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Summary
This Appendix contains five separate attachments that support the plan formulation for the Brandon Road flushing lock. Project measures are described in more detail in the main report.

Attachment 1, Site Selection of Brandon Road Lock and Dam as the Southernmost Control Point to Prevent the Migration of Asian Carp into the Chicago Area Waterway System Via Aquatic Pathways provides background and evaluation of the three considered locations considered for the southernmost control point: Dresden Island, Brandon Road, and Lockport Lock and Dam.

Attachment 2, H&H Information for Brandon Road Lock (GLMRIS-BR), contains pertinent data and physical attributes of the lock and gates, flow durations and modeling discharges, and the range of headwater and tailwater elevations. Available water supply is also identified as a potential limitation for lock flushing. The document investigates potential bypass locations around Brandon Road Lock and Dam, and the potential for fish passage through the dam’s headgates. The document also summarizes preliminary hydraulic modeling completed to determine potential water surface impacts because of the current design.

Attachment 3, GLMRIS-BR H&H Bypass Assessment, was conducted to determine whether hydraulic bypasses and/or connections around Brandon Road Lock and Dam (BRLD) could facilitate ANS passage around an ANS control point located at BRLD. The investigation included a search for potential connections to the Des Plaines River Watershed from the DuPage and Fox River Watersheds.

Attachment 4, GLMRIS Lock [flushing lock at Brandon Road Lock and Dam], Reducing Risk of Aquatic Nuisance Species Transfer through Brandon Road Lock, Analytical and Numerical Model Study, presents a detailed description of the Brandon Road Lock chamber flushing, along with numerical modeling procedure and results for the evaluation of four lock flushing systems. These systems include use of the existing culvert, two alternatives requiring modifications to the lock structure, and finally one that would provide a continuous supply of clean water to the lower lock approach to prevent ANS from reaching the lock chamber.

Attachment 5, Reverse Flows in Brandon Road Lock Approach Channel, describes a reverse flow in the approach channel downstream of Brandon Road Lock. This reverse flow has been observed by lock personnel, and measured by the United States Geological Survey (USGS). These reverse flows could make measures less effective by transporting ANS through deterrent measures. This document describes the development and use of a hydraulic model developed to simulate these reverse flows and compares these reverse velocities to those measures by USGS. The model may be used in the future to evaluate potential mitigating measures or operation changes.

USACE is undertaking its climate change preparedness and resilience planning and implementation in consultation with internal and external experts using the best available—and actionable—climate science. As part of this effort, the USACE has developed concise reports summarizing observed and projected climate and hydrological patterns, at a HUC2 watershed scale cited in reputable peer-reviewed literature and authoritative national and regional reports. Trends are characterized in terms of climate threats to USACE business lines.

Attachment 6, Climate Change, describes how climate change may affect the environment and function of various measures at Brandon Road.
Attachment 1:

Site Selection for Brandon Road Lock and Dam as the Southernmost Control Point to Prevent the Migration of Asian Carp into the Chicago Area Waterways System via Aquatic Pathways
Title: Site Selection of Brandon Road Lock and Dam as the Southernmost Control Point to prevent the Migration of Asian Carp into the Chicago Area Waterway System Via Aquatic Pathways

This white paper reviews three locations along the Des Plaines River for acceptability as a downstream control point to prevent upstream migration of Asian Carp into the Chicago Area Waterway System (CAWS) and ultimately into the Great Lakes. Three locations have been proposed as potential control points for the Mississippi River Basin (MRB) aquatic nuisance species (ANS):

- Lockport Lock and Dam;
- Brandon Road Lock and Dam; and
- Dresden Island Lock and Dam.

Modifications to the lock chambers and to aquatic pathways around the lock chambers are considered at all three locations. For the purposes of this white paper, an aquatic pathway is defined as a surface water connection between the MRB and the Great Lakes Basin (GLB) chambers which allow for the potential transfer of ANS in various life stages. See Figure 1 for the locations of Lockport, Brandon Road, and Dresden Island Lock and Dams. A pool is the water impounded upstream of a dam and maintained for navigation. See Figure 2 for a profile of the normal pool elevations for the Illinois Waterway system including the pools created by the Lockport, Brandon Road, and Dresden Island Locks and Dams.

Figure 1: Map of the CAWS Noting the Location of the Lockport Brandon Road, and Dresden Island Locks & Dams

Figure 2: Profile of Normal Pool Elevations for the Illinois Waterway System Including Pools Created by Lockport, Brandon Road, and Dresden Island Locks and Dams.
Removal of Dresden Island Lock and Dam from Consideration

Prior to completing a full analysis comparing each of the three proposed lock locations based on the above listed criteria, Dresden Island was immediately removed from consideration as an appropriate control point location. Significant Bighead and Silver Asian Carp populations are known to inhabit the Des Plaines and Kankakee Rivers between the Brandon Road and Dresden Island Locks and Dams.

- There is little head difference across Dresden Island Lock and Dam during large flow events and there is no structural barrier to prevent the passage of Asian Carp.
- Asian Carp have been observed on the Des Plaines River 10 miles upstream of Dresden Island Lock and Dam the Rock Run Rookery.
- While the full extent of Asian Carp along the Kankakee River is not known at this time, the USGS has observed Asian carp upstream of the Wilmington dam, located 10.3 miles upstream of the confluence\(^1\).

Due to the little head difference across Dresden Lock and Dam, and the wide extent of habitation of the Asian Carp, Dresden Island Lock is not considered to be a feasible option.

The Lockport and Brandon Road Lock chambers both are considered to be potential locations to successfully reduce the likelihood of Asian Carp transfer into the Great Lakes Basin because these

\(^1\) US Geological Survey, NAS – Nonindigenous Aquatic Species. Web 6 April 2015. 
chambers are located i) downstream of the current electrical dispersal barrier; and ii) upstream of the confirmed large population of Asian carps.

**Potential Aquatic Pathways- Effectiveness of controlling the upstream transfer of ANS**

A companion White Paper to this document, “Potential Modifications to Lockport and/or Brandon Road Lock Chambers to Prevent the Migration of Asian Carps into the Chicago Area Waterway System Via Aquatic Pathways” includes a thorough description of the Lockport and Brandon Road Lock and Dam Facilities including background, photographs, and upstream aquatic pathways associated with each. A summary of each facility is included here with a highlight of potential aquatic pathways which would need to be addressed if either site should be selected as a downstream control point. Based on results presented in that White Paper, the Chicago District had identified fewer and less complex aquatic pathways around the Brandon Road Lock and Dam.

**LOCKPORT LOCK**

As noted in Figure 3 below, Lockport Lock and Dam consists of one lock chamber, a dam and powerhouse, and an abandoned lock. Upstream of the Lockport Lock and Dam is the Lockport Controlling Works. See *Plate A* for an Illinois Waterway Navigation Chart of the area in the vicinity of the Lockport Lock and Dam.

![Figure 3: Lockport Lock and Dam (looking upstream)](image)

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Originally constructed in 1933 and recently rehabilitated in 1989, the Lockport Lock is comprised of a single lock chamber having a width of 110 feet and length of 600 feet. The lock’s average filling time is 22.5 minutes, and the average emptying time is 15 minutes.

Six potential aquatic pathways have been identified:

1. Lockport Powerhouse and Dam, which consists of a dam, two (2) turbines, and nine (9) sluice gates. Based on available data, the Chicago District estimates the normal head difference on either side of the powerhouse and dam equals approximately 40 feet. See Figure 3. Velocity through the powerhouse gates is estimated to be equal to or greater than 11 feet per second.

2. Lockport Controlling Works, which is located upstream of the Lockport Lock and Dam and connects the CSSC to the Des Plaines River. See Figure 4 for a photo of this structure. The Lockport Controlling Work’s primary purpose is to control flooding by allowing overflow relief for the CSSC into the Des Plaines River, and its secondary purpose is to maintain CSSC’s elevations for navigation. Additionally, activities at the controlling works are also coordinated with downstream powerhouse activities to maximize electricity production. The Lockport Controlling Works consists of seven (7) operational vertical lift sluice gates, which are 20 feet

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high by 30 feet wide. MWRDGC reports the controlling works’ gates are opened six (6) to ten (10) times a year and water velocity traveling through these gates can be minimal.\(^6\)

The head difference between the Des Plaines River and CSSC at Lockport Controlling Works reduces while the storm event progresses. Effectively, the water level on the CSSC reduces as a result of canal drawdown, and the water level on the DPR rises as the flood wave passes through it. Gates often remain open until water levels on the Des Plaines River and CSSC equalize and the flow through the gates is nearly stagnant.

3. During 100-year flood events, another potential aquatic pathway is created around the Lockport Lock. This potential pathway occurs in the Lockport Pool between the CSSC and Deep Run. Deep Run is a waterway that i) runs parallel to the CSSC and ii) connects with the Des Plaines River, downstream of the Lockport Lock and Dam. See Plates B and C for the Federal Emergency Management Agency’s 100-year flood maps of this area. During flood events, Deep Run could potentially connect to the CSSC and creates an aquatic pathway around the Lockport Lock. A hydraulic study would be required to make this determination.

4. An open aquatic pathway to the Des Plaines River exists downstream of Lockport near the Brandon Road Lock and Dam location. Several overflow locations from the Des Plaines River to the CSSC upstream of the dispersal electric barrier were identified as part of the Chicago District’s The Dispersal Barrier Efficacy Study- Interim I. As these locations were upstream of the existing dispersal (electric) barrier system, if the ANS carps were to traverse the Des Plaines River upstream to these overflow areas, and flooding occurs, they could easily gain access to the CSSC and disperse freely to Lake Michigan. A Des Plaines River Barrier was constructed in 2010 to address this overflow concern. The Des Plaines River Barrier consists of jersey barriers and 1/4” mesh fence, which ranges in height from 4’ to 8’. The barrier is constructed along the Centennial Trail which begins at 135th Street in Romeoville, IL (Will County) extends through Cook and DuPage County and terminates north of I294 in Cook County. Figure 5 indicates the location of this barrier system. The barrier system was constructed at the 100-year flood elevation with 3 levels of free board. The ¼” wire mesh of the chain link fence is designed to prevent all fish greater than a ¼” in girth from bypassing. The current threat of Asian carps dispersal is from large adults. If eggs and larvae were present in the Des Plaines River they will likely be swept downstream to below barrier reaches within hours since they have no swimming capability\(^7\).

While this open aquatic pathway through the Des Plaines River to the CSSC has been mitigated, a potential for this system to overtop or to fail may be present.

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\(^6\) Information obtained from Chicago District phone conversations with MWRDGC’s Department of Maintenance and Operations staff and Rock Island District H&H staff.

5. The Dispersal Barrier Efficacy Study- Interim I also identified that an open aquatic pathway to the I&M Canal existed through a connection to Deep Run, which has a direct connection to the Des Plaines River downstream of Lockport. If ANS were able to traverse upstream in the I&M Canal, culverts between the I&M Canal and the CSSC posed a potential aquatic pathway. A blockage berm was placed within the I&M canal at an identified flow divide location in Lemont, IL to prevent upstream transfer through the Canal’s culverts into the CSSC. The berm was constructed at the calculated 100-year flood elevation.

6. Lastly, another potential aquatic pathway around the Lockport Lock and Dam may exist at a small abandoned lock immediately adjacent to the lock and dam. See Figure 3. This lock has been bulkheaded.

II. BRANDON ROAD LOCK

The Brandon Road Lock and Dam is located in Joliet, Illinois on the Des Plaines River at Mile 286. See Figure 6. USACE regulates, operates and maintains this lock. Originally constructed in 1933 and rehabilitated from 1984 to 1987, the Brandon Road Lock and Dam contains one lock chamber and a dam. See Plate D for an Illinois Waterway Navigational Chart of the area in the vicinity of this lock and dam. The Rock Island District coordinates operations of the Brandon Road pool with MWRDGC. The inflow of this pool is dependent on the outflow from the MWRDGC facility at Lockport, Illinois.9

8 Lockport. . .Master Reservoir Regulation Manual. Abandoned lock noted on the bottom of Plate 2-4
The dimensions of the lock chamber are 600 feet long by 110 feet wide. The lock’s average filling time is 19 minutes, and the average emptying time is 15 minutes.

Three potential aquatic pathways have been identified:

1. Brandon Road Dam is an aquatic pathway around the Brandon Road Lock. This dam contains eight (8) operational headgates and 21 tainter gates. Water velocity through the dam’s headgates ranges from approximately 28 to 42 feet per second.\(^\text{10}\) The nominal lift between the Brandon Road Pool and the Dresden Island Pool equals approximately 34 feet.\(^\text{11}\)

2. An inoperable lock is located northwest of the Brandon Road Lock and Dam. See Figure 7 for the location of the inoperable lock. This inoperable lock connects the Des Plaines River with the Illinois and Michigan Canal (I&M Canal), and it contains a sluice gate. The pathway between the Des Plaines River and the I&M Canal occurs when the sluice gate is occasionally opened to allow water from the river to flow into the canal.

3. A potential bypass has been identified around the Brandon Road Lock and Dam through an open aquatic pathway in the DuPage River basin. The potential pathway follows the DuPage River to the I&M Canal and through the Rock Run Tributary.

Figure 6: Bird’s Eye View of Brandon Road Lock and Dam

\(^{10}\) US Army Corps of Engineers, Inland Navigation Design Center (INDC), H&H information for Brandon Road ANS Lock (GLMRIS), 28July 2015.

Comparison of Aquatic Pathways at Lockport and Brandon Road:

Potential aquatic pathways at Brandon Road are less complex and less geographically expansive than the pathways at Lockport.

Specifically, modifications to the structure and/or operations of the Lockport Controlling works would require significant coordination with MWRDGC and modifications to flood control operations may be difficult to adopt due to the direct relationship to flood risk in the Chicagoland area. Additionally, the potential hydrologic bypass (flanking) from the Des Plaines River to the CSSC and from Deep Run to the CSSC that may occur during periods of high flow presents an aquatic pathway for ANS to transfer to the GL basin which may be difficult to fully address.

Due to the complexity of pathways at Lockport Lock and Dam and challenges associated with eliminating those pathways in comparison to the challenges associated with the pathways at the Brandon Road location, Brandon Road is considered to be preferable to Lockport for this criterion.

Review and Recommendation:

Specifically, a review of potential aquatic pathways indicates that selection of Lockport Lock and Dam location as a control point would include numerous challenges related to hydraulic separation at the Controlling Works and at locations where flanking between the CSSC, the Des Plaines River, and Deep Run may occur during high water events occurs. Due particularly to these challenges, in addition to the review of all other considered criteria, the Chicago District recommends that the Brandon Road Lock and Dam facility be selected as the optimal location for the implementation of a downstream control point.
Plate A: Illinois Waterway Navigation Chart in the Vicinity of Lockport Lock and Dam

Attachment 1:
Site Selection
Plate B: 100 year Flood Plain Maps - Aquatic Pathway Connecting the CSSC with Deep Run and Possible Aquatic Pathway Connecting the CSSC to the Des Plaines River - Area Located Between Lockport Lock and Dam and Lockport Lockport Controlling Works
Plate C: 100 year Flood Plain Maps - Aquatic Pathway Connecting the CSSC and Deep Run - Area Located Opposite the Lockport Controlling Works
Plate D: Illinois Waterway Navigation Chart in the Vicinity of Brandon Road Lock and Dam
Attachment 2:

H&H Information for Brandon Road Lock  
(GLMRIS-BR)
H&H Information for Brandon Road ANS Lock (GLMRIS)

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1) General Purpose

The H&H analyses in this summary report were performed by the Rock Island District at various times from 2014 to 2018. The information was compiled using readily available data to inform the study team of ways to better prohibit Aquatic Nuisance Species (ANS) from passing upstream of Brandon Road Lock and Dam and to address water related concerns of team members from various disciplines. Navigation on the Illinois Waterway carries with it the potential for ANS to follow, drift, or attach themselves to the hull; these three types of ANS are called swimmers, floaters, and hitch-hikers, respectively. The goal of the study team is to reduce or eliminate the risk for these ANS to pass upstream through Brandon Road Lock and Dam.

The work during Preconstruction Engineering and Design (PED) will need to evaluate empty barges and potential operational changes needed for recreation vessels. The vessels are assumed to be moored downstream of the lower miter gates, as is currently done for upstream lockages thereby taking a more conservative (safe) approach.

To narrow the focus of tailwater impacts on the study measures, three river discharges—15,000, 11,200, and 1,260 cfs—were selected to capture the range of elevations which would be used to evaluate alternatives and balance the level of detail needed for the feasibility study to compare/screen alternatives. Cold weather impacts were not considered in the study alternatives, as this is anticipated to be an operational alteration based on the risk of ANS movement during cold weather periods.

2) Pertinent Data for Brandon Road Lock & Dam

**Datum Conversions**

1929 = 1912 – 0.45 ft  
1988 = 1929 – 0.2 ft  
1988 = 1912 – 0.65 ft

Original drawings use 1912 datum and many newer texts use 1929 datum. River data is collected in 1929.

**Lock Information (elevations 1929)**

Nominal Length of Chamber    600 ft  
Pintle to Pintle Length of Chamber  671 ft  
Chamber Width  
Flat Pool  538.5 ft  
Flat Tail  504.5 ft  
Minimum Tail  504.1 ft  
Operational Pool (High):  538.9 ft  
Operational Pool (Low):  538.4 ft

Upper Miter gate leakage  60 cfs (includes valve leakage, valve is shut) USGS measured  
Lower Miter gate leakage  185 cfs (also includes valve leakage when valve is shut)
Upper Miter Gate Height      20 ft
Lower Miter Gate Height      50 ft

Sill Elevation (upper)       520.7 ft
Sill Elevation (lower)       490.75 ft
Chamber Floor:               489.7 ft

**Lock Volume Calculations**

** Pintle to Pintle distance (671 ft) used for volume computations:
Normal Pool Depth in the Chamber   48.8 ft  (538.5 – 489.7)
Minimum Depth in the Chamber       14.8 ft  (504.5 – 489.7)
Typical Lift                       34.0 ft  (48.8 – 14.8)
Lock Volume without vessel displacement:  2,509,540 cubic feet  (34.0 * 110 * 671)
Lock Volume with 3x3 barges in chamber drafting 9 ft:  1,943,280 cubic feet
Typical Filling Time               19 min
Typical Emptying Time              15 min
Average Inflow while Filling       1710 cfs
Average Discharge while Emptying   2159 cfs
Peak Discharge while Emptying      7120 cfs  (measured by USGS on Dec 8-10, 2014)

Max Flow during a Flushing Operation 1350 cfs (fill valves at 1/4 open by USGS)
Minimum Depth over Lower Sill      13.75 ft  (504.5 – 490.75)
Velocity over Lower Sill while Flushing  0.87 ft/s (estimated)

Note that during a flushing operation, the discharges are less than instantaneous discharges from a pool to tail chamber emptying discharge. This is because the filling valve (upstream) has a limited range of opening due to pinning forces at full head when used for flushing.

**Dam Information**

Tainter crest of skin plate (gate closed) 539.4 ft (1929) overtopping coefficient 3.5
Bottom Elevation of Open Tainter Gate  539.5 ft (1929)
Crest of Ogee Spillway under Tainter Gates  536.25 ft (1929) overtopping coefficient 3.3
Tainter gates (21 total) 50 ft wide by 2’ 3.5” high
Each Tainter Gate releases 550 cfs discharge when fully opened; partial openings are not used.

Head Gate Sill  510.5 ft  (1929)
Head Gates 15 ft wide by 15.75 ft high, 8 operational head gates (8 inoperable – concrete sealed)
Head Gate capacity 450 cfs per foot of opening, or 6800 cfs fully opened
3) Modeling Discharges
This section describes the background information for the three H&H teams (MVR, ERDC, LRC) so that the modelers have the same understanding of what flow rates should be used for various aspects of the project. The goal is to develop three reasonable tailwater elevations to evaluate during the design of the study features. It was concluded that “Low”, “Medium”, and “High” flow rates should be considered as 1260 cfs, 11200 cfs, and 15000 cfs for Brandon Road, respectively. These will be used for design work into the future. Medium and low flow rates were obtained from duration analysis (see Table 1) and the high flow came from the navigational restriction of 15,000 cfs, when lockages are generally halted on the Illinois Waterway at Brandon Road Lock.

3.1. Flow and Elevation Durations
A number of data sources were available from previous studies and the team relied on readily available information when possible. As the feasibility study progressed additional analysis was completed and incorporated into the report. The ANS measures are most influenced by the tailwater stage which is a function of flow. As a result three tailwater conditions were selected for use during model studies. The tailwater conditions were termed low, medium, and high. The information will need to be revisited and refined when the final design effort takes place. The information below contains the rating curves, flow durations and elevation duration curves for Brandon Road (Table 1, Figures 1-3).

Table 1. Summary of the Three Tailwater Conditions That Were Selected for the Model Studies

<table>
<thead>
<tr>
<th>Percent Exceedance</th>
<th>Elevation (NGVD 29)</th>
<th>Flow (CFS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8%</td>
<td>507.2</td>
<td>15,000 (high flow, nav stop)</td>
</tr>
<tr>
<td>2.5%</td>
<td>506.6</td>
<td>11,200 (medium flow)</td>
</tr>
<tr>
<td>5.0%</td>
<td>506.2</td>
<td></td>
</tr>
<tr>
<td>25%</td>
<td>505.4</td>
<td></td>
</tr>
<tr>
<td>50%</td>
<td>505.2</td>
<td></td>
</tr>
<tr>
<td>75%</td>
<td>504.9</td>
<td></td>
</tr>
<tr>
<td>95%</td>
<td>504.6</td>
<td></td>
</tr>
<tr>
<td>97.5%</td>
<td>504.5</td>
<td>1,260 (low flow)</td>
</tr>
</tbody>
</table>

The 50% Flow Duration was originally used for a medium flow based on ranking daily data from the past 10 years of record at the USGS gage 5537980 (Route 53 at Joliet). For a comparison, the 1940-1976 Flow Duration (as contained in the 1990 Navigation Study report appendix) for 50% is 3200 cfs (versus 3160 cfs for the past 10 years), and the stage duration for 1935-2011 is 505.2 ft which matched the past 10 years of data. The medium flow was changed from the 50% duration to the 2.5% duration because initial modelling results showed no significant difference from 1260 cfs (low flow) to 3200 cfs (old medium flow). The medium flow was chosen as 11200 cfs and the high flow was changed from the 500-year event discharge of 36000 cfs to the maximum flow where navigation halts 15000 cfs.
Figure 1. Rating Curve for the Brandon Road Lock Tailwater & Rated Dam Flows. Developed using data from 2008-2017. Trend line manually added for visualization of the estimated median values.

Figure 2. Duration curve (1935 to 2011).

Attachment 2:
H&H Information for Brandon Road Lock (GLMRIS-BR)
Figure 3. Duration curve (1987-2017). The duration information for the last 30 years indicates a slight downward trend when compared to the period 1935 to 2011.

3.2. Annual Chance Exceedance (ACE) Flows & Elevations

Brandon Rd Lock is unique in that it does not officially close to navigation during high water events. However, there are bridge clearance issues that may restrict the movement of vessels at high flows that generally result in navigation traffic stopping until flows recede. The head is greater than 20 feet at a 500-year event. When the tailwater is higher, more volume will be needed to flush floaters before upstream lockages occur. This may have a significant impact on navigation delays or the risk reduction afforded by a set flushing duration. The frequency data in Table 2 and Figure 4 is from the 2004 Upper Mississippi River Flood Frequency Study (UMRFFS) at the Brandon Road Lock and Dam.

Table 2. Flow Frequency Relationship at Brandon Road Lock and Dam

<table>
<thead>
<tr>
<th>Annual Exceedance Probability</th>
<th>Recurrence Interval</th>
<th>Discharge (CFS)</th>
<th>Tailwater Stage (1929)</th>
<th>Pool Stage (1929)</th>
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<td>0.5</td>
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<td>12,000</td>
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<td>538.5</td>
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<td>0.32 (interpolated)</td>
<td>3.8-year</td>
<td>15,000</td>
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<td>0.2</td>
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<td>0.002</td>
<td>500-year</td>
<td>36,000</td>
<td>514.5</td>
<td>538.5</td>
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Figure 4. Plot of the 2004 UMRFFS Published flow frequency relationship at Brandon Road Lock and Dam - downstream (RM 285.9)
Table 3. Record High Stages at Brandon Road Tailwater

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<td>511.90</td>
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<td>12/4/1982</td>
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<td>513.30</td>
<td>7/13/1957</td>
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</table>

4) Head Gate Velocities
This analysis was performed because of the concern that ANS swimmers might be able to pass upstream through the head gates at Brandon Road Dam. The concern of debris passage was also considered because of the thought that large debris might prevent the head gates from closing. To be able to address these concerns, the water velocity of the jet coming through the opened head gate must be estimated and compared to the estimated burst speed of the ANS swimmers, especially Asian Carp. It was concluded by a panel of experts (November 2015) that the velocity coming through the head gates (28 ft/s) was too strong for ANS passage.

Head Gate Velocities and velocities downstream of the lock were estimated using the as-built drawings and hand calculations. Figure 5 is a drawing of the Head Gate geometry after the major rehabilitation work in the 1980’s. There are currently eight head gates that are 15 ft wide by 16 ft high, five of which are raised and lowered by a mobile crane. The opening height is 15.75 ft high, and the gate sill elevation is 510.5 ft.
Velocity calculations were made using an orifice equation (Roberson/Crowe, Engineering Fluid Mechanics, Fifth ed.): 

$$ Velocity = \sqrt{2gH} \quad (1) $$

Normal Pool Elevation: 538.5 ft 1929 datum
Top of Gate Opening = 526.75 ft (1912 datum) or 526.3 ft (1929 datum)
Sill Elevation of Head Gates = 511 ft (1912 datum) or 510.55 ft (1929 datum)
Tailwater at High Flow Elevation (3.8-year ACE or 15,000 cfs) = 509.9 ft (1929)

Estimates of velocity through head gates at 20 ft head gate are 28 to 42 ft/s depending on the location within the water column of the gate opening (Head Gate sill to the Top of the gate opening). Discharge and velocity calculations appear in Tables 5, 6, and 7.
Table 5. Head Gates Discharges at Brandon Road Dam

Head Gate Computations

<table>
<thead>
<tr>
<th>RI</th>
<th>freq elevs</th>
<th>depth (ft) on sill</th>
</tr>
</thead>
<tbody>
<tr>
<td>500</td>
<td>514.5</td>
<td>4</td>
</tr>
<tr>
<td>200</td>
<td>513.5</td>
<td>3</td>
</tr>
<tr>
<td>100</td>
<td>512.6</td>
<td>2.1</td>
</tr>
<tr>
<td>50</td>
<td>512</td>
<td>1.5</td>
</tr>
<tr>
<td>25</td>
<td>511.2</td>
<td>0.7</td>
</tr>
<tr>
<td>10</td>
<td>510.8</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Flow Equation Q

\[
Q \approx \frac{CA\sqrt{2gH}}{} (2)
\]

| Qcoeff (C) | 0.8 |
| Height Gate | 15.75 ft |
| Width Gate | 15 ft |
| Opening Area (A) | 236.25 ft^2 |
| 2g | 64.4 ft/s^2 |
| pool | 538.55 ft 1929 |
| sill | 510.5 ft 1929 |
| Head ave (H) | 20.1 ft |
| Q (head gate discharge full open) | 6800 cfs |

Table 6. Velocity Distribution of Vertical Water Column in fully open head gate opening

<table>
<thead>
<tr>
<th>Location of Spot Velocity</th>
<th>Elevation</th>
<th>Head</th>
<th>Velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of Opening (Full open gate)</td>
<td>526.25 ft</td>
<td>12.3 ft</td>
<td>28.1 ft/s</td>
</tr>
<tr>
<td>Middle of Opening</td>
<td>518.38 ft</td>
<td>20.18 ft</td>
<td>36.4 ft/s</td>
</tr>
<tr>
<td>Bottom of Opening (Sill)</td>
<td>510.5 ft</td>
<td>28.05 ft</td>
<td>42.5 ft/s</td>
</tr>
</tbody>
</table>
### Table 7. Incremental Gate Opening Velocity Analysis, Velocity Equation (1) is used

<table>
<thead>
<tr>
<th>Gate Opening (ft)</th>
<th>Coef</th>
<th>A</th>
<th>H (ft)</th>
<th>Total Q (cfs)</th>
<th>Ave Q per foot</th>
<th>Frequency</th>
<th>Average Velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.73</td>
<td>0</td>
<td>28.05</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.3</td>
<td>0.73</td>
<td>4.5</td>
<td>27.90</td>
<td>139</td>
<td>464</td>
<td>25-yr</td>
<td>30.9</td>
</tr>
<tr>
<td>0.7</td>
<td>0.73</td>
<td>10.5</td>
<td>27.70</td>
<td>324</td>
<td>462</td>
<td>25-yr</td>
<td>30.8</td>
</tr>
<tr>
<td>1</td>
<td>0.73</td>
<td>15</td>
<td>27.55</td>
<td>461</td>
<td>461</td>
<td>50-yr</td>
<td>30.6</td>
</tr>
<tr>
<td>1.5</td>
<td>0.73</td>
<td>22.5</td>
<td>27.30</td>
<td>689</td>
<td>459</td>
<td>100-yr</td>
<td>30.4</td>
</tr>
<tr>
<td>2</td>
<td>0.73</td>
<td>30</td>
<td>27.05</td>
<td>914</td>
<td>457</td>
<td>200-yr</td>
<td>30.6</td>
</tr>
<tr>
<td>2.1</td>
<td>0.73</td>
<td>31.5</td>
<td>27.00</td>
<td>959</td>
<td>457</td>
<td>200-yr</td>
<td>30.4</td>
</tr>
<tr>
<td>3</td>
<td>0.74</td>
<td>45</td>
<td>26.55</td>
<td>1377</td>
<td>459</td>
<td>200-yr</td>
<td>30.6</td>
</tr>
<tr>
<td>4</td>
<td>0.74</td>
<td>60</td>
<td>26.05</td>
<td>1819</td>
<td>455</td>
<td>500-yr</td>
<td>30.3</td>
</tr>
<tr>
<td>5</td>
<td>0.75</td>
<td>90</td>
<td>25.05</td>
<td>2711</td>
<td>452</td>
<td>30.1</td>
<td>30.1</td>
</tr>
<tr>
<td>6</td>
<td>0.76</td>
<td>105</td>
<td>24.55</td>
<td>3173</td>
<td>453</td>
<td>30.2</td>
<td>30.2</td>
</tr>
<tr>
<td>7</td>
<td>0.77</td>
<td>120</td>
<td>24.05</td>
<td>3636</td>
<td>455</td>
<td>30.3</td>
<td>30.3</td>
</tr>
<tr>
<td>8</td>
<td>0.77</td>
<td>135</td>
<td>23.55</td>
<td>4048</td>
<td>450</td>
<td>30.0</td>
<td>30.0</td>
</tr>
<tr>
<td>9</td>
<td>0.78</td>
<td>150</td>
<td>23.05</td>
<td>4508</td>
<td>451</td>
<td>30.1</td>
<td>30.1</td>
</tr>
<tr>
<td>10</td>
<td>0.79</td>
<td>165</td>
<td>22.55</td>
<td>4967</td>
<td>452</td>
<td>30.1</td>
<td>30.1</td>
</tr>
<tr>
<td>11</td>
<td>0.8</td>
<td>180</td>
<td>22.05</td>
<td>5426</td>
<td>452</td>
<td>30.1</td>
<td>30.1</td>
</tr>
<tr>
<td>12</td>
<td>0.81</td>
<td>195</td>
<td>21.55</td>
<td>5884</td>
<td>453</td>
<td>30.2</td>
<td>30.2</td>
</tr>
<tr>
<td>13</td>
<td>0.82</td>
<td>210</td>
<td>21.05</td>
<td>6340</td>
<td>453</td>
<td>30.2</td>
<td>30.2</td>
</tr>
<tr>
<td>14</td>
<td>0.82</td>
<td>225</td>
<td>20.55</td>
<td>6630</td>
<td>442</td>
<td>29.5</td>
<td>29.5</td>
</tr>
<tr>
<td>15</td>
<td>0.8</td>
<td>236.25</td>
<td>20.175</td>
<td>6813</td>
<td>426</td>
<td>28.8</td>
<td>28.8</td>
</tr>
<tr>
<td>16</td>
<td>0.8</td>
<td>236.25</td>
<td>20.175</td>
<td>6813</td>
<td>426</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The average velocity through the head gates using a conservative water depth (H) of 20 ft is 30 ft/s. A maximum velocity threshold for ANS transfer has not yet been specified, so it is currently unknown what velocity is too great for ANS transfer. Therefore, Head Gate velocities were looked at more closely to determine if initial velocity estimates could at all be lowered by such factors as debris, downstream submergence, and gate operations. These further analyses found that 28 ft/s is the lowest spot velocity possible through the head gates. The 28.8 ft/s in Table 7 is an average velocity of a fully opened gate in the incremental gate opening analysis. In Table 6, the 28.1 ft/s is the lowest estimated velocity in the water column of the fully opened gate, representing the most conservative (lowest) velocity for consideration of ANS passage. The highest estimated velocity is 42.5 ft/s located right above the head gate sill, this area being the most likely location for fish to first attempt to pass through the head gate opening.

#### 4.1. Debris Concern

The consideration of debris causing pockets of lower velocity has been examined. The head gates are 15 ft wide and 16 ft tall. They can be operated with partial openings and the approximate discharge is 450 cfs per foot of opening. From experience at the site, there has never been trouble with debris in the head gates; debris gets caught on the Tainter gates because the debris is floating and a debris and ice wall is in place at this dam. If debris was pulled downward and through the head gates it would most likely occur during a time the gates are fully opened and able to pass such debris. A 16 ft gate opening could then pass debris below it due to the high velocities produced by the head pressure.
4.2. Head Gate Tailwater Submergence

A literature search was conducted to address the condition where tailwater submergence would decrease the velocities through the head gates. The following three journal articles were used to conduct a submerged velocity analysis given the geometry at Brandon Road Dam:


Figure 6 shows the location of submergence at h1 in the upper portion of the head gate opening.

Figure 6. Domain and Boundary Conditions for 2D RANS simulation of the submerged sluice gate

Spreadsheet calculations show that as the tailwater rises and submerges the head gate jets, the average velocity through the gate becomes smaller. Also, as the difference between Pool and Tail decreases, the average velocity through the gate decreases. However, site conditions at Brandon Road do not permit this difference to become too small to significantly decrease velocities through the head gates. Velocity decreases of 2 ft/s were seen at various elevations through the water column. The minimum velocity through the Head Gates at Brandon Road Dam should be considered to be 28 ft/s.

During the literature research for sluice gate discharge under submerged tail conditions, all the figures showed that the maximum velocity on the vertical velocity distribution curves is near the channel bottom both upstream and downstream of the sluice gate. Therefore, it is possible to increase the average velocity through the head gates by opening more head gates and keeping them at smaller openings; however the increase in average velocity through these head gates would be small (approximately 2 ft/s). The maximum burst velocity of ANS must be 28 ft/s or higher for upstream passage of ANS to occur through the Head Gates of Brandon Road Dam.
4.3. Head Gate Operation

The following procedure will be used to operate the dam and regulate the upper pool. This procedure will allow the dam to be regulated to conform to “Brandon Road Dam Design” dated 23-Sept-2009.

1. The Tainter Gates will be used as the main gates to regulate the pool.

2. After the tainter gates are at full capacity, head gates will be operated in the following order:
   Head Gate #14, 16, 13, 15, then 12 to 9. At most, 3 of the 8 head gates get used, but all of them are operational.

3. Head Gate #14 is opened first and can be remotely operated from the lock house.

4. Head Gate #16 should be opened next. This gate should only be opened after Head gate # 13 is at full capacity. As this gate is right next to the dam building, there are concerns of scouring in the area.

5. The traveling head gate hoist at Head Gate #13 will be used after Head Gate #16 is at full capacity. Head Gate #13 has had new seals installed on the gate and should operate normally. Using a gate operated with the movable hoist has a risk of one hook on the strongback releasing from the gate and the gate being opened with one hook. This places all of the weight of the gate on two cables instead of four increasing the possibility of a cable being overstressed and breaking. Raising the gate in this manner also causes the gate to be pulled to one side and binding can occur.

6. Head Gate #15 should be opened next. This used to be the first head gates opened but #14 was later used as the first opened head gate.

7. After Head Gate # 15 is opened, Head Gates # 12 thru 9 will be opened in that order in the event of a major flow event.

5) Filling Valves

The purpose of this analysis is to quantify the amount of water that enters the lock chamber during lockage and flushing operations. These calculations were made manually and were verified by direct measurement by the USGS on December 9, 2014. This information was given to ERDC to help their modeling of 3D flushing of ANS particles (floaters) in the lock chamber. This ultimately led to the decision to select a 15-minute flushing time for the study.

Valve height 8.5 ft in valve assembly drawing (and 7’- 9” in the water control manual)
Drawing “Valve Well Steelwork General” shows 9”- 0” high. 9’x 9’
Culvert width under the valve is 9 ft
Valve type: vertical lift gate (sluice gate), sill elevation, Flat Pool elevation 538.5 ft
Sill elevation of lift gate 506 ft (upper valve) and 492.7 ft (lower valve).

When the upper valves are greater than 1/4 open and there is a normal tailwater elevation, head pressure on the upstream side of the valve cause the gate to vibrate or get stuck when closing.
Typically the lock filling valve is under 32.5 ft of head (538.5 – 506.0), and the tailwater is at its flat pool elevation of 504.5 ft 1929 datum. A simplified diagram appears in Figure 7, along with equations and discharge calculations in Table 8. The discharge coefficients are taken from the Hydraulic Design Criteria (HDC) 320-1 and shown in the graph on Figure 8.

Free Discharge Equation: \[ Q = C \times A \times \sqrt{2gH} \times (\% \text{ Gate Opening}) \]

![Figure 7. Sluice Gate Discharge, free discharge and submerged discharge](image)

**Table 8. Discharge Calculations of Lock Chamber Filling Valves at Brandon Road Lock**

<table>
<thead>
<tr>
<th>Sluice Flow (cfs)</th>
<th>C</th>
<th>A</th>
<th>H</th>
<th>Gate Opening percent</th>
<th>Opening ft</th>
<th>ho/h1</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.73</td>
<td>0</td>
<td>32.50</td>
<td>0.0%</td>
<td>0</td>
<td>0.00</td>
</tr>
<tr>
<td>335</td>
<td>0.73</td>
<td>10.125</td>
<td>31.99</td>
<td>12.5%</td>
<td>1.125</td>
<td>0.03</td>
</tr>
<tr>
<td>665</td>
<td>0.73</td>
<td>20.25</td>
<td>31.43</td>
<td>25.0%</td>
<td>2.25</td>
<td>0.07</td>
</tr>
<tr>
<td>1002</td>
<td>0.74</td>
<td>30.375</td>
<td>30.86</td>
<td>37.5%</td>
<td>3.375</td>
<td>0.10</td>
</tr>
<tr>
<td>1342</td>
<td>0.75</td>
<td>40.5</td>
<td>30.30</td>
<td>50.0%</td>
<td>4.5</td>
<td>0.14</td>
</tr>
<tr>
<td>1706</td>
<td>0.77</td>
<td>50.625</td>
<td>29.74</td>
<td>62.5%</td>
<td>5.625</td>
<td>0.17</td>
</tr>
<tr>
<td>2080</td>
<td>0.79</td>
<td>60.75</td>
<td>29.18</td>
<td>75.0%</td>
<td>6.75</td>
<td>0.21</td>
</tr>
<tr>
<td>2495</td>
<td>0.82</td>
<td>70.875</td>
<td>28.61</td>
<td>87.5%</td>
<td>7.875</td>
<td>0.24</td>
</tr>
<tr>
<td>2754</td>
<td>0.80</td>
<td>81</td>
<td>28.05</td>
<td>100.0%</td>
<td>9</td>
<td>0.28</td>
</tr>
</tbody>
</table>
The average velocity in the chamber would be approximately 0.85 ft/s when flushing continuously using a valve opening of 25%. At this valve opening each filling culvert has a discharge of 665 cfs. The valve opening of 25% is used because, by experience, this is the maximum opening the valve can be before pinning forces on the gate become a concern during existing debris flushing operations. It is possible for these valves to be re-designed to accommodate larger openings, which will be looked at by the design team in the future.

Flushing is accomplished through the existing side filling ports of the lock, which directs flow into the chamber and out of the opened miter gates downstream. This procedure leaves a “recirculation zone” at the upstream portion of the lock chamber that may not be able to be flushed. Various alternatives were considered by the study team to reduce or eliminate this recirculation zone, and a 3D model was used by ERDC to give recommendations of the best alternatives.

During flushing operations, the lower miter gates must be secured in their recesses using straps or latches to avert the possibility of being pulled into the current and off their hinges. (Miter gates falling off in this fashion are rare but have occurred resulting in long periods of navigation shutdown while the gates are being replaced and gate connections are being repaired.) Securing the miters was done during the USGS data collection period Dec 8-10, 2014. It is recommended that future anti-ANS flushing operations use a permanent latching system for the downstream miter dates (which must be constructed).
Figure 8. Discharge Coefficients of Lock Filling Valve Conduits, Hydraulic Design Criteria (HDC) 320-1
6) ANS Transfer via I&M Canal

The I&M Canal was built to increase commerce by connecting Lake Michigan to the Mississippi River. The canal was 6 ft deep, 60 ft wide, 96 miles long, and had 15 locks. Construction was completed in 1848. Its function was largely replaced by the wider and shorter Chicago Sanitary and Ship Canal in 1900 and it ceased transportation operations with the completion of the Illinois Waterway in 1933. Since then, the canal has been developed for recreation. The I&M Canal Lock has been closed with a permanent concrete bulkhead placed where the former lock miter gates were located. Current photos of the I&M Canal Lock are shown below; Figure 9 is looking upstream and Figure 10 is looking downstream. The valve shown in Figure 9 has been permanently sealed off by concrete.

Figure 9. I&M Canal Lock at Brandon Road closed by a permanent concrete bulkhead
The purpose of this analysis is to investigate whether or not ANS transfer at the I&M Canal Lock is possible from any of three locations. All of these pathways have been found to be highly unlikely but will be further analyzed during PED (preconstruction, engineering, & design). Details of these analyses are given in the following sections below.

The possibility of ANS has been investigated to determine:

1) whether the Brandon Road Pool can rise and overtop the I&M Canal’s Lock Bulkhead providing a pathway for ANS,

2) whether a high tailwater on the Illinois Waterway can bring the water level in the I&M Canal above the invert elevation of a valve that penetrates the permanent concrete bulkhead, assuming this valve would be open at the time, or

3) whether overland flow could allow fish access from the I&M Canal to the pool during extreme flood conditions, either over the I&M Canal Lock bulkhead or around it.

6.1. Brandon Road Pool Fluctuations

The Brandon Road pool is held at elevation 538.5 ft throughout the year, and even during flood events. It is possible at this site to hold pool during flood events because there are an adequate number of head gates that can be opened to pass incoming flood flows. A large flood flow of 36,000 cfs (1/500 ACE event) can be passed by four head gates fully open, and there are eight head gates available on the dam. Therefore, the pool rising and overtopping the I&M Canal bulkhead (crest 542 ft) will not occur. See Figure 10 for level of Brandon Road Pool compared to the crest of the I&M Lock Bulkhead.

Figure 10. I&M Canal Lock at Brandon Road (looking downstream)
6.2. High Tailwater on the Illinois Waterway
Normal tailwater is at an elevation of 504.5 ft and can rise ten feet to 514.5 during the 1/500 Annual Chance Exceedance (ACE) Flood. The I&M Canal is separated from the Illinois Waterway by a canal berm and a roadway berm; the I&M Canal does not have a direct connection to the Illinois Waterway near Brandon Road, and does not experience the same fluctuations in water levels. The tailwater on the Illinois Waterway connects to the I&M Canal approximately 10 miles downstream from Brandon Road Lock, and water would have back up into the I&M Canal 10 miles for water levels to be impacted near the I&M Canal Lock; this cannot occur even during a 1/500 ACE flood.

The separation of the I&M Canal from the IWW can be seen in Figure 11. This figure is a GIS generated map of 1 ft LIDAR data showing a close-up view of the elevations in the area of Brandon Road Lock and Dam. Highway 6 is approximately 6 ft higher than the 1/500 ACE event in the IWW tailwater at its lowest point (see Figure 12).

Figure 11. LIDAR elevations near Brandon Road Lock and the I&M Canal Lock
6.3. I&M Canal Overland Flow
The potential pathway of ANS using overland flow on Thorn Creek to bypass Brandon Road Lock & Dam at the I&M Canal lock was investigated. The Flood Insurance Rate Map shows a flood elevation of 537 ft (1/100 ACE event) at the junction of Thorn Creek and the I&M Canal (Figure 13). The 1/500 ACE event is 537.2 ft on the corresponding profile graph (Figure 14).
The crest elevation of the permanent concrete bulkhead on the I&M Canal Lock is 542 ft, so a major flood from Thorn Creek cannot overtop it (537.2 ft is the 500-Year Flood on FIRM profile) and this pathway is not viable.

From Figure 10, the invert of the culvert is well below 537.2, so a 500-year flood on Thorn Creek, for example, could raise water levels in the I&M Canal above the invert of this culvert. This could lead to a direct pathway for ANS if the valve inside the culvert is operational. Little information exists about this valve but from Figure 9 it is known that the valve is currently closed. Further investigation in the field show this valve to have been concreted off, although some minor leakage is visible.

The analysis above focuses on if the tailwater were to rise, could ANS go into the tributary; it does not evaluate the stormwater drainage network adjacent to the I&M canal to see if ANS could then get to the pool upstream of the lock. An interior drainage study would need to be conducted to accurately assess the I&M canal water levels during different precipitation events and how the storm drainage network might function as a pathway. However, such a study is not expected to produce a pathway for upstream transport of ANS because of the high elevation of the physical embankment separating the Brandon Road pool (539 ft) from the I&M canal. Such overland floodwater is likely to enter the I&M Canal and not allow a direct connection during extreme flood events.

7) Water Supply during Low Flow Periods

Brandon Road Lock and Dam is a navigation dam and is not authorized or designed to store water for other purposes. The term “water supply” in this document refers to the inflow of water from the Illinois Waterway plus the possible use of natural water volume within a 0.5 foot operational band for maintaining navigable depths in the pool. A major portion of the inflow to Brandon Road Dam comes
from releases from the hydropower plant at Lockport, located five miles upstream of Brandon Road Lock and Dam. The Lockport Hydropower Plant is operated by the Metropolitan Sanitary District of Greater Chicago (MSDGC). The pool extends from Brandon Road Lock and Dam to Lockport Lock and Dam. If the flushing demand is less than the water supply, water can be used to flush ANS neutrally buoyant particles (floaters) out of the chamber prior to each lockage.

Daily average discharges in low flow periods vary, but 1400 cfs is considered a typical value for low flow discharge at Brandon Road (see Table 9). The month of November has the greatest potential for low flows, and 981 cfs is considered a minimum daily average during this month (although lower flows can occur for short periods of time). Table 9 shows monthly values and other statistics based on the USGS gaging station at Ruby Street upstream of Brandon Road Lock and Dam.

### Table 9. USGS Flow Statistics at the Ruby Street Gaging Station
(USGS 05537980 DES PLAINES RIVER AT ROUTE 53 AT JOLIET, IL 2005-2014 data)

<table>
<thead>
<tr>
<th></th>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>Jun</th>
<th>Jul</th>
<th>Aug</th>
<th>Sep</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min Daily Ave</td>
<td>1412</td>
<td>1519</td>
<td>2209</td>
<td>1886</td>
<td>1913</td>
<td>1811</td>
<td>1774</td>
<td>1642</td>
<td>1652</td>
<td>1442</td>
<td>981</td>
<td>1434</td>
</tr>
<tr>
<td>10th Percentile</td>
<td>n/a</td>
<td>1540</td>
<td>2209</td>
<td>1886</td>
<td>1913</td>
<td>1811</td>
<td>1774</td>
<td>1642</td>
<td>1652</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>25th Percentile</td>
<td>1816</td>
<td>1960</td>
<td>2879</td>
<td>2808</td>
<td>2492</td>
<td>2550</td>
<td>2318</td>
<td>2296</td>
<td>2109</td>
<td>1753</td>
<td>1303</td>
<td>1787</td>
</tr>
<tr>
<td>Mean of Monthly</td>
<td>3060</td>
<td>3410</td>
<td>4820</td>
<td>4920</td>
<td>4080</td>
<td>4300</td>
<td>3660</td>
<td>4230</td>
<td>3810</td>
<td>2720</td>
<td>2110</td>
<td>3300</td>
</tr>
<tr>
<td>75th Percentile</td>
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#### 7.1. Navigation during Low Flow Periods
The Brandon Road pool is very narrow and does not have overbank areas. Because of this, water supply is limited and it is possible for the water level to drop even if all of the Tainter gates at the dam are closed due to leakage and lockage. Flushing of the lock chamber prior to each lockage requires a certain volume of water depending on the duration of flushing and the size of valve opening made to discharge water to the lock chamber. The pool must be controlled within a maximum of 0.5 ft from normal pool (elev 538.5 ft) to stay within authorized navigational limits. Tainter gate operations on the dam need to be coordinated with ANS flushes, especially in times of low flow or drought.

The maximum flushing discharge is currently of 1350 cfs (currently the valve opening maximum is 25% during flushing operations). This flow was measured by the USGS on Dec 9, 2014. This represents the flushing demand, so any river discharge below 1350 cfs will not meet this demand. A new valve design is being considered to allow larger valve openings during flushing, which could reduce the flush time needed prior to each lockage.

The average filling discharge of 1700 cfs was calculated using a 19 minute fill time and the dimensions of the lock chamber containing a 3x3 barge cut assembly drafting 9 ft. During this portion of the lockage, any river discharge below 1700 cfs will not meet this demand.

Flushing during periods of low flow conditions may be difficult if the flushing demand is greater than the water supply. In times of prolonged low flow periods, it may be necessary to reduce the duration of
flushing (to less than 15 minutes) or reduce the valve opening (to less than 25% open), if not preclude flushing altogether. The number of vessels that can be flushed and locked were analyzed for different flow conditions. Statistics were analyzed by the PCX Navigation Economics appendix and discharge data came from USGS gage 05537980 DES PLAINES RIVER AT ROUTE 53 AT JOLIET, IL. A summary of these water supply analysis results are shown in Table 10.

Table 10. Water Supply Analysis Results (water budget spreadsheet calculations)

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<thead>
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<th>River Discharge</th>
<th>Effect on Water Supply</th>
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<td>0 cfs (no flow)</td>
<td>Approx. 2 lockages can be made before cutting back on flushing duration or valve opening. No Flow occurrences are short duration events.</td>
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<tr>
<td>981 cfs (November Min Daily Ave flow)</td>
<td>Approx. 6 lockages can be made before cutting back on flushing duration or valve opening</td>
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<td>1400 cfs (Typical Low Flow)</td>
<td>Approx. 22 lockages can be made before cutting back on flushing duration or valve opening</td>
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<td>1700 cfs and higher</td>
<td>Adequate water supply for 15-min flushing operations</td>
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Brandon Rd Lock and Dam has a high head compared to the rest of the dams on the Illinois Waterway. It never shuts down during high flow, although barges may tie off upstream of the lock during high flows because the upstream lock (Lockport) is closed. 15,000 cfs is typically when navigation ties off and lockages cease at Brandon Road Lock. More information on Navigation Traffic is located in the PCX Navigation Economics appendix.

Downstream tie off locations for tows are necessary during flushing periods so that ANS floaters can be pushed out of the lock chamber and further downstream. The closer to the lock the tow ties off, the less volume of water that is needed to be flushed. The potential location of an electric barrier may limit where the tie-off locations can be, and future tests of the electric field will likely need to be done to determine this. More information to be found in the PCX Navigation Economics appendix.

7.2. Required Number of Flushes

The required number of flushes is the number of upbound lockages, both commercial and recreational lockages, plus the number of downbound double lockages. The reason for flushing downbound traffic is due to “return water” that comes into the chamber when the first downbound cut exits the lock. The return water can carry ANS floaters which could then be transported upstream when the second half of the vessel enters the chamber.

The lowest flow month is typically November (Table 9) when hydropower operations change upstream. Based on past navigation traffic during this month, there are typically between 5-7 lockages per day. Navigation statistics are located in the PCX Navigation Economics appendix.

7.3. USGS Lock Chamber Velocity and Discharge Measurements

During the data collection on Dec 8-9, 2014, two Tainter gates were opened at the beginning of the collection. When the pool fell to its lower limit, gate #2 was shut; however, gate #1 could not be shut due to a large tree stuck in the gate. The tree was too large for small scale removal techniques so a crane will be needed. Debris on the Tainter gates occurs frequently at this site, so it is recommended that at least two gates be sheltered by a debris boom so they can be reliably operated in the future during ANS flushing.
The stage hydrograph in Figure 15 shows the water levels in Brandon Road Pool during the USGS site visit December 8-9, 2014. The drop in pool from Max operation to Min operation in less than 2 hours can be best seen on Dec 9th from 4:00 pm to 6:00 pm. Valve discharge of 1350 cfs was continuously flushing through chamber during this time while the USGS collected velocity data in the lock chamber. One Tainter gate was stuck open during this test due to a large stump which could not be removed. The discharge through this gate was approximately 500 cfs (estimated visually from 550 cfs per tainter that is typically discharged by one gates). The total discharge, loss to the pool, was 1850 cfs during the USGS tests. The pool can drop from maximum to minimum (0.5 ft operational band) in 2 hours with 1850 cfs.

**Figure 15. Pool Hydrograph at Ruby Street Gaging Station upstream of Brandon Road Dam during time of USGS field tests, December 8-9, 2014**

7.4. Conclusions on Water Supply
Flushing the lock chamber before each lockage at Brandon Road Lock is being considered to reduce the likelihood of Aquatic Nuisance Species (ANS) passing upstream into the Great Lakes. The effectiveness of flushing increases with both the flushing duration and flushing discharge. There may be periods of low flow in which flushing operations must be reduced in order to preserve navigation or ceased altogether (Ref Table 10). These procedures would preserve the available water supply in order to preserve (or extend) navigation in times of drought. It is recommended that at least two gates be sheltered by a debris boom so they can be reliably operated in the future during ANS flushing.

8) Floodplain Modeling

8.1. Background and Purpose
Preliminary hydraulic modeling was completed to determine potential water surface impacts as a result of the current design. This section presents the modeling methodology and results of the analysis. The design for the engineered channel and rock placement will continue to evolve as the design progresses. As a result the hydraulic modeling will need to periodically be revisited to determine and assess the potential changes to the water surface elevations as a result of the project.

The 2005 Floodway HEC-RAS unsteady flow model for Illinois Waterway, Dresden Island Lock & Dam navigation pool served as the starting point for this modeling effort. HEC-RAS software version 5.0.4 was used to compute the frequency flood events presented in this analysis. This analysis considers the effects of the rock placement and engineered channel. No other existing or proposed construction was identified to include in the analysis. The hydraulic model extends from tailwater of Brandon Road Dam (RM 285.9) to the Pool of Dresden Island Lock and Dam (RM 271.5). The Floodway model base, or existing condition, geometry was developed from the same terrain, bathymetric data and cross section
locations used in the hydraulic models of the 2004 Upper Mississippi River System Flow Frequency Study (UMRSFFS). During the 2005 Floodway Study the base geometry was adjusted, mainly using Manning’s roughness values, to closely reproduce the UMRSFFS frequency stage for the 1% Annual Chance Event, when applying the UMRSFFS 1% Event discharges.

The model boundaries were developed from the 2004 UMRSFSS Flood Event flows and stages for each of the eight frequency flood events between the 50% annual chance events (2-year) and 0.2% (500-year). The upstream inflow boundary represents the computed discharge at the tailwater for Brandon Road Dam as computed during the UMRSFFS for each frequency event. The downstream stage boundary represents the computed stage at the pool of Dresden Island Dam, converted to NAVD88 elevations, as computed during the UMRSFFS for each frequency event. Additional lateral inflows were added to account for the inflow hydrographs of the Kankakee River, the Du Page River and an estimate of ungaged inflows. These inflow hydrographs were also applied during the UMRSFFS.

The existing Floodway HEC-RAS model cross sections were modified to incorporate existing condition and proposed condition cross sections developed by the Brandon Road Project Delivery Team. These cross sections are included in the plan set, 451617_BrandonRoad_Draft Plan Set_31 MAY 18-Backcheck Set.pdf. The plan set cross sections represented only the portion of the river section near the approach channel and dike, and did not extend across the full width of the river. Each plan set cross section was digitized from the plan drawings and inserted into the existing HEC-RAS cross sections, overwriting the necessary portion of the existing model cross sections. This created an Existing Condition and a Project Condition set of HEC-RAS cross sections to be used for the comparative analysis.

This information is considered preliminary and will continue to evolve as the design progresses. The modeling and results will be coordinated with the ILDNR for floodplain permitting requirements.

8.2. Figures and Results
The following figures and tables show the model extents, cross section locations, and model results for four selected frequencies (0.2%, 1%, 10%, 50% annual chance exceedance). Additional design or channel modifications may be required for compliance with State of Illinois Floodplain Regulations. Coordination with the IDNR and other stakeholders will continue through the design phase.
Figure 16. Model Extents and Cross Section Locations
Figure 17. Project Vicinity Cross Section Location, Floodway, and Rock Placement Areas.
Figure 18. HEC-RAS Cross Section showing existing and proposed conditions.

Table 11. 0.2% Annual Chance Event Summary of Results

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<th>W.S. Elev. (ft)</th>
<th>W.S. Incr. (ft)</th>
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### Table 13. 10% Annual Chance Event Summary of Results

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Attachment 3:

H&H Information for Brandon Road Lock (GLMRIS-BR)
GLMRIS – BR – H&H Bypass Assessment

Basis for the Analysis
The Des Plaines River originates in Racine County in southern Wisconsin and flows in a general southerly direction to its confluence with Salt Creek in Riverside, Illinois. It then flows southwesterly to its confluence with the CSSC near Lockport, Illinois. A portion of this reach that flows to the southwest is situated parallel and adjacent to the CSSC, and the two waterways are separated by a strip of land only a few hundred feet across. The strip of land between the Des Plaines River and CSSC accommodates industrial plants, navigation facilities and recreational bike trails. It can be accessed through small access roads. There were two large spoil banks, mostly consisting of the debris left from the canal construction, which existed on this strip of land near Romeoville. These spoil banks functioned as a levee that prevented the Des Plaines River water from overflowing to the CSSC during flood events. The spoil banks were removed in the 1990s, and overflows into the CSSC have been observed several times during flood events. The water surface elevation on the CSSC is mainly controlled by the Lockport Lock and Dam. The stage on the Des Plaines River can significantly rise during flood events, but the stage on the CSSC will rise by a much lesser degree due to canal operations.

The construction of the Des Plaines Bypass Barrier, recommended in Interim I, Efficacy Study, was completed in 2010. The bypass barrier composed of a 13 mile jersey barrier/fence upstream of the CSSC-EB and is an interim risk reduction measure to reduce the probability of fish bypass of the CSSC-EB. The bypass could occur when the Des Plaines River overflows to the CSSC upstream of the CSSC-EB control point. The Draft Fish and Wildlife Coordination Act Report indicated that there is still a concern of flood bypass from the Des Plaines River to the CSSC upstream of the CSSC-EB for various life stages of ANS.

In order to formulate complete alternatives, an H&H assessment was conducted to determine whether hydraulic bypasses and/or connections around Brandon Road Lock and Dam (BRLD) could facilitate ANS passage around an ANS control point located at BRLD. The investigation included a search for potential connections to the Des Plaines River Watershed from the DuPage and Fox River Watersheds. The FEMA 100-year floodplain extents were reviewed to identify locations where hydraulic connections between the various watersheds could potentially exist during periods of high water. This analysis of the waterway connections in and around the BRLD concluded that aquatic pathways around BRLD generally only exist for flood events estimated to be at or between the 100-year and 500-year flood events. Areas reviewed as part of this analysis are discussed in more detail, on watershed by watershed basis, below. Identified possible connections are shown on Figure H&H Bypass Assessment - 1. A summary of the identified connection points, including the findings from the analysis and estimated annual exceedance frequency are contained in Table H&H Bypass Assessment - 1.
Figure 1. Assessed Hydraulic Connections
Table 1. Assessed Hydraulic Connections, Estimated at the 500-Year Event

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<th>Aquatic pathway starts at DuPage River at confluence with Des Plaines River</th>
<th>Over/around Channahon Dam</th>
<th>Continue upstream to Rock Run</th>
<th>Overland flow connection, at 100-year event, through wetland area to Des Plaines River watershed, Crest Hill</th>
<th>Aquatic pathway to Des Plaines River upstream of Brandon Road Lock and Dam</th>
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<td>Continue upstream to East Branch DuPage River</td>
<td>Overland flow connection, at 500-year event, with Westwood Creek, Lombard</td>
<td>Westwood Creek discharges to Salt Creek</td>
<td>Salt Creek discharges to Des Plaines River</td>
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<td>Over/through Dayton Dam</td>
<td>Continue upstream to Poplar Creek</td>
<td>Overland flow connection, at 500-year event, with Tributary D, Paul Douglas Forest Preserve</td>
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<td>Overland flow connection, at 500-year event, Spring Brook, Bloomingdale 1</td>
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Attachment 4:
GLMRIS – BR – H&H Bypass Assessment
DuPage River Bypass Analysis

DuPage River-I&M Canal-Rock Run Tributary-Des Plaines River. A potential hydraulic bypass via the DuPage River, I&M Canal, Rock Run Tributary and other intermediary connections was identified during the initial stages of a feasibility study to assess flooding risk for the DuPage River watershed in DuPage and Will Counties, Illinois. The potential bypass is called “DuPage River/Rock Run Connection” on Figure H&H Bypass Assessment – 2 with the specific connection points listed below highlighted.

- **Connection Point #1.** DuPage River at confluence with Des Plaines River
- **Connection Point #2.** Over or around Channahon Dam on the DuPage River
- **Connection Point #3.** From the DuPage River to the Illinois and Michigan (I&M) Canal through:
  - **Connection Point #3a.** The open (inoperable) historic I&M Canal Lock 7; or
  - **Connection Point #3b.** An open floodplain connection during high water events
- **Connection Point #4.** From the DuPage River to Rock Run Tributary at confluence
- **Connection Point #5.** Through the Rock Run Tributary across DuPage River/ Des Plaines River watershed divide in a wetland area to Tributary A to Des Plaines River
- **Connection Point #6.** Through Tributary A to Des Plaines River near Lockport Lock and Dam

Figure 2. Potential hydraulic bypass of the Brandon Road Lock and Dam Site.

A detailed site investigation and inspection was completed in April 2016 to assess the possibility for fish to swim the potential aquatic pathway identified and successfully bypass a control point located at Brandon Road Lock. Based on the site investigation and H&H assessment, the aquatic pathway is
estimated to exist at or above the 100-year flood event. However, it was determined the likelihood for Asian carp or other fish passage through these numerous connections is very low. This location was screened out of consideration for the implementation of an ANS control measure based on the aquatic pathway estimated to be at or above a 100-year flood event. Although a hydraulic passage has been identified during the 100-year flood event, fish would have difficulty navigating between the watersheds during flood flows due to a labyrinthine pathway full of constricting drainage structures, dense vegetation, and a highly variable flow regime. Additionally, this pathway offers less incentive to attract fish movement than existing areas of the waterway. Therefore, the likelihood that ANS may transfer through this aquatic pathway was rated as low and did not require a control point. The site investigation and resulting passage analysis was shared with the Asian Carp Regional Coordinating Committee Monitoring and Response Workgroup, and the current monitoring plan has incorporated actions to account for this low risk. A summary report on the connection as well as the field evaluation is attached to this bypass assessment (Attachment H&H Bypass Assessment-1).

**DuPage River-Salt Creek Tributaries-Des Plaines River.** Further upstream, there are other potential locations that could possibly connect the DuPage River to the Des Plaines River at an estimated 500-year level or greater via overland flow or through possible sewer connections due to the significant inundation associated with a 500-year event. A possible DuPage River bypass is located in Oak Brook where surface flow across the Midwest University Campus could enter Ginger Creek, a tributary to Salt Creek. See “Midwest University” on Figure H&H Bypass Assessment –1. Salt Creek is a tributary of the Des Plaines River. The next two possible bypass sites are located in Bloomingdale. See “Bloomingdale 1 and 2” on Figure H&H Bypass Assessment –1. The potential connection from the DuPage River could occur at two locations on Spring Brook, a tributary to Salt Creek. Salt Creek connects to the Des Plaines River north of Brandon Road and Lockport Locks and Dams. The final potential bypass point identified is located in Lombard near North Avenue. See “Lombard” on Figure H&H Bypass Assessment –1. At this location, overland flow from the DuPage River can potentially connect to Westwood Creek, also tributary to Salt Creek, and then to the Des Plaines River.

These overland connections to the Des Plaines River are all located geographically north of BRLD. As noted, these connections are possible for a very large, infrequent event, estimated to be equal to or greater than the 500 year event. For these locations a site investigation was not conducted. In addition, review of any detailed topography and/or sewer maps was not conducted as part of the assessment. These locations were screened out for implementation of an ANS control measure because an aquatic pathway is estimated to be created at or above the 500-year flood event, which is the design event for GLMRIS.

**Fox River Bypass Analysis**

The Fox River watershed is located to the west of the DuPage River, flowing from Wisconsin to its confluence with the Illinois River near Ottawa, IL. The Dayton Dam is located on the Fox River approximately 5.5 miles upstream of the confluence with the Illinois River and serves as a downstream barrier for the watershed. This 29.6 foot high concrete structure includes a hydroelectric powerhouse. The dam is the lowermost of eleven dams on a nearly 77 mile stretch of the Fox River. This large structure is considered to act as a barrier to upstream fish and ANS passage into the Fox River Watershed due to its height. Furthermore, fish are not considered to be able to swim upstream through the powerhouse turbines. Unless this structure is modified in the future to facilitate fish passage, the likelihood of transit of swimming ANS such as Asian carp from the Illinois River to the Fox River is
considered very low. The dam is located downstream of the portion of the Fox River shown in Figure H&H Bypass Assessment – 1.

For completeness, the watershed upstream of the Dayton Dam was reviewed for potential hydraulic connections to the Chicago River or Des Plaines River watersheds during large flood events. Upstream of the Dayton Dam, one location that could possibly connect during a 500 year flood event or through possible overland connections was identified in the Paul Douglas Forest Preserve, near Hoffman Estates. See Paul Douglas Forest Preserve on Figure H&H Bypass Assessment – 1. If the dam was breached or modified to allow for fish or ANS passage, an overland aquatic pathway could occur during very infrequent conditions, estimated to be at or above the 500-year event. A site investigation for this location was not conducted as part of the assessment, nor was a review of any detailed topography and/or sewer maps. This location was also screened out for consideration for implementation of a structural ANS control because it was at or above the design event for GLMRIS.

Finally, the McHenry Dam, also referred to as the Stratton Lock and Dam, is the most upstream of the eleven dams and serves as the passageway between the Fox Chain of Lakes and the Fox River. This area would seem to provide the greatest chance for potential hydraulic connections however the floodplain for the Fox Chain of Lakes was reviewed and no direct hydraulic connections were identified and therefore is not shown on Figure H&H Bypass Assessment – 1.

**Summary and Recommendations**
A hydrographic analysis of the tributary watersheds in the CAWS and Upper IWW was completed to determine whether alternative pathways exist that could allow MRB ANS to bypass a control point at Brandon Road. The analysis identified six pathways that could connect the Des Plaines River below BRLD to the Des Plaines River above BRLD at or below the 500-year flood event. The bypasses are created by events estimated to be at or above the 100-year event, but in some cases, the aquatic pathway would include passage over dams and travel through infrastructure such as culverts, retention basins and storm sewer passages. Based on the results of this hydraulic and hydrologic investigation, these locations were screened out as locations requiring a structural control measure to address upstream transfer of MRB ANS to the GLB because they met and exceeded the GLMRIS design event.
**Introduction**
A potential hydraulic bypass around the Brandon Road Lock and Dam has been identified through and open aquatic pathway in the DuPage River Watershed in Will County, IL.

The potential Bypass is portrayed in Figure 1 with the specific connection points listed below highlighted.

- **Connection Point #1.** DuPage River at confluence with Des Plaines River
- **Connection Point #2.** Over or around Channahon Dam on the DuPage River
- **Connection Point #3.** From the DuPage River to the Illinois and Michigan (I&M) Canal through:
  - **Connection Point #3a.** The open (inoperable) historic I&M Canal Lock 7; or
  - **Connection Point #3b.** An open floodplain connection during high water events
- **Connection Point #4.** From the DuPage River to Rock Run Tributary at confluence
- **Connection Point #5.** Through the Rock Run Tributary across DuPage River/Des Plaines River watershed divide in a wetland area to Tributary A to Des Plaines River
- **Connection Point #6.** Through Tributary A to Des Plaines River near Lockport Lock and Dam

![Figure 1. - Potential hydraulic bypass of the Brandon Road Lock and Dam Site.](image)

A site visit of the potential aquatic pathway was conducted by LRC and MVR staff on April 12, 2016 to assess the potential for fish to swim the potential pathway identified and successfully bypass the proposed barrier to upstream swimming fish at the Brandon Road Lock & Dam location. The visit was conducted by 3 fish biologist (Mark Cornish, Matt Shanks, and Nick Barkowski) and one hydraulic engineer (Erin Maloney). This document includes a summary of the conditions observed during the site visit as well as an assessment of expected fish passage during flood flow conditions.

Attachment 4:
GLMRIS – BR – H&H Bypass Assessment
**Assessment**

**Fish Passage into the DuPage River Waterway**
The DuPage River has a direct connection to the Des Plaines River via an open water confluence approximately 1 mile downstream of the Channahon Dam (Connection Point #1). Swimming fish have direct access to the DuPage River via this open aquatic pathway.

The Channahon Dam is an 11.5 ft high run-of-the-river dam that likely prevents movement of upstream swimming fish during normal flow conditions. However, according to Federal Emergency Management Agency (FEMA) data, the Dam is fully submerged at the 0.1-percent annual chance flood event (10 year) and any greater event (Figure 2). During periods of high flow, fish will have the ability to swim and/or jump upstream across the dam to enter the DuPage River Watershed (Connection Point #2). In addition, the dam has a small bypass pipe that may provide fish passage. The design and specifications of the bypass raceway are unknown and will be further investigated.

![Channahon Dam on the DuPage River near the Channahon Parkway State Park. The bypass raceway that may provide year round passage is in the foreground of the image.](image)

In addition to the Channahon Dam, there is a potential for fish passage into the DuPage River from the I&M Canal from the South at I&M Canal Lock 7 and the adjacent I&M Canal Feeder Gate. Figure 3 includes an oblique aerial imagery of Channahon Dam and the connection of the I&M Canal to the DuPage River via I&M Canal Locks 6 and 72.
Figure 3.— Oblique Aerial Imagery of Channahon Dam and I&M Canal lock connectivity. Inset picture is of small weir located approximately 3.3 miles south of the Channahon Dam. The weir allows water to spill over from the I&M into the Lower Des Plaines River and may be a connection during flood stage.
I&M Lock 7 is currently inoperable and sealed shut with concrete and the gate valve was closed on the date of the site visit (Figure 4). Head differential between the downstream and upstream end of Lock 7 was at least 6 ft and the head differential is likely to remain at least 6 ft during flood conditions. This location is considered to have no or very low chance of fish passage when the gate valve remains in its close position.

Figure 4.— I&M Lock 7 from downstream end. The lock has been bulkheaded shut and an operable gate valve for culvert flow is in place. The Channahon Dam pool is visible in the photo’s background.

The I&M Canal Feeder Gate, which provides a level of base flow from the DuPage River into the I&M Canal downstream, is located immediately West of I&M Lock 7. At the time of the site visit, the lifting mechanisms of the feeder gate appeared to be inoperable and the stoplogs were failing (Figure 6). Moderate flows were observed through the gaps in the stoplogs with approximately 2 ft of head between the upstream and downstream ends of the gate. Potential of fish passage at this location is considered to be moderate to high, even during normal flow conditions. Note that the I&M Canal is hydraulically disconnected from the Des Plaines River waterway but may be connected to it during flood conditions at various locations. Approximately 3.3 miles downstream of I&M Canal Lock 7 is a small weir that allows overflow from the I&M to spill into the Des Plaines River across from Haborside Marina in Wilmington, Illinois. The 100 yr flood elevation appears to be greater than the height of the weir according to Will County DTM. However, flood elevations for the I&M are not known and may limit and or prevent potential fish passage.
Fish Passage into the I&M Canal Waterway

From the DuPage River, two hydraulic connection points between the DuPage River and the I&M Canal exist. During high water events, fish may be able to enter the I&M Canal at the location of the historic I&M Canal Lock 6, which is currently inoperable (Figure 6). The location of Lock 6 is indicated in Figure 3. The water level above the lock is maintained by a concrete weir spillway, however this weir is nearly submerged at the 0.1-percent annual chance flood event (10-year), potentially providing fish access to swim or jump over the weir into the I&M canal (Connection Point #3a). During the site visit, the team had a chance to interview the site manager of the Channahon Parkway State Park, located at the lock and dam site. The site manager mentioned that water levels in the past 10 years have never exceeded the height of the lock. The site managers also stated that water velocities are very high during high flood water and he does not believe fish could swim through the increased currents. It is important to note, however, that the Asian Carp species of concern have a high swimming speed and burst rate.

Figure 6.— I&M Lock 6 from downstream end. The lock has been bulkheaded shut and the gate valve for culvert flow is likely inoperable. The I&M Canal going North is visible in the photo's background.
An additional potential pathway between the DuPage River and the I&M Canal may exist during flood events, during which a combined floodplain area for the waterways is indicated (Connection Point #3b; Figure 7). At this location, about a 0.75 mile stretch adjacent to S Canal Road in unincorporated Will County near Channahon, FEMA data indicates that the floodplain of the DuPage River reaches the elevation and overtops the berm separating the DuPage River from the I&M Canal1. Based on a review of 2014 Will County digital elevation data3, several locations at which the 0.01-percent annual chance (100 year) flood elevation is anticipated to overtop the berm by 0-2 feet have been identified. Although a hydraulic passage has been identified during extreme flow conditions, due to the dense vegetation on the top of the berm, fish may have difficulty navigating between the DuPage River and the I&M Canal during flood flows and would likely have little to no incentive to do so.

Figure 7.—Floodplain connection between the Indiana and Michigan (I&M) and the DuPage River. The DuPage river is approximately 10 ft below the I&M canal during normal flow conditions but may have connection under very high flows.

Fish Passage into the Rock Run Tributary Waterway
The I&M Canal has a direct connection to the Rock Run Tributary, via an open water confluence near Channahon, IL (Connection Point #4; Figure 8).
Further upstream on the Rock Run Tributary, the creek flows through a large wetland that is dominated by cattails, which would result in a wide dispersion of the water prior to continuing downstream towards the I&M Canal (Figure 9). The cattail marsh is approximately 0.75 miles long and small defined channel exists through about half of the marsh. According to FEMA data, the entire marsh is expected to be inundated during a 0.01-percent annual chance flood event (100 year) event and the depth of water in this area may be as high as 2-feet. Under current conditions, fish would have difficulty navigating the dense marsh of thickly vegetated cattails and other vegetation, even at a 2-foot ponding depth, and would likely have no incentive to do so.
Figure 9.— Cattail marsh upstream of the Rock Run Creek. No to little establish channel exists through this field

On the upstream end of the marsh a large box culvert travels under Plainfield Road and several concrete parking lots (Figure 10a). The culvert is approximately 0.10 miles long and has a trash rack on the upstream portion of the culvert with 4 to 6 inch gaps between the bars (Figure 10b).

Figure 10.— A) The downstream end of the box culvert that travels approximately 0.10 miles under Plainfield Road and several parking lots and buildings. B) The upstream end of the culvert, which has a trash rack on the entranced with 4 to 6 inch gaps between the rebar.

Fish Passage across DuPage and Des Plaines Watershed Divide
Traveling upstream of the box culvert, the creek is further dispersed amongst a large cattail marsh. The marsh is approximately 1.2 miles in length and does not appear to show any distinguished channel, however the marsh does contain a few ponds (Figure 11). Within the 1.2 mile stretch of cattail marsh the watershed divide between the DuPage River Watershed and the Des Plaines River Watershed exists (Connection Point #5). The area is a very flat and does not have a clearly defined flow path. Surface
water in this area may drain to either tributary, depending on meteorological and hydraulic flow conditions. According to FEMA data, the entire marsh is expected to be inundated during a 0.01-percent annual chance flood event (100 year) event and the depth of water in this area may be as high as 2-feet. Under current conditions, fish would have difficulty navigating the dense marsh of thickly vegetated cattails and other vegetation, even at a 2-foot ponding depth, and would likely have little to no incentive to do so.

Figure 11.— Cattail marsh upstream of the box culvert shown in Figure 9. The marsh expands approximately 1.2 miles long and has a few isolated ponds.

Fish Passage in the Des Plaines River Tributary A Waterway
The corners of Sak Dr. and Oakland Ave. in Crest Hill, Illinois is on the east side of the cattail marsh depicted in Figure 11. At the time of the site visit, water was observed flowing in the direction of the Des Plaines River through Tributary A (Figure 12). Tributary A flows underneath Oakland Ave. through a box culvert and then alongside a railroad yard. A shallow open water connection may exist during normal conditions, allowing access of swimming fish from the DuPage River Watershed via Rock Run Creek to the Des Plaines River Watershed via Tributary A (Connection Point #5). Approximately 0.75 miles upstream of Oakland Ave., Tributary A flows underneath Highway 7 and then flows through two, 3 ft diameter by 75 ft long culverts (Figure 13). Finally, Tributary A flows approximately 0.30 miles where it connects with the Des Plaines River near Lockport Lock and Dam.
Figure 12.— Box culvert under Oakland Ave. in Crest Hill, Illinois. The water at this point is flowing towards the Des Plaines River. For clarification, water originates behind the photo and flows toward the direction of the culvert.

Figure 13.— The 3ft diameter by 75 ft culverts next to Highway 7 that Tributary A flows through.
## Summary:

<table>
<thead>
<tr>
<th>Location</th>
<th>Potential at 100 yr flood stage</th>
<th>Fish Passage at normal flow</th>
<th>Considerations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Channahon Dam Area (41°25’19.24”N; 88°13’43.19”W)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dam</td>
<td>Moderate</td>
<td>None</td>
<td>• Spillway too high for fish to swim up or jump over</td>
</tr>
<tr>
<td>Dam Sluice</td>
<td>Unknown</td>
<td>Unknown</td>
<td>• Velocities through the sluice are high</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Gate valve may prevent movement.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Intake may have a screen below surface.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Intake was not identified at the base of the sluice valve and could be pulling from further away in deeper water</td>
</tr>
<tr>
<td>I&amp;M Canal Lock 6</td>
<td>Low</td>
<td>None</td>
<td>• Concrete closure</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Gate valve closed</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Motivation for fish to attempt lock passage instead of heading up main stream DuPage is low</td>
</tr>
<tr>
<td>I&amp;M Canal Spill Way (above lock 6)</td>
<td>None</td>
<td>None</td>
<td>• Crest elevation significantly higher than that of Lock 6</td>
</tr>
<tr>
<td>I&amp;M Canal Lock 7</td>
<td>Low</td>
<td>None</td>
<td>• Concrete closure</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Gate valve closed</td>
</tr>
<tr>
<td>I&amp;M Canal Lock 7 Controlling Works</td>
<td>High</td>
<td>Moderate</td>
<td>• Lifting mechanisms inoperable and stoplogs failing.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Moderate flows coming through upper gate (~2 ft. of head)</td>
</tr>
<tr>
<td>I&amp;M Canal</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DuPage River/ I&amp;M Canal Overtopping (41°27’29.93”N; 88°12’59.88”W)</td>
<td>Low</td>
<td>None</td>
<td>• DuPage River flood debris four feet below the crest of the canal levee</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Canal levee approximately 50ft from DuPage River</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Backwater at closest point</td>
</tr>
<tr>
<td>Rock Run Creek/ I&amp;M Canal Junction (41°28’44.45”N; 88°10’47.02”W)</td>
<td>High</td>
<td>High</td>
<td>• Open Connection</td>
</tr>
<tr>
<td>Rock Run</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RightWay Auto Sales (41°33’24.97”N; 88°7’45.99”W)</td>
<td>Low-Moderate</td>
<td>None</td>
<td>• Poor/no channel through portions of cattail Marsh. Water spreads across the marsh to the south.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• 100 yr flood may provide a more defined channel through the marsh.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Controlled burn management evident on southern half of marsh</td>
</tr>
<tr>
<td>Behind Clothes Mentor (41°33’25.54”N; 88°7’35.02”W)</td>
<td>Low</td>
<td>None</td>
<td>• Metal gratings over culverts (~6 inch spacing)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Box culverts were large enough for fish passage but discrepancy with water levels on either end indicate further dynamics within the culverts</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Poor channel through cattail marsh. Water spreads.</td>
</tr>
<tr>
<td>Flow Divide-Woodland of Crest Hill Apartments Tennis Courts (41°33’30.02”N; 88°6’42.74”W)</td>
<td>Low</td>
<td>None</td>
<td>• Poor/no channel through portions of cattail Marsh. Water spreads across the marsh 2-3 ft deep during 100 yr flood.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Pockets of ponded water occur during normal flow conditions; depth unknown but likely shallow.</td>
</tr>
<tr>
<td>Bus Barn (First Student, Inc.) (41°33’46.12”N; 88°6’8.71”W)</td>
<td>High</td>
<td>Moderate</td>
<td>• Well-defined flowing channel</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Box culvert identified under roadway with minimal water flowing during normal conditions but passage possible under slightly elevated water depth.</td>
</tr>
<tr>
<td>Crest Hill Wastewater Treatment Rd. (41°33’50.95”N; 88°5’12.50”W)</td>
<td>High</td>
<td>Low</td>
<td>• 30 inch perched culvert impedes fish movement under normal flow conditions.</td>
</tr>
</tbody>
</table>
Future Considerations:
This assessment of risk of fish passage provides an assessment of risk at the time of the site inspection. Various changes in conditions could increase risk of fish passage including increase in flood elevations and/or frequencies or a change in physical conditions of the waterways.

Specifically, drainage and flood conditions at and near the basin divide in Crest Hill (Connection point #5) have been identified as problems by the city. Future mitigation actions are identified and await funding opportunities. Additionally, the Forest Preserve District of Will County, which owns most of the marshland, periodically conducts controlled burns of these areas. These actions could potentially further open aquatic connections and allow for more accessible channels during normal flow conditions within the marshes identified above.

References:


Attachment 4:

GLMRIS Lock, Reducing Risk of Aquatic Nuisance Species Transfer Through Brandon Road Lock, Analytical and Numerical Model Study
Great Lakes and Mississippi River Interbasin Study

Analytical and Numerical Model Study on Reducing the Risk of Aquatic Nuisance Species Transfer through Brandon Road Lock

E. Allen Hammack, David S. Smith, Richard Styles, and Richard L. Stockstill

June 2018

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Analytical and Numerical Model Study on Reducing the Risk of Aquatic Nuisance Species Transfer through Brandon Road Lock

E. Allen Hammack, David S. Smith, Richard B. Styles, and Richard L. Stockstill

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Final report

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Under Project 451647, “Brandon Rd - Great Lakes/Miss RVR Interbasin Study (GLMRIS)”
Project 451617, "Brandon Rd. Lock Flushing Numerical Model"
Project 114597, "Interbasin Control GL, MR Species"
Abstract

The Great Lakes and Mississippi River Interbasin Study (GLMRIS) is a U.S. Army Corps of Engineers study to evaluate methods of preventing the movement of aquatic nuisance species (ANS) movement between the Great Lakes and Mississippi River basins through aquatic connections. This report is an assemblage of ideas, preliminary hydraulic calculations, and numerical model evaluations that serve as part of the development of an ANS flushing system for Brandon Road Lock on the Illinois Waterway. Four flushing system designs and operations are presented. An analytical description of each lock flushing system design and numerical model results of those designs when applied to Brandon Road Lock are presented. Further, justifications and considerations of a physical model study of any lock flushing design that is chosen for construction are presented. This report is an overall commentary on design ideas and considerations for modeling the flushing rate of the lock chamber. The hydraulic details of lock flushing are outlined with the significant parameters of each lock flushing alternative highlighted. Numerical model results are presented to quantify the effectiveness of each lock flushing concept considered in this study.

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Preface

This study was performed for U.S. Army Corps of Engineers, Rock Island and Chicago Districts, by the Navigation Branch (CEERD-HNN) of the Navigation Division (CEERD-HN), U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory (ERDC-CHL), under the following projects: “Brandon Rd - Great Lakes/Miss RVR Interbasin Study (GLMRIS),” "Brandon Rd Lock Flushing Numerical Model," and "Interbasin Control GL, MR Species." During the study, Ms. Tracy Leeser was Chief, CEERD-HNN.

At the time of publication, Mr. Tim Shelton was Chief, CEERD-HNN; Dr. Jackie S. Pettway was Chief, CEERD-HN; and Mr. W. Jeff Lillycrop was the ERDC Technical Director for Navigation (CEERD-HZT).

The Acting Deputy Director of ERDC-CHL was Mr. John T. Tucker III, and the Acting Director was Mr. Jeffrey R. Eckstein.

COL Brian S. Green was the Commander of ERDC, and Dr. David W. Pittman was the Director.
# Unit Conversion Factors

<table>
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<th>To Obtain</th>
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<tr>
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<td>0.02831685</td>
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<td>feet</td>
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</tr>
<tr>
<td>inches</td>
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<td>meters</td>
</tr>
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<td>miles (U.S. statute)</td>
<td>1,609.347</td>
<td>meters</td>
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<td>square feet</td>
<td>0.09290304</td>
<td>square meters</td>
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<tr>
<td>tons (force)</td>
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<td>newtons</td>
</tr>
<tr>
<td>tons (force) per square foot</td>
<td>95.76052</td>
<td>kilopascals</td>
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1 Introduction

1.1 Background

The Great Lakes and Mississippi River Interbasin Study (GLMRIS) is a multi-agency effort aimed at preventing the spread of aquatic nuisance species (ANS) from the Mississippi River basins to the Great Lakes. According to the Aquatic Nuisance Species Agency website, aquatic nuisance species are “nonindigenous species that threaten the diversity or abundance of native species, the ecological stability of infested waters, or any commercial, agricultural, aquacultural or recreational activities dependent on such waters. ANS include nonindigenous species that may occur within inland, estuarine or marine waters and that presently or potentially threaten ecological processes and natural resources. Further, invasive species are any species or other viable biological material (including its seeds, eggs, spores) that is transported into an ecosystem beyond its historic range, either intentionally or accidentally, and reproduces and spreads rapidly into new locations, causing economic or environmental harm or harm to human health. Synonyms for invasive species include introduced, foreign, exotic, alien, non-native, immigrant and transplants.”

The GLMRIS – Brandon Road effort is an assessment of the viability of establishing a single point to control the one-way, upstream transfer of ANS from the Mississippi River basin into the Great Lakes basin near Brandon Road Lock and Dam located in Joliet, IL. The Brandon Road control point was identified in the GLMRIS analyses as the only single location that can address upstream transfer of Mississippi River species through all Chicago Area Waterway System (CAWS) pathways. Implementation of technologies at the Brandon Road control point was a feature of three of the six structural alternatives presented in the GLMRIS Report (http:// glmris.anl.gov/glmris-report/).

The Brandon Road site (shown in Figures 1 and 2) is located downstream of the confluence of the Des Plaines River and the Chicago Sanitary and Ship Canal (CSSC). In Figure 1, the red box shows the area of the United States where the GLMRIS study is focused, and Figure 2 shows the

---

1 https://www.invasivespeciesinfo.gov/aquatics/main.shtml
details of the area inside that box. Brandon Road Lock is Item 10 (the green circle) in Figure 2. Previous investigations have indicated that a potential hydrologic bypass can occur during periods of high precipitation from the Des Plaines River to the CSSC. A one-way control point at the Brandon Road site would significantly lessen the likelihood of bypass of Mississippi River ANS into the Great Lakes basin during flood events.

Figure 1. GLMRIS study area.
Figure 2. Des Plaines River showing Brandon Road Lock and Chicago area.

A project at the Brandon Road site is likely to reduce a number of previously identified adverse impacts to existing waterway uses and users significantly. These impacts include but are not limited to increased potential for flooding or degradation of water quality. These impacts contributed significantly to the lengthy timeframes and significant costs of the structural alternatives presented by the GLMRIS Report.
The physical configuration of Brandon Road Dam (Figure 3) prevents the upstream transfer of Mississippi River ANS. There is a minimum 25-foot (ft) difference in water elevation from the downstream side of the dam to the upstream side, which effectively limits upstream transfer and promotes the use of gravity for flushing operations. Lock operation at this location currently provides the only known aquatic pathway that allows transfer of Mississippi River ANS to the Great Lakes through the CAWS.

**Figure 3. Aerial view of Brandon Road Lock.**

### 1.2 Objective

Preventing ANS, present in the lower pool, from reaching the upper pool requires that the chamber be flushed prior to each lock filling operation. The empty chamber (water surface at tailwater elevation) must be flushed prior to filling regardless of the presence of a tow in the chamber. Once a lock chamber is flushed and the miter gates and operation valves are closed, the chamber can be filled in a normal manner with clean water.
from the upper pool entering the chamber. Filling the lock in preparation for a down-bound tow approaching the lock must be preceded by a flushing cycle.

Different designs have been proposed for a flushing system of Brandon Road Lock. These flushing system designs require structural and operational changes to the lock. Further, these designs introduce flushing flow into the lock chamber in different locations and at different rates, so the effectiveness and efficiency of each design must be determined before any decision can be made on which flushing system design is best suited for significantly reducing the transfer of ANS across Brandon Road Lock. The objective of this project is to determine the dominant hydraulic mechanisms for each of the proposed flushing system designs and to complete numerical models to determine how each one performs.

1.3 Approach

To evaluate each proposed flushing system design, the hydraulics of each proposed flushing system design have been explored, and numerical hydraulic models of each system as applied to Brandon Road Lock have been completed. The literature has been reviewed to determine the relevant hydraulic considerations for each design. This information provides qualitative estimates of how each design will perform. Three-dimensional (3D) numerical hydraulic models of each flushing system have been performed to determine how effectively and efficiently each system introduces flushing water into the lock chamber. The numerical model results of each design have been compared, and a recommended design is provided.

1.4 Other flushing considerations

This study does not address all the relevant hydraulic considerations of the modification of Brandon Road Lock for ANS transfer. Other considerations include the safety associated with navigation. Safety assurance will most likely require longer flushing times when a tow is in the chamber as compared to an empty chamber. These questions are best answered with a physical model study that includes hawser force measurements.

This report presents design concepts for flushing Brandon Road Lock as part of the overall study to answer how a navigation lock and dam can be
used as a barrier to the upstream passage of ANS. These ideas were generated during discussions among personnel of the U. S. Army Corps of Engineers (USACE), Chicago District; Rock Island District; Inland Navigation Design Center (INDC); and the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL). Initial estimates of flushing efficiency for various design ideas are provided. Before implementation, any chosen design should be further evaluated with a physical model study. The physical model will provide the flushing information and ensure that navigation safety is maintained after modifications are made to the lock’s filling and emptying system. Therefore, a description of the physical model is also provided in this report.

This report is a commentary on design ideas and considerations for modeling the flushing of a lock chamber. First, previous studies that may provide design ideas are reviewed. This literature review is followed by a brief description of Brandon Road Lock. Then the mechanics of hydraulic mixing with application to flushing a lock chamber are discussed. Five design and operation ideas are presented with estimates of component sizes and efficiencies. Finally, descriptions of further evaluation needed for design refinement are presented including but not limited to a physical model study, the associated modeling considerations, and safety concerns associated with the operation of a modified lock.
2 Literature Review of Lock Flushing Ideas

Operations of navigation locks are hindered when floating or submerged substances in the water require consideration. The most common substances are floating objects such as debris and ice, which must be flushed from the chamber to allow room for vessel passage. Salt water is another substance that must be addressed daily at certain projects. Locks are used to arrest salt-water wedges at some projects that separate the forebay’s fresh water from the tailwater’s saline water. Studies have been directed toward developing operation strategies for flushing ice and debris, as well as limiting salt water advance with bubble plumes and various structures placed on the lock floor.

Prohibiting passage of neutrally buoyant particles, such as ANS, presents a new challenge to lock operators. Ice and debris floating on the water surface will be transported from the chamber once enough flow is introduced into the chamber to develop a water-surface gradient. However, ANS flushing is more complicated because the turbulent flow will disperse the entities. Therefore, previous studies are of limited benefit to the problem faced by USACE operators of the Illinois Waterway navigation projects. However, modifications made to the Eisenhower and Snell Locks on the St. Lawrence Seaway are used in this report to demonstrate the feasibility of adding culverts in the lock upper sill and tied into one of the filling culverts.

2.1 Previous lock flushing ideas

Investigations have been conducted to determine how a navigation lock may be used as a conduit to facilitate passage of substances such as ice and debris. Ice and debris studies (e.g., Tuthill et al. 2004; Tuthill 2003; Tuthill and Gooch 1997) have focused on passing materials that tend to float on the water surface. Numerous studies are documented in the literature regarding how a lock may serve as a barrier to salt water intrusion. Salt water intrusion studies (e.g., Parchure et al. 2000; Mausshardt and Singleton 1995; Abraham et al. 1973; Bastian 1971; Wood 1970; Boggess 1970) have focused on preventing salt water from entering the lock chamber. The salt water problem focuses on the density differences of the fresh and salt water bodies that are to remain separated.
2.2 Considerations for flushing aquatic nuisance species (ANS)

The current study differs from previous research in that the objective is to prevent passage of ANS, which, for the purposes of the current study, are assumed to be neutrally buoyant particles. The exchange of upstream and downstream waters for the CAWS is complicated by the fact that the mixing of water from these bodies is to be limited even though natural mixing processes occur during normal operations. The simple act of opening the lock gates generates turbulent mixing of the fluids on either side of the gate. Also, vessels entering and exiting the chamber generate mixing as return currents and propeller wash mix large quantities of water. These mixing processes make maintaining the ANS concentration at near-zero levels difficult.

2.3 Brandon Road Lock details

Brandon Road Lock and Dam is being considered for modification to make the project serve as a barrier to ANS. Brandon Road Lock and Dam is the first project downstream of the Lockport Lock and Dam, and the ANS are assumed to exist on the downstream side of Brandon Road Lock. The objective of the GLMRIS is to prevent ANS from entering the CAWS from the Lower Des Plaines River via Brandon Road Lock.

Brandon Road Lock and Dam is located 286 miles above the confluence of the Illinois River with the Mississippi River (Figure 1. GLMRIS study area. and Figure 2). Brandon Road Dam, located on the Des Plaines River just below the city of Joliet, IL (approximately 27 miles southwest of Chicago), is a fixed concrete structure, 1,569 ft long. The dam is 2,391 ft long (exclusive of fixed embankment and river wall). The water-surface elevation of the pool and discharge past the dam are controlled by twenty-one 50 ft tainter-type crest gates that hold the normal pool 27 inches above the crest of the masonry. Six openings through the dam, previously controlled by sluice gates, have been sealed and are no longer used. A 320 ft section of head gates, which was designed for the future addition of a powerhouse, contains eight operating head gates used for passing water. An ice chute and two sections of earth embankment complete the dam. Most of the short pool is contained between flood walls. These walls vary with a maximum height of 35 ft.

Brandon Road Lock, opened in 1933, is of the sidewall port design filling and emptying system (HQUSACE 2006) as are the majority of locks
operated by the USACE. It operates under a nominal lift of 34 ft with an average 19-minute (min) lock chamber fill time and a 15 min emptying time. The lock chamber is nominally 600 ft long and 110 ft wide (HQUSACE 2006).

The layout of the lock filling and emptying system is shown in Figure 4. The lock features a redundant upstream miter gate and vertical-lift valves for flow control. The intakes and outlets are immediately upstream and downstream of the upper and lower miter gates, respectively. The chamber is filled and emptied with 12 ft-diameter (diam) culverts in each lock wall. Each sidewall manifold has ten ports, 5.0 ft wide by 3.5 ft tall. The port spacing varies from 35 ft to 115 ft along the chamber length. The ports in each wall are positioned directly opposite rather than staggered as specified in current lock design criteria (HQUSACE 2006). The chamber floor is at elevation (el) 489.7 ft with 19 ft wide aprons at el 490.7 (NGVD 29) adjacent to either lock wall.

The ratio of the sum of the cross-sectional area of the ports to the cross-sectional area of the culvert (port-to-culvert area ratio) is 1.55 for Brandon Lock whereas 0.95 is the current design criteria for sidewall port systems based on USACE guidance (HQUSACE 2006). If the sum of the cross-sectional area of the ports is larger than the cross-sectional area of the culvert, the flow into the lock chamber is culvert controlled instead of port controlled, and poor distribution of flow from the port manifold will result. During peak discharge of a filling operation, flow can be drawn from the lock chamber by the upstream ports (HQUSACE 2006). Conversely, if the port-to-culvert area ratio is too small, filling time will be sacrificed without a noticeable improvement in conditions in the lock chamber.
Figure 4. Brandon Road Lock, sidewall port filling and emptying system (elevations are in feet referred to mean sea level 1912).
Simple volume exchange calculations can provide order-of-magnitude estimates of mixing attributed to a vessel entering or leaving a lock chamber. For the case in which a tow exits the lock into ANS-contaminated water, a first approximation is to assume the volume of water displaced by the tow in the lock will be replaced by ANS-contaminated water as the barge leaves the chamber. The calculations below are based on geometric parameters of the lock chamber (listed in Tables 1 and 2) and the design tow. The design tow is assumed to be a $3 \times 3$ flotilla of jumbo barges, each barge being 35 ft wide by 195 ft long and drafted at 9 ft. The total dimensions of the $3 \times 3$ tow are the beam width ($b = 105$ ft), the length ($l = 585$ ft), and draft ($d = 9$ ft).

<table>
<thead>
<tr>
<th>Elevation Description</th>
<th>Elevation (ft, NGVD 29*)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Pool Normal</td>
<td>538.5</td>
</tr>
<tr>
<td>Upper Pool Minimum</td>
<td>537.2</td>
</tr>
<tr>
<td>Upper Pool Maximum</td>
<td>540.5</td>
</tr>
<tr>
<td>Lower Pool Normal (no flow)</td>
<td>504.5</td>
</tr>
<tr>
<td>Lower Pool Minimum</td>
<td>501.1</td>
</tr>
<tr>
<td>Lower Pool Maximum</td>
<td>513.5</td>
</tr>
<tr>
<td>Chamber Floor (Average)</td>
<td>490.0</td>
</tr>
</tbody>
</table>

*National Geodetic Vertical Datum of 1929

The floor of Brandon Road Lock chamber is rock at el 489.7 ft with a concrete apron at el 490.7 ft that is 19 ft wide adjacent and along either chamber wall. The average elevation of the chamber floor is el 490.0 (72 ft at el 489.7 and 38 ft at el 490.7). The chamber is 110 ft wide by 671 ft long, pintle-to-pintle.

Although the volume to be exchanged will be less when the tailwater is at normal or minimum lower pool elevation, the higher head may be most critical regarding hawser forces if a tow is present. River conditions that provide maximum lower pool elevation will have the largest volume and the least head, both of which result in a longer flushing time. This report does not consider the volume of water in the culverts, but the volume of potentially contaminated water residing in the culverts could be included in the exchange-time determinations in a physical model study.

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2 All elevations included in this report are in feet referenced to NGVD 1929 datum.
### Table 2. Brandon Road Lock, lock particulars.

<table>
<thead>
<tr>
<th>Lock Information at Normal Upper Pool (el 538.5) and Lower Pool (el 504.5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lock filling and emptying system</td>
</tr>
<tr>
<td>Chamber width</td>
</tr>
<tr>
<td>Chamber length</td>
</tr>
<tr>
<td>Culvert diam</td>
</tr>
<tr>
<td>Port size</td>
</tr>
<tr>
<td>Number of ports (each culvert)</td>
</tr>
<tr>
<td>Port-to-culvert area ratio</td>
</tr>
<tr>
<td>Filling time</td>
</tr>
<tr>
<td>Emptying time</td>
</tr>
<tr>
<td>Chamber depth when filled</td>
</tr>
<tr>
<td>Volume of “filled” lock</td>
</tr>
<tr>
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</tr>
<tr>
<td>Volume of “empty” lock</td>
</tr>
<tr>
<td>Normal lift</td>
</tr>
<tr>
<td>Normal lift volume</td>
</tr>
</tbody>
</table>
3 Lock Flushing – Analytic Description

Flushing of Brandon Road Lock chamber will be accomplished by introducing clean water from the upper pool into the chamber, diluting ANS-contaminated water by mixing, and transporting ANS-contaminated water into the lower pool from the chamber through opened lower miter gates. This chapter provides the analytic evaluation required to estimate the time and space consequences of flushing the lock chamber.

This study focuses on concepts for flushing the lock using gravity, thus avoiding the large expenses of mechanical pumping. The energy and other operation costs as well as construction and maintenance costs over the lifespan of the pump can be avoided if a gravitational system can be developed. Flushing will bring upper pool water into the chamber, which will be at lower pool level, so the energy available will be the head from the pool differences.

The introduction of clean upper pool water at the upstream end of the chamber can be considered as either a point or line source. Schematics of each of these systems are provided in Figure 5. The red dots indicate the presence of ANS, and the blue lines indicate clean water. In the upper image, clean upper pool water is introduced at a single location, and that flow spreads into the lock chamber as a jet. In the lower image, clean upper pool water is introduced as a line of point sources. The introduction of clean water is essentially a uniform plug.

Modeling the point source conditions requires knowledge of both the lateral and the longitudinal dispersion coefficients. Point source evaluation further requires the inclusion of lateral diffusion and a multidimensional advection-diffusion equation for analysis. Rather than speculating about the effectiveness of a single outlet, this analytical evaluation will consider the clean water inflow as a steady-state line source as illustrated in Figure 5. The alternatives will be further evaluated by the design team to compare cost, operation and maintenance issues, and overall efficiency of the alternatives.
3.1 Advection-diffusion equation

Flushing of the lock chamber using a line source of clean water is a complex hydro-transport problem but can be explained using the simplified one-dimensional (1D) transport equation if the assumption is that flow characteristics do not change perpendicular to the flow direction. The concentration relative to position is quantified with the 1D advection-diffusion equation. The advection-diffusion equation with a conservative constituent is used to estimate the rate of longitudinal dispersion. The 1D advection-diffusion equation is

$$\frac{\partial C}{\partial t} + U \frac{\partial C}{\partial x} - D \frac{\partial^2 C}{\partial x^2} = 0$$  \hspace{1cm} (1)
where:

\[ C = \text{cross-sectional average concentration [ML}^{-3}] \]
\[ T = \text{time [T]} \]
\[ U = \text{cross-sectional average velocity [LT}^{-1}] \]
\[ x = \text{longitudinal direction of flow [L]} \]
\[ D = D_x + D_t + E_x = \text{the longitudinal dispersion coefficient [L}^2\text{T}^{-1}] \]
\[ D_x = \text{x-direction molecular diffusion} \]
\[ D_t = \text{turbulent (eddy) diffusion (time-averaged)} \]
\[ E_x = \text{x-direction (mechanical) dispersion coefficient (space-averaged)}. \]

The molecular diffusion, \( D_x \), is the random motion of particles; the eddy diffusion, \( D_t \), is the turbulent mixing of particles; and the mechanical dispersion, \( E_x \), is the mixing caused by variations in velocities. Diffusion is the process where a constituent moves from a higher concentration to a lower concentration whereas dispersion is mixing caused by physical processes.

The flushing process consists of the initial condition that at \( t = 0, C = C_0 \) for all \( x \) and the boundary condition that the concentration at the inflow boundary is constant, \( C = 0 \) at \( x = 0 \) or

\[ C_{x,0} = C_0 \text{ for } x \geq 0 \]  \hspace{1cm} (2)
\[ C_{0,t} = 0 \text{ for } t \geq 0 \]  \hspace{1cm} (3)
\[ C_{\infty,t} = C_0 \text{ for } t \geq 0 \]  \hspace{1cm} (4)

The analytical solution for the advection-diffusion equation with these initial and boundary conditions is (e.g., Kumar et al. 2011; Socolofsky and Jirka 2005; Runkel 1996)

\[ C_{x,t} = \frac{C_0}{2} \left[ 1 - \text{erf} \left( \frac{x-Ut}{\sqrt{4D_t}} \right) \right] \]  \hspace{1cm} (5)

where \( \text{erf}() \) is the Gauss error function defined as the following:
The difficulty of solving the spatial and temporal concentration variation using this equation is that the longitudinal dispersion coefficient \( D \) is unknown. Numerous researchers have developed methods to quantify the longitudinal dispersion coefficient. Yet, the discrepancies between the values of the observed and predicted longitudinal dispersion coefficients range from one to three orders of magnitude, and existing methods, in general, underestimate the dispersion coefficient (Deng et al. 2002).

Mixing in the lock chamber will be driven by free shear such as a jet from the clean-water source. Farther from the clean-water source, the flushing will approach uniform flow, and boundary friction will then be the primary source of shear. The longitudinal dispersion coefficient for boundary shear is estimated from the friction velocity, \( U_* \), which is

\[
U_* = \sqrt{gRS_f} = \sqrt{\frac{f}{8}} U^2
\]

where:
- \( g \) = acceleration due to gravity
- \( S_f \) = friction slope
- \( R \) = hydraulic radius
- \( F \) = Darcy friction factor
- \( U \) = average flow velocity.

The most commonly used method of determining the longitudinal dispersion coefficient is the Fischer equation (Fischer et al. 1979), which is

\[
\frac{D}{hU_*} = 0.011 \left( \frac{B}{h} \right)^2 \left( \frac{U}{U_*} \right)^2
\]

This equation is popular because it gives the longitudinal dispersion coefficient, \( D \), in terms of readily available hydraulic variables; the width-to-depth ratio \( (B/h) \); and friction term \( (U/U_*) \). The left-hand side of the Fischer equation is commonly referred to as the dimensionless dispersion coefficient. Seo and Cheong (1998) used regression analysis to develop an
empirical form of the hydraulic and geometric variables of the Fischer equation to better represent observed values.

\[ \frac{D}{hU_*} = 5.915 \left( \frac{B}{h} \right)^{0.62} \left( \frac{U}{U_*} \right)^{1.428} \]  

(9)

### 3.2 Turbulence and mixing

Without knowledge of the longitudinal dispersion coefficient, the problem can be bounded as one of advection-dominated flow and one in which the flow is better characterized as dispersion dominated. Evaluation requires determination of the importance of dispersion relative to the transport of a concentration (ANS). This is done using the Peclet number, \( Pe \), which is the relative advection-to-dispersion ratio and is given as

\[ Pe = \frac{UL}{D} = \frac{U^2t}{D} \]  

(10)

Note that the Peclet number is sometimes given as the ratio of dispersion to advection (reciprocal of what is defined here). As presented here the Peclet number is large when the flow is advection dominated and small when dispersion dominates. If the flow is dispersion dominated, the Peclet number goes to zero. In the case of advection domination, the Peclet goes toward infinity, and the transport is similar to plug flow.

Some simple water quality models can be developed for special cases where either advection or dispersion is dominant. As the Peclet number becomes large, the longitudinal dispersion can be neglected, and the system behaves as a plug-flow chamber.

#### 3.2.1 Plug flow

The plug-flow concentration is shown at a particular position for various times in Figure 6. The concentration, \( C \), is normalized by the initial concentration, \( C_0 \), and the distance from the clean water source, \( x \), is related to the lock chamber length, \( L \).
The time required flushing water into and from the chamber, \( T_f \), is the volume to be exchanged, \( V_c \), divided by the volumetric flow rate, \( Q \).

\[
T_f = \frac{V_c}{Q} \tag{11}
\]

where:

\( T_f \) = time required to flush the lock chamber assuming plug flow  
\( V_c \) = the volume of the lock chamber when the water surface is at tailwater.

The fastest time is limited by the maximum allowable discharge. Of course, the actual flow conditions in the lock chamber will not be plug flow, but the plug-flow equation provides the absolute shortest time and least volume of water required to flush the lock chamber. The actual flow volume required to flush Brandon Road Lock chamber in a reasonable time will produce high-shear turbulent conditions in the lock chamber. The turbulent dispersion in the lock chamber will require a longer time and larger volume of water to flush as compared to the plug-flow condition.
3.2.2 Well-mixed flow

Dispersion-dominated problems can be treated as a well-mixed system. The dispersion-dominated case is analogous to a continuously stirred tank. Flow that enters the chamber is assumed to instantaneously mix throughout the full chamber volume. This situation is referred to as the well-mixed case wherein conservation of mass means that

\[
\frac{\partial CV}{\partial t} = -Q \left( C_{in} - C_{out} \right) \tag{12}
\]

In the case at hand, where the chamber has an ANS concentration of \( C(t) \), the chamber is flushed with clean inflow having a concentration of \( C_{in} = 0 \). The volume of water in the chamber is constant because the volumetric flow rate into the chamber equals that flowing from the chamber, so

\[
\frac{\partial C}{\partial t} = -\frac{Q}{V} C_{out} \tag{13}
\]

The well-mixed case means that the concentration of water flowing from the chamber is equal to the concentration in the chamber, \( C(t) \). The solution of this differential equation is

\[
C(t) = \exp\left( -\frac{Q}{V} t \right) \tag{14}
\]

For an inflow concentration of zero, the concentration in the chamber decreases exponentially for the well-mixed case, wherein dispersion dominates advection.

The time required to flush 95% of the chamber concentration is the time required to reduce the concentration from 1.0 to 0.05. The time required is

\[
t = -\frac{V}{Q} \ln C = -\frac{V}{Q} \ln 0.05 \tag{15}
\]

Temporal variation of relative concentration at a particular distance from the clean-water source is illustrated for the well-mixed case in Figure 7. This case represents the longest time required to flush the chamber, and complete flushing is theoretically never obtained because the concentration varies logarithmically (hence asymptotically) with time.
3.2.3 Advection-dispersion flow

A third case of flushing the lock chamber will cause the ANS to be transported downstream and their concentration dispersed (i.e., advection-dispersion flow). The problem is theoretically bound between the plug-flow situation, which is the quickest flushing time and the well-mixed case which requires the most time to flush. The actual response to the introduction of clean water via momentum jets is illustrated by the concentrations in Figure 8.
3.3 **Momentum jets**

Regardless of how clean water is introduced into Brandon Road Lock chamber, the flow will enter the chamber as a momentum jet or a set thereof. Albertson et al. (1950) describe the mechanics of a submerged jet using the assumptions of steady (but turbulent) flow, quiescent ambient fluid, and that the receiving fluid has the same density as the discharge fluid. The jet development is classified as being in two zones, the zone of flow establishment and the zone of established flow (ZEF) as illustrated in Figure 9. Further assumptions are that the jet grows linearly, that the pressure distribution is hydrostatic, and that the velocity profile is Gaussian.
The Albertson et al. (1950) experiments were conducted at $Re \approx 5 \times 10^4$, so the results are valid for turbulent flow.

3.3.1.1 Round (circular) jet

Expressions for velocity and discharge in the ZEF for a round (circular) are

$$\frac{u_m}{U_0} = 6.2 \left( \frac{D_0}{x} \right)$$  \hspace{1cm} (16)

$$\frac{Q}{Q_0} = 0.32 \left( \frac{x}{D_0} \right)$$  \hspace{1cm} (17)

where:

- $u_m$ = maximum velocity within the jet
- $U_0$ = jet velocity at the port face
- $D_0$ = inflow culvert diam
- $x$ = distance from the port face.

As the jet spreads, it entrains flow from the surrounding fluid, growing linearly. The centerline velocity also grows linearly. The plot shown in Figure 10 illustrates the jet velocity and discharge growth. The blue lines indicate the motion of the surrounding fluid as it is entrained into the jet. Note that nominal boundaries of the submerged circular jet expand by a ratio of 1 lateral to 5 longitudinal.
3.3.1.2 Two-dimensional (2D) momentum jet

The velocity distribution for a 2D jet produced from a channel of width $B_0$ is illustrated in Figure 11. Albertson et al. (1950) determined the upstream limit of the ZEF to be $\frac{x}{B_0} = 5.2$ and that the velocity and discharge are given as

$$\frac{u_m}{U_0} = 2.28 \sqrt{\frac{B_0}{x}}$$

$$\frac{Q}{Q_0} = 0.62 \sqrt{\frac{x}{B_0}}$$

The free shear attributed to submerged jets will be the primary source of dispersion during lock flushing operations.
3.4 Stagnant regions (dead zones)

The jet diffusion sketches (Figure 9–Figure 11) illustrate that, even with the entrainment currents induced by the jet shear, there can be regions within the ambient fluid that remain unmoved, such as areas outside the spreading jet and to the immediate sides of the jet outlet. ANS will be trapped in stagnant regions referred to as “dead zones” or “storage zones” (Fernando 2013). Therefore, reduction of dead zones will in turn provide a more efficient flushing system.
4 Lock Flushing Concepts

4.1 Hydraulic design of flushing systems

Hydraulic flushing systems have been studied and designed for many years, and basic hydraulic loss coefficients are often known.

The energy loss $H_L$ through each component can be expressed as

$$H_L = K_i \frac{V_i^2}{2g}$$

where:

$K_i$ = loss coefficient for component $i$

$V_i$ = velocity through component $i$.

Loss coefficients for many hydraulic components are well established and are readily available in the literature (e.g., Miller 1990). However, lock culvert system components are often unique to a particular project, and the loss coefficients have not been determined for lock components of any size or configuration.

4.2 Lock flushing systems

The flushing process can be described as introducing clean water from the upper pool into an empty chamber. The lock flushing process that provides clean water from the upper pool into an empty chamber (chamber water surface at lower pool elevation), either using the existing filling and emptying system or a new culvert system designed specifically for flushing ANS, will be analyzed in detail for several alternatives.

Four basic concepts have been identified. The first lock flushing concept (Type 1) relies on the existing filling and emptying system to flush the lock. The second lock flushing concept (Type 2) adds a lateral manifold from one of the filling culverts across the lock chamber immediately downstream of the upper sill. The third lock flushing concept (Type 3) adds culverts from the upper pool to the chamber through the upper sill. A final concept (Type 4) considered is not designed to flush the lock chamber but rather provides a continuous flow of clean water flushing the lower
lock approach to prevent ANS from entering the lock chamber. A fifth lock flushing concept (Type 5) has also been proposed. The Type 5 lock flushing concept is similar to the Type 1 concept in that it relies on the filling and emptying system to flush the lock. However, the Type 5 concept uses a filling and emptying system that has been modified to closely conform to current hydraulic design guidance.

### 4.2.1 Type 1 lock flushing concept (existing filling and emptying system)

The Type 1 lock flushing concept (shown in Figure 12) uses the existing lock filling and emptying system to flush the lock chamber. The system setup would have the upper miter gates closed, the lower miter gates open, the fill valves opened (perhaps partially), and the emptying valves closed. This scheme would input clean water with zero concentration along the length of the chamber, which would respond more as a well-mixed system wherein dispersion dominates the flow. The movement of flushing flow through the system is indicated by the blue arrows in the figure. The lower miter gates may need to be retrofitted with a means to secure them in the open position as the chamber is flushed.

![Figure 12. Type 1 (existing) lock flushing concept schematic.](image)

The lock coefficient for a standard design sidewall port filling system is approximately 0.80 (McCartney et al. 1998). Loss coefficients for lock
filling and emptying systems are customarily given in terms of the velocity head in the culvert at the valve (i.e., valve fully open). Estimates of loss coefficient values for Brandon Road Lock filling system components are provided in Table 3. These coefficients are for a standard sidewall port filling system (e.g. Murphy 1975; McCartney et al. 1998; HQUSACE 2006). Since the design standards were not developed until decades after Brandon Road Lock was constructed, the loss coefficient values will need to be validated with field or laboratory data. The head loss as flow passes a partially opened vertical-lift valve is a function of the shape of the valve lip and the valve opening. The head loss varies during a valve operation. The Hydraulic Design Chart 320-1 (HQUSACE 1988) is a plot of discharge coefficient as a function of valve position. The relation between the discharge coefficient, $C_v$, and a head loss coefficient for the valve, $K_v$, can be determined by equating the change in head across the valve:

$$K_v = \left(\frac{b}{B}\right)^2 C_v^{-2} \quad (21)$$

where:

- $b$ = valve opening
- $B$ = culvert height at the valve, 9 ft for Brandon Road Lock.

The discharge coefficient for a valve opening of 25% is given in Hydraulic Design Criteria 320-1 as 0.73 (HQUSACE 1988). This yields a loss coefficient of 30, which is the same value given in Miller (1990) for vertical lift valves opened 25%.

<table>
<thead>
<tr>
<th>Lock Component</th>
<th>Representative area (valve area), $A_v$</th>
<th>Total Loss Coefficient, $K$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper pool to valve</td>
<td>9 ft by 9 ft = 81 ft$^2$</td>
<td>0.45</td>
</tr>
<tr>
<td>Through open valve</td>
<td>81 ft$^2$</td>
<td>0.11</td>
</tr>
<tr>
<td>Through valve 25% open</td>
<td>81 ft$^2$</td>
<td>30</td>
</tr>
<tr>
<td>Valve to lock chamber</td>
<td>81 ft$^2$</td>
<td>1.05</td>
</tr>
</tbody>
</table>

The sum of head loss at normal pool conditions is 1.61 (see Table 2), so the discharge is estimated to be
This discharge calculated from Equation 21 is the estimated discharge per culvert with the vertical-lift gates fully open. Because there are two culverts, the total discharge into the chamber is estimated to be 5,970 cubic feet per second (ft³/sec). This method of flushing will require that the filling valves are able to close in flowing water at the project’s full 34 ft head. Project operation personnel have indicated that the filling valves can only be opened 25% during flushing. One reason for concern is that during flushing, the open lower miter gates may slam shut. Therefore, a device to hold the lower miter gates open will need to be installed. Another consideration is that the existing vertical-lift valves may either require modification or replacement with valves that are designed specifically for flow control and are heavy enough to close under full flow. The flushing discharge with the fill valves opened 25% rather than 100% is calculated using the head loss coefficient for a vertical-lift valve opened 25% as $K_v = 30$ (Miller 1990). Then, the total loss coefficient with the valve opened 25% is 31.5, so the discharge through each culvert is estimated to be

$$Q_{25\%} = A_v \sqrt{\frac{2gH}{K_v}} = 81 ft^2 \sqrt{\frac{2 \times 32.2 \frac{ft}{sec^2} \times 34 \frac{ft}{1.61}}{31.5}} = 675 \frac{ft^3}{sec} \quad (23)$$

per culvert for a total flushing discharge of about 1,350 ft³/sec.

Concentration histories for various discharges are shown in Figure 13. The concentrations presented correspond to a 504.5 ft normal lower pool where the concentration can be calculated with Equation 14.
The Type 1 concept will perform more as a well-mixed flow field wherein the flushing times for various discharges are illustrated in Figure 13. Flushing of the well-mixed chamber can be evaluated as the time required for dilution to a particular concentration. The flushing times required to reduce the ANS concentration by various amounts ranging from 50% to 99% are shown in Figure 14. The flushing times shown correspond to various dilutions at the normal tailwater elevation of 504.5 ft where the time is given by Equation 12.
4.2.2 Type 2 lock flushing concept (lateral flushing manifold)

The Type 2 lock flushing concept (Figure 15) is a new culvert and manifold perpendicular to the lock walls. The movement of flushing flow through the system is indicated by the blue arrows in the figure. The flushing operation would have the upper miter gates closed, the lower miter gates open, and the filling and emptying valves closed. This scheme would require a valve on the lateral flushing manifold that would be opened during the flushing operation. This flushing manifold would be placed immediately downstream of the upper sill so that most of the chamber is downstream of the lateral’s discharge and would extend into the lock chamber normal to the culvert-side lock walls. This culvert would join one of the filling culverts (either right or left wall) upstream of that culvert’s fill valve. The existing culvert system of Brandon Road Lock would have to be significantly altered to accommodate this design. The culverts make rectangular-to-circular transitions, so connecting a lateral flushing manifold to an existing filling and emptying (F/E) culvert will be difficult during construction. Also, the fill valve will have to be moved downstream (as indicated in the plan image of Figure 15) to allow flow from the upper pool to reach the flushing manifold but not the F/E ports. The first port on
the lock F/E manifold may have to be closed to make room for the new fill valve and lateral culvert. Closing the first port may actually have beneficial consequences for Brandon Road Lock since the sum of the port areas is approximately 1.5 times that of the culvert area. Sidewall port design criteria calls for a ratio of 0.95. Chamber performance during filling is enhanced when the flow control is at the ports. This requires that the port-to-culvert area ratio be less than or equal to 1.0, which is currently not the case for Brandon Road Lock.

Figure 15. Type 2 design flushing system schematic.

For the purposes of this report, culvert sizes and configurations were taken from the St. Lawrence Seaway Development Corporation design for the Eisenhower Lock (Appendix A). These values were chosen because a properly functioning flushing system is already in place at Eisenhower Lock, and the proposed flushing system at Brandon Road Lock is similar. The flushing culvert is 10 ft in diam, and the lateral manifold has five pipes that tee into the lateral flushing manifold. Each pipe is 4 ft in diam and serves as a port resulting in a sum of the port-to-culvert area ratio of 0.8.

The sum of head loss at normal pool conditions is 34 ft. Equating this head loss to the losses listed in Table 4 and using Equation 21, the discharge is calculated to be 3,540 ft³/sec. This is the estimated discharge from the Type 2 lock flushing concept at normal pool conditions with the F/E valve
(vertical-lift valve) fully open. This concept will act somewhat as an advection-dominated system wherein the water moves downstream as plug flow. The flushing time for this system is provided in the following section.

<table>
<thead>
<tr>
<th>System Component</th>
<th>Loss Coefficient, $K$</th>
<th>Representative Area, ft$^2$</th>
<th>Coefficient Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lock Intake</td>
<td>$K_i$</td>
<td>216</td>
<td>0.24</td>
</tr>
<tr>
<td>90 deg bend</td>
<td>$K_b$</td>
<td>216</td>
<td>0.24</td>
</tr>
<tr>
<td>Open vertical-lift valve (old)</td>
<td>$K_{gv}$</td>
<td>81</td>
<td>0.11</td>
</tr>
<tr>
<td>90 deg T junction</td>
<td>$K_{tee}$</td>
<td>78.5</td>
<td>0.23</td>
</tr>
<tr>
<td>Open flow-control (butterfly) valve</td>
<td>$K_{bw}$</td>
<td>78.5</td>
<td>0.2</td>
</tr>
<tr>
<td>Dividing flow manifold</td>
<td>$K_m$</td>
<td>78.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Outlets</td>
<td>$K_o$</td>
<td>12.6</td>
<td>1.0</td>
</tr>
</tbody>
</table>

The Type 2 design will require at least 10.5 ft of excavation below the existing lock floor elevation for Brandon Road Lock to provide enough clearance required by USACE guidelines (HQUSACE 2006). This limestone excavation is required for placement of the lateral culvert beneath the lock floor.¹

4.2.3 Type 3 lock flushing concept (culverts through sill)

The Type 3 lock flushing concept (Figure 16) is the addition of conduits through the upper sill. These pipes require valves to control the flushing flow. The movement of flushing flow through the system is indicated by the blue arrows in the figure. Snell Lock on the St. Lawrence Seaway has undergone similar modifications to facilitate ice flushing from the chamber. The plug-flow analogy would be a more reasonable representative of the flushing than the well-mixed case.

¹ The Type 2 flushing concept was removed from consideration before the free-surface numerical modeling effort described in detail in Chapters 5, 6, and 7. A numerical model was performed on Type 2 in an earlier stage of the modeling effort, and the flow solutions of that model are shown in Appendix C.
The new pipes will be long enough to pass from the upstream to downstream face of the upper sill (approximately 85 ft). Culverts passing through the sill will experience intake losses, friction losses, loss at the opened butterfly valve, and exit losses. The pertinent loss coefficient characteristics and values for the Type 3 lock flushing concept are listed in Table 5. The total discharges possible for Type 3 with a 34 ft lift are shown in Table 6. Multiple pipes are required to prevent reverse eddies and motionless areas in the upper corners of the chamber (Oswalt 1976), and the total discharge for various pipe configurations and sizes are computed.

### Table 5. Loss coefficients for Type 3 lock flushing concept.

<table>
<thead>
<tr>
<th>Conduit Segment</th>
<th>Loss Coefficient, $K$</th>
<th>Characteristic Dimension(s)</th>
<th>Coefficient Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inlet</td>
<td>$K_i$</td>
<td>$A = \frac{\pi D^2}{4}$</td>
<td>0.1</td>
</tr>
<tr>
<td>Wall friction</td>
<td>$f \frac{L}{D}$</td>
<td>$L = 85$ ft</td>
<td>$0.014 \times 85/D$</td>
</tr>
<tr>
<td>Open butterfly valve</td>
<td>$K_v$</td>
<td>$A = \frac{\pi D^2}{4}$</td>
<td>0.2</td>
</tr>
<tr>
<td>Exit</td>
<td>$K_e$</td>
<td>$A = \frac{\pi D^2}{4}$</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Table 6. Calculated discharge for Type 3 lock flushing concept, 34 ft normal lift.

<table>
<thead>
<tr>
<th>Pipe Diam, D, ft</th>
<th>Number of Pipes</th>
<th>Total Discharge, ft³/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>1</td>
<td>460</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>930</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1,390</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1,860</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>2,320</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>740</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1,480</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>2,230</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>2,970</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>3,710</td>
</tr>
<tr>
<td>6</td>
<td>1</td>
<td>1,080</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2,170</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>3,250</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>4,340</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>5,420</td>
</tr>
</tbody>
</table>

Intakes in the upper sills of locks have led to vortex formations during lock filling. Numerous physical model studies have been conducted to reduce vortex tendencies (e.g., Ables 1979; Hite 1999; Hite 2000; Hite and Tuthill 2005; Hite and Bislip-Morales 2012). During previous model studies, conducted with through-the-sill intakes, modifications in the approach were developed to reduce the tendency for strong vortices to form. Streamlining the flow into the intakes by modifying the miter gate recesses have helped reduce vortex formation. Reducing the approach velocities by deepening the approach also helps improve flow conditions. Therefore, the Type 3 concept will most likely induce vortices in the upper approach during flushing operations. The tendency for vortex formation is due to the relatively small intake area in conjunction with the culvert intakes being relatively shallow.

Vortices will not only inhibit the efficiency of the culverts thereby reducing the discharge, they can also be a safety hazard and draw debris down to the culvert intakes. Intake trash racks can then become clogged with submerged debris, further restricting the intake area and discharge.
The Type 3 concept, as with Type 2, will serve more as advection-dominated flow fields, and the plug-flow analogy provides an order of magnitude estimate of the time required to flush ANS from the lock chamber. The flushing times for multiple flushing discharges are shown in Figure 17. The discharges shown are given by Equation 10.

![Flushing Time for Plug Flow](image)

**Figure 17.** Plug-flow system flushing time for various tailwater elevations.

### 4.2.4 Type 4 lock flushing concept (continuous flow below chamber)

The Type 4 concept is not a configuration of Brandon Road Lock but rather relies on keeping ANS from ever entering the lock chamber. The barrier is a lateral manifold across the lower approach channel downstream of the lock providing continuous flow. The clean water is taken from the upper pool, and the flow is distributed across the channel with a multi-ported manifold as illustrated by the sketch in Figure 18.

One shortcoming of this concept is that it does not address the ANS that can be carried into the lock chamber as upbound and downbound tows traverse the lock. Propeller wash from tow boats and return currents generated as downbound tows leave the chamber may transport ANS over the continuous flow manifold. Also, the water pushed ahead of upbound
tows may also overcome the hydraulic gradient that the manifold maintains. Once ANS are upstream of the manifold, there is no means to flush them using the Type 4 concept.

Figure 18. Type 4 concept schematic.

A physical model can be designed to accommodate experiments to evaluate the Type 4 concept. Particular questions can be answered such as the effectiveness of the Type 4 concept as tows pass over it.

4.2.5 Type 5 lock flushing concept (redesigned filling and emptying system)

The Type 5 lock flushing concept, shown in Figure 19, uses a redesigned lock filling and emptying system to flush the lock chamber. This system will be set up and operate similarly to the Type 1 system. For details of this setup, see Section 4.2.1. This concept basically adheres to the current USACE hydraulic design guidance for lock filling and emptying systems (HQUSACE 2006). Each culvert consists of twelve 3.54 ft tall ports that are 2.54 ft wide at the culvert with a 1-on-20 taper to the culvert. These ports have a 28 ft spacing (center to center). Unlike the ports with the existing design that face one another directly, the ports in the redesigned system are offset by 14 ft. This offset is included so the jets extending from the ports on one culvert do not interfere with those from the ports in the opposite culvert. Removing any such interference should reduce any bulking at the chamber water surface. Triangular flow deflectors are
included in the lock chamber for the first four ports on each culvert. These deflectors are included on the upstream third of the ports on each culvert to direct the jets (from each port) toward the opposite lock chamber wall instead of more toward the downstream gates. Having the jets oriented that way reduces the likelihood of bulking at the lock chamber surface, which would increase the hawser forces on a vessel in the chamber.

The only deviation from the current design guidance is the inclusion of a 6 × 6 ft port (shown in Figure 24 in Section 6.5) at the upstream end of each filling and emptying culvert. These two ports are included in the design solely to improve the flushing performance of the upstream end of the lock chamber. These ports are positioned at the location of the first port in the existing filling and emptying system (Figure 20 in Section 6.5).

Figure 19. Type 5 concept schematic.
5 **Numerical Modeling Process**

Dead zones, defined as regions that are not directly affected by the momentum exchange from the flushing jets, are an issue that the total loss and discharge calculations previously performed cannot determine. However, these regions can be recreated with numerical models. Therefore, a numerical model has been developed for each of four lock flushing concepts to determine the location and size of dead zones produced by each lock flushing concept. Each numerical model includes details of the flushing evaluation such as the complete geometry, something not used in the purely analytic approach. Also, only the numerical model produces flow distributions and patterns that are needed to better evaluate flushing efficiency.

A 3D Navier-Stokes (non-hydrostatic) numerical flow model of the lock is a useful predictive tool to explore lock flushing concepts. The ERDC 3D Navier-Stokes module of the Adaptive Hydraulics (AdH) code has been used to model the complicated turbulent exchange processes as flow passes into and from the lock chamber and is an appropriate tool for this modeling effort. AdH produces time-varying flow solutions, and steady-state solutions are obtained by simulating time until the dynamic variation in the flow field ceases.

5.1 **Governing equations**

The Reynolds-Averaged Navier-Stokes (RANS) equations are employed to model the flow field approaching, interacting with, and passing by hydraulic structures. The RANS equations are 3D with 4 degrees of freedom: the pressure and the three components of fluid velocity. These equations make no assumptions about pressure distributions. Since many hydraulic flow models assume the flow is hydrostatic, RANS models are referred to as non-hydrostatic models.

The RANS equations are derived from the conservation of mass and conservation of momentum applied to fluid flow by decomposing the instantaneous flow velocity into a mean component, \( \mathbf{U} \), and a fluctuating component, \( \mathbf{u} \), and averaging these equations over time periods that are long compared to the periods of the fluctuations. Mathematically, the conservation of mass for an incompressible fluid is described as
\( \nabla \cdot \mathbf{U} = 0 \) \hspace{1cm} (24)

and the conservation of momentum is given as

\[
\rho \left( \frac{\partial \mathbf{U}}{\partial t} + \mathbf{U} \cdot \nabla \mathbf{U} \right) - \nabla \cdot \sigma + \nabla \cdot (\rho \mathbf{uu}) = 0
\]

where:

\[
\begin{align*}
  t &= \text{time} \\
  \rho &= \text{fluid density} \\
  \sigma &= -p \mathbf{I} + \tau \\
  \mathbf{I} &= \text{identity matrix} \\
  \tau &= 2\mu \Gamma \\
  \Gamma &= \frac{1}{2} (\nabla \mathbf{u} + (\nabla \mathbf{u})^T) \\
  \mu &= \text{fluid viscosity}.
\end{align*}
\]

The RANS equations are written in terms of the mean velocity, \( \mathbf{U}(x, t) \), and pressure, \( p(x, t) \), to reduce the modeling of turbulence to a set of quasi-steady-state equations that incorporate terms to model the effects of turbulence on the main flow. In a RANS approach, the term \( \nabla \cdot (\rho \mathbf{uu}) \) is used to represent the effect of turbulence on the mean flow.

Following the suggestion of Boussinesq, an eddy viscosity is added to the molecular viscosity in the momentum equations to account for the effects of turbulence. A constant eddy viscosity model was used to replicate the turbulent effects. The eddy viscosity value was reduced until the velocity magnitudes no longer changed with decreasing values of the eddy viscosity. This threshold value of eddy viscosity was used for all simulations.

The effectiveness and efficiency of each lock flushing concept is modeled by direct calculation of the concentration of the flushing flow throughout the flow domain. These concentrations are treated as being composed of neutrally buoyant concentrations. The behavior of the concentration is described by the advection-diffusion equation (shown in the 1D form as Equation 1 in Section 3.1). For the numerical modeling, the 3D form of the advection-diffusion equations assuming a constant diffusion coefficient is used:
\[
\frac{\partial C}{\partial t} + \mathbf{U} \cdot \nabla C - D \nabla^2 C = 0
\]  

(26)

where \( \mathbf{U} \) = mean velocity vector at a point.

The diffusion coefficient, \( D \), is assumed to be equal in all directions.

### 5.2 Modeling procedure

Before the equations of motion can be applied, the domain must be discretized into numerical elements. This process includes the construction of a 3D computer-aided design (CAD) representation of the flow boundaries including the geometric features of the hydraulic structure, the bathymetry of the approaching river, and the water surface. The CAD model is then used as input for a mesh generator.

A computational mesh is constructed to fill the volume enclosed by the CAD model surfaces. For AdH simulations, the computational mesh must only sufficiently describe the boundaries of the flow domain because automatic mesh refinement is used to ensure that the flow features interior to the domain are reproduced correctly. The mesh of the CAD surface will be composed of individual faces of the elements that form the lock boundaries and the water surface. The boundary conditions such as velocity, discharge, and pressure are needed on these faces and their nodes to determine a particular solution to the governing partial differential (RANS) equations.
6 Numerical Model Setup

A numerical model was created for each lock flushing concept, except Type 4 since it only includes changes to the flow downstream of the lock itself (the lock is bypassed). For each lock flushing concept, the upper pool elevation is 538.5 ft NGVD, and the lower pool (chamber) elevation is 504.5 ft NGVD. These elevations are the average normal pool elevations present at Brandon Road Lock from 2005–2014. Additional information about the computational meshes is included in Appendix B.

Each model has two flux boundaries — one inflow and one outflow. The model discharge, listed in Table 7, is applied as an average inflow velocity at the inflow boundary. Initially for Type 3, a flushing discharge of 3,000 ft³/sec is used. This discharge was based on the maximum available flushing discharge outlined in Section 4.2.3 (Table 6). However, this discharge produced such high velocities in the upstream end of the chamber near the flushing pipe outlets that the water surface drawdown in that area was approximately 5 ft. Such a large drawdown will likely not satisfy safety concerns when barges are moored in the lock chamber, so the flushing discharge is reduced to 1,000 ft³/sec. The new flushing discharge produces average velocities in the flushing pipes that are similar to the largest average velocities through the filling and emptying ports in Type 1. An additional concept, Type 3r, is based on the Type 3 design and includes rectangular flushing pipes. This concept allows for the higher 3,000 ft³/sec discharge to be tested.

The 2,600 ft³/sec flushing discharge for Type 5 is based on a redesigned (stronger) intake valve and gate that can be opened to half open when under full head; currently the existing valves can only be lifted a quarter open under full head without excessive vibration and chance of being pinned shut.

<table>
<thead>
<tr>
<th>Table 7. Lock flushing model discharges</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flushing Concept</td>
</tr>
<tr>
<td>-------------------</td>
</tr>
<tr>
<td>Type 1</td>
</tr>
<tr>
<td>Type 2</td>
</tr>
<tr>
<td>Type 3</td>
</tr>
<tr>
<td>Type 3r</td>
</tr>
<tr>
<td>Type 5</td>
</tr>
</tbody>
</table>
For each model, a hydrostatic pressure distribution was applied to the nodes on the downstream boundary such that zero pressure was applied to the nodes at the water surface on the downstream boundary. Details of each lock flushing concept configuration are discussed in Chapter 4. The diffusion coefficient of 0.0003 ft$^2$/sec was used for all simulations.

### 6.1 Type 1 lock flushing concept model geometry

The Type 1 lock flushing concept is the existing filling and emptying system. Figure 20 shows a CAD model of Brandon Road Lock filling and emptying system. The CAD model was constructed from the line drawings provided by the INDC. The flow domain includes a portion of the lock upstream of the upper miter gates, both filling and emptying culverts with both the intakes and ports, and the lock chamber. The downstream end of the flow domain is located at the pintle of the downstream miter gates. The culvert is terminated downstream of the last (tenth) port.
6.2 Type 2 lock flushing concept model geometry

The Type 2 lock flushing concept is an additional lateral flushing manifold positioned near the upstream end of the lock chamber. The CAD model for this concept is shown in Figure 21. The flushing manifold is connected to the right of the existing filling and emptying culvert at the first port. This manifold culvert has a uniform 10 ft circular diam cross section with five 4 ft diam ports. These ports are connected to the top of the flushing culvert and are directed vertically. The center of the first port is 15 ft from the lock wall, and the ports are at 20 ft spacings (center to center). To satisfy USACE design criteria on clearance, the lock flushing culvert is 10.5 ft below the lock chamber floor. The flushing ports connect the top of the flushing culvert to the lock floor. The flushing manifold requires all the flow entering one of the existing filling and emptying culverts to be directed completely through the manifold, so the filling and emptying culvert downstream of the flushing manifold is not included in the flow domain. The opposite filling and emptying culvert (the one not involved in the lock chamber flushing) is also excluded from the flow domain.

Figure 21. CAD model of Type 2 lock flushing concept.
6.3 Type 3 lock flushing concept model geometry

The Type 3 lock flushing concept is a series of pipes in the upstream gate sill that connect the upper pool with the lock chamber. The configuration chosen for the computation model, shown in Figure 22, has four 5 ft diam pipes positioned laterally at a 19.2 ft spacing (center to center) along the gate sill over the deepest portion of the lock chamber. The center of these pipes is at el 494, which corresponds to a submergence of 10.4 ft when the lower pool water surface is at el 504.5. The filling and emptying culverts play no role in the lock flushing for this concept, so they are not included in the flow domain. The Type 3 numerical model flow domain includes a portion of the lock upstream of the upper miter gates, the gate sill pipes, and the lock chamber terminated at the downstream miter gates. The lock chamber for the Type 3 concept differs from that of Types 1 and 5 in that a portion of the upstream end (upstream of the first port in the existing filling and emptying system) is removed. This removal is a structural requirement for the flushing pipes.

Figure 22. CAD model of Type 3 lock flushing concept.
6.4 Type 3r lock flushing concept model geometry

The Type 3r lock flushing concept, shown in Figure 23, was developed as a result of the reduced discharge in the Type 3 model. For the Type 3 numerical model, the discharge was reduced because of the high velocities in the upstream side of the lock chamber. For the Type 3r model, the size of the conduit that introduces flushing flow into the lock chamber was increased such that the average flushing velocity introduced into the lock chamber is close to 15 ft/sec. This average velocity corresponds to the largest average velocity of flow through the filling and emptying ports in the Type 1 lock flushing concept. This new conduit, referred to as the “rectangular slot,” is a constant 72 ft wide by 3 ft tall cross section that connects to the lock chamber at the same centerline elevation as the gate sill pipes in Type 3. The rectangular slot is centered laterally in the lock chamber. Constructing such a large conduit though the upstream gate sill is highly improbable, so this lock flushing concept is largely just to show how well a gate sill lock flushing concept could perform if only the hydraulics of the system are considered.

Figure 23. CAD model of Type 3r lock flushing concept.
6.5 Type 5 lock flushing concept

The Type 5 lock flushing concept, shown in Figure 24, is a redesign of the filling and emptying system at Brandon Road Lock. The flow domain includes a portion of the lock upstream of the upper miter gates, both filling and emptying culverts with both the intakes and ports, and the lock chamber. The downstream end of the flow domain is located at the pintle of the downstream miter gates. The culvert is terminated downstream of the last (twelfth) port. One of the flushing ports mentioned in Section 4.2.5 is shown in Figure 24, Detail A. The upstream four ports on either side of the lock include deflectors as shown in Detail C. The port positioning and deflector geometry follow the guidelines set forth in EM 1110-2-1604 (HQUSACE 2006).

Figure 24. CAD model of Type 5 lock flushing concept.
7 Numerical Model Results

The results of each numerical model are shown and discussed in this chapter. Contour plots of the flow velocity and the original lock water concentration of each lock flushing concept during a simulated lock flushing operation are presented.

The flow results are presented with the velocity magnitude, \( V \), which is defined as

\[
V = \sqrt{u^2 + v^2 + w^2}
\]

(27)

where:

- \( u \) = \( x \)-component of flow velocity
- \( v \) = \( y \)-component of flow velocity
- \( w \) = \( z \)-component of flow velocity.

The simulation results are shown via contour plots that show the spatial distribution of the flow variables during lock flushing. These contour plots are presented for each of three different vertical slices in the lock chamber. As indicated in Figure 25, these three slices are located 3 ft from the lock chamber floor, 10 ft from the lock chamber floor, and at the lock chamber surface. Figure 25 has been stretched vertically by a factor of five, so the different slice locations can be seen more easily.

Figure 25. Simulation contour plot elevations.
The effectiveness and efficiency of each flushing operation is quantified by calculating the reduction of the original lock chamber water concentration during the flushing operation. The volume of the lock chamber where the original lock chamber water concentration reduces to pre-chosen levels is shown as different curves on the dilution plots. These flushing volume results are reported as percentages of the total lock chamber volume throughout the lock flushing operation.

Figure 26 shows an example plot of how the lock flushing performance is quantified. The horizontal axis represents the flushing time, and the vertical axis represents the percentage of the lock chamber volume that is reduced to certain concentration levels. In the example plot, the green curve shows the volume of the lock chamber that has been flushed to 60% of the concentration of water in the lock chamber during flushing. Two points on the curve are indicated. The red point on the plot represents 10 min of flushing flow. Moving vertically from the horizontal axis at 10 min to the red point, then proceeding to the left to the vertical axis shows that 27% of the lock chamber volume has been reduced to 60% concentration of the original lock chamber water. The blue point on the plot represents 15 min of flushing flow. Moving vertically from the horizontal axis at 15 min to the blue point, then proceeding to the left to the vertical axis shows that 92% of the lock chamber volume has been reduced to 60% concentration of the original lock chamber water.

Figure 26. Sample lock flushing volume plot — single curve explanation.
Additionally, the plots can be read to show how much of the lock chamber has been reduced to multiple concentration levels at a single flushing time. In Figure 27, the blue, red, and green curves represent 70%, 60%, and 50% concentrations, respectively, of water in the lock chamber during flushing. The black dashed line indicates 10 min of flushing flow. The plot is read by picking a flushing time and moving vertically from the horizontal, flushing time axis to each time the black dashed line intersects a concentration curve. For each concentration curve, move left to the horizontal axis to read the percentage of the lock chamber that has been flushed to the concentration indicated by the intersected concentration curve. For instance, the black dashed line first intersects the blue line, which indicates that after 10 min of flushing flow, 4% of the lock chamber has been reduced to 70% of the original concentration. Similarly, the red curve indicates that 25% of the chamber is reduced to 60% of the original concentration in 10 min of flushing. Also, the green curve indicates that 74% of the lock chamber has been reduced to 50% of the original concentration in 10 min of flushing.

Figure 27. Sample lock flushing volume plot — single flushing time explanation.
7.1 Type 1 lock flushing concept

The contour plots of the velocity magnitudes for the Type 1 lock flushing concept are shown in Figure 28–Figure 30. In each figure, the velocity contours are shown at the beginning of flushing, at 5 min of flushing, and at 10 min of flushing. The flushing discharge remains constant throughout the simulation. Flushing flow is introduced into the lock chamber at several locations via the filling and emptying ports. Viewing the contours closest to the chamber floor, the velocity magnitudes vary in both time and space. The jets that extend from each port have a maximum velocity of approximately 4 ft/sec. Each jet extends approximately halfway across the lock chamber. The jets are directed more toward the downstream miter gates for the ports that are farthest downstream. The contours 10 ft from the chamber floor show that the variation in velocity magnitude is much smaller farther away from the ports. At that elevation the maximum velocity magnitude is approximately 3 ft/sec. At the lock chamber surface, the velocity magnitudes vary more than near the center of the lock chamber water column. The largest velocity magnitudes at the water surface are approximately 3 ft/sec.
Figure 28. Type 1 velocity magnitude contours at 3 ft from chamber floor.

Type 1 Lock Flushing Concept
Existing Filling and Emptying System
Velocity Magnitudes Three Feet from Chamber Floor

Flush Time (min) = 0

Flush Time (min) = 5

Flush Time (min) = 10
Figure 29. Type 1 velocity magnitude contours 10 ft from chamber floor.

Type 1 Lock Flushing Concept
Existing Filling and Emptying System
Velocity Magnitudes Ten Feet from Chamber Floor

Flush Time (min) = 0

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

Flush Time (min) = 5

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

Flush Time (min) = 10

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
Figure 30. Type 1 velocity magnitude contours at lock chamber surface.
Contour plots of the original lock chamber water concentration for the Type 1 flushing concept are shown in Figure 31–Figure 33. The purpose of these figures is to show how much the concentration in the lock chamber varies in both time and space in the lock chamber during a lock filling operation. At the beginning of flushing (flushing time = 0), the entire lock chamber is orange/red indicating a uniform concentration of 100% of the original lock chamber water. During flushing, the contours in the lock chamber change from orange/red to green to blue. These changes show that the lock chamber flushing is reducing the concentration of original lock chamber water. Since flushing flow is introduced at multiple locations in the lock chamber, the original lock chamber water concentration is reduced gradually throughout the lock chamber. At 3 ft from the chamber floor, the effect of the ports is noticeable, and the reduction in chamber concentration varies dramatically in both time and space. Moving farther up the water column, the reduction in concentration is more gradual. After 15 min of lock flushing, the concentration of original lock chamber water for each elevation is approximately 50% for the entire chamber. Upstream of the first filling port, the original lock chamber concentration is even higher.
Figure 31. Type 1 original lock chamber water concentration contours 3 ft from chamber floor.

Type 1 Lock Flushing Concept
Existing Filling and Emptying System
Original Lock Chamber Concentration Three Feet from Chamber Floor

Flush Time (min) = 0

Flush Time (min) = 5

Flush Time (min) = 10
Figure 32. Type 1 original lock chamber water concentration contours 10 ft from chamber floor.
Figure 33. Type 1 original lock chamber water concentration contours at lock chamber surface.

**Type 1 Lock Flushing Concept**
Existing Filling and Emptying System
Original Lock Chamber Concentration at Water Surface

Flushing Time (min) = 0

<table>
<thead>
<tr>
<th>Original Lock Chamber Concentration (%)</th>
<th>0</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
</tr>
</thead>
</table>

Flushing Time (min) = 5

<table>
<thead>
<tr>
<th>Original Lock Chamber Concentration (%)</th>
<th>0</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
</tr>
</thead>
</table>

Flushing Time (min) = 10

| Original Lock Chamber Concentration (%) | 0 | 10 | 20 | 30 | 40 | 50 | 60 | 70 | 80 | 90 | 100 |
The flushing effectiveness and efficiency for Type 1 are shown in Figure 34 and Table 8. In the figure, the original lock water concentration is plotted against the flushing time. The different curves indicate how much of the lock chamber has reached different levels of original lock chamber water concentration during a flushing operation, with red indicating at most a 10% reduction in the original lock chamber concentration, green indicating at most a 50% reduction, and dark blue indicating a 99.9% reduction (essentially portions of the chamber where the water has been completely replaced by flushing water).

Most of the curves show a slow volume change initially, a period of rapid volume change, and finally a return to a slow volume change as the curves approach 100% of the lock chamber. The desired amount of flushing (99.9% reduction) as indicated by the dark blue line (which is essentially on top of the horizontal axis) is not attained in 40 min of flushing. After 15 min, indicated by the dashed black line, 80% reduction of the flow has only occurred in 4% of the lock chamber. Lock chamber volume percentages at 5 min increments of flushing are listed in Table 8. The values listed correspond to values that can be read directly from Figure 33, but the table values provide more precision in the percent volumes.
Table 8. Type 1 chamber flushing performance — 5 min intervals.

<table>
<thead>
<tr>
<th>Flushing Time (min)</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
</tr>
</thead>
<tbody>
<tr>
<td>99.9%</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
</tr>
<tr>
<td>90%</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>1</td>
<td>2</td>
<td>7</td>
<td>54</td>
<td>76</td>
<td>87</td>
</tr>
<tr>
<td>80%</td>
<td>&lt;1</td>
<td>1</td>
<td>4</td>
<td>57</td>
<td>85</td>
<td>92</td>
<td>96</td>
<td>99</td>
</tr>
<tr>
<td>70%</td>
<td>1</td>
<td>4</td>
<td>57</td>
<td>91</td>
<td>94</td>
<td>99</td>
<td>100</td>
<td>100</td>
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<td>60%</td>
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<td>100</td>
<td>100</td>
</tr>
<tr>
<td>50%</td>
<td>7</td>
<td>74</td>
<td>96</td>
<td>99</td>
<td>100</td>
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<tr>
<td>40%</td>
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<td>100</td>
<td>100</td>
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<tr>
<td>30%</td>
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<td>100</td>
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<tr>
<td>20%</td>
<td>87</td>
<td>99</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>10%</td>
<td>96</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

7.2 Type 2 lock flushing concept

The Type 2 lock flushing concept was simulated in a previous phase of numerical modeling work for the GLMRIS project. Between the completion of the first phase of numerical modeling effort and the beginning of the phase being reported in this report, the Type 2 lock flushing concept was removed from further consideration. The decision to remove the Type 2 concept is based on a combination of factors including high vertical velocities it produced in the lock chamber and construction (excavation) requirements for the new lateral manifold. The Type 2 numerical models used a fixed-lid boundary condition instead of a free-service boundary, and contour plots of the velocity magnitudes are included in Appendix C. No direct calculation of original lock water concentration was included in that phase of the numerical modeling work. The velocity magnitude contour plots for the Type 2 concept would perform, but these results should not be used as a direct comparison with the results shown and discussed in this chapter because extra degrees of freedom were included in those models, which can significantly affect the flow solution.

7.3 Type 3 lock flushing concept

The contour plots of the velocity magnitudes for Type 3 concept are shown in Figure 35–Figure 37. In each figure, the velocity contours are shown at
the beginning of flushing, at 5 min of flushing, and at 10 min of flushing. The flushing discharge is constant throughout the simulation. flushing flow is introduced into the lock chamber at the upstream end via four pipes through the gate sill. The outlets of these pipes are near the lock chamber floor. The jets that extend from each pipe have a maximum velocity of approximately 15 ft/sec. The contours closest to the chamber floor show that the velocity magnitudes vary in both time and space in the upstream third of the lock chamber. Farther downstream, the variation of velocity magnitude is much smaller, and the maximum flow velocities are approximately 2 ft/sec. The contours 10 ft from the chamber floor show that the variation in velocity magnitudes is smaller farther away from the ports but is still largely restricted to the upstream third of the lock chamber. At that elevation, the maximum velocity magnitude is approximately 7 ft/sec. At the lock chamber surface, the velocity magnitudes vary more than near the center of the lock chamber water column. The strong effect of introducing the flushing flow through the pipes is still very apparent at the chamber surface. The largest velocity magnitudes at the water surfaces are approximately 7 ft/sec.

Contour plots of the original lock chamber water concentration for Type 3 are shown in Figure 38–Figure 40. The purpose of these figures is to show how much the concentration in the lock chamber varies in both time and space in the lock chamber during a lock filling operation. At the beginning of flushing (flushing time = 0), the entire lock chamber is orange/red indicating a uniform concentration of 100% of the original lock chamber water. During flushing, the contours in the lock chamber change from orange/red to green to blue. These changes show that the lock chamber flushing is reducing the concentration of original lock chamber water during flushing. Since flushing flow is introduced at the upstream end of the lock chamber, the original lock chamber water concentration is reduced first at the upstream end of the lock chamber. The original lock chamber water concentration is reduced throughout the lock chamber as the flushing flow moves toward the downstream miter gates. There is no strong vertical variation in the original lock chamber water concentration. After 15 min of lock flushing, the original lock chamber concentration at the upstream end of the lock chamber has already been reduced to approximately 10%. These concentration contours indicate that Type 3 is more efficient than Type 1. Further, Type 3 does not produce any areas of the lock chamber that take significantly longer to flush than other areas.
Figure 35. Type 3 velocity magnitude contours 3 ft from chamber floor.

Type 3 Lock Flushing Concept
4, 4-ft Gate Sill Pipes
Velocity Magnitudes Three Feet from Chamber Floor

Flushing Time (min) = 0

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

Flushing Time (min) = 4

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

Flushing Time (min) = 10

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
Figure 36. Type 3 velocity magnitude contours 10 ft from chamber floor.

Type 3 Lock Flushing Concept
4, 4-ft Gate Sill Pipes
Velocity Magnitudes Ten Feet from Chamber Floor

Flushing Time (min) = 0

Flushing Time (min) = 4

Flushing Time (min) = 10
Figure 37. Type 3 velocity magnitude contours at lock chamber surface.

Type 3 Lock Flushing Concept
4, 4-ft Gate Sill Pipes
Velocity Magnitudes at Chamber Surface

Flushing Time (min) = 0

Flushing Time (min) = 4

Flushing Time (min) = 10
Figure 38. Type 3 original lock chamber water concentration contours at 3 ft from chamber floor.

**Type 3 Lock Flushing Concept**
**4, 4-ft Gate Sill Pipes**
Original Lock Chamber Concentration Three Feet from Chamber Floor

- **Flushing Time (min) = 0**
- **Flow →**

- **Original Lock Chamber Concentration (%):** 0 10 20 30 40 50 60 70 80 90 100

- **Flushing Time (min) = 5**
- **Flow →**

- **Original Lock Chamber Concentration (%):** 0 10 20 30 40 50 60 70 80 90 100

- **Flushing Time (min) = 10**
- **Flow →**

- **Original Lock Chamber Concentration (%):** 0 10 20 30 40 50 60 70 80 90 100
Figure 39. Type 3 original lock chamber water concentration contours at 10 ft from chamber floor.

**Type 3 Lock Flushing Concept**
*4, 4-ft Gate Sill Pipes*
*Original Lock Chamber Concentration Ten Feet from Chamber Floor*

Flushing Time (min) = 0

Flushing Time (min) = 5

Flushing Time (min) = 10
Figure 40. Type 3 original lock chamber water concentration contours at lock chamber surface.

Type 3 Lock Flushing Concept
4, 4-ft Gate Sill Pipes
Original Lock Chamber Concentration at Water Surface

Flushing Time (min) = 0

Flushing Time (min) = 5

Flushing Time (min) = 10
The flushing effectiveness and efficiency for Type 3 are shown in Figure 41 and Table 9. In the figure, the original lock water concentration is plotted against the flushing time. The different curves indicate how much of the lock chamber has been flushed to different levels of original lock chamber water concentration during a flushing operation with red indicating at most a 10% reduction in the original lock chamber concentration, green indicating at most a 50% reduction, and dark blue indicating a 99.9% reduction (essentially portions of the chamber where the water has been completely replaced by flushing water).

The curves corresponding to at least 70% dilution show a slow volume change initially, a period of rapid volume change, and finally a return to a slow volume change as the curves approach 100% of the lock chamber. Essentially complete flushing (99.9% reduction) as indicated by the dark blue line farthest to the right in the figure is only achieved in over 1% of the lock chamber after 20 min of flushing. However, 90% reduction of the original lock chamber concentration has occurred in 44% of the lock chamber after 15 min of flushing (indicated by the dashed black line). Lock chamber volume percentages at 5 min increments of flushing are listed in Table 9. The values listed correspond to values that can be read directly from Figure 41, but the table values provide more precision in the percent volumes.
Figure 41. Type 3 lock chamber flushing performance.

### Table 9. Type 3 chamber flushing performance — 5 min intervals.

<table>
<thead>
<tr>
<th>Flushing Time (minute)</th>
<th>Flushed Chamber Volume (% of total lock chamber volume)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dilution of Original Lock Chamber Water</td>
<td>5</td>
</tr>
<tr>
<td>99.9%</td>
<td>&lt;1</td>
</tr>
<tr>
<td>90%</td>
<td>&lt;1</td>
</tr>
<tr>
<td>80%</td>
<td>1</td>
</tr>
<tr>
<td>70%</td>
<td>15</td>
</tr>
<tr>
<td>60%</td>
<td>28</td>
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<td>50%</td>
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<td>30%</td>
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</tr>
<tr>
<td>20%</td>
<td>44</td>
</tr>
<tr>
<td>10%</td>
<td>54</td>
</tr>
</tbody>
</table>
7.4 Type 3r lock flushing concept

The contour plots of the velocity magnitudes for the Type 3r lock flushing concept are shown in Figure 42–Figure 44. In each figure, the velocity contours are shown at the beginning of flushing, at 5 min of flushing, and at 10 min of flushing. The flushing discharge is constant throughout the simulation. Flushing flow is introduced into the lock chamber at the upstream end via a rectangular slot through the gate sill. The jet that extends from the rectangular slot has a maximum velocity of approximately 15 ft/sec. Viewing the contours closest to the chamber floor, the velocity magnitudes vary in both time and space in approximately the upstream half of the lock chamber. The flow velocity in the jet halfway down the lock chamber is approximately 7 ft/sec. Farther downstream, the variation of velocity magnitude is much smaller, and the maximum flow velocities are approximately 4 ft/sec. The contours 10 ft from the chamber floor show that the velocity magnitudes are much smaller farther away from the rectangular slot but are still largely restricted to the upstream half of the lock chamber. At that elevation, the maximum velocity magnitude is approximately 7 ft/sec in the upstream half of the chamber and 4 ft/sec farther downstream. At the lock chamber surface, the velocity magnitudes vary more than near the center of the lock chamber water column. The jet issuing from the rectangular slot reaches the surface approximately halfway down the lock chamber creating 7 ft/sec flows at the surface.

Contour plots of the original lock chamber water concentration the Type 3r are shown in Figure 45–Figure 47. The purpose of these figures is to show how much the concentration in the lock chamber varies in both time and space in the lock chamber during a lock filling operation. At the beginning of flushing (flushing time = 0), the entire lock chamber is orange/red indicating a uniform concentration of 100% of the original lock chamber water. During flushing, the contours in the lock chamber change from orange/red to green to blue. These changes show that the lock chamber flushing is reducing the concentration of original lock chamber water. Since flushing flow is introduced at the upstream end of the lock chamber, the original lock chamber water concentration is reduced first at the upstream end of the lock chamber. The original lock chamber water concentration is reduced throughout the lock chamber as the flushing flow moves toward the downstream miter gates. There is no strong vertical variation in the original lock chamber water concentration. After 15 min of lock flushing, the original
lock chamber concentration at the upstream end of the lock chamber has already been reduced to approximately 10%.

Figure 42. Type 3r velocity magnitude contours 3 ft from chamber floor.
Figure 43. Type 3r velocity magnitude contours 10 ft from chamber floor.
Figure 44. Type 3r velocity magnitude contours at lock chamber surface.

Type 3r Lock Flushing Concept
Gate Sill Rectangular Slot
Velocity Magnitudes at Chamber Surface

Flushing Time (min) = 0

Flushing Time (min) = 5

Flushing Time (min) = 10
Figure 45. Type 3r original lock chamber water concentration contours 3 ft from chamber floor.

Type 3r Lock Flushing Concept
Gate Sill Rectangular Slot
Original Lock Chamber Concentration Three Feet from Chamber Floor

Flushing Time (min) = 0

Original Lock Chamber Concentration (%): 0 10 20 30 40 50 60 70 80 90 100

Flow →

Type 3r Lock Flushing Concept
Gate Sill Rectangular Slot
Original Lock Chamber Concentration Three Feet from Chamber Floor

Flushing Time (min) = 5

Original Lock Chamber Concentration (%): 0 10 20 30 40 50 60 70 80 90 100

Flow →

Type 3r Lock Flushing Concept
Gate Sill Rectangular Slot
Original Lock Chamber Concentration Three Feet from Chamber Floor

Flushing Time (min) = 10

Original Lock Chamber Concentration (%): 0 10 20 30 40 50 60 70 80 90 100

Flow →
Figure 46. Type 3r original lock chamber water concentration contours 10 ft from chamber floor.

Type 3r Lock Flushing Concept
Gate Sill Rectangular Slot
Original Lock Chamber Concentration Ten Feet from Chamber Floor

Flushing Time (min) = 0

Flushing Time (min) = 5

Flushing Time (min) = 10
Figure 47. Type 3r original lock chamber water concentration contours at lock chamber surface.

Type 3r Lock Flushing Concept
Gate Sill Rectangular Slot
Original Lock Chamber Concentration at Water Surface

Flushing Time (min) = 0

Flushing Time (min) = 5

Flushing Time (min) = 10
The flushing effectiveness and efficiency for Type 3r are shown in Figure 48 and Table 10. In the figure, the original lock water concentration is plotted against the flushing time. The different curves indicate how much of the lock chamber has been flushed to different levels of the original lock chamber water concentration during a flushing operation with red indicating at most a 10% reduction in the original lock chamber concentration, green indicating at most a 50% reduction, and dark blue indicating a 99.9% reduction (essentially portions of the chamber where the water has been completely replaced by flushing water).

The curves corresponding to at least 70% dilution show a slow volume change initially, a period of rapid volume change, and finally a return to a slow volume change as the curves approach 100% of the lock chamber. Essentially, complete flushing (99.9% reduction) as indicated by the dark blue farthest to the right has only been achieved in approximately 4% of the lock chamber after 20 min of flushing, although nearly 80% of the lock chamber is flushed after 25 min. A 95% reduction of the concentration has occurred in the entire lock chamber after 15 min of flushing (indicated by the dashed black line). Lock chamber volume percentages at 5 min increments of flushing are shown in Table 10. The values listed correspond to values that can be read directly from Figure 48, but the table values provide more precision in the percent volumes.

Figure 48. Type 3r lock chamber flushing performance.
Table 10. Type 3r chamber flushing performance — 5 min intervals.

<table>
<thead>
<tr>
<th>Flushing Time (minute)</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
</tr>
</thead>
<tbody>
<tr>
<td>99.9%</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>79</td>
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<td>100</td>
<td>100</td>
</tr>
<tr>
<td>90%</td>
<td>8</td>
<td>86</td>
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<td>100</td>
<td>100</td>
<td>100</td>
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<td>100</td>
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<tr>
<td>80%</td>
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<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>70%</td>
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<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>60%</td>
<td>80</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>50%</td>
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<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
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<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>30%</td>
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<td>100</td>
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<td>100</td>
<td>100</td>
</tr>
<tr>
<td>20%</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>10%</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

7.5 Type 5 lock flushing concept

The contour plots of the velocity magnitudes for the Type 5 lock flushing concept are shown in Figure 49–Figure 51. In each figure, the velocity contours are shown at the beginning of flushing, at 5 min of flushing, and at 10-min of flushing. The flushing discharge remains constant throughout the simulation. Flushing flow is introduced into the lock chamber at several locations via the filling and emptying ports. Viewing the contours closest to the chamber floor, the velocity magnitudes vary in both time and space. The jets that extend from each port have a maximum velocity of approximately 5 ft/sec. Each jet extends approximately halfway across the lock chamber. The jets are directed more toward the downstream miter gates for the ports that are farthest downstream. The flow deflectors on the first four ports on each culvert drastically reduce the distance the corresponding jets extend toward the opposite lock chamber wall. The contours 10 ft from the chamber floor show that the variation in velocity magnitudes is much smaller farther away from the ports. At that elevation, the maximum velocity magnitude is approximately 3 ft/sec. At the lock chamber surface, the velocity magnitudes vary more than near the center of the lock chamber water column. The largest velocity magnitudes at the water surfaces are approximately 3 ft/sec. The velocity magnitudes in the lock chamber for Type 5 are noticeably larger throughout the lock chamber than with Type 1.
Contour plots of the original lock chamber water concentration for Type 5 are shown in Figure 52–Figure 54. The purpose of these figures is to show how much the concentration in the lock chamber varies in both time and space in the lock chamber during a lock filling operation. At the beginning of flushing (flushing time = 0), the entire lock chamber is orange/red indicating a uniform concentration of 100% of the original lock chamber water. During flushing, the contours in the lock chamber change from orange/red to green to blue. These changes show that the lock chamber flushing is reducing the concentration of original lock chamber water during flushing. Since flushing flow is introduced at multiple locations in the lock chamber, the original lock chamber water concentration is reduced gradually throughout the lock chamber. At 3 ft from the chamber floor, the effect of the ports is noticeable, and the reduction in chamber concentration varies dramatically in both time and space. Moving farther up the water column, the reduction in concentration is more gradual. Note that after 15 min of lock flushing, the concentration of original lock chamber water for each elevation is approximately 30% for the entire chamber. Upstream of the flushing port and upstream of the filling ports, the original lock chamber concentration is even higher.
Figure 49. Type 5 velocity magnitude contours 3 ft from chamber floor.

Type 5 Lock Flushing Concept
Redesigned Filling and Emptying System
Velocity Magnitudes Three Feet from Chamber Floor

Flushing Time (min) = 0

Flow →

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

Flushing Time (min) = 5

Flow →

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

Flushing Time (min) = 10

Flow →

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
Figure 50. Type 5 velocity magnitude contours 10 ft from chamber floor.
Figure 51. Type 5 velocity magnitude contours at lock chamber surface.

Type 5 Lock Flushing Concept
Redesigned Filling and Emptying System
Velocity Magnitudes at Chamber Surface

Flushing Time (min) = 0

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

Flushing Time (min) = 5

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

Flushing Time (min) = 10

Velocity Magnitude (ft/s): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
Figure 52. Type 5 original lock chamber water concentration contours 3 ft from chamber floor.

**Type 5 Lock Flushing Concept**
Redesigned Filling and Emptying System
Original Lock Chamber Concentration Three Feet from Chamber Floor

Flushing Time (min) = 0

<table>
<thead>
<tr>
<th>Original Lock Chamber Concentration (%)</th>
<th>0</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
</tr>
</thead>
</table>

Flushing Time (min) = 5

<table>
<thead>
<tr>
<th>Original Lock Chamber Concentration (%)</th>
<th>0</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
</tr>
</thead>
</table>

Flushing Time (min) = 10

| Original Lock Chamber Concentration (%) | 0 | 10 | 20 | 30 | 40 | 50 | 60 | 70 | 80 | 90 | 100 |
Figure 53. Type 5 original lock chamber water concentration contours 10 ft from chamber floor.

Type 5 Lock Flushing Concept
Redesigned Filling and Emptying System
Original Lock Chamber Concentration Ten Feet from Chamber Floor

Flushing Time (min) = 0

Flushing Time (min) = 5

Flushing Time (min) = 10
Figure 54. Type 5 original lock chamber water concentration contours at lock chamber surface.

Type 5 Lock Flushing Concept
Redesigned Filling and Emptying System
Original Lock Chamber Concentration at Water Surface

Flushing Time (min) = 0

Flushing Time (min) = 5

Flushing Time (min) = 10
The flushing effectiveness and efficiency for Type 5 are shown in Figure 55 and Table 11. In the figure, the original lock water concentration is plotted against the flushing time. The different curves indicate how much of the lock chamber has been flushed to different levels of the original lock chamber water concentration during a flushing operation with red indicating at most a 10% reduction in the original lock chamber concentration, green indicating at most a 50% reduction, and dark blue indicating a 99.9% reduction (essentially portions of the chamber where the water has been completely replaced by flushing water).

The curves corresponding to at least a reduction of 50% of the original lock chamber concentration show a slow volume increase initially, a period of rapid volume increase, and finally a return to a slow volume increase as the curves approach 100% of the lock chamber. This improvement in efficiency is indicated in the concentration volume solution (Figure 52 through Figure 54) lying farther to the left and above the analogous curves for Type 1 (Figure 34). The desired amount of flushing (99.9% reduction) as indicated by the dark blue line that is essentially on top of the horizontal axis is not attained even in 40 min of flushing. After 15 min (indicated by the dashed black line) 90% reduction of the flow has only occurred in 45% of the lock chamber. Lock chamber volume percentages at 5 min increments of flushing are shown in Table 11. The values listed correspond to values that can be read directly from Figure 55, but the table values provide more precision in the percent volumes.
Figure 55. Type 5 lock chamber flushing performance.

Table 11. Type 5 chamber flushing performance – 5 min intervals.

<table>
<thead>
<tr>
<th>Flushing Time (min)</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
</tr>
</thead>
<tbody>
<tr>
<td>99.9%</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
<td>&lt;1</td>
</tr>
<tr>
<td>90%</td>
<td>&lt;1</td>
<td>1</td>
<td>45</td>
<td>66</td>
<td>78</td>
<td>91</td>
<td>96</td>
<td>100</td>
</tr>
<tr>
<td>80%</td>
<td>1</td>
<td>54</td>
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<td>92</td>
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<tr>
<td>60%</td>
<td>47</td>
<td>89</td>
<td>96</td>
<td>100</td>
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<td>100</td>
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</tr>
<tr>
<td>50%</td>
<td>74</td>
<td>93</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>40%</td>
<td>85</td>
<td>97</td>
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<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>30%</td>
<td>93</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
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<tr>
<td>20%</td>
<td>96</td>
<td>100</td>
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<td>100</td>
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<td>100</td>
<td>100</td>
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</tr>
<tr>
<td>10%</td>
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<td>100</td>
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</tr>
</tbody>
</table>
7.6 Lock flushing numerical model results summary

The numerical models provide information that can be used to make some direct comparisons of the relative performance of the lock flushing concepts. Two graphical representations of the performance of each flushing concept are shown: contour plots showing the spatial variation concentration during a flushing operation and lock chamber flushing performance scatter plots (Figure 28 through Figure 55).

The contour plots of concentration show how each type of system introduces water into the chamber. The convection-dominated systems gradually reduce concentration throughout the lock chamber during a flushing operation. However, the advection-dominated systems reduce the concentration of the upstream end of the lock chamber (where the flushing flow enters the chamber) and pushes the original lock chamber water downstream and out of the chamber. The contour plots also give a qualitative indication of how well each system flushes the lock chamber.

The lock chamber flushing performance plots provide a more macroscopic perspective of the lock chamber performance throughout a flushing operation. Viewed separately, the lock chamber flushing performance plots (Figure 34, Figure 41, Figure 48, and Figure 48Figure 55) indicate how effectively and how efficiently each flushing concept performs. For each plot, a shift in any curve to the left (x-axis, higher flushing efficiency) and upward (y-axis, larger portion of the lock chamber) indicates better lock flushing performance. Viewed together, those plots show which flushing concepts provide the most feasible option for flushing the lock chamber of ANS. Another insight from the lock flushing performance plots is whether a convection-dominated system (like Type 1 or Type 5) or an advection-dominated system (like Type 3 or Type 3r) would flush the lock chamber most effectively and efficiently.

The Type 1 performance results (Figure 34) show how well the Brandon Road Lock can be flushed with the existing F/E system. These results can be used to determine if using the existing structure sufficiently satisfies the goal of the lock flushing portion of GLMRIS. These results also serve as the point of comparison for any lock flushing concept that requires a change to Brandon Road Lock. The Type 1 concept does not fully flush the lock chamber (at least 99.9% concentration reduction) in 40 min of flushing.
The Type 5 performance results (Figure 41) show that the Type 5 lock flushing concept does perform significantly more effectively and efficiently than Type 1. The improved performance is largely due to the increase in the flushing discharge allowed by the resizing and new placement of the F/E ports. While the performance of the Type 5 concept is significantly better than the Type 1, the Type 5 still does not fully flush the lock chamber (at least 99.9% concentration reduction) in 40 min of flushing. Implementing the Type 5 concept would also yield a significantly shorter lock chamber filling and emptying time. This expectation is taken from the filling and emptying times of locks that are designed according to the current USACE guidelines (HQUSACE 2006).

The Type 3 performance results show that the advection-dominated concepts perform differently than the convection-dominated concepts. Beginning after approximately 11 min of lock flushing, the original concentration in the lock chamber is reduced by 95% in more than 5% of the lock chamber. For Types 1 and 5, 95% reduction is not achieved in 5% of the lock chamber until after approximately 32 min and 17 min, respectively. Another important aspect of the Type 3 performance is that noticeable portions of the lock chamber are fully flushed after approximately 27 min of flushing. Note that the flushing discharge that produces the improved flushing performance for Type 3 requires a lower flushing discharge (1,000 cubic feet per second [cfs]) than both Types 1 and 5 (1,350 cfs and 2,600 cfs, respectively).

The Type 3r indicate how well an advection-dominated flushing system could work if the maximum allowable discharge could be introduced in a way that did not produce excessively high velocities that would cause severe variations in the lock-chamber water surface. Over 5% of the lock chamber would reach at least 95% original concentration reduction after 7 min of flushing. Also, noticeable portions of the lock chamber are fully flushed after approximately 20 min of flushing, and 100% of the lock chamber is fully flushed after approximately 27 min of flushing. The Type 3r configuration is not feasible because the slot (as presented in Figure 23) will not fit through the upstream gate sill because of another (mechanical) components of the lock in the area. The Type 3r models were produced as an indication of flushing performance of the advection-dominated case when only the system hydraulics were considered.
8 Physical Model Considerations

A complete evaluation of the structural and hydraulic modifications of Brandon Road Lock cannot be performed using solely a numerical model. Further, current USACE policy does not allow construction of a modification to a navigation lock that is outside of current design guidance without a physical model study that investigates the ramifications of the design changes — particularly to safety related to vessels that would traverse the lock. Therefore, the construction of a physical model of the Brandon Road Lock with its existing F/E system and at least one of the lock flushing concepts has been proposed to follow the numerical modeling efforts. This physical model would provide the necessary information on the hydraulic performance of the lock related to vessels (hawser forces) and navigation safety that is required before any modifications could be made to Brandon Road Lock. This chapter outlines the considerations for designing the physical model and how to use the information gained from the physical model study to inform the GLMRIS team on how Brandon Road Lock would perform if one of the lock flushing concepts is implemented.

The primary similitude consideration in hydraulic modeling of navigation locks is that the scale is large enough to reduce the scale effects to an understandable level. A 1:25-scale model is the current practice for evaluating the performance of a lock chamber (HQUSACE 2006).

8.1 Kinematic similitude

Kinematic similarity is an appropriate method of modeling free-surface flows in which the viscous stresses are negligible. Kinematic similitude requires that the ratios of the inertial forces ($\rho V^2 L^2$) to the gravitational forces ($\rho g L^3$) in the model are equal to those of the prototype. Here, $\rho$ = fluid density, $V$ = fluid velocity, $L$ = a characteristic length, and $g$ = acceleration due to gravity. This ratio is generally expressed as the Froude number, $Fr$.

$$Fr = \frac{V}{\sqrt{gL}}$$  \hspace{1cm} (28)

Here, $L$, the characteristic length, is usually taken as the flow depth in open-channel flow.
The Froude number can be viewed in terms of the flow characteristics. Because a surface disturbance travels at the celerity of a gravity wave, \( \sqrt{gh} \) where \( h \) = flow depth, the Froude number describes the ratio of advection speed to the gravity wave celerity. Evaluation of the lock chamber performance generally focuses on modeling the hawser forces on moored barges during filling and emptying operations. During normal locking operations, the skin friction drag on the vessel is insignificant because horizontal fluid velocities are small. Hawser forces are generated primarily by slopes of the lock chamber water surface. The tow’s bow-to-stern water-surface differentials are the result of long period seiches in the lock chamber. Seiching is the process of gravity waves traveling in the longitudinal direction from the upper service gates to the lower service gates. Therefore, equating Froude numbers in the model and prototype is an appropriate means of modeling the lock chamber to ensure that the model measurements accurately describe (and can be scaled up to) the behavior of the prototype.

8.2 Dynamic similitude

Physical models are often used to model forces. Appropriate scaling of viscous forces requires the model be dynamically similar to the prototype. Dynamic similarity is accomplished when the ratios of the inertia forces to viscous forces \( (\rho v L) \) of the model and the prototype are equal. Here, \( v \) is the kinematic viscosity of the fluid. This ratio of inertia to viscous forces is usually expressed as the Reynolds number

\[
Re = \frac{VL}{\nu}
\]

and in pressure flow analysis, the culvert hydraulic diam is usually chosen as the characteristic length, \( L \). The Reynolds number quantifies the flow’s viscous forces relative to advection forces. As the Reynolds number increases, the flow is less affected by viscous shear (friction).\(^1\)

8.3 Similitude for lock models

Modeling lock filling and emptying systems is not entirely quantitative. The system is composed of pressure flow conduits and open-channel flow components. Further complicating matters, the flow is unsteady.

\(^1\) See Section 8.3 for further information on why reducing the effect of shear in a scaled model of a navigation model is important.
Discharges (therefore $Re$ and $Fr$) vary from no flow at the beginning of an operation to peak flows within a few minutes and return to no flow at the end of the cycle. Current physical model studies of lock designs employ 1:25-scale Froudian models in which the viscous differences (between the model and the prototype) are small and can be estimated based on previously reported model-to-prototype comparisons. Setting the model and prototype Froude numbers equal yields the relations between the dimensions and hydraulic quantities shown in Table 12, assuming a 1:25 length scale relationship.

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Dimension</th>
<th>Scale Relation Model:Prototype</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>$L_r = L_r$</td>
<td>$1:25$</td>
</tr>
<tr>
<td>Pressure</td>
<td>$P_r = L_r$</td>
<td>$1:25$</td>
</tr>
<tr>
<td>Area</td>
<td>$A_r = L^2$</td>
<td>$1:625$</td>
</tr>
<tr>
<td>Velocity</td>
<td>$V_r = L_r^{1/2}$</td>
<td>$1:5$</td>
</tr>
<tr>
<td>Discharge</td>
<td>$Q_r = L_r^{5/2}$</td>
<td>$1:3.125$</td>
</tr>
<tr>
<td>Time</td>
<td>$T_r = L_r^{1/2}$</td>
<td>$1:5$</td>
</tr>
<tr>
<td>Force</td>
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<tr>
<td>Reynolds number</td>
<td>$Re_r = L_r^{3/2}$</td>
<td>$1:125$</td>
</tr>
</tbody>
</table>

These relations are used to transfer model data to prototype equivalents and vice versa.

Complete similitude in a physical model is attained when geometric, kinematic, and dynamic similitudes are satisfied. Physical models of hydraulic structures with both internal flow (pressure flow) and external flow (free-surface flow) typically are scaled using kinematic (Froudian) similitude at a large enough scale that the viscous effects in the scaled model can be neglected.

Boundary friction losses in lock culverts are empirically described using the smooth-pipe curve of the Darcy-Weisbach friction factor where the head loss is expressed as

$$H_f = f \frac{LV^2}{D2g}$$

(30)
where:

\[ H_f = \text{head loss due to boundary friction} \]
\[ f = \text{Darcy-Weisbach friction factor} \]
\[ L = \text{culvert length} \]
\[ D = \text{culvert diam}. \]

The Darcy-Weisbach friction factor for turbulent flow in smooth pipes is given in an implicit form as (Vennard and Street 1982)

\[
\frac{1}{\sqrt{f}} = 2.0 \log \left( Re \sqrt{f} \right) - 0.8 \tag{31}
\]

The USACE has investigated more than 50 model and 10 prototype studies of lock filling and emptying systems (Pickett and Neilson 1988). The majority of these physical model studies were conducted at a scale of 1:25, although early studies sometimes used a 3:100 scale. Lock models constructed to a scale of 1:25 have maximum Reynolds numbers at peak discharges on the order of $10^5$ while the corresponding prototype values are on the order of $10^7$. This difference is illustrated in results from physical model (Ables 1978) and field (McGee 1989) experiments on the Whitten (Bay Springs) Lock presented on the filling curves in Figure 56. Because the friction factor decreases with increasing Reynolds number, the model is hydraulically too rough as compared to the prototype. The scaled friction losses in the model will therefore be larger than those experienced by the prototype structure. Consequently, the scaled velocities (and discharges) in the model will be lower, and the scaled pressures within the culverts will be higher than those of the prototype. Lower discharges result in longer filling and emptying times in the model than the prototype. Prototype filling and emptying times for similar designs will be smaller than those measured in a 1:25-scale lock model.
Boundary friction decreases with increasing Reynolds number. As mentioned previously, lock model velocities scaled using kinematic similitude (model Froude number equal to prototype Froude number) in a 1:25-scale model have maximum Reynolds numbers at peak discharges on the order of $10^5$, yet the corresponding prototype values are on the order of $10^7$. Therefore, scaled friction losses in the model are larger than those experienced by the prototype structure. (The model is said to be hydraulically too rough.) Consequently, the scaled velocities (and discharges) in the model are smaller, and the scaled pressures within the culverts are higher than those of the prototype. Even in lock systems in which low pressures are not a particular concern, the lower discharges cause longer scaled filling and emptying times in the model than those experienced by the prototype.

Even though a prototype lock filling-and-emptying system is normally more efficient than predicted by its hydraulic model, EM 1110-2-1604 (HQUSACE 2006) states that the difference in efficiency is acceptable as far as most of the modeled quantities are concerned (hawser forces, for example) and can be accommodated empirically for others (filling time and overtravel, specifically).

## 8.4 Model-prototype comparison

Direct component-to-component comparison of differences in model and prototype performance is difficult due to the lack of accurate data of the turbulent, unsteady flow. However, comparison of the overall lock
The lock coefficient, $C_L$, for model and prototype structures provides a dimensionless parameter that describes the hydraulic efficiency integrated over the flow cycle of locking operations. The nondimensional term is also convenient because rarely do data exist for the same pools in model and prototype. The lock coefficient is defined by the relation commonly referred to as the Pillsbury equation (Pillsbury 1915):

$$T - k_v t_v = \frac{2A_L}{\sqrt{2gC_L A_c}} \sqrt{H + d} - \sqrt{d}$$

or

$$C_L = \frac{2A_L}{\sqrt{2gA_c \left( T - k_v t_v \right)}} \sqrt{H + d} - \sqrt{d}$$

where:

- $C_L$ = overall lock coefficient
- $A_L$ = plan area of the lock chamber
- $H$ = initial head or lift
- $d$ = lock-chamber water level over-travel (undertravel for lock emptying)
- $A_c$ = sum of culvert area at each operation valve
- $T$ = filling or emptying time
- $t_v$ = valve operation time
- $k_v$ = valve coefficient (which generally ranges from 0.45 to 0.55 and is taken as 0.5 for the present study.)

The lock coefficient for existing locks ranges from about 0.45 for relatively slow operation to approximately 0.90 for very efficient systems that provide rapid operation (HQUSACE 2006).

Physical model and field data suggest that the coefficient from a 1:25-scale model can range from 11% to 17% less than the prototype equivalent during filling and 12% to 19% less during emptying. As previously discussed, the prototype structures are relatively more efficient because of the differences in viscous forces. Some examples from studies listed in the references are presented in Table 13. Equation 32 shows that the lock coefficient quantifies a lock’s filling and emptying performance using several lock parameters that, in turn, are related to other lock
characteristics. These “hidden” lock parameters, particularly the viscous effects, are a primary driver in the value of the lock coefficient. The lock coefficient should not be used to determine a proper scale for a physical model study.

<table>
<thead>
<tr>
<th>Lock Project</th>
<th>Filling Operations</th>
<th>Emptying Operations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Model</td>
<td>Prototype</td>
</tr>
<tr>
<td>Bankhead</td>
<td>0.66</td>
<td>0.78</td>
</tr>
<tr>
<td>Lower Granite</td>
<td>0.77</td>
<td>0.93</td>
</tr>
<tr>
<td>Bay Springs</td>
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<td>0.75</td>
</tr>
<tr>
<td>Bonneville</td>
<td>0.61</td>
<td>0.72</td>
</tr>
<tr>
<td>Barkley*</td>
<td>0.75</td>
<td>0.84</td>
</tr>
<tr>
<td>Greenup*</td>
<td>0.57</td>
<td>0.62</td>
</tr>
</tbody>
</table>

*Barkley and Greenup locks were tested in 3:100- (1:33.3)-scale models.

### 8.5 Similitude for mixing models

Hydraulic modeling of mixing requires not only geometric and kinematic similitude but also dynamic similitude so that model observations are representative of prototype behavior. The most important phenomenon that must be modeled is the interplay between momentum of the issuing jet and the ambient fluid. This behavior dominates the flow both near and to some distance from the discharge point. To achieve dynamic similitude, the values of Reynolds number and Froude numbers in the model and prototype must be the same. This cannot be achieved unless the model is full scale (1:1-scale) or a different fluid is used in the model. The Reynolds number is always delegated to secondary importance with the provision that its value in the model is sufficiently large to achieve turbulent flow (Fischer et al. 1979). This turbulent flow requirement means that the Reynolds number of the discharge jet based on the jet flushing velocity and diam should be at least 2,000 (Fischer et al. 1979). This Reynolds number would produce a turbulent jet in the model. Therefore, a 1:25-scale Froude-scaled model with Reynolds number of 100,000 — well above the turbulent threshold — will be large enough to provide a reasonable representation of prototype mixing.
8.6 Hawser forces during flushing

Navigation locks are designed and operated to ensure safety for vessel operators and project personnel. Safety is viewed in terms of lock chamber performance, which is evaluated based on surface currents and turbulence. Conditions in a lock chamber cannot be hazardous to small craft but must not cause excessive forces the mooring line forces required to hold the design vessel in place. Those mooring forces, referred to as hawser forces, are limited to 5 tons by USACE lock design criteria (HQUASC 1995, 2006).

The hawser forces can be determined if the motion of a moored system (barge tow or other vessel) is known. If the buoyant force balances the barge weight and if the pitch motion is assumed negligible, then the single-degree-of-freedom equation of motion for such a moored system is

$$1 + C_a m_v \ddot{s} + C_h \dot{s} + K_0 + k s = F$$

(34)

where:

- $s$ = surge displacement of the barge
- $C_a$ = added mass coefficient
- $m_v$ = mass of the barge tow
- $C_h$ = hydrodynamic damping coefficient
- $K_0$ = initial tension in the hawser
- $K$ = hawser spring constant
- $F$ = the total force acting on the barge ($F_s + F_r + F_p$)
- $F_s$ = difference in hydrostatic force between the bow and stern
- $F_r$ = force due to shear stress
- $F_p$ = hydrodynamic response (force required to accelerate the fluid) (Kalkwijk 1973).

The overscript dot indicates differentiation with respect to time.

Steady-state forces acting on a moored vessel can be determined from the right-hand side of Equation 33, which is the sum of the external forces acting on the system. In equation form

$$F_s = \rho g b d l S_s$$

(35)
\[ F_r = \frac{1}{2} C_f \rho A V |V| \]  
\[ F_p = \frac{1}{2} \rho bd C_p V |V| \]

where:

- \( b \) = beam width of barge
- \( d \) = barge draft
- \( l \) = barge length
- \( S_s \) = slope of the water surface
- \( g \) = acceleration due to gravity
- \( \rho \) = fluid density
- \( C_f \) = friction coefficient
- \( A \) = wetted area of the hull
- \( C_p \) = pressure coefficient
- \( V \) = mean velocity of fluid relative to the vessel.

Although this single-degree-of-freedom model only simulates the vessel’s surge, Natale and Savi (1994) demonstrate its accuracy in modeling barges moored in a lock chamber. The single-degree-of-freedom equation of motion (Equation 33) is a second-order, nonhomogeneous, ordinary differential equation for a damped system with external forcing. In mooring applications, the system is generally underdamped, and the displacement of the moored vessel oscillates with an exponential decay in amplitude.

A model of forces exerted in hawsers mooring a vessel in a lock chamber has been used in conjunction with a numerical flow model of lock filling systems to optimize lock operations (Natale and Savi 1994, 2000). Information on such parameters as the hawser spring constant is available from sources such as Naval Facilities Engineering Command (1986a,b). Added mass and hydrodynamic damping coefficients needed to model the mooring of a barge tow in a lock chamber are given in Stockstill (2003).

Often, the only force considered is the hydrostatic force since it is much larger than the shear due to friction or the hydrodynamic response. However, using the equation of motion provides a more accurate answer. Equation 33 can be used in conjunction with a numerical flow solution to estimate the forces exerted during flushing operations when a tow is moored in the chamber. This modeling system could also be used to
estimate hawser forces on vessels moored downstream of the lock in the lower approach during flushing. This process would require values for the added mass and damping coefficients, which could be determined using physical model experiments. This one-dimensional modeling approach in conjunction with numerical modeling could provide estimates of the hawser forces, but a physical model would still be required for any modeling with a vessel inside a lock chamber.

If the inertia is neglected, then the longitudinal flow is uniform from one end of the chamber to the other. The water-surface slope in the lock chamber can be estimated using the Darcy-Weisbach equation.

$$\frac{H_L}{L} = S_f = S_s = \frac{f Q^2}{8R gA^2}$$

where:

\( S_f \) = friction slope of the water surface  
\( Q \) = discharge  
\( A \) = flow area = \( Bh \)  
\( R \) = hydraulic radius of the channel cross-section; flow area divided by wetted perimeter \( R = \frac{Bh}{2h+B} \).

The dimensions of Brandon Road Lock are included in Table 1 and Table 2 in Section 2.3.

The water-surface slope can also be estimated using the Manning’s equation:

$$S_s = \frac{n^2}{C_m R^{\frac{3}{2}}} \left( \frac{Q}{A} \right)^2$$

where:

\( n \) = Manning’s roughness coefficient  
\( C_m \) = a dimensional constant (\( C_m = 1 \) for SI units and 2.208 for U.S. customary units).
A reasonable value of Manning’s coefficient for a lock chamber with concrete walls and rock floor is 0.015, depending on the roughness of the lock floor.

The information provided in this chapter summarizes the aspects of the lock flushing system that must be considered for and during a physical model study. Without a physical model study of the lock flushing system, the navigation safety concerns for vessels in the lock chamber during flushing cannot be addressed.
9 Further Research – Prototype and Physical Model Testing

9.1 Prototype tests

A field study of Brandon Road Lock has been proposed by the U.S. Geological Survey. The field study would provide data on the flow distribution in the lock chamber for the existing F/E system. These field data will be used to validate future numerical and physical models. The prototype test plan includes measuring velocities in the lock chamber during steady flow conditions through the filling system, similar to the Type 1 lock flushing concept. To avoid any safety issues, these experiments will be conducted without tows in the lock chamber. If the validation results find that the physical model reproduces field conditions with an empty chamber, then experiments with a tow present in the chamber can be conducted with confidence.

9.2 Physical model experiments

A physical model is required for the lock flushing concept that is being considered for implementation after the numerical modeling phase. Before a final decision is made on the lock flushing concept to be constructed at Brandon Road Lock, a physical model would be needed to answer questions related to safety, because of existing USACE regulations, and uncertainties that currently exist in the numerical modeling of navigation locks and the related flow situations.

9.2.1 Safety

Vessel movement within the lock chamber during flushing is the primary safety concern. Fluid and vessel forces are derived from many sources including hydraulic forces acting on the vessel and mooring line forces. The processes involved in calculating vessel movement within the lock chamber during flushing are very complex and include poorly constrained parameters (e.g., added mass, hydraulic damping coefficients, turbulence intensity, scales of motion). Further research is needed, especially with regard to large vessels operating in confined spaces as well as vessel displacement sensitivities to some input parameters, to determine appropriate values for use in analytical and numerical models.
Hawser criteria are based on the hydrodynamic vessel forces and are designed to provide a high degree of safety during lock operations. The vessel is expected to experience the maximum safe forces for an extended period during the flushing process. Flushing produces a longitudinal water surface slope that will generate substantial forces on vessels. Since the hawser forces must be known for safe lock operation when vessels are in the chamber, the only way to develop safe operating criteria is to measure these forces in a physical model.

9.2.2 U.S. Army Corps of Engineers (USACE) regulations

USACE regulations require physical models for lock designs that do not follow the design criteria directly. EM 1110-2-1604 *Hydraulic Design of Navigation Locks* (HQUSACE 2006) and EM 1110-2-2602 *Planning and Design of Navigation Locks* (HQUSACE 1995) describe the USACE requirements for lock design and construction including navigation criteria. EM1110-2-1604 (HQUSACE 2006) defines the criteria for maximum hawser forces on moored vessels based on physical model studies. Therefore, USACE guidance standards require a physical model to determine the maximum safe force. EM 1110-2-2602 (HQUSACE 1995) states that “physical model studies... are a traditional and necessary part of the planning and design phase for most navigation facilities.” While the document mentions the use of numerical models, they currently cannot sufficiently reproduce the flow conditions and subsequent physical quantities to address new designs or extensive modifications to existing projects. The hydraulic engineering community does not have a thoroughly validated modeling system capable of providing the accuracy required to meet the USACE hawser safety criteria prescribed in EM 1110-2-1604 (HQUSACE 2006). Existing numerical models are appropriate for low-level screening to determine relative differences (not absolute) between some design alternatives but are inappropriate for exploring the extensive modifications proposed in the Brandon Road lock flushing study.

9.2.3 Numerical modeling uncertainties

Numerical models (including those presented in this report) can include a tracer or particle tracking feature to simulate chamber flushing, but the numerical model results are only as good as the values of the empirical constants that appear in the equation governing longitudinal dispersion in a flowing fluid (Equations 1 and 25). The tracer algorithms are based on turbulent diffusion theory that includes diffusion coefficients that are
derived from turbulence closure methods. These diffusion coefficients are subject to some degree of uncertainty as the mixing algorithm is specific to a particular set of equations, which are discretized in a particular manner. These equations include empirically determined coefficients and scaling parameters that are calculated from physical model or field data and often must be adjusted when applied in a new situation. There is no real randomness in the numerical model, so the result is an average or synthetic representation of the physical system. Furthermore, the added complexity of the alternative flushing systems combined with the presence of a vessel in the chamber has not been explored previously. Numerical model results cannot be verified to be representative of Brandon Road Lock until the values of parameters such as the diffusion coefficient for the flushing flows are determined in either a field study or a physical model study. Because of the uncertainty associated with the known diffusion coefficients for the flow situation encountered with navigation locks, tracer and dye studies are the only practical method to quantify lock flushing with higher certainty than 3D numerical models.

### 9.2.4 Physical model tests

A physical model can be constructed at the ERDC such that a suite of filling and emptying tests will be conducted to determine flushing rates for the lock chamber. A 1:25-scale physical model of Brandon Road Lock filling and emptying system can be used to measure flushing with and without tows to directly determine residence time within the lock chamber and the culverts. This will unequivocally establish the lock operation procedures required to reduce the lock ANS concentration to various dilution levels (e.g., 95%, 99%). In addition to emptying and filling without vessels, experiments using a remote-controlled tow with different barge configurations can determine the effects of vessel lockage on exchange. Vessel blockage during the locking process can either increase (exiting) or decrease (entering) the volume within the lock. Localized mixing along the hull as well as propeller wash can further complicate the exchange mechanism, so the case with tows is significantly more complex and critical to understanding the role of navigation in lock flushing. Questions of vessel effects such as (1) “What is the role of vessels during the locking process in enhancing/hindering the exchange flow,” (2) “How does vessel-induced turbulence and propeller wash modify residence time,” and (3) “What role does vessel blockage play in modifying the exchange flow rate” can be quantified. Experiments can explore the consequences of upbound and downbound tows with different barge configurations.
Rhodamine dye can be used to track the water mass within the lock chamber to quantify turbulent dispersion coefficients and flushing rates. Rhodamine dye has been used extensively in the marine environment as a water mass tracer, and accurate methodologies to measure the mixing rates and dispersion are well developed. Rhodamine fluorescence can be easily measured using inexpensive fluorometers, thus providing residence time and flushing efficiency within the lock chamber and culvert. Confetti can be used to measure surface water exchange and flow visualization techniques, such as high-speed digital photography, provide direct measurements of particle velocity and rotation to evaluate water mass exchange dynamics.

Particular attention can be paid to evaluating the Type 1 concept (existing filling and emptying system). If Brandon Road Lock can serve as the necessary barrier using the existing filling and emptying system, then the project modification expenses will be limited to ensuring that the filling valves can close under flowing conditions and that the lower miter gates are secured open during flushing. The barrier expense will then be limited to the additional flushing time required prior to each filling operation, which is common to each design alternative considered.

The effectiveness of the Type 4 concept (continuous flushing downstream of the lock) can be evaluated in the same physical model if it is constructed to include a sufficient reach of the lower approach. Experiments with tows passing over the Type 4 concept manifold can determine how the flushing currents interact with the tow’s return currents and propeller wash for both upbound and downbound tows, thus indicating possible impacts to navigation.

### 9.3 Tow effects

The currents generated as an upbound tow enters a lock chamber are illustrated in Figure 57. As the vessel enters the chamber, the displacement forces the same volume of water from the chamber. The resulting flow is referred to as the return current. A barge tow leaving the lock draws water into the chamber to replace the vessel’s displacement volume (Figure 58). Vessel speed and squat, and thus the return currents, are influenced by the depth over the sill and chamber floor (Maynord 2000). These return currents provide a means for ANS contamination through Brandon Road Lock. The flows near a barge tow as it transits into Brandon Road Lock should be considered when developing or evaluating a lock flushing concept or during any physical model study.
For the case where the vessel is exiting the lock into ANS-contaminated water, a rough approximation is to assume that the volume of water initially displaced by the tow will be replaced by ANS-contaminated water as the tow exits the chamber. In practice, the propeller wash will contribute significantly to mixing and increase the risk of ANS-contaminated water entering the chamber, particularly as the tow exits the chamber.
The work completed during this study and presented herein provides a good tool for determining the most likely lock flushing concepts for reducing ANS concentration in at prevent transfer of ANS upstream of Brandon Road Lock. However, the analytical and numerical model results provided in this report must be supplemented by a physical model study of the lock flushing system to ensure that the USACE lock operation and navigation safety concerns are met.
10 Summary and Recommendations

Ideas and preliminary hydraulic calculations have been presented as part of the development of an ANS flushing system for Brandon Road Lock. Five flushing system designs and operations have been drafted. The Type 1 lock flushing concept uses the existing culvert system. Types 2 and 3 lock flushing concepts require modifications to the lock structure. The Type 4 concept is not actually a lock flushing concept but rather a design that provides a continuous supply of clean water in the lower lock approach, thus preventing ANS from ever reaching the lock chamber. The Type 5 lock flushing concept is a redesigned F/E system.

Five lock flushing concepts (Types 1, 2, 3, 3r, and 5) have been tested in a 3D numerical model. Each model has a lock chamber without tows only. The Type 1 lock flushing concept shows highly unsteady flow fields. Original lock chamber water dilution happens throughout the lock chamber. The area upstream of the first filling and emptying ports flushes considerably faster than the other areas of the chamber.

The Type 2 flushing concept was simulated in the first phase of modeling work but was removed from further consideration because of construction and cost concerns. The numerical models of this concept did not include direct calculation of original lock dilution.

In the Type 3 lock flushing concept, flow is introduced at the upstream end of the chamber, and lock flushing occurs generally upstream to downstream. Because of lock chamber surface drawdown, the flushing discharge for the Type 3 concept must be significantly reduced from what flushing discharge is possible given the head differential between the upper and lower pools. If tows are not present in the lock chamber during flushing, the flushing discharge could remain at the maximum possible value. Flushing flow is also introduced in the upstream end of the lock chamber for the Type 3r concept. The conduit through which this flushing flow is introduced into the chamber is much larger than with the Type 3 concept to reduce the flushing velocities inside the chamber. The flushing behavior for this concept is also generally upstream to downstream. However, the flushing is much faster for Type 3r due predominantly to the higher flushing discharge. The introduction of flushing flow over a larger area of the upstream end of the lock chamber also increases the flushing efficiency. The main drawback
to the Type 3r design is that structural, mechanical, and construction considerations may significantly limit how much space is available for the flushing conduit through the upstream gate sill.

The Type 5 flushing concept behaves similarly to the Type 1 concept in that flushing flow is introduced through the filling and emptying system ports at multiple areas of the lock chamber. In the Type 5 concept, the filling and emptying system conforms to current USACE hydraulic design guidance standards. The ports have been resized and repositioned. The only deviation in this flushing concept from current design guidance is the inclusion of a flushing port on each culvert upstream of the filling and emptying ports. These flushing ports are located at the same station as the first filling and emptying port in the Type 1 concept. The sole purpose of these ports is to increase the flushing rate in the upstream end of the lock chamber. The Type 5 concept flushes the lock chamber more efficiently than the Type 1 concept but not as efficiently or effectively as either the Type 3 or Type 3r flushing concepts. The Type 5 concept has the benefit of reducing the lock filling time due to the improved filling and emptying system performance. Such a reduction in filling time would at least partially offset any lockage time increases due to flushing and would be a benefit to navigation.

The Type 1 lock flushing concept should be tested in a physical model. This concept would be the most economical one to implement because it will not require modifications to the lock structure and should be tested to determine the flushing efficiency using the existing filling system. A decision should be made whether the Type 3 (or some variation) or the Type 5 lock flushing concept should be tested. This decision should be based primarily on structural, mechanical, and cost engineering considerations. Placing pipes through the gate sill may require lower construction costs, but this concept (Type 3) may require flow control valves being submerged.

The Type 4 idea (lateral manifold downstream of lock) could be refined once information on the lock approach bathymetry and engineered channel is developed. The physical model should be designed to accommodate experiments to evaluate the Type 4 concept. Experiments can determine the effectiveness of the Type 4 concept as tows pass over it.

The numerical models that have been created and discussed in this report provide insight into how the different lock flushing concepts will flush the
lock chamber. However, much information for the flushing concepts related to vessel traffic including safety, vessel forces, and vessel motion cannot be adequately addressed in a numerical model with the current state of numerical model in the engineering community. If a lock flushing concept is chosen for implementation at Brandon Road Lock, a physical model study must be performed to evaluate the safety concerns to navigation traffic during the flushing operation before a final decision is made.
References


Appendix A: Eisenhower Lock Flushing System Drawings

Figure 59. Plan and section of ice flushing system added to Eisenhower Lock, St. Lawrence Seaway.
Figure 60. Section and elevation of ice flushing system added to Eisenhower Lock, St. Lawrence Seaway.
Appendix B: Computational Meshes

Before the lock flushing numerical simulations can be conducted, the geometry defined by the CAD model of each lock flushing concept has to be divided into a computational mesh. Each geometry is divided into meshes that are composed of tetrahedral elements. The sizes of these elements differ throughout the flow domain for each geometry. Generally, the size of the element depends on the complexity and size of the component in that area of the geometry. For instance, the filling and emptying ports and culverts require smaller elements to define the wall curvatures than elements in the lock chamber and areas upstream of the upstream gate sill. The geometry and the computation mesh for each lock flushing concept are shown in Figure 61–Figure 75. The number of nodes and tetrahedral elements for each mesh is listed in Table 14.

<table>
<thead>
<tr>
<th>Flushing Concept</th>
<th>Nodes</th>
<th>Elements</th>
</tr>
</thead>
<tbody>
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<td>Type 1</td>
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<td>1,874,851</td>
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<td>Type 2</td>
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<td>Type 3</td>
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<td>Type 3r</td>
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<td>Type 5</td>
<td>365,312</td>
<td>1,865,316</td>
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</table>
Figure 61. Type 1 geometry and computational mesh.

Figure 62. Type 1 geometry and computational mesh – Zoom 1.
Figure 63. Type 1 geometry and computational mesh – Zoom 2.

Figure 64. Type 2 geometry and computational mesh.
Figure 65. Type 2 geometry and computational mesh – Zoom 1.

Figure 66. Type 2 geometry and computational mesh – Zoom 2.
Figure 67. Type 3 geometry and computational mesh.

Figure 68. Type 3 geometry and computational mesh – Zoom 1.
Figure 69. Type 3 geometry and computational mesh – Zoom 2.

Figure 70. Type 3r geometry and computational mesh.
Figure 71. Type 3r geometry and computational mesh - Zoom 1.

Figure 72. Type 3r geometry and computational mesh - Zoom 2.
Figure 73. Type 5 geometry and computational mesh.

Figure 74. Type 5 geometry and computational mesh – Zoom 1.
Figure 75. Type 5 geometry and computational mesh – Zoom 2.
Appendix C: Type 2 Fixed Lid Model Results

The Type 2 lock flushing concept was simulated in a previous phase of numerical modeling work for the GLMRIS project. The numerical modeling process during that phase included fixed lid models of the lock flushing concepts with no direct calculation of original lock water concentration. The velocity magnitude contour plots (Figure 76 and Figure 77) are included in this appendix to give an idea of how the Type 2 concept would perform, but these results should not be used as a direct comparison with the results shown in Chapter 7 of this report because the displacement and velocity of the water surface were included in those models, which can significantly affect the flow solution.
Figure 76. Type 2 velocity magnitude contours at el 494.6.
Figure 77. Type 2 velocity magnitude contours at el 499.6.
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The Great Lakes and Mississippi River Interbasin Study (GLMRIS) is a U.S. Army Corps of Engineers study to evaluate methods of preventing the movement of aquatic nuisance species (ANS) movement between the Great Lakes and Mississippi River basins through aquatic connections. This report is an assemblage of ideas, preliminary hydraulic calculations, and numerical model evaluations that serve as part of the development of an ANS flushing system for Brandon Road Lock on the Illinois Waterway. Four flushing system designs and operations are presented. An analytical description of each lock flushing system design and numerical model results of those designs when applied to Brandon Road Lock are presented. Further, justifications and considerations of a physical model study of any lock flushing design that is chosen for construction are presented. This report is an overall commentary on design ideas and considerations for modeling the flushing rate of the lock chamber. The hydraulic details of lock flushing are outlined with the significant parameters of each lock flushing alternative highlighted. Numerical model results are presented to quantify the effectiveness of each lock flushing concept considered in this study.

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Attachment 5:

Reverse Flows in Brandon Road Lock Approach Channel

(GLMRIS-BR)
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Reverse Flows in Brandon Road Lock Approach Channel

1. Data Collection

The GLMRIS Study Team recognized that water is frequently observed moving from the downstream end of the approach channel upstream toward the lock chamber. To better understand the frequency and magnitude of this, United States Geological Survey (USGS) installed an Acoustic Doppler Velocity Meter (ADVM) in the approach channel to detect and measure velocity reversals in the approach channel. The ADVM (USGS 05538020 Des Plaines River at Rockdale, IL) is located approximately 1,150 feet downstream of the downstream lock gates. The instrument is located on a mooring structure on the left side (looking downstream) of the approach channel. The ADVM measures velocity in nine bins to produce horizontal velocity profiles across the channel. Figure 1 shows the location of the ADVM.

![Figure 1: Acoustic Doppler Velocity Meter (ADVM) in the Brandon Road Approach Channel.](image)

The gage displays average stream velocity in the x and y direction at 5-minute intervals. USGS Illinois-Iowa Water Science Center provided provisional 1-minute data for time windows between 13-18 June 2015, and 8-11 December 2014. The June 2015 time window represents a high flow condition and the December 2014 time window represents a moderate flow condition. Figure 2 shows an example of the flow reversals observed in the approach channel for the bin closest to the mooring structure. The strong positive velocity in the approach channel is the immediate response to a lock empty. The short duration, large discharge through the approach channel creates a reverse gradient along the approach channel, resulting in the negative flow towards the lock chamber.
2. Model Development

2.1 Hydraulic Model

An integrated one-dimensional, two-dimensional model of the Des Plaines River downstream of Brandon Road Lock and Dam was created using HEC-RAS 5.0. The purpose of the model is to identify factors contributing to velocity reversals in the approach channel, and to evaluate how velocity reversals are affected by potential geometry changes. A one-dimensional HEC-RAS geometry developed by the US Army Corps of Engineers – Rock Island District was used to create the one-dimensional portion of the model. The one-dimension portion of the model extends from River Station 284.1 downstream to River Station 271.5 (Dresden Island Lock and Dam). This model domain fully encompasses the Dresden Island Pool. The cross-section at River Station 284.1 connects the one-dimensional and two-dimensional portions of the model. The computational mesh used in the two-dimensional area uses a variable grid mesh. In the approach channel where higher resolution required, the grid has a 10-foot spacing. Downstream of Brandon Road Dam and downstream of the approach channel, the grid size increases to 50 ft. In the two-dimensional flow area, a constant manning’s n value of 0.022 was used to be consistent with the one-dimensional model developed by Rock Island District. Figure 3 shows the extent of the HEC-RAS model developed for Brandon Road Lock and Dam. Figure 4 shows the two-dimensional portion of the model.

Figure 2: Provisional ADVM 1-minute data showing the x-velocity in the approach channel.
Figure 3: Model extent for the HEC-RAS model developed for Brandon Road to evaluate flow reversals in the approach channel.

Figure 4: Two-dimensional flow area in HEC-RAS model developed for Brandon Road to evaluate flow reversals in the approach channel.
2.2 Hydrology

The model includes a large number of boundary conditions forcing flow into and out of the model domain. The major inflow into the upstream portion of the model domain is from releases at Lockport Lock and Dam upstream of Brandon Road. Discharges at Brandon Road Lock and Dam are regulated through 21 tainter gates and during flood flows, through 16 pairs of headgates. The approach channel experiences flow pulses through the empty valves during lock empties, and lower sustained discharges from the empty valve leakage. Boundary conditions were added to the two-dimensional mesh to represent each of the 21 tainter gates, head gates and lock empty valves. Two Joliet Generating Stations withdraw cooling water from the Des Plaines River and return the flow downstream. The larger station (on the north side of the river) withdrawals/discharges an average of 1,660 cfs. The smaller station (on the south side of the river) withdrawals/discharges an average of 588 cfs. Boundary conditions on the mesh domain represent these withdrawals and discharges. Unregulated discharges from Hickory Creek enter Dresden Pool immediately downstream of Brandon Road Dam on the left (looking downstream), and Sugar Run enters the pool 1700 feet downstream. Additional boundary conditions on the mesh domain represent these unregulated flows. The Kankakee and DuPage Rivers, along with Grant, Jackson and Cedar Creeks also enter Dresden Pool in the one-dimensional portion of the HEC-RAS model. A downstream stage boundary condition is used.

Boundary conditions come from a variety of sources. During the December 2014 and June 2015 data collection period, the USGS installed a stage hydrograph in the lock chamber. This stage time series was used, along with the lock chamber dimensions, to compute a flow time series for the lock empties. The time series generated with this data shows that the flow rates from each lockage can be highly variable (Figure 5). During the two time periods simulated, the left (looking downstream) empty valve was not operational due to maintenance, so the full lock empty was performed with the right empty valve. This operational change was model to the model by using only the right valve boundary condition.

Figure 5: Computed discharge from two lock empties on June 14, 2015.
For the tainter gates and the head gates, gate opening time series were used along with the gate rating curves, to generate flow hydrographs for each gate. Hickory Creek, the DuPage River and Hickory Creek are gaged, so gage data was used with a scaling factor to compensate for drainage area. Smaller ungaged tributaries were scaled from the Hickory Creek gage (05539000). Table 1 contains the inflow boundary conditions and the source for inflows.

Table 1: HEC-RAS boundary conditions

<table>
<thead>
<tr>
<th>Boundary Condition</th>
<th>Location</th>
<th>Source</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tainter and Head Gates</td>
<td>2D mesh domain</td>
<td>Rivergages.com and Rating Curves</td>
<td></td>
</tr>
<tr>
<td>Lock Empty Valves</td>
<td>2D mesh domain</td>
<td>Lock Chamber stage hydrograph with lock chamber dimensions</td>
<td></td>
</tr>
<tr>
<td>Hickory Creek</td>
<td>2D mesh domain</td>
<td>USGS (05539000)</td>
<td>Scaled 1.01</td>
</tr>
<tr>
<td>Sugar Run</td>
<td>2D mesh domain</td>
<td>USGS (05539000)</td>
<td>Scaled 0.12</td>
</tr>
<tr>
<td>Joliet Generating Station (right)</td>
<td>2D mesh domain</td>
<td>NPDES Permit (monthly/average)</td>
<td>- for withdrawal, + for return</td>
</tr>
<tr>
<td>Joliet Generating Station (left)</td>
<td>2D mesh domain</td>
<td>NPDES Permit (monthly/average)</td>
<td>- for withdrawal, + for return</td>
</tr>
<tr>
<td>Kankakee River @ Wilmington</td>
<td>RS 272.65</td>
<td>USGS (05527500)</td>
<td></td>
</tr>
<tr>
<td>DuPage River @ Shorewood</td>
<td>RS 276.5</td>
<td>USGS (05540500)</td>
<td>Scaled 1.16</td>
</tr>
<tr>
<td>Grant Creek</td>
<td>RS 275.0</td>
<td>USGS (05539000)</td>
<td>Scaled 0.16</td>
</tr>
<tr>
<td>Jackson Creek</td>
<td>RS 279.5</td>
<td>USGS (05539000)</td>
<td>Scaled 0.49</td>
</tr>
<tr>
<td>Cedar Creek</td>
<td>RS 280.2</td>
<td>USGS (05539000)</td>
<td>Scaled 0.13</td>
</tr>
<tr>
<td>Ungaged Uniform</td>
<td>Distributed uniform</td>
<td>USGS (05539000)</td>
<td>Scaled 0.25</td>
</tr>
<tr>
<td>Downstream Stage</td>
<td>RS 271.5</td>
<td>Rivergages.com</td>
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</tr>
</tbody>
</table>

The underlying terrain was developed from a variety of sources. A bathymetric survey performed by the Rock Island District is used for the channel bottom of the lock chamber, approach channel and the Des Plaines River downstream of the approach channel, and the scour area immediately downstream of Brandon Road Dam. A survey performed by the United States Geological Survey of the wide, flat section of the Des Plaines River between downstream of Brandon Road Dam and the approach channel was used for the bathymetry of this portion of the Des Plaines River. The survey was performed with an Acoustic Doppler Current Profiler (ADCP) and Real-Time Kinematic Global Positioning System (RTK-GPS) in shallow water depth areas. The Will County countywide LIDAR was used for overbank area, including the lock wall structures. Figure 6 shows the terrain used in the HEC-RAS two-dimensional flow area.
Figure 6: Terrain used for the two-dimension flow area downstream of Brandon Road.

2.3 Continuous Simulation Model Runs

Model simulations were performed for two runs between 14JUN2015 00:00 and 18JUN2015 24:00 and 08DEC 2014 12:00 and 11DEC2014 24:00 to allow a comparison between simulated and observed velocities in the approach channel where the ADVM was installed. The current version of the model runs the two-dimension flow area with a 1-second time step with mixed flow regime. The current version of HEC-RAS 5.0 does not allow a user to directly access x and y velocity components from the results, so the application ‘h5Dump’ was used to extract the x and y velocity components from the hdf output file. The velocity components were rotated to align parallel (x) and perpendicular (y) to the approach channel. Figure 6 shows the model results for two lock empties on 14 June 2015. The model provides a good representation of velocities in the approach channel for some lock empties, but the comparison is not as favorable for others. It may be beneficial for future modeling to collect vessel positioning in the approach channel, to better understand how the presence of vessels may affect the hydrodynamics. Figure 7 through 16 shows a comparison of observed and modeled velocities across the approach channel for the June 2015 model run.

The comparison is not as favorable for Bin 9, on the side of the approach channel opposite the ADVM instrument. It is possible that the distance across the approach channel begins to approach the maximum distance for the instrument. The plots also shows quite a bit of disturbance in the approach channel outside of lock empties. From available data, it is not possible to know whether these disturbances are from waiting or moving vessels, wind, or other forces.
Figure 7: Model results showing the comparison of channel velocities at Bin 1 (near mooring structure) between the HEC-RAS model results (red) and the observed ADVM data (blue).

Figure 8: Model results showing the comparison of channel velocities at Bin 1 (B1) between the HEC-RAS model results (red) and the observed ADVM data (blue) for the June 2015 run.
Figure 9: Model results showing the comparison of channel velocities at Bin 2 (B2) between the HEC-RAS model results (red) and the observed ADVM data (blue) for the June 2015 run.

Figure 10: Model results showing the comparison of channel velocities at Bin 3 (B3) between the HEC-RAS model results (red) and the observed ADVM data (blue) for the June 2015 run.
Figure 11: Model results showing the comparison of channel velocities at Bin 4 (B4) between the HEC-RAS model results (red) and the observed ADVM data (blue) for the June 2015 run.

Figure 12: Model results showing the comparison of channel velocities at Bin 5 (B5) between the HEC-RAS model results (red) and the observed ADVM data (blue) for the June 2015 run.
Figure 13: Model results showing the comparison of channel velocities at Bin 6 (B6) between the HEC-RAS model results (red) and the observed ADVM data (blue) for the June 2015 run.

Figure 14: Model results showing the comparison of channel velocities at Bin 7 (B7) between the HEC-RAS model results (red) and the observed ADVM data (blue) for the June 2015 run.
Figure 15: Model results showing the comparison of channel velocities at Bin 8 (B8) between the HEC-RAS model results (red) and the observed ADVM data (blue) for the June 2015 run.

Figure 16: Model results showing the comparison of channel velocities at Bin 9 (B9) between the HEC-RAS model results (red) and the observed ADVM data (blue) for the June 2015 run.
3. Production Runs

In addition to the two continuous simulation periods, model simulations were performed to evaluate flow reversals for lock empties, head gate openings and head gate openings. The two different operation changes affect flow conditions differently. Average flow conditions were used in the channel. Future work could include an evaluation of a range of high and low flows.

3.1 Lock Empty

When the lock chamber empties, the volume of the lock chamber between the headwater and tailwater elevation empties in approximately 15 minutes through two values on the left and right bank downstream of the lock chamber. As previously described from the continuous simulation model runs, the peak and duration of discharge can be highly variable. A production simulation was performed to isolate the effects of a lock empty. A simplified lock empty flow time series was created based on measured peak discharges observed by USGS, the volume of water in the lock chamber, and the typical duration of a lock empty. Figure 17 shows the flow hydrograph from the simplified lock empty.

![Figure 17: Flow hydrograph for a simplified lock empty from the Brandon Road lock chamber. In the HEC-RAS model, the hydrograph is split evenly between the right and left empty valves.](image)

The lock empty process produces short duration, high flow conditions in the approach channel. The highest flow rate occurs several minutes after the lock empty is initiated when the head difference between the lock chamber and downstream approach channel is the greatest. As the water level in the lock chamber drops and the water level in the approach channel rises from the large inflow of water, the flow rate decreases. During a typical lock empty, this process takes approximately 15 minutes.

Figures 18 through 20 show the progression of water surfaces and velocities in and downstream of the approach channel. Figure 18 shows the water surface 3 minutes after the lock empty has been initiated. The blue shading and contours show the upper portion of the approach channel is approximately 1.5 foot higher than downstream end of the approach channel. Flow velocities are in a downstream direction across the entire channel cross-section (Figure 19). After the lock empty is complete, the simulation shows reverse flows in the approach channel.
Figure 18: Approximately 3 minutes after the lock empty is initiated, the upper portion of the approach channel is approximately 1.5 feet higher than the downstream end and flow velocities are in the downstream direction across the full channel cross-section.

Figure 19: Approximately 5 minutes after the lock empty is initiated, the channel downstream of the approach channel is elevated and the at the elevated velocities have moved downstream.
Figure 20: Approximately 2 minutes after the lock empty complete, the channel downstream of the reverse flows in the approach channel appear.

3.2 Head Gate Operations

During flood operations, a series of head gates are opened to release flood flows downstream to Dresden Pool. When this occurs, flow discharges increase quickly downstream of Brandon Road Road Dam. A production simulation was performed to isolate the effects of head gate operations. An average flow condition downstream of Brandon Road was increased by 12,400 cfs to simulate the opening of two head gates. The head gates cause an elevation of water surface downstream of the approach channel. This results in a flow and velocity reversals into the approach channel.

Figure 21: Before the head gates are opened with average flow downstream of Brandon Road Dam.
Figure 22: Approximately 8 minutes after the head gates are opened, channel downstream of approach channel becomes elevated and reverse flow through approach channel is observed in the simulation.

4. Summary

The one-dimensional, two-dimensional model shows reasonable agreement between the observed velocities in the approach channel during USGS’s data collection period. The velocity data collected during this period and the modeling confirm that frequent flow reversals in the approach channel occur. The majority of these reversals occur during routine and daily lockages. The production runs also show that the operation of head gates, and the rapid increase in discharge associated with these gate operations, can also induce flow reversals in the approach channel.

The simulation runs performed to evaluate the reverse flows in the approach channel resulting from the lock empties and gate operating only were performed using average flow conditions in the channel. Future work with the model could include an evaluation of high and low flows to determine if the flow conditions in the channel significantly affect the magnitude and frequency of reversals. This future work could also evaluate how withdrawal flow rates from the power plants affects the reversal. Future work may also include an evaluation of alternatives to mitigate the impacts of the reversal through operation changes to the existing gate operating or proposed features.
Attachment 6:

Climate Change
(GLMRIS-BR)
GLMRIS – BR – Climate Change

Literature Review

USACE is undertaking its climate change preparedness and resilience planning and implementation in consultation with internal and external experts using the best available — and actionable — climate science. As part of this effort, the USACE has developed concise reports summarizing observed and projected climate and hydrological patterns, at a HUC2 watershed scale cited in reputable peer-reviewed literature and authoritative national and regional reports. Trends are characterized in terms of climate threats to USACE business lines. The reports also provide context and linkage to other agency resources for climate resilience planning, such as downscaled climate data for sub-regions, and watershed vulnerability assessment tools.

The USACE literature review report focused on the Great Lakes Region was finalized in April, 2015 (USACE, April 2015) and the USACE literature review focused on the Upper Mississippi Region was finalized in June, 2015 (USACE, June 2015). The Des Plaines River Watershed is located in the Upper Mississippi Region, but is near the Great Lakes Region, so climatic information from both literature reviews are relevant to the Des Plaines River Watershed. Figure 1, taken from the Great Lakes report, portrays the National Climate Assessment’s (NCA) reported summary of the observed change in very heavy precipitation for the U.S., defined as the amount of precipitation falling during the heaviest 1% of all daily events. The NCA results indicate that 37% more precipitation is falling in the Great Lakes Region now as compared with the first half of the 20th century, and that the precipitation is concentrated in larger events.

Figure 1. Percent changes in precipitation falling in the heaviest 1% of events from 1958 to 2012 for each region (Walsh et al., 2014).
The USACE literature review document summarizes and consolidates several studies which have attempted to project future changes in hydrology. Based on a review of four studies, the projected total annual precipitation is expected to have a small increase when compared to the historic record and the precipitation extremes are projected to see a large increase. It is noted that consensus between the studies is low, and although most studies indicate an overall increase in observed average precipitation, there is variation in how these trends manifest both seasonally and geographically. Figures 2 and 3, taken from the USACE Climate Change and Hydrology Literature Reviews, summarizes observed and projected trends for various variables reviewed.

For both the Upper Mississippi and Great Lakes Regions, increase in temperatures have been observed and additional increases in temperature are predicted for the future. For the Great Lakes Region, “nearly all studies note an upward trend in average temperatures, but generally the observed change is small. Some studies note seasonal differences with possible cooling trends in fall or winter.” For the Upper Mississippi Region, increasing trends were more uniformly reported by multiple studies. There is a strong consensus within the literature that temperatures are projected to continue to increase over the next century.

Increases in streamflow have been observed and projections for streamflow rates are variable. For the Great Lakes region, trends in low and annual streamflow were variable, with slight increases observed at some gages but other gages showing no significant changes. “Significant uncertainty exists in projected runoff and streamflow, with some models projecting increases and other decreases. Changes in runoff and streamflow may also vary by season. Projections of water levels in the Great Lakes also have considerable uncertainty, but overall lake levels are expected to drop over the next century.” For the Upper Mississippi Region, “a strong consensus was found showing an upward trend in mean, low, and peak streamflow in the study region.” There is no clear consensus on projected streamflow trends, “with some studies projecting an increase in future streamflow (as a result of increased precipitation) in the study region, while others project a decrease in flows (a result of increased evapotranspiration).” In general, projections suggest increased flow are expected in the winter and spring and decreased flows expected in the summer.
Figure 2. Great Lakes Region - Summary matrix of observed and projected climate trends and literary consensus. (USACE, 2015)
First Order Statistical Analysis & Nonstationarity Analysis

The closest USGS gage on the Des Plaines River near Brandon Road is located at Route 53 in Joliet (05537980). The gage has less than 13 years of record and is not included in the Corps of Engineers Non-Stationarity Tool. In addition to the short record, flows at this location are heavily influenced by regulation upstream at Lockport and at the lakefront control structures (Chicago Lock, Wilmette, and O’Brien). Below is a first order statistical analysis and non-stationarity analysis of the DuPage River, which is located adjacent and is tributary to the Des Plaines River downstream of Brandon Road.

There are a total of seven USGS stream gages within the DuPage watershed, as show on Figure 4 below. However, only four of those gages have a period of record greater than 30 years and therefore will be the focus of this analysis. These four stream gages include: 05539900 – West Branch DuPage River near West Chicago, IL, 05540095 – West Branch DuPage River near Warrenville, IL, 05540160 – East Branch DuPage River near Downers Grove, IL and 05540500 DuPage River at Shorewood, IL.
The drainage area for gage 05539900, West Branch DuPage River near West Chicago, IL, is 28.5 square miles and approximately 25 percent of the watershed is impervious cover. The gage has a period of record from July 1961 to present day for stream discharge and in 1994, the gage height also began being recorded.

The drainage area for gage 05540095, West Branch DuPage River near Warrenville, IL, is 90.7 square miles and approximately 25 percent impervious cover within the watershed. The gage has a period of...
record from October 1968 to present day for stream discharge and in 1993, the gage height also began being recorded. In addition, this gage was moved in 2011 and then again in early 2015.

The drainage area for gage 05540160, East Branch DuPage River near Downers Grove, IL, is 26.6 square miles and approximately 31 percent impervious cover within the watershed. The gage has a period of record from October 1989 to present day for stream discharge and in 1993, the gage height also began being recorded.

The drainage area for gage 05540500, DuPage River at Shorewood, IL, is 324 square miles and approximately 25 percent impervious cover. The gage has a period of record from October 1940 to present day for stream discharge and in 1993, the gage height also began being recorded.

**Linear Trend Analysis**
As outlined in ECB No. 2016-25, an investigation of the trends in the annual maximum flow gage data was performed to qualitatively assess impacts of climate change within the watershed using the USACE Climate Hydrology Assessment Tool. For the DuPage River, Figures 5 through 8 below show the observed, instantaneous peak streamflow obtained from the USGS website for four gages within the watershed that have a period of record greater than 30 years. There are no significant sources of regulation within DuPage River watershed. The figures depict an increasing trend in annual peak streamflow for the period of record at two sites. The two gages on the West Branch DuPage River, both have p-values smaller than 0.05 (the generally accepted threshold for significance) which indicates that the trends are statistically significant. No statistically significant trends are apparent within the peak streamflow records on the mainstem and East Branch of the DuPage River. Figure 9 displays the projected annual maximum monthly trends from the USACE Climate Hydrology Assessment Tool. As expected for this type of qualitative analysis, there is a considerable, but consistent spread in the projected annual maximum monthly flows. This spread is indicative of the uncertainty associated with climate changed hydrology. The trend in the mean projected annual maximum monthly streamflow indicates an increase over time. This increase for the West Branch DuPage River is statistically-significant (p-value < 0.05) and suggests the potential for future increases in flow relative to current conditions.
Figure 5. Annual Peak Streamflow Time Series, West Branch DuPage River near West Chicago, IL

Figure 6. Annual Peak Streamflow Time Series, West Branch DuPage River near Warrenville, IL

Figure 7. Annual Peak Streamflow Time Series, East Branch DuPage River near Downers Grove, IL

Figure 8. Annual Peak Streamflow Time Series, DuPage River at Shorewood, IL
Stationarity, or the assumption that the statistical characteristics of hydrologic time series data are constant through time, enables the use of well-accepted statistical methods in water resources planning and design in which the definition of future conditions relies primarily on the observed record. However, recent scientific evidence shows that climate change and human modifications of watersheds are undermining this fundamental assumption, resulting in nonstationarity (Friedman, et. al, 2016). An assessment of historic gage records was performed to determine if nonstationarity exists within the watershed by carrying out a nonstationarity detection analysis using the USACE’s nonstationarity detection tool.

Using the web-based Nonstationary Detection Tool, four stream gages within the watershed with a period of record of 30 years or more were investigated for nonstationarities. Of the four gages, two showed evidence of nonstationarities in annual instantaneous peak streamflow datasets.

For USGS 055399000 the West Branch DuPage River at West Chicago gage, both smooth and abrupt nonstationarities were detected, as shown in Figure 10. Nonstationarities were detected at three different points within the period of record: 1980, 1993 and 2006. In 2006, there is consensus between the Kolmogorov-Smirnov and LePage tests for distribution and Mood test for variance. For 1980 there is consensus between the Kolmogorov-Smirnov test for distribution and the Mann-Whitney test for mean. Finally for 1993 there is consensus between the Cramer-Von-Mises test for distribution and the Lombard Wilcoxon and Petitt tests for mean. The nonstationarities detected in 1980 and 1993 are indicated by...
statistical tests that target changes in mean and overall distribution, and the nonstationarity detected in 2006 is indicated by statistical tests that target changes in variance/standard deviation and overall distribution. Therefore the nonstationarities can be considered robust. In 2006, the nonstationarities detected correspond to changes of about 258 cubic feet per second (cfs) in the standard deviation. In 1980 and 1993, there is a 146 cfs and 215 cfs change in mean, respectively. For 1980, the nonstationarity can be considered robust and represents change in the mean associated with the data. For 1993, the nonstationarity demonstrates consensus and also represents change in the mean with the data. For 2006, the nonstationarity demonstrates consensus, can be considered robust, and represents change in the variance/standard deviation associated with the data. In addition, from the period of 1961-2008 there was a smooth nonstationarity being detected by the Lombard Wilcoxon Test, represented by the blue shading, which means there has been a gradual change in the mean recorded at the USGS gage site. Based on these results one can conclude that nonstationarities within the dataset exist. This is further supported when assessing monotonic trends within the record, as shown in Figure 11 through 14. Monotonic trends are assessed based on the entire period of record and for subsets of the period of record, after detected nonstationarities greater than 10 years in length (1981-2014 and 1994-2014). Increasing monotonic trends are detected in both the period of record dataset, as well as within the subset of data collected post 1981. No monotonic trends were detected within the data collected between 1994 to 2014.

For USGS gage 05540095, the West Branch DuPage River at Warrenville gage, abrupt nonstationarities were detected in 1992 and 1993, as shown in Figure 15. Since these occur within a five year period, they could be considered as one nonstationarity, in 1993. For this changepoint there is consensus between the Lombard Wilcoxon, Pettitt and Mann-Whitney tests. In addition, the nonstationarity detected in 1993 is indicated by statistical tests that target changes in mean and overall distribution. Therefore, the nonstationarity can be considered robust. In 1993, the nonstationarities detected correspond to changes of about 810 cfs in the mean of the annual instantaneous peak streamflow. Therefore, since the nonstationarity in 1993 demonstrates consensus, can be considered robust, and represents a significant change in the mean associated with the data, one can conclude that nonstationarities within the dataset exist. This is further supported when assessing monotonic trends within the record, as shown in Figure 16. Monotonic trends are assessed based on the entire period of record and for the period of record, after detected nonstationarities greater than 10 years in length (1993-2014). Increasing monotonic trends are detected in the period of record dataset but not within the data collected between 1993 to 2014.

There are no nonstationarities or statistically significant monotonic trends detected in the remaining two gages, as show in Figures 17 through 19 below. The East Branch DuPage River gage was shown for project completeness but was not be used for nonstationarity detection because there are significant gaps in the period of record and renders the results incorrect.
Figure 1. Nonstationarity Analysis, West Branch DuPage River near West Chicago, IL
Figure 2. Trend Analysis for West Branch DuPage River near West Chicago, IL

Figure 3. Trend Analysis (1980-2014) for West Branch DuPage River near West Chicago, IL
Figure 4. Trend Analysis (1993-2014) for West Branch DuPage River near West Chicago, IL
Figure 5. Nonstationarity Analysis, West Branch DuPage River near Warreenville, IL
Figure 6. Trend Analysis for West Branch DuPage River near Warrenville, IL

Figure 7. Trend Analysis (1993-2014) for West Branch DuPage River near Warrenville, IL
Figure 8. Nonstationarity Analysis, East Branch DuPage River near Downers Grove, IL
Figure 9. Nonstationarity Analysis, DuPage River at Shorewood, IL
Vulnerability Assessment Tool

The USACE Vulnerability Assessment Tool was applied for the 0712-Upper Illinois HUC-4 to assess the basin’s vulnerability to climate change impacts relative to the other 201 HUC-4 watersheds within the United States. The 0712-Upper Illinois HUC-4 includes both the DuPage and Des Plaines River. The USACE Watershed Climate Vulnerability Assessment (VA) Tool facilitates a screening level, comparative assessment of the vulnerability of a given HUC 04 watershed to the impacts of climate change relative to a maximum of 202 (depending on which business line is specified) HUC04 watersheds within the continental United States (CONUS). Assessments using this tool identify and characterize specific climate threats and sensitivities or vulnerabilities, at least in a relative sense, across regions and business lines. Ecosystem Restoration is the primary business line being assessed as part of this study.

The Watershed Vulnerability tool uses the Weighted Order Weighted Average (WOWA) method to represent a composite index of how vulnerable (vulnerability score) a given HUC04 watershed is to climate change specific to a given business line by using a set of specific indicator variables which relate to a particular business line. The HUC04 watersheds with the top 20% of WOWA scores are flagged as
vulnerable. All vulnerability assessment analyses were performed using the National Standard Settings.

Indicators considered within the WOWA score for Ecosystem Restoration include low flow reduction, mean annual runoff and flood magnification. Additional information about each of these indicator variables and how they are used to determine a WOWA score is described in the Vulnerability Assessment User Manual.

The USACE Climate Vulnerability Assessment Tool makes an assessment for two 30-year epochs centered at 2050 and 2085 to judge future risk due to climate change. These two epochs are selected to be consistent with many other national and international analyses related to climate. The Vulnerability tool assesses climate change vulnerability for a given business line using climate changed hydrology based on a combination of projected climate outputs from the general circulation models (GCM) and representative concentration pathway (RCPs) of greenhouse gas emissions resulting in 100 traces per watershed per time period. The top 50% of the traces is called “wet” and the bottom 50% of traces is called “dry.” Meteorological data projected by the GCMs is translated into runoff using the Variable Infiltration Capacity (VIC) macroscale hydrologic model. The VIC model applied to generate the results used by the Vulnerability Assessment Tool was developed by the U.S. Bureau of Reclamation and is configured to model unregulated basin conditions.

There is a great deal of uncertainty with the climate changed hydrology given by the vulnerability assessment tool. Each of the inputs to the vulnerability assessment tool has uncertainty associated with it. The vulnerability tool relies on projected, climate changed hydrology. The uncertainty associated with projected hydrologic data includes error in temporal downscaling, error in spatial downscaling, errors in the hydrologic modeling, errors associated with emissions scenarios, and errors associated with GCMs. Some of the uncertainty associated with the tool can be visualized because the tool separates results for each of the scenarios (wet versus dry) and epochs (2050 versus 2085) combinations rather than presenting a single, aggregate result (USACE, 2016). Beyond the uncertainties associated with the inputs to the vulnerability assessment tool, the analysis also contains substantial uncertainty inherent in the exact level of risk aversion selected (ORness factor) and the importance weights applied. Some users may elect to use a higher level of risk aversion while others may not. The importance weights of the indicator variables used to compute the WOWA (vulnerability) scores are subjective and there is no way to quantify which indicator variables are more important than others when making projections about vulnerability. The user should note that the uncertainty with climate changed hydrology projects is high and not currently, readily quantifiable.

For the Ecosystem Restoration business line, the project did not show vulnerability to either the Wet or Dry scenario. There was a 1.7% change in the WOWA score for the Dry scenario and a 1.6% change in the WOWA score for the Wet scenario in the HUC-4 Region with an Ecosystem Restoration business line as shown below.
Figure 20. Vulnerability Score, Dry Scenario HUC-4:0712-Upper Illinois

Figure 11. Vulnerability Score, Wet Scenario HUC-4:0712-Upper Illinois
REFERENCES


**Electric Barrier**

- Achieve a electric field of 20 V/ft at top of water surface.
- Stage Seasonal Precipitation; Temperature (snow melt)? Yes
- Electric Barrier: Repairs predict increased intensity in precipitation resulting in more frequent flood events, which has the potential to increase flood stages. Repairs also predict increased duration of dry periods. Increased flow (a higher environment) and increased depth may change the system and make it more susceptible to failure. Electric barrier should be designed for a range of flood conditions. The increased frequency of higher flood stages should not affect performance.
- Changes should be reviewed and fall within the original expected uncertainties of stage and flow. Electric barrier should be designed for a range of flood conditions. The increased frequency of higher flood stages should not affect performance.

**Complete Nutrient Management**

- Achieve target nutrient and optimum delivery angle* for a given duration.*
- Stage, Flow; Temperature (snow melt)? Yes
- Yes
- Water jet: Repairs predict increased intensity in precipitation for all nutrient events, which has the potential to increase flood stages. Repairs also predict increased duration of dry periods. Low - Operation of the water jet system can be adjusted after installation to accommodate changes in stage or flow. Increased flow (a higher environment) and increased depth may change the system and make it more susceptible to failure. Electric barrier should be designed for a range of flood conditions. The increased frequency of higher flood stages should not affect performance.

**Prevent swimming**

- ANS from transferring from the Mississippi River basin to the Great Lakes Basin through Brandon Rd Lock and Dam.
- Stage Seasonal Precipitation; Temperature (snow melt)? Yes
- Water jet: Repairs predict increased intensity in precipitation for all nutrient events, which has the potential to increase flood stages. Repairs also predict increased duration of dry periods. Low - Operation of the water jet system can be adjusted after installation to accommodate changes in stage or flow. Increased flow (a higher environment) and increased depth may change the system and make it more susceptible to failure. Electric barrier should be designed for a range of flood conditions. The increased frequency of higher flood stages should not affect performance.

**Engineered Channel**

- Prevent water from entering the engineered channel above the lock and dam.
- Stage Seasonal Precipitation; Temperature (snow melt)? Yes
- Yes
- Engineered Channel: Repairs predict increased intensity in precipitation for all nutrient events, which has the potential to increase flood stages. Repairs also predict increased duration of dry periods. Medium - Increased tailwater elevation of engineered channel will be adjusted after installation to accommodate changes in stage or flow. Increased flow (a louder environment) or increased depth may change the system and make it more susceptible to failure. Electric barrier should be designed for a range of flood conditions. The increased frequency of higher flood stages should not affect performance.

**Low - Operation of the water jet system can be adjusted after installation to accommodate changes in stage or flow. Increased flow (a higher environment) and increased depth may change the system and make it more susceptible to failure. Electric barrier should be designed for a range of flood conditions. The increased frequency of higher flood stages should not affect performance.

**High - there is a significant height/depth differential between the pools at the lock. At tailwater depths. Perform testing of electric barrier upon installation to verify operational performance.

**Medium - increased tailwater elevation of engineered channel will be adjusted after installation to accommodate changes in stage or flow. Increased flow (a louder environment) or increased depth may change the system and make it more susceptible to failure. Electric barrier should be designed for a range of flood conditions. The increased frequency of higher flood stages should not affect performance.

**None**

**Modeling still in development for these measures.**

<table>
<thead>
<tr>
<th>Objective</th>
<th>Measure</th>
<th>Design Considerations</th>
<th>Important Hydrograph</th>
<th>Driving Climate Variables</th>
<th>To be Integrated With Branch of Water Management</th>
<th>Engineer Considerations</th>
<th>Potential for Fluctuation</th>
<th>Design Considerations</th>
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<tbody>
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