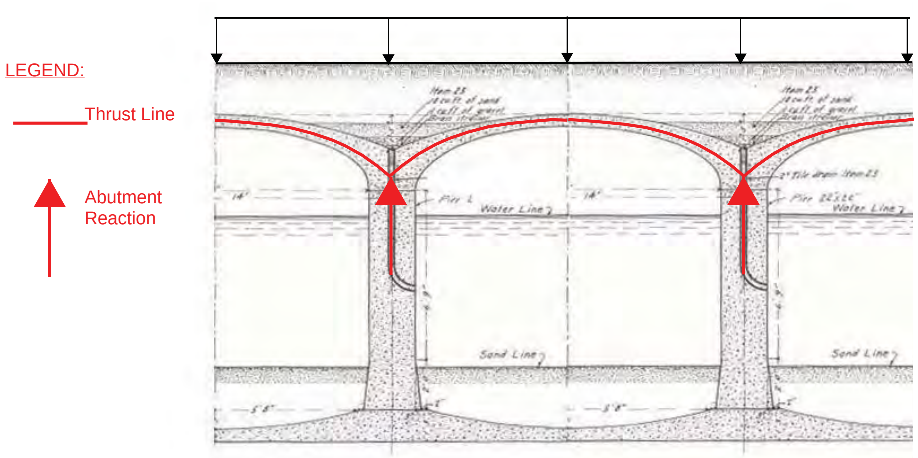


FIG. E.1 - LOCATION OF THE VAULT THRUST LINE UNDER BALANCED LOADING CONDITIONS

In every vault there can be traced a catenary curve within the wall thickness, described by the thrust force lines which represent the force of gravity acting on vault segments. Using the assumption that the applied loads (soil weight and self-weight of the concrete vault) are uniformly distributed, the first step of the analysis is to divide the arch into a series of similar segments and convert the uniform loadings into singular concentrated loads placed at the center of each segment. These segments are identified in figure E.2 as P1 thru P5 and P1' thru P5'. For symmetrical loading it is only necessary to consider half of the ceiling vault thrust through the ceiling vault.

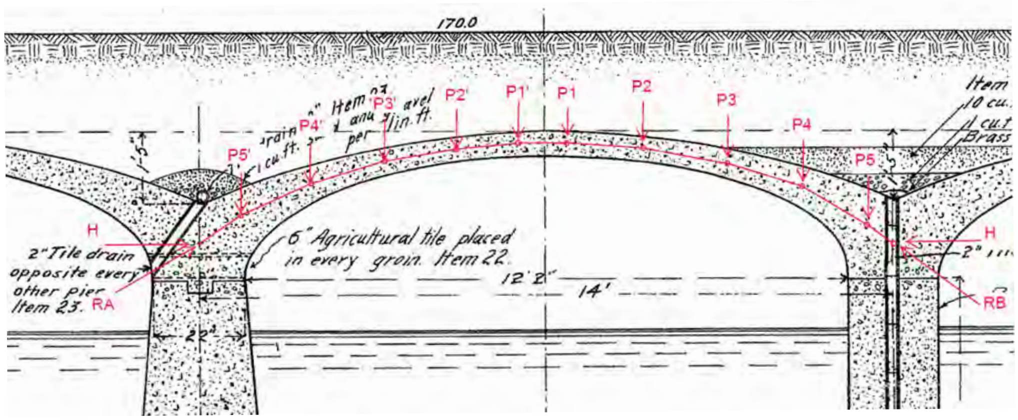


FIG. E.2 - GRAPHIC ANALYSIS METHOD SEGMENTAL DIVISION OF UNREINFORCED CONCRETE VAULT

Generally, each arch segment exhibits three forces: the thrust which is received from the segment above it; the applied gravity loadings (P1, etc.); and the reaction of the resultant thrust. The points where these forces are applied are the pressure centers and the connecting lines between each segment are the lines of resistance or thrust lines.

The ceiling vault is evaluated to the position of the thrust line to maintain the ceiling vault in compression and to estimate the thrust at a given segment pressure center by catenary action between ceiling vaults segments where the slope and magnitude of the thrust at a determined segment of the ceiling vault is graphically estimated. Accordingly the vertical reactions at the support as well as the horizontal thrust can be calculated by structural static analysis.

When the thrust line does not coincide with the geometric center line of the vault, as depicted in Figure E.3 internal stresses are redistributed.

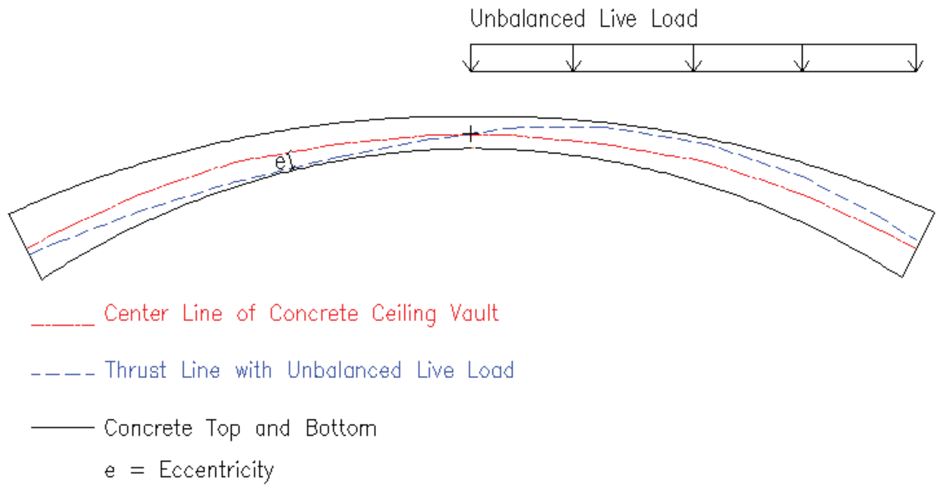


FIG. E.3 - UNBALANCED LIVE LOAD ANALYSIS CASE

Figure E.4 depicts the typical stress diagram for different cases of eccentricities between the center line of the vault and the thrust line. Under ideal conditions, that is balanced loading, uniform soil bearing capacity, and no end wall or footing movement, the thrust line coincides with the geometric center line of the vault and there is uniform compression stress (Figure E.4a). The realities of uneven soil bearing capacities, unbalanced loads, or perimeter wall movement make the thrust line move away from the geometric center line of the vault, creating eccentricities 'e'. As the eccentricity between the center line of the vault and the thrust line increases, the compressive stresses increase on the side where the thrust line is located and decreases on the other (Figure E.4b, E.4c).

When the thrust line falls outside of the middle third of the vault section, the face opposite to the thrust line begins to experience tensile stresses. Because unreinforced concrete is very weak in tension, cracks will soon form, effectively reducing the thickness of the cross section that can carry load (Figure E.4d).

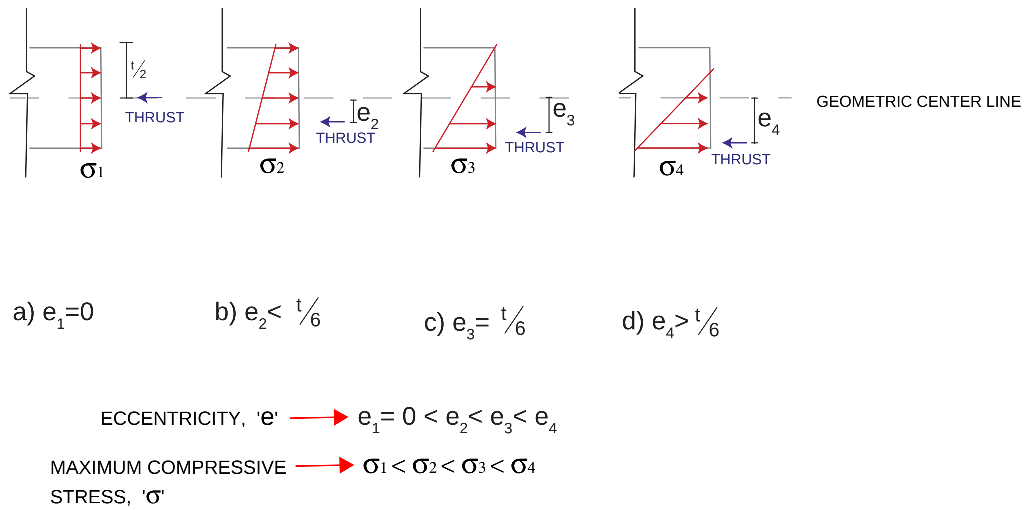


FIG. E.4 - STRESS DISTRIBUTION DUE TO ECCENTRIC LOADING

The typical interior bays are supported in such a way that two-way action results. When loaded, the load path of the vault takes both principal directions, unlike the half barrel vault where internal forces curve in one direction. Accordingly, interior bay analysis is based on a simplified method that assumes a groin vault assembly made up of two intersecting segments of paraboloids. The method entails calculating a diagonal horizontal thrust with components in both directions. The structural behavior of the interior bays is similar to the end bays, but with the benefit of two-way force distribution and added stiffness due to the double curvature of intersecting barrel arch forms along the thrust-carrying groins. Because of these structural advantages of the interior bays, and the prevalence of damage in the exterior bays, the analysis focuses primarily on the end wall bays as the limiting component in the overall vault system.

The following load conditions were analyzed for the exterior bays:

- Uniform distributed load
- Unbalanced loads
- Support displacement – spreading supports
- Support displacement – support settlement

The loads applied to the vault include:

- Self-weight of the concrete.
- Soil above vault.
- Surface live load.

The allowable compressive strength of the unreinforced concrete was estimated to be 22.5% of the ultimate compressive strength (f'c), which was common practice at the time the slow sand filters were designed and built. With an ultimate concrete strength (f'c) estimated at 4000 psi, an allowable compressive stress (f'c) equal to 900 psi was estimated. The assumed 4000 psi specified concrete strength is then considered a more realistic f'c for the analysis of this type of structure. The tensile capacity of the unreinforced concrete was assumed to be negligible for the purposes of this analysis, which is consistent with the general design approach for unreinforced concrete both historically and today.

Under uniformly distributed loads the ceiling vault is intended to sustain pure compressive axial stresses. The horizontal reactions from the resultant thrust forces at the pier to ceiling vault interface, 'T1h' and' T2h' depicted in Figure E.5 are equal on either side of the pier.

Given that the surface live loadings are typically transient and not uniform; this idealized scenario represents the upper limit of potential capacity. Preliminary analysis results demonstrate the vault configuration provides adequate capacity to sustain significant uniform live loads in addition to permanent loads. RSA employed traditional equilibrium methods to calculate the structural capacity of both the end bays and interior bays. The end bay analysis was based on a thrust line calculation using graphical statics.

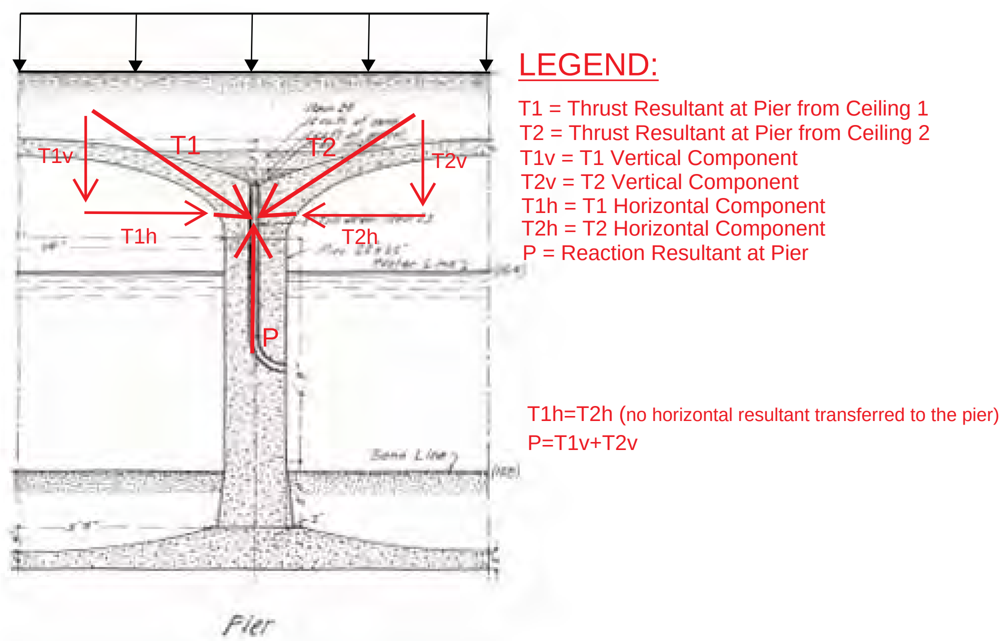


FIG. E.5 - FREE BODY DIAGRAM AT PIER WHEN VAULT IS SUBJECTED TO BALANCED LOADS

The calculation of the thrust translates to a compressive stress in the concrete, which is compared to the allowable stress for the material. Under the idealized case of uniform distributed loading, analysis results indicate that the end vaults can reach an allowable uniform surface live load capacity of approximately 2,800 psf., before reaching the unreinforced concrete allowable compressive strength estimated at 900 psi. This value is much greater than typical code-required live load. However, as noted above, this represents a theoretical upper limit of capacity which the reality of unbalanced loadings, irregular geometry, pier settlement, or wall movement begins to quickly reduce.

Similar to the exterior bay analysis, the calculation of the horizontal thrust at interior bays translates to a compressive stress in the concrete, which is compared to the allowable stress for the material. Under the idealized case of uniform distributed loading, groin vault analysis results indicate that the interior vaults reach a theoretical allowable surface live load capacity of approximately 2,840 psf, which is almost identical to the calculated live load capacity of the end bays following the graphical statics method.

In keeping with the variable nature of live loads, the second case considered in the slow sand filter analysis is to assume unbalanced live loads by applying a uniform live load at only half the span of the vault. The variation of the load from the base uniform assumption shifts the thrust line away from the center of the vault section, resulting in increasing compression stresses and potential cracking.

Under such unbalanced load conditions, the horizontal reactions from the resultant thrust forces at the pier to ceiling vault intersection are opposite in direction but not equal in magnitude on either side of the pier, thus there is transfer of lateral load to the pier, which undergoes compressive stresses and bending stresses (Fig. E.6). Subjected to bending, the piers are susceptible to rotation and cracking.

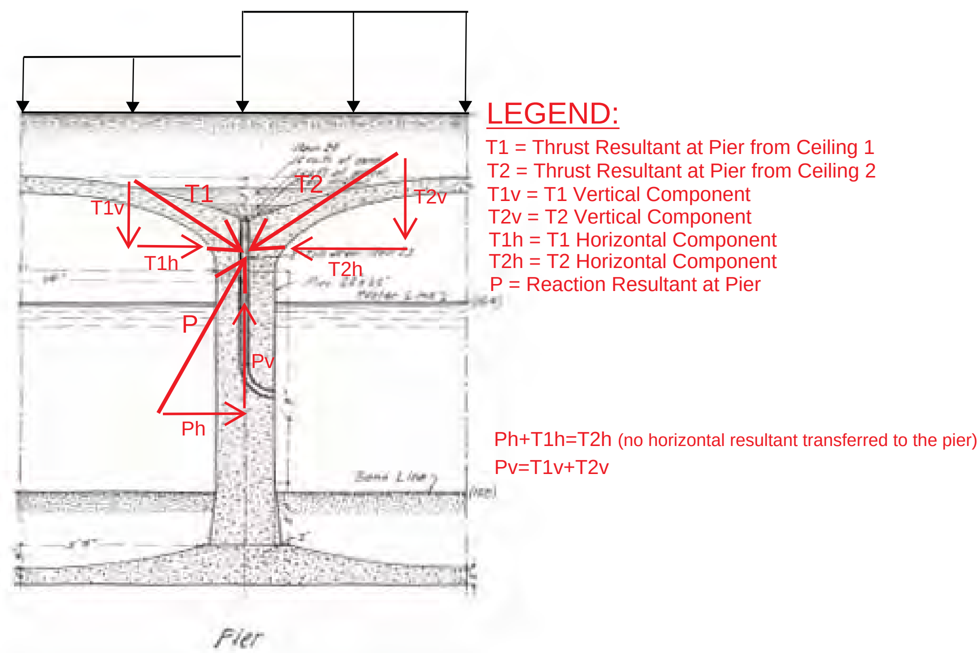


FIG. E.6 - FREE BODY DIAGRAM AT PIER WHEN VAULT IS SUBJECTED TO UNBALANCED LOADS

Analysis results for this case indicate that the end vaults can reach a theoretical allowable surface live load capacity of approximately 900 psf, before reaching the unreinforced concrete allowable compressive strength estimated at 900 psi. This value is still greater than common code-required live loads, however, other adverse conditions such as pier settlement, or wall movement would quickly reduce capacity even more. RSA performed stability investigations to estimate the influence of these conditions on upper bound load capacity estimates.