Appendix 7: Structural/Geotechnical Engineering Evaluation of the McMillan Filter Site

ZONING COMMISSION
District of Columbia
CASE NO.13-14
EXHIBIT NO.93

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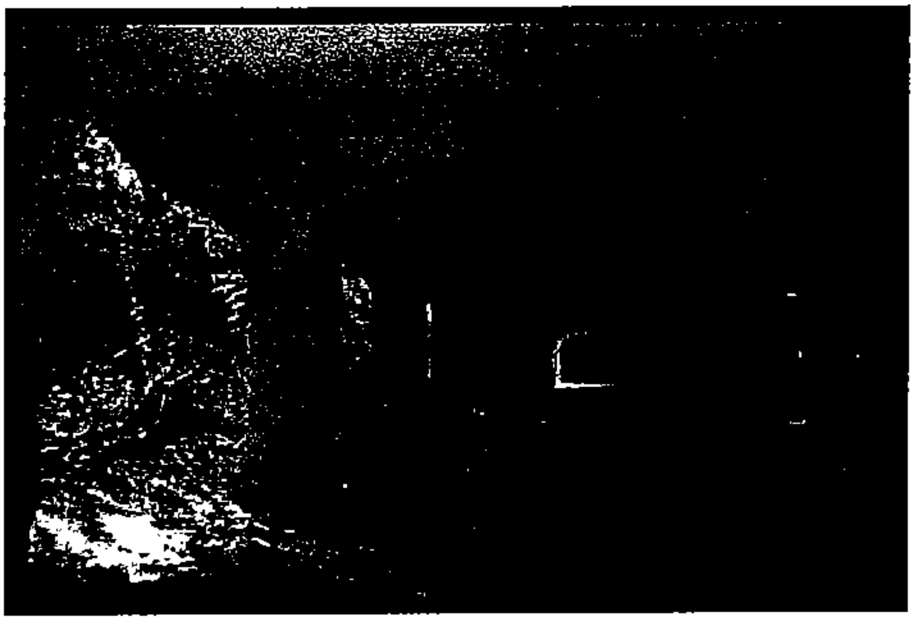
STRUCTURAL/GEOTECHNICAL ENGINEERING EVALUATION of the McMILLAN FILTER SITE

Prepared for the
D.C. DEPARTMENT OF HOUSING
AND COMMUNITY DEVELOPMENT



AUGUST 25, 2000

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Structural/Geotechnical Engineering Evaluation of the McMillan Filter Site

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Structural/Geotechnical Engineering
Evaluation of the McMillan Filter Site

EXECUTIVE SUMMARY

C.C. Johnson & Malhotra, P.C. along with it sub-consultant E2CR Inc., performed a field survey that evaluated the structural conditions of 20 filters cells at the McMillan Filter Site. Geotechnical data was also obtained.

The purpose of gathering the field data was to determine what would be required to allow the site to be developed as open space, a site for single-story buildings, or a site that could support 4-story buildings.

Once the project team reviewed the field data, it was obvious that the cells varied significantly in their physical condition. Therefore the cells were categorized into three types of cells:

Type I: Significant Deterioration: Potentially dangerous. Cracks of to two inches and larger, joint separation of up to four inches, and collapsed portions of the ceiling. A significant amount of structural damage has occurred since a 1967 survey performed by the Corps of Engineers. The cells within this category are mostly in the south east corner of the site and were constructed on earth fill.

Type II: Moderate Deterioration: Cracks up to one inch and joint separations of up to two inches. Joints were increasingly loose and cracks appear wider and longer than original survey. These defects were generally noted in the vicinity of the exterior walls, and it was only in the vicinity of the exterior walls that significant deterioration was observed to have occurred since the 1967 survey.

Type III: Stable: There are only hairline cracks and less than 1/8 inch joint separations. No significant deterioration has occurred since the 1967 survey. The cells within this category are generally the interior cells

Table 8.1 Summarizes the costs for the various combinations of type of cell and type of development. Development costs range from $440,000 per cell to 2.6 million dollars per cell.

TABLE 8-1 STRUCTURAL/GEOTECHNICAL REQUIREMENTS FOR DEVELOPMENT OF McMILLAN FILTER SITE			
DESIGNATION	CELL DESIGNATION		
	TYPE I	TYPE II	TYPE III
CELLS	19,22,23,24,26,27,28,29	10,11,12,13,14,15,20,25	16,17,18,21
DESCRIPTION	Built on fill, active cracking, some failures, additional failures likely	Built in cut areas, active cracking observed around perimeter	Interior cells, built in cut areas, no apparent new cracking has occurred in last 30 years
CONDITION	Unstable, Unsafe	Stable except at edges	Stable
OPEN SPACE			
PRESERVE FILTERS			
Struct. Req'ments	Not Feasible	Reinforced top slab and exterior walls	Reinforced top slab
Geotech. Req'ments	N/A	None	None
Cost Estimate	N/A	$2,020,000 per cell	$1,790,000 per cell
DEMOLISH FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	None	None	None
Cost Estimate	$860,000 per cell	$860,000 per cell	$860,000 per cell
FILL FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	None	None	None
Cost Estimate	$440,000 per cell	$440,000 per cell	$440,000 per cell
SINGLE STORY BUILDING			
PRESERVE FILTERS			
Struct. Req'ments	Not Feasible	Reinforced top slab, columns and exterior walls	Reinforced top slab and columns
Geotech. Req'ments	N/A	Spread footers	Spread Footers
Cost Estimate	N/A	$2,250,000 per cell	$2,020,000 per cell
DEMOLISH FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	Pile Foundation	Spread Footers	Spread Footers
Cost Estimate	$1,330,000 per cell	$1,240,000 per cell	$1,240,000 per cell
FILL FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	Pile Foundation	Spread Footers	Spread Footers
Cost Estimate	$920,000 per cell	$790,000 per cell	$790,000 per cell
FOUR STORY BUILDING			
PRESERVE FILTERS			
Struct. Req'ments	Not Feasible	Reinforced top slab, columns and exterior walls.	Reinforced top slab and columns
Geotech. Req'ments	N/A	Spread Footers	Spread Footers
Cost Estimate	N/A	$2,560,000 per cell	$2,330,000 per cell
DEMOLISH FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	Pile Foundation	Spread Footers	Spread Footers
Cost Estimate	$2,000,000 per cell	$1,370,000 per cell	$1,370,000 per cell
FILL FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	Pile Foundation	Spread Footers	Spread Footers
Cost Estimate	$1,610,000 per cell	$920,000 per cell	$920,000 per cell

CHAPTER 2

BACKGROUND

In 1852, the U.S. Congress authorized the U.S. Army Corps of Engineers to develop a drinking water system for Washington, DC. Construction began in 1853 and the first of the system was placed in service in 1864. The early construction was delayed four times due to funding shortages and the Civil War.

The early facilities consisted of a conduit from Great Falls on the Potomac River to a receiving reservoir near Little Falls (now known as the Dalecarlia Reservoir); a distribution reservoir (now known as the Georgetown Reservoir); and a system of cast iron pipe to distribute the water from the Georgetown Reservoir to the City. Most of the original facilities remain in service today.

Many problems were encountered with the original water system. These included: 1) Inadequate pipe sizing from the Georgetown Reservoir to provide sufficient volumes of water. 2) Insufficient water pressure in many parts of the city and 3) The quality of the water was not always good, particularly during periods of heavy rainfall. As a result, the system was expanded to include a tunnel connecting the Georgetown Reservoir to a new reservoir, (now known as the McMillan Reservoir) located on the grounds of Howard University. The tunnel and reservoir were placed in service in 1903. This solved the inadequate water quantity and pressure problems, but it did not improve the quality of the water.

To improve the quality of the water, twenty-nine (29) slow sand filters were constructed adjacent to the reservoir. The construction of the filters began in 1903 and were placed in service in 1905. At the time of completion, the filters were the largest slow sand filtration installation in the country. These filters remained in service until 1986 when a rapid sand filter facility was placed in service.

Twenty of the original 29 filters (or filter cells) were constructed east of First Street, N.W., in the block bounded on the north by Michigan Avenue, the west by North Capitol Street and on the South by Channing Street. When the new rapid sand filters went into service the Corps of Engineers declared the property and facilities east of First Street to be excess. The District of Columbia Government purchased the property at that time. Appendix A shows the overall site location and cell numbering system.

Prior to construction of the filters, the site for the filters was of gently rolling topography. There was a ridgeline running in a northeasterly direction across the site at elevations just over 180. There was also a stream that cut across the southeast corner of the site. The elevation of the stream was as low as 120. The bottom of the filters is at elevation 155; therefore approximately one third of the facility was built upon earth fill, which was as much as 35 feet deep. The other two thirds of the site were built in cut areas as much as 25 feet deep. Figure 2-1 depicts the fill under a portion of the filters.

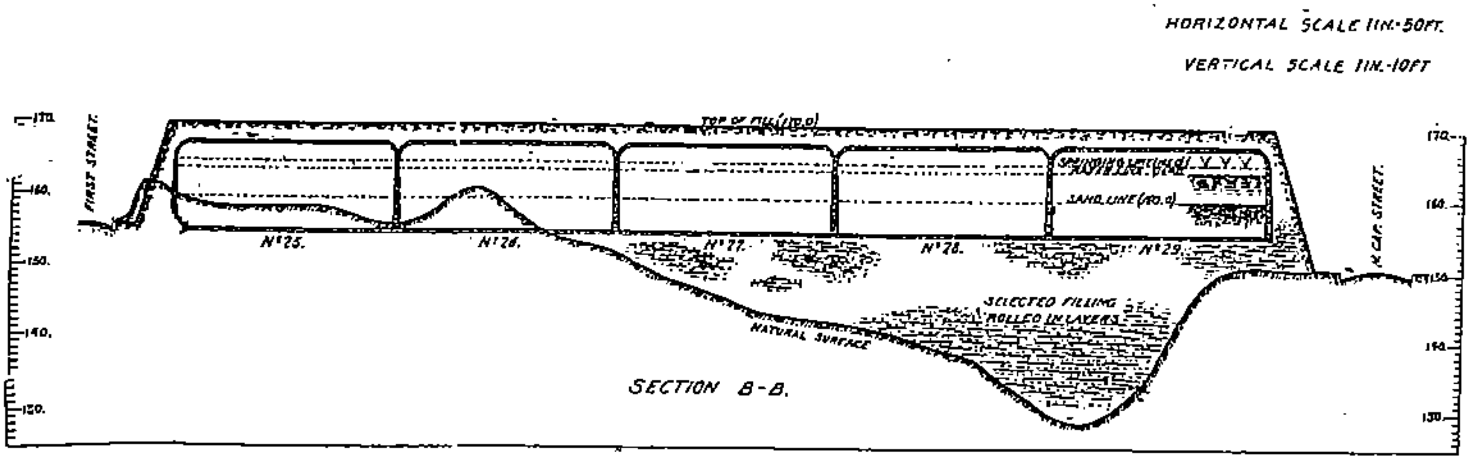


Figure 2-1
. Extract From 1902 Design Drawings

Each filter cell is approximately 150 feet in the east-west direction and 300 feet in the north-south direction. Each cell occupies approximately 45,000 sq. ft. Several cells adjacent to Michigan Avenue, however, do deviate in length and width.

The filter cells are buried concrete vault-like structures constructed entirely of non-reinforced concrete. The structural design consisted of an arch shaped top slab supported on 22 inch square columns spaced 14 feet on center. At the high point of the arches, centered between the columns, were manhole openings. These manholes were for light, ventilation, and for placing clean sand into the filters. The walls of the filter cells were of various configurations varying in widths from three feet to seven feet. Figure 2-2 illustrates the existing cell configuration.

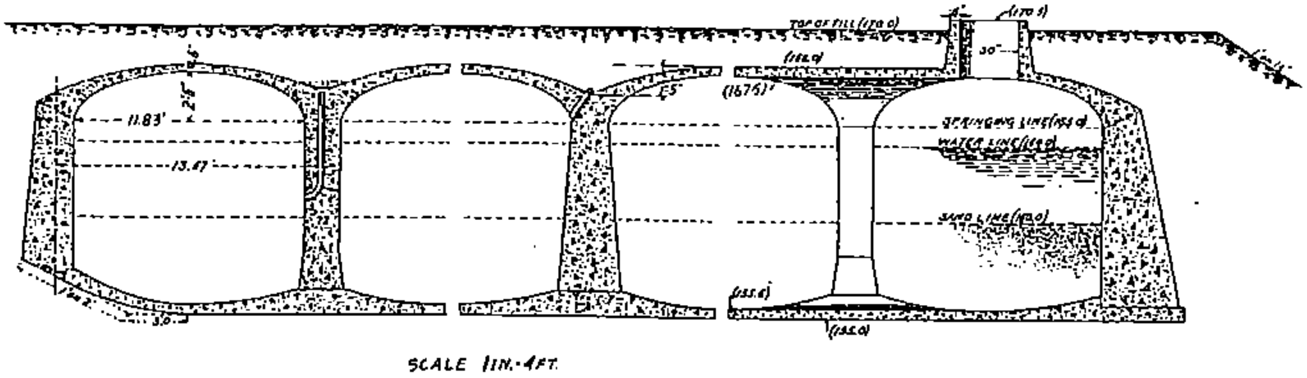


Figure 2-2
Extract From 1902 Design Drawings

The roof of each cell is approximately elevation 168 and is covered with approximately 2 feet of soil; thus making the existing surface grade over the filter cells at elevation 170. The cells have a floor to ceiling height of about 12 feet at the highest point in each cell. The floor slab in each cell is approximately elevation 155 and is 6 inches thick at its thinnest point. The floor is covered with about a foot of gravel overlain by about 3 ½ feet of sand (the elevation of the top of sand is 160).

Two east-west courtyards approximately 100 feet wide traverse the width of the site dividing the site into thirds. The courtyards are paved with 6 inches of concrete and have a surface grade of approximately elevation 163. Concrete and masonry sand bins, sand washers, and regulator houses are present in each courtyard. Numerous underground pipes and conduits also present under each courtyard extending up to 19 feet below grade.

The grades around the site vary from a high along Michigan Avenue of about elevation 180 to a low Channing St. of about elevation 150.

In July 2000, CCJM performed a Phase I Environmental Site Assessment of the McMillan Filter Site, in conformance with the scope and limitations of American Society for Testing and Materials (ASTM) Practice E-1527-94. The assessment revealed no evidence of recognized environmental conditions in connection with the McMillan Filter Site. A copy of the Environmental Assessment is included in Appendix D.

CHAPTER 3

FIELD SURVEY

The field condition survey consisted of a detailed visual inspection of each of the 20 filter cells. The condition survey was performed between June 26 and July 11, 2000. Engineers from CC Johnson & Malhotra, P.C. walked through each of 20 the filter cells to inspect the condition of concrete columns, walls, and ceiling. The results of the survey are presented in the following three sections – an overview of the survey process, a general description of what was observed, and a detail summary of the condition of each cell.

Overview

CCJM utilized as a base map, a 1967 the Corps of Engineers condition survey of all of the filters. Individual drawings of each filter cell map the location and size of all cracks and other defects. CCJM utilized that condition survey as a base line for the field survey to identify where new cracks developed, the change in existing cracks, areas of collapse, joint separations, and any new structural deficiencies. Drawings showing the results of the field survey are included in Appendix A.

General Observations

Most structural deficiencies were observed in the arched ceiling in the form of cracks, sagging ceilings, and joint separations. The damage to the ceiling ranges from hairline cracks to the collapse of large areas.

Perimeter walls and partition walls (common wall between two cells) have only minor hairline to 1/8" cracks and the construction joints appear to be in good condition. Several of the perimeter walls had tree or plant roots penetrating the cracks and in some cases, light through the cracks could be seen.

Most concrete columns or piers appear to be in good shape. A notable exception is in cell No. 27 where columns near the entrance ramp are so severely displaced that steel collars and rods were used to confine them. The ceiling continues to crack and the joint separations are wider.

An 8' x 8' area (test pit # 2 – see Appendix B) on top of filter cell No. 23 was excavated to inspect the condition of the top of the ceiling slab. There is a 1/8" wide crack indicated on the plan of condition survey. (This crack is presently a ¼" wide crack). After excavation, it was revealed that this crack is continuous through the depth of the ceiling slab and is also ¼" wide on the topside of ceiling slab.

The topside of the ceiling appears to be curved as shown on plan and the two adjacent ceiling slabs butt together at the crown of the curve. The topside of ceiling slab appears to be in good shape except for the crack, probably caused by the long term loading of the two feet of fill.

Another 8'x8' area (test pit #1, see Chapter 5) on top of filter cell No. 18 revealed a similar condition of top face of ceiling slab.

Specific: Observations for Each Cell

Based on the results of the field survey each of the twenty cells was placed into one of three categories as defined below:

Type I:	Significant Deterioration: Potentially dangerous. Cracks of up to two inches and larger, joint separations of up to four inches, and collapsed portions of the ceiling. A significant amount of structural damage has occurred since the 1967 survey. The cells within this category are mostly in the south east corner of the site and were constructed on earth fill.
Type II:	Moderate Deterioration: Cracks up to one inch and joint separations of up to two inches. Joints were increasingly loose and cracks appear wider and longer than original survey. These defects were generally noted in the vicinity of the exterior walls, and it was only in the vicinity of the exterior walls that significant deterioration was observed to have occurred since the 1967 survey.
Type III:	Stable: There are only hairline cracks and less than 1/8 inch joint separations. No significant deterioration has occurred since the 1967 survey. The cells within this category are generally the interior cells.

A drawing of the site depicting the locations of these cell categories is included in Appendix A.

Following are detailed descriptions the field inspection observations for cells 24, 23 and 19. These three cells, all Type 1, show the biggest change of conditions since the 1967 survey. Table 3.1, which follows the discussion of these 3 cells, summarizes the findings for 20 cells. Detail drawings that map the defects in each of the cells are in Appendix A.

Cell No. 24

Ceiling

This is the only cell where the ceiling has collapsed. According to Corps of Engineer records, the collapsed area was measured to be approximately 106 square feet on January 5, 1969. Since that time, the collapsed area has increased to about 900 square feet. *Note: In the discussion on the photos, lines are referenced by number and letter. The designations are on the diagrams in Appendix A.*

Photo 24-1: Early collapse in 1969 expanded from 106 SF to about 900 SF.

An additional 60 feet south of the above ceiling has collapsed, which adds another 500 feet of collapsed ceiling. No documentation was provided to CCJM as to when the additional failures took place.

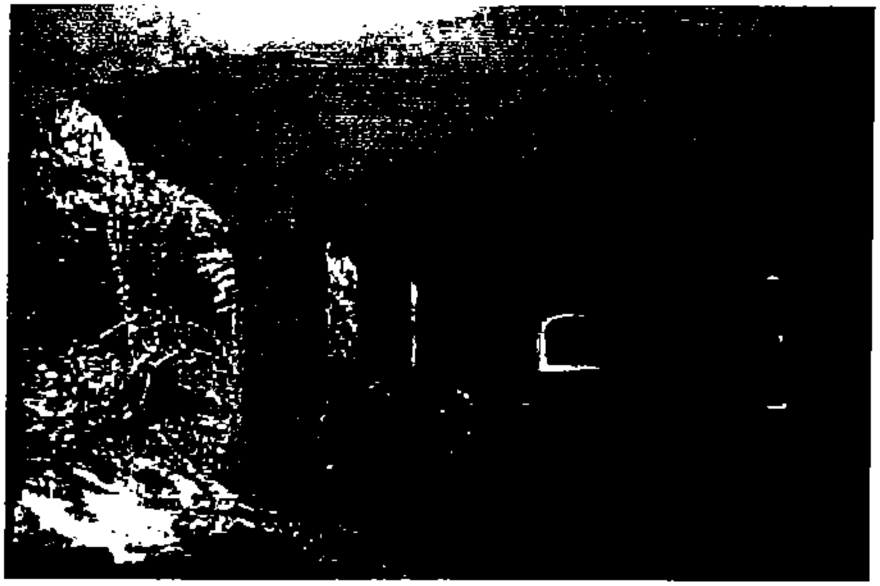


Photo. 24-2: Second collapsed ceiling in Cell No. 24

The collapse of both ceilings have similar patterns. Structural cracks develop at about 1.5 feet to 2 feet from the face of the column, the cold joint between different concrete pours separate, and settle differentially. The strength of an arched ceiling is compromised and the crown collapses.

At locations near the ceiling collapse, dangerously wide cracks and joint separation were observed. The arched ceiling is no longer a smooth curve and further collapse appears imminent. Presently there is as much as four inches of differential settlement at the joints, and 2-3" wide cracks at locations near the existing collapsed areas.

Photo 24-3: Sagging ceiling that is out of alignment at the edge row next to North Capital Street.

Early retrofitting with steel plate and bolts did not prevent further sagging of the separated joints nor widening of structural cracks.

Photo 24-4: Earlier structural repair did not stop the cracks from expanding.

At the east side of the entrance ramp, the joint settlement on the ceiling increased from two inches to three inches.

Photo 24-5: Even with steel strap, cracks extend and joint separation continues.

A crack near line 1 and A widened from ¼" to ½" and settled even with the strap in place to restrain it.

Photo 24-6: Crack development continues even though the strap is in place.

A crack near column 5 and M widened from ¼" to ½" even with the strap in place to limit further widening.

Photo 24-7: Crack development continues even though the strap is in place.

At the manhole near line 1 and B, a ½" joint separation documented in 1969 increased to 3" although the steel strap was placed in the 1960's to slow widening of the crack. With further crack development and joint separation, this piece of ceiling is likely to fall.

Photo 24-8: A piece of ceiling surrounded by cracks and separated joint. The circular opening is a manhole.

Concrete has fallen out of a crack near line 2, 4 and 7. A few new cracks appear at line A and 10, 7/9 and C, and 2/3 and H. In general, where the ceiling slab was repaired in 1960's, there is increased settlement and wider cracks.

Column (Pier)

No cracks are observed on the column although at several locations near the entrance ramp the cold joint between the top of column, and the base the of arched ceiling are slightly separated.

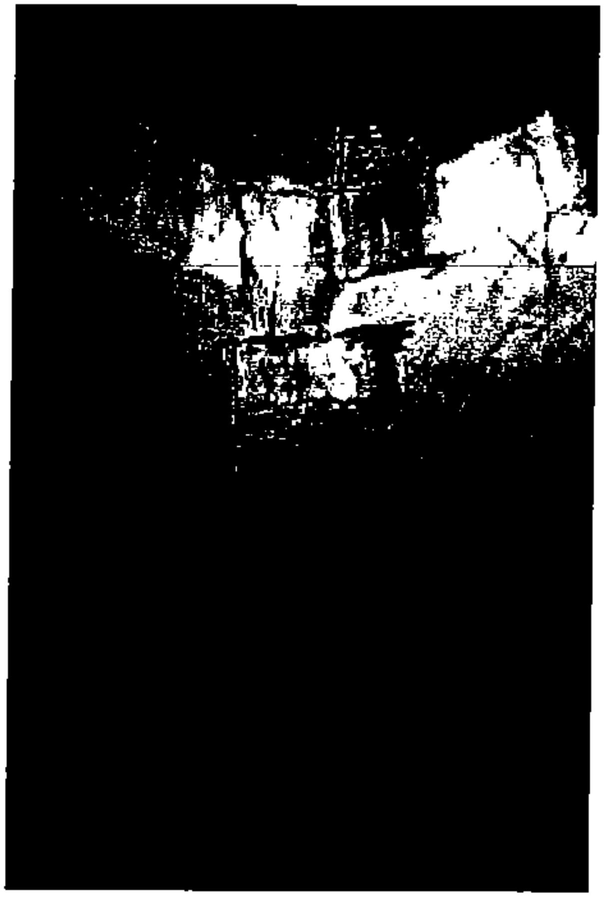
Walls

The South wall shows four new hairline cracks between line 6 and 8. The West wall shows a new hairline crack near line L. One crack is observed on the North wall at line 8.

Cell No. 19

Ceiling

Cell No. 19 is an edge cell next to North Capital Street. It is North of cell No. 24. It shows major cracks and joint separations on the ceiling besides cell No. 24 where the collapse occurred. At the manhole between line Q/R and 10, the ceiling slab has separated from the bottom of the manhole wall for 3" – 4" and is only restrained by a row of steel straps. A ½" wide compound crack developed along the settled joint.



Photo, 19-1 and 19-2: Ceiling shows 3"- 4" settlement and compound cracks parallel to the joint.

Not far above the sagging ceiling, there are a few 1" – 2" wide cracks in the ceiling. These were recorded in 1967 as a ¼" wide single crack. They are now much wider and compounded into multiple cracks next to each other. The slabs are strapped together with steel plates. Since the ceiling slab at the crown is about 6" thick, further settlement of the ceiling may crumble the ceiling slab-causing collapse without any advance warning.

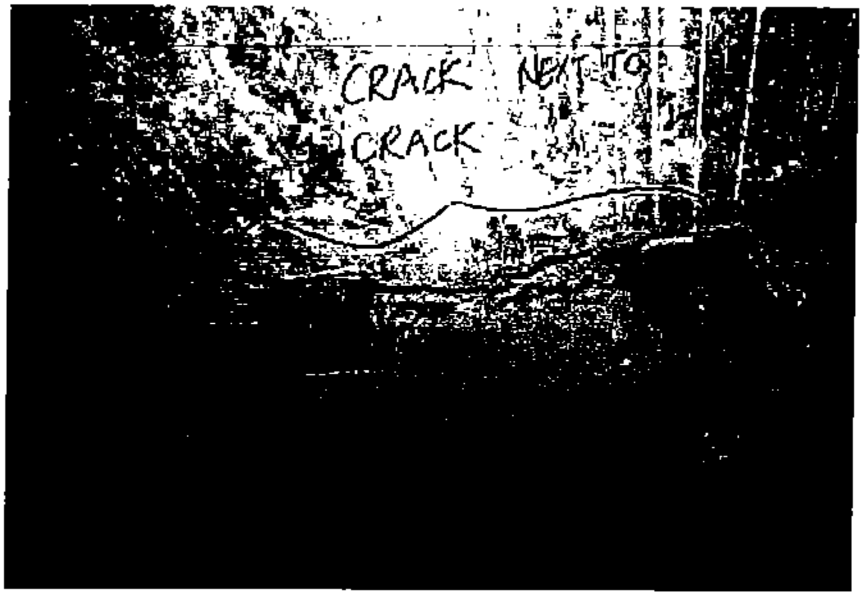


Photo 19-3: Wide cracks at ceiling near line 10.

Concrete has started to fall out at a crack near line K and 10. In general, where the ceiling slab was repaired in the 60's, there is development of increased settlement and wider cracks.

Column (Pier)

No cracks were observed on columns although at several locations the cold joint between top of the column the base of the arched ceiling shows signs of separation. This is obvious at the entrance along line 4 and 5.

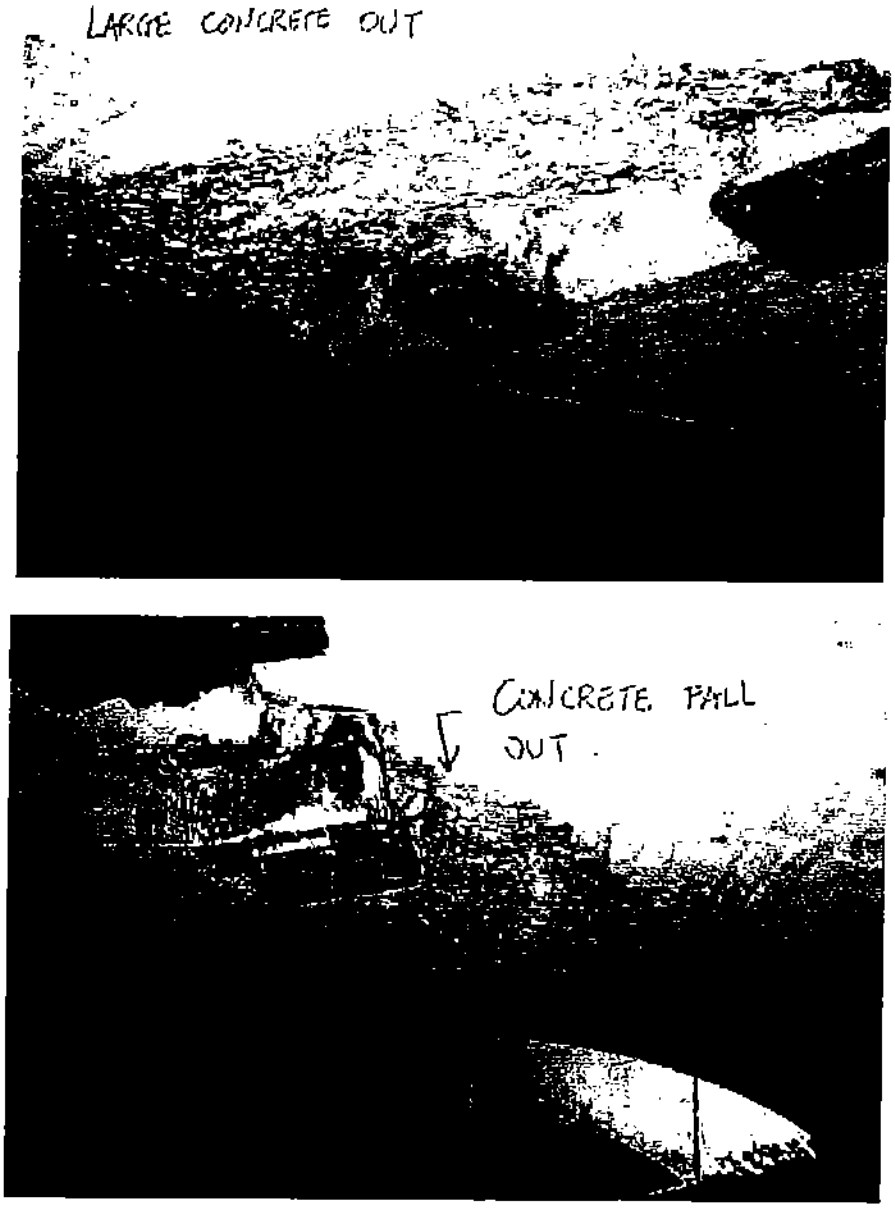
Walls

A new horizontal crack has appeared on the South wall and the East wall. A few vertical cracks show on the South wall.

Cell No. 23

Ceiling

A large piece of concrete (7' x 2'- 6") with thickness ranging from 0-6" has fallen out near line A and 6. This appears to have happened after the steel plates were placed to prevent its fall.



Photos 23-1 &2. 7'x2'-6"x up to 6" deep concrete loss.

Photo 23-3 shows further joint settlement and crack development from the 1967 survey, near a manhole along line 1 and P.

Photo 23-3: Further settlement at the crack and joint around manhole.

In general, where the ceiling slab was repaired in the 1960's, there is more settlement and wider cracks.

Column (Pier)

No cracks are observed on columns although at several locations the cold joint between top of the column and the base of arched ceiling show signs of separation. This is obvious at the entrance along line 4 and 5.

Walls

Three vertical cracks were found on the South wall between line 8 and 10.

Other photos below show some of the structural deficiencies in cells 17, 22, 26, 28.

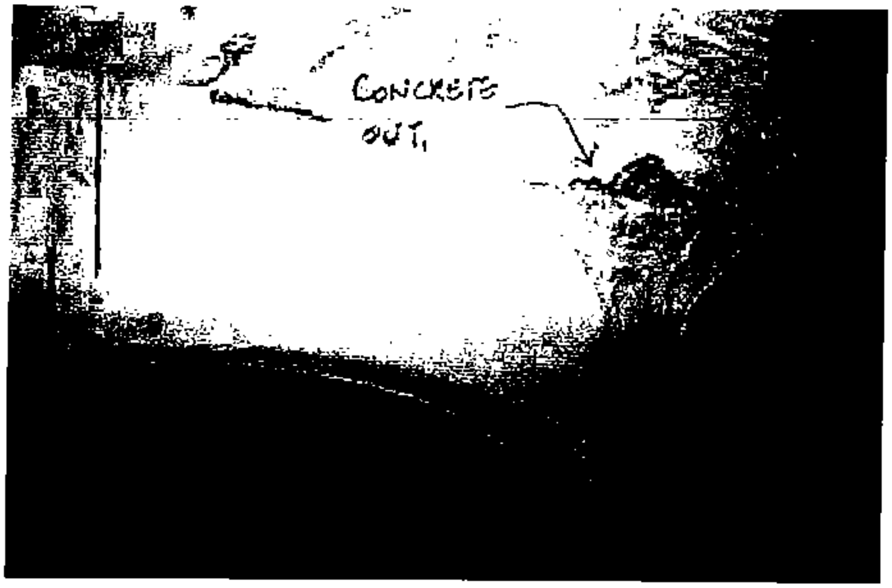
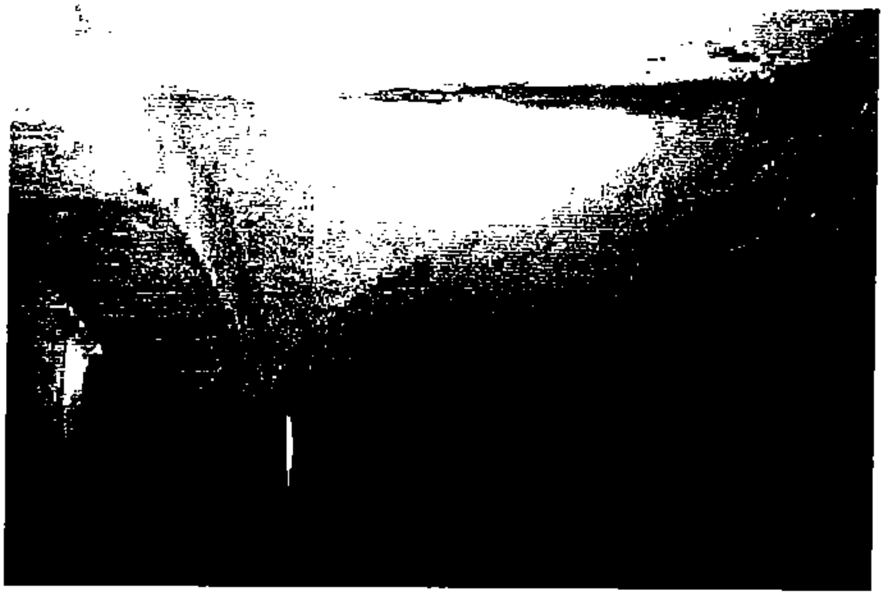
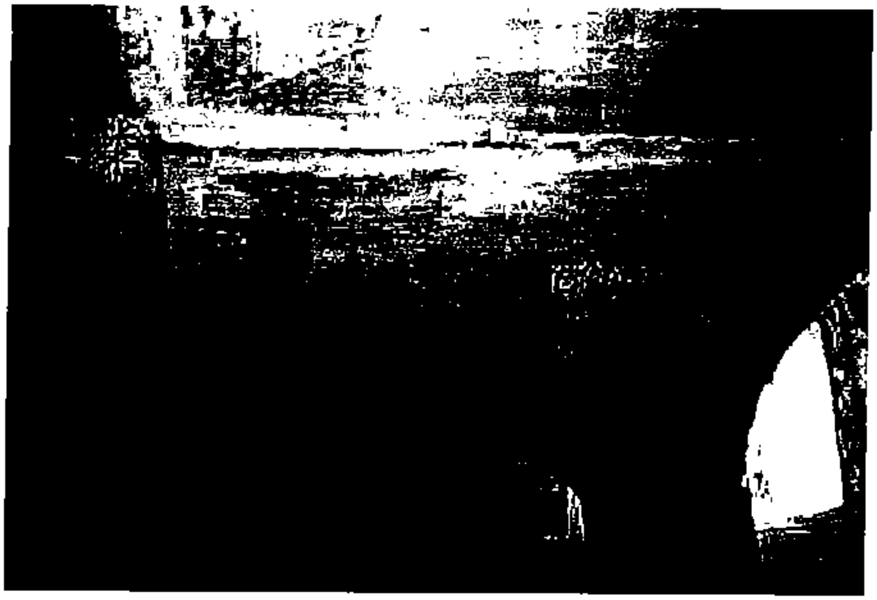


Photo 22-1: A piece of concrete fell from the cracked ceiling near column D5.





Photos 22-2 and 3: Ceiling slab rotated and dropped out of its original place at the butt joint.

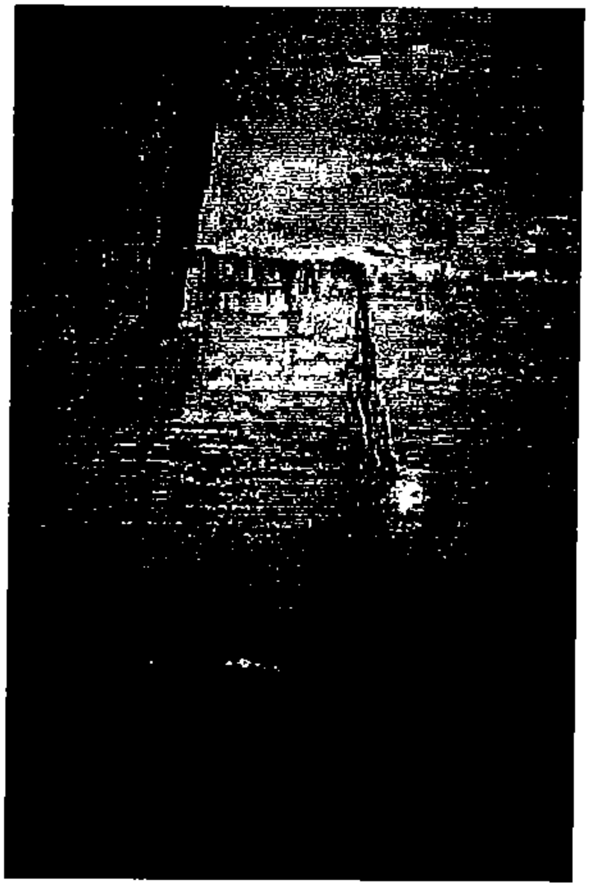


Photo 26-1: Root growth out of the structural cracks.

Photo 28-1

Photo 28-2

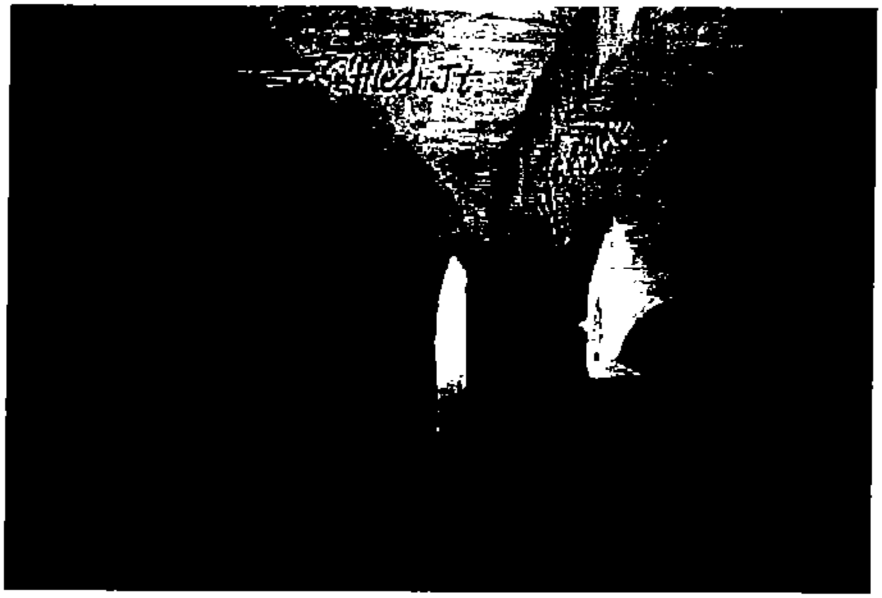


Photo 28-3

Photos 28-1,2 &3: Joint settled differentially.

Table 3-1: Summary of Field Inspection

Cell No.	Soils Below Base Slab	Description of Observation	Special Notes	Category of Damage
10	Firm Material	Refer To Plan CS-2. 1. A few new hairline cracks on the ceiling and walls. 2. Further settlement of the ceiling near column C5 and a piece of concrete fell out at this location. 3. Active water migration along north wall and ceiling nearby.	Water migrating into the filter was collected and sent to the Corps of Engineers for testing for chloride and fluoride to determine if the water comes from water main leakage or surface runoff. Results are .24 mg/l fluoride which is above local background levels and below target levels for distribution system.	**TYPE II**
11	Firm Material	Refer To Plan CS-3. 1. A few new hairline cracks on the ceiling and north wall. 2. Further settlement of the ceiling at joint near column A7 and a piece of concrete fell out at this location. Bottom of column A5 shows red brick surface.	There is no documentation as to why or when red brick on column 5 was used in the filter construction. This is not typical.	**TYPE II**
12	Firm Material	Refer To Plan CS-4. 1. A few new hairline cracks on the ceiling at the entrance. 2. One new hairline crack on North wall.		**TYPE II**

Cell No.	Soils Below Base Slab	Description of Observation	Special Notes	Category of Damage
13	Firm Material	Refer To CS-5 1. A few new hair line cracks on the ceiling and North wall. 2. New settlement of up to ½" near column A5 and A6. 3. Hair line crack at C10 widens to 1/8" and a small piece of concrete is falling off at the crown under manhole wall.		**TYPE II**
14	Firm Material	Refer To CS-6 1. New hairline cracks on the North and south walls. 2. New hairline cracks near column X1. 3. New settlement at manholes near C1 and F1.		**TYPE II**
15	Firm Material	Refer To Plan CS-7 1. Northeast corner appears to have widened cracks and more joint settlement at several locations. 2. Widened cracks at the ceiling above the entrance ramp. New hairline cracks on the ceiling and North and South walls.		**TYPE II**

Cell No.	Soils Below Base Slab	Description of Observation	Special Notes	Category of Damage
16	Firm Material	Refer To Plan CS-8 1. Two new hairline cracks on the ceiling at C6 and A1. 2. Minor joint settlement near A4. 3. A few new hair line cracks on the North and South walls.		**TYPE III**
17	Firm Material	Refer To Plan CS-9 1. Six new hairline cracks on West side of north wall. 2.		**TYPE III**
18	Firm Material	Refer To Plan CS-10 1. Along East side of line A, lower joint separation, new and wider cracks. 2. Crack on top of column B5. 3. A small piece of concrete fell off a crack near column E2.	Test pit No. 1 at column D4 over a nearby 1/8" wide crack: 1/8" wide crack was sprayed with water on the topside. Water was observed below thereby indicating full-dept crack.	**TYPE III**
19	Partially on fill	Refer To Plan CS-11 1. Condition similar to cell No. 24 at the Southeast corner. Cracks widened and joint separation worsened. 2. Joint separation of 1" found near line A.	SE corner appear to be in pre-collapse condition. Unsafe to walk under or above it.	**TYPE I**

Cell No.	Soils Below Base Slab	Description of Observation	Special Notes	Category of Damage
20	Firm Material	Refer To Plan CS-12 1. Numerous new cracks on ceiling between line B and H. 2. New cracks on top of column at A6. 3. New cracks on North and South walls.		**TYPE II**
21	Firm Material	Refer To Plan CS-13 1. At column I7, a hairline crack developed into a 1" wide crack.		**TYPE III**
22	On Fill in south east corner	Refer to Plan CS-14 1. Column cracks on top at the ramp entrance. 2. Red brick found in column B5. 3. Joint settled out of plan at the crown for up to 2 ½". 4. A piece of concrete fell off ceiling near column D5.		**TYPE I**
23	Mainly on top of fill	Refer to Plan CS-15 1. As shown in photos 23-1 thru 23-3..		**TYPE I**
24	Mainly on top of fill	Refer to Plan CS-16 1. As shown in photos 24-1 thru 24-8.		**TYPE I**

Cell No.	Soils Below Base Slab	Description of Observation	Special Notes	Category of Damage
25	Partially on top of fill at south east corner	Refer to Plan CS-17. Refer to photo numbers 1. Joint settle for up to 2" along line S between 2 and 3. 2. New cracks found. 3. Root growth out of crack on East wall near line A. 4. Crack found on top of column U2.	This is an edge cell subject to damage caused by the end wall displacement along the edge. It appears the edge cell shows more new crack development than the interior cell.	**TYPE II**
26	Mainly on top of fill	Refer to Plan CS-18 1. Concrete fell out at two joints at the crown, near U6 and A5. 2. Earlier cracks repaired with straps in 60's appear to have expanded and widened. 3.		**TYPE I**
27	Entirely on top of fill material	Refer to Plan CS-19 1. At the entrance, vertical cracks on the columns and arch ceiling bottom is severe. Steel collar and tie rod to retrofit these columns and ceiling corroded and lost more than 50% of its size. 2. Severe vertical cracks in column A5, B6, C6 and D6. 3. Earlier cracks repaired with straps in 60's appear expand and widened.	Early repair to the cracked column and arched ceiling fell apart.	**TYPE I**

Cell No.	Soils Below Base Slab	Description of Observation	Special Notes	Category of Damage
28	Entirely on top of fill material	Refer to Plan CS-20 1. Earlier cracks repaired with straps in 60's appear to have expanded and widened.		**TYPE I**
29	Entirely on top of fill material	Refer to Plan CS-21 1. Earlier cracks repaired with straps in 60's appear to have expanded, widened and sagged. 2. Horiz. cracks on top of column B5, C5.		**TYPE I**

CHAPTER 4

GEOTECHNICAL INVESTIGATION AND RECOMMENDATIONS

PURPOSE AND SCOPE

The purpose of the investigation was to evaluate the subsurface conditions, on a preliminary basis. The information obtained may be used by a potential developer to determine the complexities involved in developing and/or preserving the site. The investigation was not a design phase investigation. Consequently, the scope of the investigation was rather limited and consisted of the following:

- Collect and review the available information.
- Conduct a site reconnaissance and visually inspect the condition of the structure.
- Drill 12 borings to determine the subsurface conditions.
- Evaluate the data.
- Prepare a report of our findings.

DATA REVIEWED

The following reports were made available to us and were reviewed:

- Architectural / Engineering Feasibility Study of McMillan Reservoir Site; Existing Conditions Report; Oct. 3, 1988.
- Architectural & Archaeological Survey – Eastern Portion, McMillan Water Treatment Plant – June 1990
- Conditional Survey Drawings – February 1968 through July 1968
- Contract Drawings – 1905

Review of the old topographical data indicates that a stream traversed the southeast portion of the site, in a northeast to southwest direction. The elevation at the site had varied from about El. 120 at the south central portion to El. 180 in the northwest portion. Since the cell floor is at about El. 155, it is apparent that the site was developed by cutting the north west portion and filling the south east portion. The thickness of the fill could be up to 35 feet, with the thickest section being in the south central part.

FIELD RECONNAISSANCE

Field reconnaissance was conducted on several occasions in June and July 2000. We observed the following:

- Portions of the arches have collapsed at several locations, especially near the southeast portion of the site and along the perimeter of the site.
- There are several "sink holes" at the surface, especially in the southeast portion of the site.
- Some arches appear to have experienced some movement, since there are steel plates and bolts across the cracks in the arches.

- Some concrete columns also appear to have experienced some movements.
- The conditions of the arches and the columns appear to vary considerably from incipient failure to fairly sound.
- The cell floor is covered by about 4 feet of sand.

The sand storage bin and regulator house structures appear to be in sound condition. Noticeable cracks or settlements were not visible in these structures.

FIELD INVESTIGATION

The field investigation was conducted in July 2000. A total of twelve test borings (E-1 through E-12) were drilled to depths of 15-feet to 60-feet at locations shown on Figure 1 and listed on Table 1 in Appendix B. Borings E-1 through E-6 were drilled in the Court area (at El. 164±) and borings E-7 through E-12 were drilled from top of the roof (El. 170±) through the manholes and by coring the concrete floor slab underneath. Borings E-1 through E-6 were drilled by a truck mounted drill rig and borings E-7 through E-12 were drilled by a skid mounted special light weight drill rig. The holes were advanced using hollow stem augers. Standard penetration tests were conducted and split spoon samples were obtained in every boring, at depth intervals of 2.5-feet to 5-feet. Representative portions of each sample were placed in an airtight glass jar and were appropriately marked. Bulk (bag) samples were obtained off the auger flights in some borings. Two undisturbed Shelby tube samples of cohesive soils were also obtained. The depths of the boring varied from 15-feet to 60-feet. The groundwater level was monitored in each boring during drilling and at completion of drilling. Temporary 2-inch PVC pipes were installed in seven borings to measure the long-term water levels. After the water levels were obtained, all borings were backfilled with grout. The edited log of the borings are included in Appendix B.

LABORATORY TESTING

All samples were visually inspected in the laboratory by a Geologist/Geotechnical Engineer, to corroborate and/or modify the field classifications. Selected samples were tested for their natural water content, gradation (sieve analysis), Atterberg Limits, and unconfined compressive strength. A total of 77 natural moisture, 6 Atterberg limits, 6 sieve analysis, 4 percent fines, 2 natural densities and 2 unconfined compression tests were conducted. All tests were conducted in accordance with ASTM procedures. The results of the laboratory tests are summarized in Table-2 in Appendix B.

SITE GEOLOGY

Regional Geological Maps indicate that the site is located in the Atlantic Coastal Plain Physiographic Region where the near surface soils are an alluvial formation consisting of interbedded layers of silt, sand, clay and gravel.

SUBSURFACE CONDITIONS

The subsurface stratigraphy below the cell floor generally consists of the following two major strata:

Stratum-I: This stratum consists of fill, which is composed of brown, orange to reddish brown clayey sand and sandy clay with varying amounts of gravel. The fill is basically free of organics and was encountered predominantly in the southeastern portion of the site, as shown on Figure 2 in Appendix B. The depth of fill varied from 3-ft. to 35-ft. below the cell floor. Table 1 and Figure 2 in Appendix B show the thickness of the fill below the existing grade of the roof (El. 170±), not the cell floor. Standard Penetration Resistance in the fill varies considerably from 3 blows/ft. to 50 blows/inch. It is believed that the fill is an uncontrolled/unengineered fill.

The pressure and thickness of the fill observed in the borings is corroborated by the old topographic data and by the borings drilled in 1998 by others. The B borings were taken in 1988 by Schnable Engineering Associates. Boring B-14 indicates the presence of about 35-ft. of fill below the cell floor (El. 155±). The topographic map indicates that the elevation near Boring B-14 was about El. 120. Thus, the fill should be, and is, about 35-ft. It should be noted that the quality and nature of the fill is highly variable. In Boring B-14, the N value is in excess of 20 blows/ft. in the upper 20-ft.; in E-7 it is in excess of 50 blows/ft., whereas in Boring B-12 and E-11, the N value is 8 to 10 blows/ft. below the cell floor.

Figure 2 in Appendix B shows the surface contours prior to the construction of the cells. It appears that a stream traversed through the southeast portion of the filtration plant, and the grade at the stream was about El. 120 to El. 130. The western portion of the site was at about El. 180. The cell floor slab is at about El. 155. This would indicate that cell No. 28 would have about 30-ft. of fill, and cells 15, 20 and 25 have about 26-ft. of cut. Based on the topographic map, it appears that the thickness of the fill/cut in the cells was as follows:

Cell Number	Pre-Construction Elev.	Floor Slab Elev.	Fill / Cut
15	180	155	26' cut
16	180	155	25' cut
17	180	155	25' cut
18	120 – 180	155	15' – 25' cut
19	150 – 170	155	5' fill – 15' cut
20	180	155	25' cut
21	175 – 180	155	20' – 25' cut
22	155 – 175	155	0 – 20' cut
23	130 – 155	155	0 – 25' fill
24	130 – 155	155	0 – 25' fill
25	180	155	25' cut
26	145 – 180	155	10' fill – 25' cut
27	120 – 155	155	0 - 35' fill
28	120 – 125	155	30 – 35' fill
29	130 – 145	155	10' – 25' fill

Borings drilled by Schnabel Engineering Associates in 1988 indicate the following:

Boring	Location	Bottom of Fill Elev.	Original Grade
B-9	Cell 24	142	140+
B-11	Cell 22	148	145+
B-12	Cell 24	136	130+
B-14	Cell 27	119	120
B-15	Cell 29	140	140

Figures 3, 4 and 5 shows the thickness of the fill below the floor slab.

Stratum-II: This stratum consists predominantly of brown, orange to reddish brown clayey to silty sand with interbedded layers of sandy to silty clay. Standard penetration resistance varies considerably, from about 3 blows/ft. to 50 blows/4 inches, and generally increases with depth.

The soils in the stratum are highly variable and include gravel, Sand, Silty Sand, Clayey Sand and Clay. The liquid limit of the cohesive portion is generally between 35 to 40; the plasticity index is generally between 14 to 24; the natural water content is generally between 17% to 22%.

Groundwater was encountered in several borings and measured to be generally between El. 135 to El.140, as shown in Table 1 in Appendix B.

The generalized subsurface profiles are shown on Figures 3 through 5 in Appendix B.

EVALUATION AND ANALYSIS

The available data was evaluated with respect to the proposed development and is discussed below.

Site Development

If the site is to be developed as a mix use development, we envision the following issues and possible solutions.

Foundations

The soil fill above the arches is not considered to be suitable for supporting the one to 4 story structures, especially since the integrity of the arches in some areas, is highly questionable (see Chapter 6). Structures could be safely founded at the site, using one or more of the following options:

Option A: Demolish the sape sand filters and place the new structures at El. 155 (existing floor slab of the cells).

Option B: Demolish the sand filters; raise the elevation using structural fill; and construct the new structures on the structural fill.

Option C: Fill the cells and use the filled cells to support the new structures.

Option D: Preserve the cells.

Each option is discussed below.

Option A: In this option, the existing cells would be demolished, either entirely or selectively. The new structures would be founded on the virgin soil below the existing floor slab. The soils below the slab are anticipated to vary significantly, from uncompacted fill to soft/loose virgin soils to medium dense sand, as discussed under Subsurface Conditions. Where the floor slab is underlain by fill or soft soils, and if the fill or the soft soil is less than about 10-ft. thick, then the fill or the soft soil could be undercut and replaced with compacted fill. In that case, all the structures could be founded on shallow spread footings, bearing either on the existing virgin soil, or on the new compacted fill. On a preliminary basis, an allowable bearing capacity of 3 ksf on the fill and 4 ksf on the floor slab could be used to design the footings. Since the soils below the floor slab are generally sands and clays of the Terrace or Potomac formations, and have standard penetration resistance varying from 5 blows/foot to over 50 blows/foot, some localized undercutting and backfilling should be anticipated below the slab.

If the thickness of the existing fill is more than 10-ft. then deep foundations would be needed. This condition is anticipated to occur in cells 23, 24, 27, 28 and 29 i.e. the southeast portion of the site. The length of the deep foundations would depend upon their type, method of installation and load carrying capacity. Additional and deeper borings need to be drilled in the southeast portion to determine the lengths of the deep foundations.

In our opinion, 14-inch thick diameter auger cast pile could be used as one of the deep foundation systems to support the structure columns. Allowable design capacities of 40 kips could be used, tentatively, per 14-inch diameter cast pile with tip elevations varying from El. 130 to El. 100. The individual pile capacity and the top elevation depend on the depth of fill and the consistency of the soil.

Option B: This option is similar to Option A. The existing cells could be demolished either entirely or selectively. The existing fill under the floor slab, and the soft soils under the floor slab, would need to be undercut and backfilled. As discussed under Option A, the southeast portion would still require a deep foundation system. The grade of the entire area could then be raised to the desired level using structural fill. The structures could be founded on the compacted structural fill using an allowable bearing capacity of 3 ksf.

Option C: This option is based on filling the cells.

Fill the cells with clean sand, using hydraulic filling method. The sand could be placed in a sand/water slurry, and pumped into the filter cells, from various openings at the surface. The sand fill would be in a medium dense condition, and would prevent the collapse of the arches. This approach would require a large volume of water. However, the water could be recycled.

Fill the cells using "flowable fill", such as fly ash, sand and cement mixture. The flowable fill would fill the cells, provide support for the arch, and prevent surficial subsidence.

Option D: Selectively Preserve the Cells

Strategically locate the surficial structures over those portions of the sand filters that have a "low risk" of structural failure, and place the footings of the structures over the existing columns that are structurally sound, see Chapter 6 for locations of low, moderate and high risk areas.

Provide additional columns in the cells to support the arches, if necessary.

Construct a new structural slab on top of the arches, and support the new slab on either the existing columns or new columns.

Sports Fields

If sports fields are planned in the southeast portion, they could develop "sink holes", since the existing arches could collapse. The option cited for "Foundations" above, also apply to sports fields. In addition, the risk of "sink holes" developing could be reduced by removing the upper 2+ feet of soil cover, and replacing it with geotextile reinforced fill.

Groundwater

The borings indicate that the groundwater is at or below El. 135. The existing floor slab of the cells is at about El. 155. Therefore, groundwater is at a depth of about 19+ feet below the floor slab. Since the excavation or removal of soft soils will not extend below El. 145, groundwater is not anticipated to have any impact on the construction of the new facilities.

Suitability of Existing Soil as Fill

The existing soil inside the cells, above the concrete, is generally a sand and gravel. There is some debris on the surface of the sand. If the debris is removed, the sand and gravel is considered to be suitable for use as fill, from construction considerations.

Perimeter Wall

The perimeter wall appears to be in poor structural condition and will need to be repaired or replaced in certain areas. If a new wall is planned, it could be founded on shallow spread footings at about El. 155, except in the southeast portion, where it would need to be founded on deep foundations. An alternative would be to use a soil nail wall, auger cast pile wall, or steel sheet pile wall.

CHAPTER 6

STRUCTURAL ANALYSIS

A structural analysis was performed on the existing non-reinforced concrete arched ceiling structures that comprise the underground filters at the McMillan Filter Site. The goal of the analysis was to determine what, if anything could be constructed on top of the filters depending on their condition (Type I, Type II and Type III). The loads that would be imposed on the filter cells were assumed to be open space, one-story buildings such as supermarkets, and four-story office buildings.

Since the loads imposed by all three of the proposed types of development probably exceed the original design loads of 1902, any change in the use of these structures will be subject to current load requirements as indicated in Chapter 34 of the BOCA Building Code, which is the governing building code.

Cost estimates were performed to determine an estimated unit price per cell for preparing the site for each of the three forms of development.

Existing Structure

The original 1902 design documents reveal that the filters were constructed totally of non-reinforced concrete. There are 20 cells, each of which is about one acre in size. This nearly century old structure has the following features of structural significance:

1. The finished grade of the site is elevation 170 and there are two feet of earth fill on top of the ceiling slab at the crown. The curved ceiling slab collects the surface water that percolates through the two feet of overburden and directs it into two-inch drains that run through the center of the columns and discharge onto the filter bed.

2. The ceiling slab has curved top and bottom faces that are supported on 22 inch square concrete columns that are spaced at 14 feet on center in each direction. The thickness of the ceiling slab varies from six inches at the crown to 15 inches at the supports atop the columns.

3. The slab-on-grade base slab varies from 14 inches to six inches in depth with a curved top and flat bottom. It was designed to hold approximately four feet of sand and one foot of gravel in addition to water. The clearance between the top of the base slab to the ceiling at the crown is about 12 feet.

4. The perimeter walls are tapered retaining walls that are either buried completely or partially buried in the ground. The perimeter walls vary in thickness from two feet to seven feet. The partition walls that separate the cells are also tapered walls with a varying thickness of 24 to 34 inches.

5. The connection between the ceiling slab and the columns is a shear key. The columns are rooted into the base slab without a shear key.

6. The concrete ceiling slab appears to have been poured in either 14 feet by 42 feet segments that are supported on three columns, or 14 feet by 14 feet segments supported on one column. Along the edge near the perimeter wall, the segments appear to be 14 feet by 7 feet and 42 feet by 7 feet. Each segment butts each other at the crown and is vertically supported at the columns and walls.

Analysis Approach:

The Finite Element Method (FEM) method of analysis was utilized on a three-dimensional (3D) model of the arched ceiling structures to determine:

1. What additional surface loading can the existing non-reinforced structure support in addition to its own weight and the soil fill on top; and

2. The amount and method of reinforcing of the top slab required to support the three types of development.

After the model analysis, column and footing design calculations were performed to determine the amount of subsurface work, such as retrofitting columns, that would be needed to meet development requirements.

The model analysis only considered the Type II and Type III cells. The Type I cells that were built on fill continue to experience active differential settlement under the foundations. This ongoing settlement is the cause for the continuing structural deterioration of these cells. To prevent further damage to these cells, stabilizing the below grade fill would be required. This option would be prohibitively expensive. Also, most of Type I cells are already damaged beyond reasonable repair and cannot be salvaged even if the below grade fills were somehow stabilized. CCJM recommends that the Type I cells either be demolished or filled (Figures 6-1, 6-2 and 6-3).

In the 3D model analysis, both prismatic and non-prismatic plate elements were used to mimic the dome ceiling. Three models were implemented for the ceiling structure:

1. A 42 feet by 28 feet segment of curved ceiling supported on three columns that are spaced at 14 feet apart;

2. A 7 feet by 42 feet segment of curved ceiling supported on the perimeter wall and adjacent ceiling at the crown;

3. A 7 feet by 7 feet segment of curved ceiling supported on the perimeter wall and adjacent ceiling at crown.

Two loading scenarios for the ceiling were analyzed:

1. Weight of two feet of soil and 640 pounds per square foot (psf) live load.

2. Weight of two feet of soil and HS-20-44 truck load per AASHTO code (32,000 lbs. per axle).

In the 3D model the only vertical supports (Z) were assumed to be at the columns, and the only horizontal supports (either X or Y) were assumed to be at the butt joints occurring at the crowns.

Analysis Results and Structural Recommendations

The results of the model analysis show that in an ideal situation, the domed ceilings have sufficient capacity to carry the assumed gravity load. Under current code requirements, however, the non-reinforced concrete ceilings are unsafe for public access because of their nearly zero ductility. Since the columns and domed ceilings are constructed of non-reinforced concrete, they have little capacity to resist tension loads caused either by unbalanced vertical loads or horizontal forces caused by a possible superstructure above the ground. It also performs poorly under dynamic loads caused by vehicular traffic.

To add ductility to the non-reinforced concrete ceilings so that they could support open space development or the ground floor of a building, a reinforced concrete cast-in-place slab overlaid on top of the existing ceiling will be required. A reinforced slab one-foot thick at the crown and about 30 inches at the column is adequate to support a surface live load of 640 psf and its own weight and/or an H-20 load (Figures 6-4 and 6-5).

In Type II and III cells, the columns are in relatively good condition and most of them can support the new slab without reinforcing. However, some of the columns are cracked and will require reinforcing. For the Type II cells, CCJM estimates that ten percent of the columns (20 columns) will require reinforcing. For the Type III cells it is estimated that five percent or ten columns per cell will require reinforcing.

In the Type II cells, in addition to the above described construction requirements, the exterior wall will have to be reinforced. This can be accomplished by constructing a continuous two-foot thick reinforced concrete mat directly on, and doweled into the exterior face of the walls (Figures 6-4 and 6-5).

The construction of future buildings on top of the Type II and Type III cells, with new slab and reinforced columns as described above, would be possible if the new building column spacing is in multiples of 14 feet, and if the new columns are centered over the existing columns. The existing columns can be retrofitted and converted into reinforced columns by wrapping them with six inches of reinforced concrete. The base slab can be reinforced with a square footing at each retrofitted column. The perimeter wall can be retrofitted in a similar manner to support new columns (Figure 6-5). In this report, a frame 28 feet by 42 feet was used to determine the design for retrofitting of the columns and footings. Based on a 3000 psf bearing capacity a 13'-6" square by two feet high footing above the base slab is required for a four-story building and an 8 feet square by one foot high footing for a single story building. The new footings would be tied into the existing base slab with drill and bond dowels at 18-inch spacing. Similarly, the new column wrap will be connected to the existing columns with drill and bond dowels.

In the case where the Type II and Type III cells are filled with sand, the spread footers can be placed directly on the concrete arches, or on the fill within the cells as long as the footing is below the frost line as described in the BOCA Building Code. Where the Type II and Type III cells are demolished, the bottom slab can be broken up and the footers placed thereon. Or, if required by the site development, the slabs can be fully or partially removed and the spread footers can be founded on the soil at any elevation below the bottom slab.

For building construction in the Type I cell areas, pile foundations are recommended for both one story commercial buildings and four story of office buildings (Figure 6-2 and 6-3). These piles would be driven after the cells are demolished to the base slabs or after the cells are filled with sand.

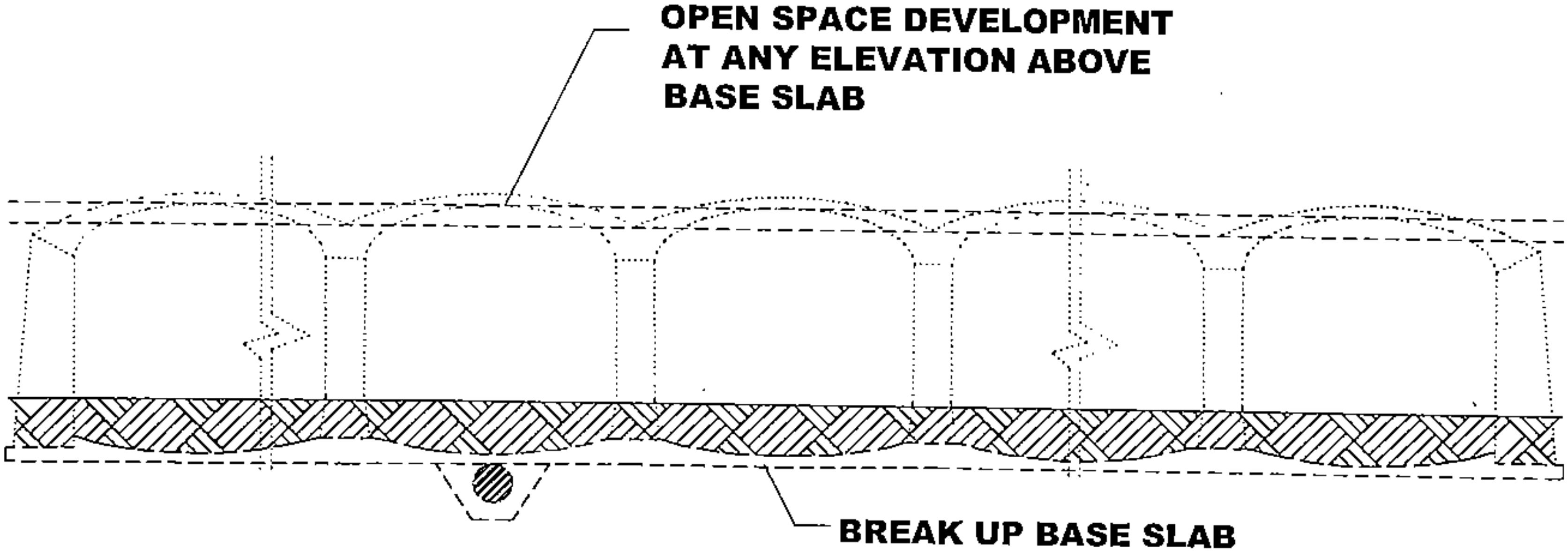


FIG. 6-1: DEMOLISH CELLS DOWN TO BASE SLAB OPEN SPACE DEVELOPMENT

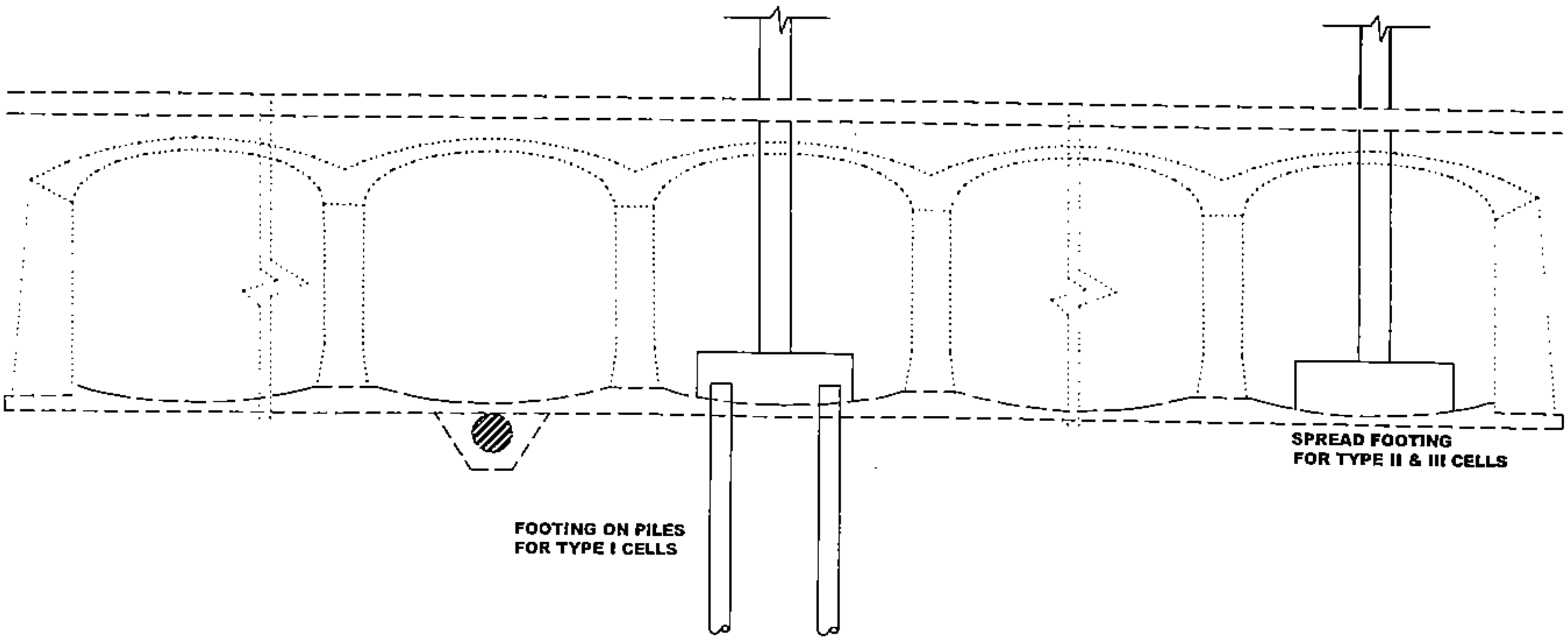


FIG. 6-2: DEMOLISH CELLS DOWN TO BASE SLAB
BUILDING CONSTRUCTION DEVELOPMENT

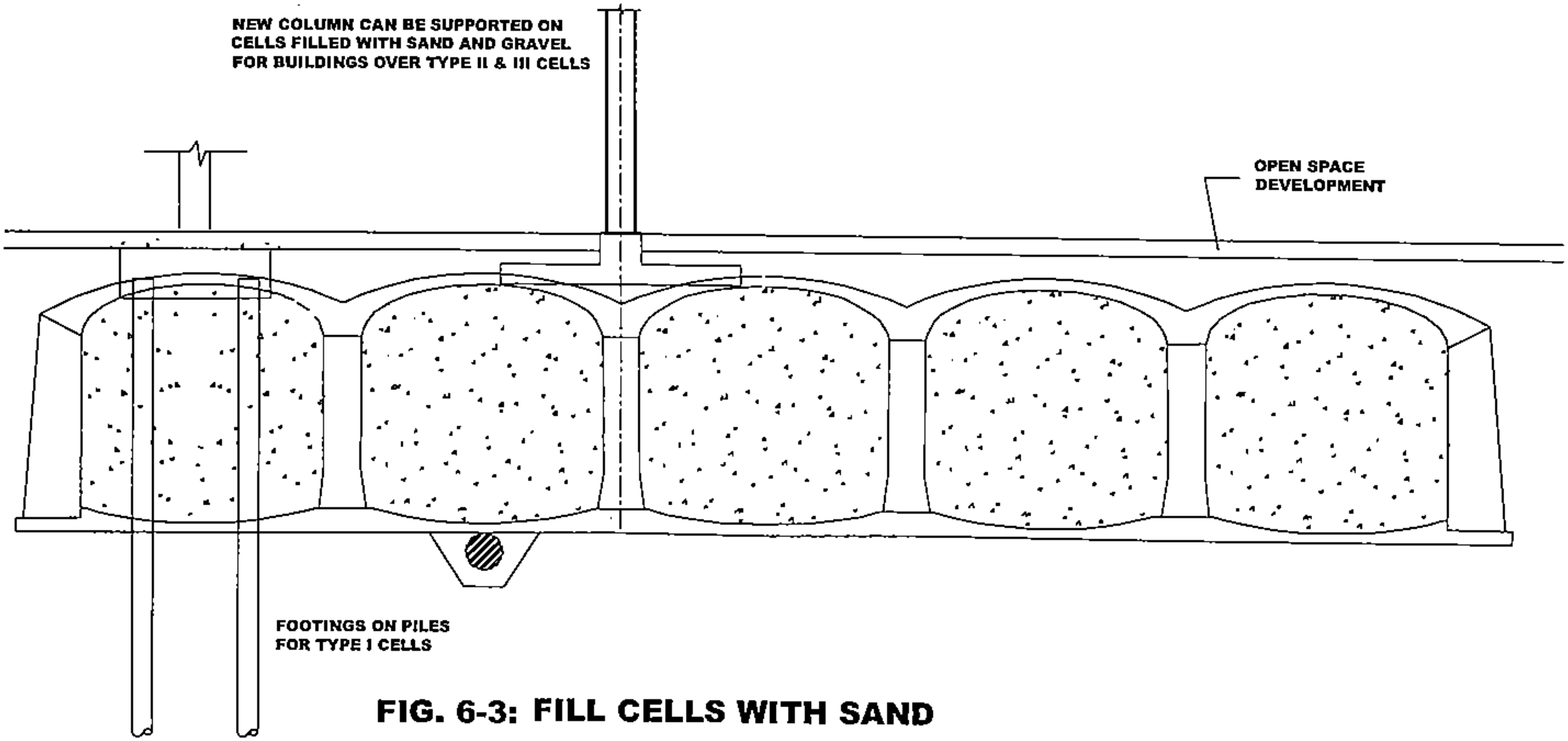


FIG. 6-3: FILL CELLS WITH SAND

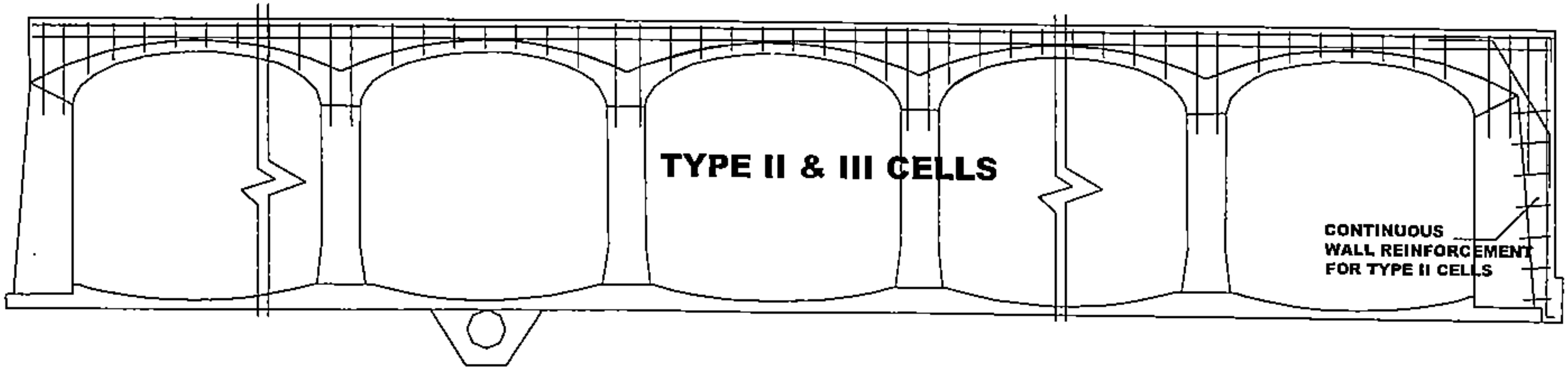


FIG. 6-4: PRESERVE CELLS
OPEN SPACE DEVELOPMENT

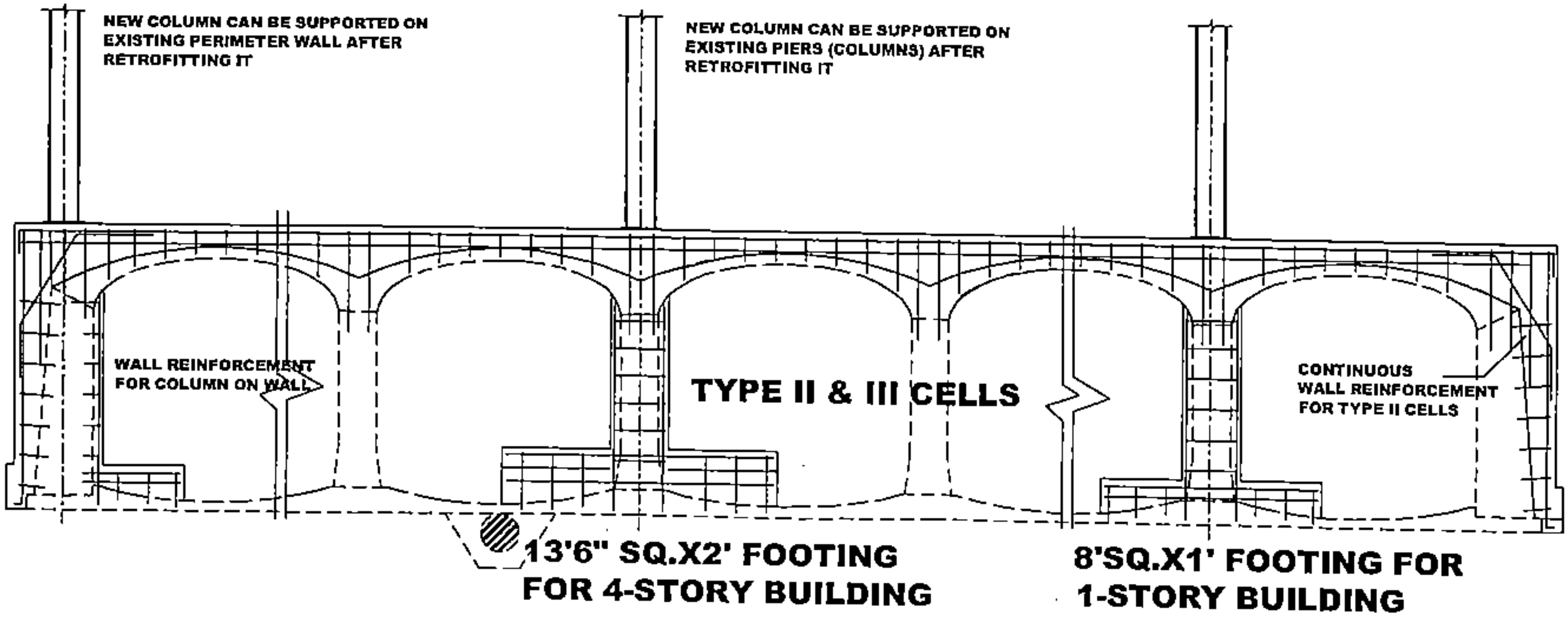


FIG. 6-5: PRESERVE CELLS
BUILDING CONSTRUCTION DEVELOPMENT

CHAPTER 7

CONSTRUCTIBILITY

Due to the age of the facility, the structural limitations of non-reinforced concrete, and the apparent relatively light surface loads considered in the original design, certain precautions should be consideration prior to construction on this site.

No traffic, pedestrian or vehicular, should be allowed within the Type I cell area. These cells should be either demolished or filled prior to any form of traffic, let alone any form of construction.

Light weight vehicles may travel over the surface of the Type II and Type III cells. In fact, equipment movement across the structures will be required in order to perform some of the construction activities required under the various options. The equipment may be steel tracked if operated on at least one foot of overburden, to protect the concrete. The equipment should be rubber tired or rubber tracked if it is to operate close to or on the concrete. In general the equipment should weigh less than 16,000 pounds, including any load and operator. Many different kinds of small construction equipment have recently become available. However prior to any construction activities, the contractor should be required to demonstrate by calculations that the stresses, moments and shears that will be created by his proposed equipment will be less than a specified uniformly distributed load. This load may vary depending on the condition of the structure, but may be as much as 100 psf to 150 psf. A uniform load is suggested as a criteria because it is impossible to predict wheel base and load distribution for the great variety of equipment that is available, but the effects of these different types can all be compared to a uniform load criteria. The contractor's analysis should also consider the effects of dynamic loading. Limiting the speed of travel may mitigate these effects.

If the option to demolish a cell is chosen, the bottom slab does not need to be demolished and removed, unless the site development is to occur at elevations lower than 155. However, since the ground water table is approximately twenty feet below the slab, the slab will form an impervious barrier to the downward migration of ground water. It is advisable, particularly with open space development, to break up the existing slab to prevent underground ponding. Currently the existing filter underdrain system prevents underground ponding of the ground water.

If the option to fill a cell is chosen, the fill material (sand, fly ash or other material) should be pumped in hydraulically through the manholes. The pumps should be located on the courts or surrounding streets. The existing under drain system can be used to drain off the excess water, except that some drain reconstruction may be required as discussed below in the existing utilities paragraph. As an option to drain reconstruction, the excess water from the filling operation could be pumped from the regulator houses into the onsite sewer which discharges into the combined sewer in North Capitol Street.

If a cell is to remain in place, traffic over the cell must be restricted as discussed above, until the new reinforced concrete top slab is in place.

The placement of concrete should be by concrete pumps. The travel of the concrete trucks and pumps should be restricted to the courts and surrounding streets.

Excavation of the overburden from over the tops of the filters will be required prior to constructing the new concrete top slab. This excavation may be done using light equipment as previously discussed. Light weight backhoes, front-end loaders, and even dozers are available, including some equipment that combines these functions, such as backhoes with dozer blades. If the weight limitations of the structures are met, these smaller types of excavation equipment may be used effectively. The difficulty arises in efficiently moving the large volumes of excavated material off of the filters, the travel distance will be as much as 300 feet. Fully loaded dump trucks will obviously exceed structural limitations. Dump trucks may be utilized if a system of timber platforms resting on cribbing at column locations is installed. A contractor should be required to submit shoring calculations for this method. Other means of transporting the material off of the filters are available such as conveyors and vacuuming equipment.

After the reinforced concrete top slabs are placed, full H-20 traffic can occur anywhere over the cells.

Any filter cell that is preserved could likely be utilized for a stormwater management facility. The filter cells could provide both detention storage as well as filtration. Modifications to the filter underdrain system would be required to carry the flow from the filters to the combined sewer in North Capitol Street.

A cursory review of records provided by the Corps of Engineers indicates that the two influent (settled) water lines (one in each court) and the two effluent (filtered) water were plugged shut by the Corps in 1989 and 1990, west of First Street. The storm sewer system in both courts collects surface water from catch basins, the drainage from the sand bins, and originally the wastewater from the sand washers. This system appears to still be functional, and is transporting these waters to the combined sewer in North Capitol Street.

The filter drain system for filters 20 through 29 (in Court 3) appears to still be functioning. This system appears to carry any and all surface waters that seep into these filters, across First Street and the Corps' Filter Plant, and discharges them into a combined sewer adjacent to the existing Reservoir. The Corps' records show that the filter drain for filters 10 through 19 was plugged in Court Two, just west of First Street, in August of 1989. This being the case, these filters cannot discharge to the sewer similar to filters 20 through 29. Since the filters are dry even though there is evidence of infiltration of surface rainwater, as well as a significant amount of seepage from an unknown source in filter 10, they must drain somewhere. Additional investigations into the existing drain system will be required if any of these filters are preserved as a part of the future site development.

Also within the courts are numerous other utilities, all of which appear to be abandoned. These include a significant amount of small pipe associated with the former sand washing operations, and several electrical ductbanks.

As mentioned above, seepage of water was observed in Filter 10 during periods of dry weather. A sample of this water was provided to the Corps of Engineers' lab. The analysis was not conclusive as to whether the water is ground water or from a leaking watermain. Since the water table is approximately 20 feet below the filter bottoms, the DC Water and Sewer Authority was notified to further investigate this matter.

CHAPTER 8

ALTERNATIVE SUMMARY

The field survey described in Chapter 3 categorized the twenty filter cells as Type I, Type II or Type III based on each cell's physical condition. Chapters 4 and 5 examined the geotechnical conditions above and below the filters. Chapter 6 examined the existing conditions and evaluated what renovations would be required to construct open space, a one-story building or a four-story building on each filter. Chapter 7 identified construction issues that could impact construction costs. Chapter 8 summarizes the information from these previous chapters and develops the cost to implement the alternatives. The costs were computed on a unit basis (per cell) and are summarize in Table 8-1. Copies of the cost estimate summaries are in Appendix E.

The assumptions used in the cost estimates are as follows:

- Open space development occurs at elevation 169 or one foot lower than the current grade over the filters.
- The elevation of the top of the floor slab for building construction is 169.
- Under the options of preserving the cells, the required new concrete slab over the cells is suitable for the floor slab for building construction.
- The cost of an on grade floor slab is included in the cost for building construction options over cells that are demolished or filled.
- Backfilling over the cells (over the bottom slab under the options for cell demolition) will be to elevation 168 for building construction, and 169 for open space construction.
- Under all options, new buildings will have columns spaced at 28 feet by 42 feet; under the options to preserve the cells, the new columns will be centered over the existing filter cell columns.
- The existing sand in the filters will remain under all the options.

The following are detailed descriptions of the alternatives presented in Table 8-1:

Cell Demolition: Under the option of demolishing the cells, there is approximately 1,900 cubic yards of concrete to be demolished and hauled off. The hauling would also include the approximate 4,600 cubic yards of overburden on each filter. The bottom slab would be broken up and left in place. Each filter would then be backfilled to elevation 169 making it suitable for open space development. The estimated cost for this subsurface work for open space development is $860,000 per cell. If the open space development occurred at elevation 160, the top of the sand, the cost would be $480,000.

To construct a new building, the superstructure would be supported on new columns and spread footers in the Type II and Type III cell areas. In the Type I area, the new building would have to be supported on pile foundations. The additional subsurface cost, over that for open space development, for constructing a one story building over a demolished Type II of Type III cell is $80,000 per cell for the columns and footers and $300,000 for the on grade floor slab. The resulting total cost is $1,240,000 per cell. The additional cost for the foundations and slab for a four-story building is $140,000 per cell resulting in a total of $1,370,000 per cell.

In the Type I area, the additional cost for a one-story building is approximately $170,000 for the pile foundations and $300,000 for the floor slab, or a total cost of $1,330,000 per cell. The additional pile foundation and slab cost for a four-story building is approximately $1,140,000 or an approximate total cost of $2,000,000 for a four-story building, per cell.

Filling the Cells: Under the option of filling the cells, there is approximately 9,000 cubic yards of void space above the existing sand that would be filled with a suitable material. The two feet of overburden would need to be replaced with one foot of compacted backfill material. Open space development could then occur. The estimated cost for this subsurface work for open space development is $440,000 per cell.

Building construction in the Type II and Type III cell areas would require new columns, spread footers, and an on grade floor slab placed on the filter tops after the filters are filled. The additional cost per cell to construct a one-story building over one of these cells is $350,000 or $790,000 total per cell for a one-story building. The additional cost for footings and slab for a four-story building would be $140,000 per cell, or a total cost of $920,000 per cell.

In the Type I area, any building constructed would have to be supported on a pile foundation. The piles would be driven after the cells are filled and the new backfill is in place. The additional cost for the pile foundations and floor slab would be $490,000 for a one-story building and $1,170,000 for a four-story building. Therefore, in the Type I cell area the total cost per cell for a one story building would be $920,000, and $1,610,000 for a four-story building.

Preserving the Cells: For the Type II and Type III cells an additional option of preserving the cells and developing over top of them is available. Under this option, the overburden would first have to be meticulously removed as described in Chapter 7. The cost of this excavation and disposal of the overburden is estimated to be $320,000 per cell. Each cell would then have a reinforced concrete slab constructed over it; the slab would be doweled into the existing ceiling, columns and walls. It is estimated that for Type III cells, five percent of the existing columns (ten columns) will be cracked and need reinforcing to support the new slab. The cost of this subsurface work for open space development over a Type III cell is $1,790,000.

In addition to the above described work for a Type III cell, a Type II cell will require reinforcing of the exterior walls. The length of exterior wall varies from cell to cell. We have assumed an average value of 350 feet per cell. It is also estimated that a total of ten percent of the existing columns (20 columns) will require reinforcing to support the new slab. The cost to modify a Type II cell to support open space development will be approximately $230,000 more per cell than for Type III, or a total of $2,020,000.

To construct a building over a preserved Type II or Type III cell, as described in the two preceding paragraphs, existing columns will selectively have to be reinforced and spread footers constructed at each of these columns on the filter bottom slab. We assumed the spacing of the columns to be reinforced would be 28 feet by 42 feet. The additional subsurface cost for a single story building is $230,000 per cell, and the additional subsurface cost for a four-story building is $540,000 per cell. On a Type II cell, the total subsurface cost for a single story building is

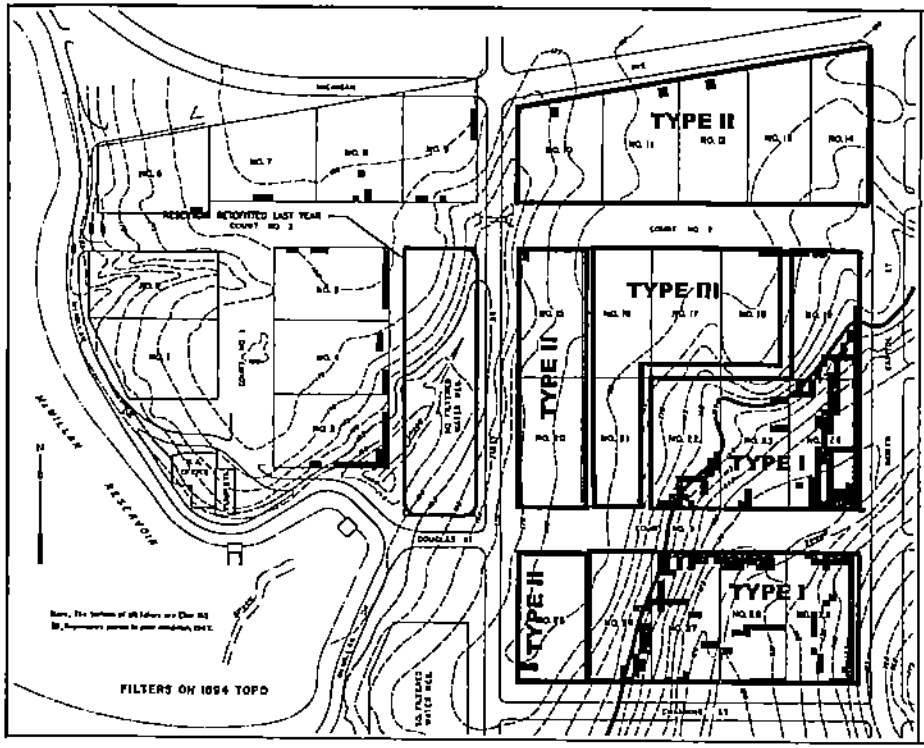
$2,250,000 and $2,560,000 for a four-story building. On a Type III cell, the total subsurface cost for a single story building will be $2,020,000, and $2,330,000 for a four-story building.

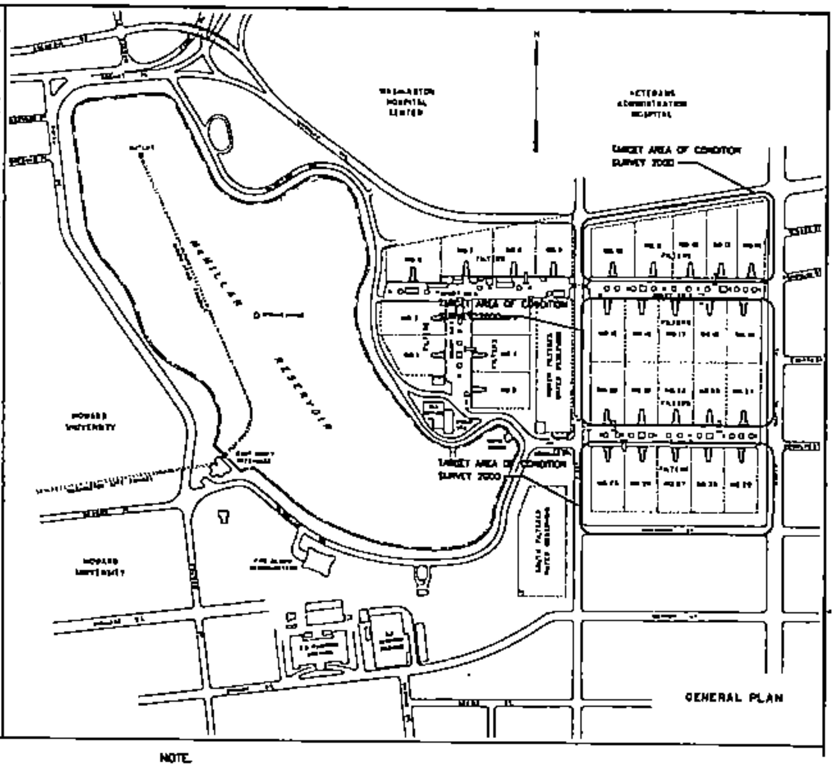
Numerous additional options and alternatives can be readily identified, such as developing at elevations other than at the assumed elevation of 169. By presenting the findings on a per cell basis, as well as presenting some of the costing considerations used, a planner can reasonably extrapolate cost estimates for most additional alternatives.

TABLE 8-1
STRUCTURAL/GEOTECHNICAL REQUIREMENTS FOR DEVELOPMENT OF McMILLAN FILTER SITE

	CELL DESIGNATION		
DESIGNATION	TYPE I	TYPE II	TYPE III
CELLS	19,22,23,24,26,27,28,29	10,11,12,13,14,15,20,25	16,17,18,21
DESCRIPTION	Built on fill, active cracking, some failures, additional failures likely	Built in cut areas, active cracking observed around perimeter	Interior cells, built in cut areas, no apparent new cracking has occurred in last 30 years
CONDITION	Unstable, Unsafe	Stable except at edges	Stable
OPEN SPACE			
PRESERVE FILTERS			
Struct. Req'ments	Not Feasible	Reinforced top slab and exterior walls	Reinforced top slab
Geotech. Req'ments	N/A	None	None
Cost Estimate	N/A	$2,020,000 per cell	$1,790,000 per cell
DEMOLISH FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	None	None	None
Cost Estimate	$860,000 per cell	$860,000 per cell	$860,000 per cell
FILL FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	None	None	None
Cost Estimate	$440,000 per cell	$440,000 per cell	$440,000 per cell
SINGLE STORY BUILDING			
PRESERVE FILTERS			
Struct. Req'ments	Not Feasible	Reinforced top slab, columns and exterior walls	Reinforced top slab and columns
Geotech. Req'ments	N/A	Spread footers	Spread Footers
Cost Estimate	N/A	$2,250,000 per cell	$2,020,000 per cell
DEMOLISH FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	Pile Foundation	Spread Footers	Spread Footers
Cost Estimate	$1,330,000 per cell	$1,240,000 per cell	$1,240,000 per cell
FILL FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	Pile Foundation	Spread Footers	Spread Footers
Cost Estimate	$920,000 per cell	$790,000 per cell	$790,000 per cell
FOUR STORY BUILDING			
PRESERVE FILTERS			
Struct. Req'ments	Not Feasible	Reinforced top slab, columns and exterior walls.	Reinforced top slab and columns
Geotech. Req'ments	N/A	Spread Footers	Spread Footers
Cost Estimate	N/A	$2,560,000 per cell	$2,330,000 per cell
DEMOLISH FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	Pile Foundation	Spread Footers	Spread Footers
Cost Estimate	$2,000,000 per cell	$1,370,000 per cell	$1,370,000 per cell
FILL FILTERS			
Struct. Req'ments	None	None	None
Geotech. Req'ments	Pile Foundation	Spread Footers	Spread Footers
Cost Estimate	$1,610,000 per cell	$920,000 per cell	$920,000 per cell

APPENDIX - A





NOTES

Condition survey of Filters No. 1, 6, 10, 16, 22 and 25 made in [illegible].
Condition survey of Filters No. 1 thru 29 made in 1967.
Condition survey of Filter No.10 thru NO.29 made in 2000.

NOTE.
CONDITION SURVEY 2000 EXCLUDES SAND TOWER AND MASONRY PUMP HOUSES IN TWO SERVICE COURTS

CONDITION OF FILTER CELLS CAN BE CATEGORIZED INTO THREE TYPES:

TYPE I: CELLS BUILT ON BACKFILL (FILL BELOW ELEVATION 155) STRUCTURALLY UNSAFE CONDITION. FOOTING HAS BEEN SAGGING. COLLAPSE HAD BEEN CONTINUOUS SINCE 1969. AREA OF COLLAPSE DEVELOPED FROM 100 SQUARE FOOT AT ONE PLACE IN 1969 TO A TOTAL OF 1,400 SF AT TWO LOCATIONS. MORE COLLAPSE IS EXPECTED

TYPE II: EDGE CELLS BUILT ON ORIGINAL GRADE. CELLS HAVE MODERATE DAMAGE AND IT HAS BEEN CONTINUOUS FOR THE PAST 31 YEARS.

TYPE III: INTERIOR CELLS BUILT ON ORIGINAL GRADE. MINOR DAMAGE IS OBSERVED THE DAMAGE APPEAR TO BE STABLE OR HAS ONLY MINOR EXPANSION DURING THE PAST 31 YEARS

CCJM

DISTRICT OF COLUMBIA
DEPARTMENT OF HOUSING AND
COMMUNITY DEVELOPMENT

McMILLAN SAND FILTER

CONDITION SURVEY
GENERAL PLAN

C1

Appendix 8: Cost Estimates

Appendix 8: Cost Estimates

Storage at McMillan Sand Filtration Site:

The following table is a summary of the estimate of probable costs for storage at the McMillan Sand Filtration Site, as described in Section 5.2.2. Stormwater would be routed to existing underground basins at McMillan via two separate sewer lines: a 72” trunk sewer along First Street NW, and a 60” trunk sewer along North Capitol Street. Costs reflect measures to rehabilitate and reinforce the existing underground basins for storage. Sewer diversion structures and interconnector pipes would also be constructed to carry out this option, and are reflected in the costs accordingly.

Description	Estimated Cost ($)
Stormwater Diversion from North Capitol Street (to northeast corner basin)	3,550,000
Stormwater Diversion from First Street NW (to middle-west basin)	3,850,000
Construction Subtotal	7,400,000
General Conditions (10%)	740,000
Construction Contingency (25%)	1,850,000
Estimated Construction Cost	9,990,000
Project Costs - Eng., PM, Legal, etc. (20%)	1,998,000
Total Estimated Project Cost	**11,998,000**

McMillan Storage and Flagler Place Pump Station:

The following tables are a summary of the estimate of probably costs for two implementation alternatives for storage at McMillan in conjunction with a Flagler Place Pump Station as described in Section 5.2.3. This option would entail stormwater and combined sewage storage at McMillan, and would divert approximately 100 MGD from the Flagler Place Trunk Sewer during peak flow. The first strategy would achieve this option through a pipe jacked force main; the second would use an open cut force main. A third strategy, which would employ a temporary bypass pump station, was not estimated due to project infeasibility.

Strategy 1 - Pipe Jacked Force Main for CSO Diversion

Description	Estimated Cost ($)
Stormwater Diversion from North Capitol Street (to northeast corner basin)	3,550,000
Stormwater Diversion from First Street NW (to middle-west basin)	3,850,000
Flagler Pump Station and CSO Diversion	13,458,000
Construction Subtotal	20,858,000
General Conditions (10%)	2,086,000
Construction Contingency (25%)	5,215,000
Estimated Construction Cost	28,159,000
Project Costs - Eng., PM, Legal, etc. (20%)	5,632,000
Total Estimated Project Cost	**33,791,000**

Strategy 2 - Open Cut Force Main for CSO Diversion

Description	Estimated Cost ($)
Stormwater Diversion from North Capitol Street (to northeast corner basin)	3,550,000
Stormwater Diversion from First Street NW (to middle-west basin)	3,850,000
Flagler Pump Station and CSO Diversion	10,485,000
Construction Subtotal	17,885,000
General Conditions (10%)	1,789,000
Construction Contingency (25%)	4,471,000
Estimated Construction Cost	24,145,000
Project Costs - Eng., PM, Legal, etc. (20%)	4,829,000
Total Estimated Project Cost	**28,974,000**

McMillan Storage and First Street Tunnel

The following table is a summary of the estimate of probably costs for storage at McMillan in conjunction with a First Street Tunnel as described in Section 5.2.4. This option would entail stormwater and storage at McMillan, and would divert stormwater and combined sewage to a new tunnel under First Street. The tunnel would be dewatered using a small temporary pump station. This option consists of accelerating construction on an already planned for portion of the DCCR tunnel system.

Description	Estimated Cost ($)
Subtotal - Storage at McMillan Site (from above estimate)	**11,998,000**
McMillan Site Demolition and Preparation	2,161,929
McMillan Shaft Construction	12,334,871
Excavate/Support First Street Tunnel	31,877,328
Adams Street Diversion and Shaft	11,782,163
First Street Diversion	5,000,000
V Street Diversion and Shaft	11,161,222
Rhode Island Avenue Dewatering Shaft	1,311,958
Final Design, Engineering During Construction, Geotechnical Instrumentation, Allowances	14,875,000
Indirect Costs	41,673,518
Subtotal - First Street Tunnel	**132,177,989**
Total Estimated Project Cost	**144,175,989**

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