Critical Infrastructure Protection and Resilience Program

Emergency Gap Closures

Joseph A. Padula, David D. Abraham, and Kevin L. Haskins

October 2010

Approved for public release; distribution is unlimited.
Emergency Gap Closures

Joseph A. Padula and Kevin L. Haskins

Geotechnical and Structures Laboratory
U.S. Army Engineer Research and Development Center
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

David D. Abraham

Coastal and Hydraulics Laboratory
U.S. Army Engineer Research and Development Center
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Final report

Approved for public release; distribution is unlimited.
Abstract: Lock gates and spillway gates are vulnerable features of navigation, flood control, and water resource projects. These structures are susceptible to damage from accidents; overloads due to natural events, lack of maintenance, or operational errors; and terrorist attack. Damage to lock or spillway gates can result in uncontrolled drawdown of pool, with possible severe impacts both upstream and downstream of the structure. Emergency gap closure systems are needed to rapidly stop flow and maintain pool and to accommodate early initiation of repairs. The literature was reviewed to identify the types of closure structures in existence and to assess their feasibility for use as emergency closures. In addition, some innovative concepts were developed, at least in preliminary form, and are presented herein.
Contents

Figures and Tables.......................................................................................................................... iv
Preface............................................................................................................................................. vi
Unit Conversion Factors.................................................................................................................... vii

1 Introduction..................................................................................................................................... 1
   Problem......................................................................................................................................... 1
   Recent emergency closures ............................................................................................................ 2
   Consequences of emergency closures ........................................................................................ 7

2 Current Practice for Maintenance Closures ............................................................................. 9
   Common maintenance closure systems ....................................................................................... 9
   Floating bulkheads ..................................................................................................................... 11
   Floating bulkheads outside the United States ............................................................................. 21

3 Emergency Closure Systems.................................................................................................... 23
   Existing emergency closure systems ........................................................................................... 23
   Objectives and challenges for emergency closure systems ...................................................... 26
   Preliminary concepts for emergency type closures .................................................................... 28
   Frame type closures .................................................................................................................. 30
   Flexible fabric structures ............................................................................................................ 33

4 Summary and Conclusions.......................................................................................................... 38

References........................................................................................................................................ 41

Appendix A: Hydraulic Forces on Bulkheads .............................................................................. 42
Appendix B: Structural Sizing of Frame Type Closure .................................................................. 53

Report Documentation Page
Figures and Tables

Figures

Figure 1. Maintenance bulkheads in place at Greenup Lock and Dam ........................................ 3
Figure 2. Barges in spillway at Bellville Dam .................................................................................. 4
Figure 3. Sunken towboat after accident at Montgomery Lock and Dam ........................................ 4
Figure 4. Failed tainter gate at Folsom Dam .................................................................................... 5
Figure 5. Downstream miter gates at Melvin Price Locks and Dam after ice passage accident .... 6
Figure 6. Greenup Locks and Dam miter gate and fractured anchor arm ....................................... 7
Figure 7. Maintenance closure systems ....................................................................................... 10
Figure 8. Cross section of the floating bulkhead for the Ice Harbor project (Walla Walla District) .... 12
Figure 9. Slots in wall to receive floating bulkhead at Ice Harbor Lock and Dam ......................... 13
Figure 10. Elevation of floating bulkhead used at the John Day project (Portland District) .......... 14
Figure 11. Floating bulkhead system after assembly (Tulsa District) ............................................ 15
Figure 12. Floating bulkhead system being positioned against faces of dam piers (Tulsa District) .... 15
Figure 13. Floating bulkhead for the Keystone Lake project (Tulsa District) ................................ 16
Figure 14. Floating caisson as it is being ballasted from horizontal to vertical position (Nashville District) ................................................................. 17
Figure 15. Floating caisson being placed into final position (Nashville District) ................................. 17
Figure 16. Bulkhead floats into position prior to placement .............................................................. 20
Figure 17. Bulkhead placement (a) with ballasted sections and bulkhead (b) in place ..................... 20
Figure 18. Floating bulkhead at Carraizo Dam in Puerto Rico ........................................................ 22
Figure 19. Emergency bulkheads for Melvin Price Locks and Dam (St. Louis District) ................. 24
Figure 20. Lift and miter gates at Greenup Locks and Dam (Huntington District) ......................... 25
Figure 21. Wicket gates used to dewater Lock No. 2 on the Lower Monongahela River (Pittsburgh District) ......................................................................................... 26
Figure 22. Bulkheads with baffles: (a) placement of first bulkhead, (b) bulkhead stack with baffles open, and (c) bulkhead stack with baffles closed ........................................... 29
Figure 23. Three hinged arch bulkhead concept ............................................................................ 30
Figure 24. Tension gate concept (New Orleans District) ............................................................... 31
Figure 25. Emplacement of framed closure structure with flexible damming surface ..................... 32
Figure 26. Small scale test of float-in frame closure system ............................................................ 32
Figure 27. Water-filled fabric lock closure concept ...................................................................... 34
Figure 28. Caisson and plug concept ............................................................................................. 34
Figure 29. Small-scale double stacked arch tube .......................................................................... 35
Figure 30. Test of small-scale arch tube ....................................................................................... 36
Figure 31. Semicircular plug concept for spillways ...................................................................... 36
Tables

Table 1. Comparison of emergency closure concepts................................................................. 37
Table A1. Iterative computation of upstream head, flow and resulting forces on bulkheads................................. 46
Table A2. Pressures under the bulkheads.................................................................................. 48
Table B1. Beam selection table.................................................................................................. 54
Preface

This study was conducted by personnel of the U.S. Army Engineer Research and Development Center (ERDC) at Vicksburg, MS, under the Critical Infrastructure Protection and Resilience (CIPR) Program of the U.S. Army Corps of Engineers (USACE). The USACE Program Manager was Yazmin Seda-Sanabria, Directorate of Contingency Operations and Homeland Security, USACE Headquarters.

The research was performed by members of the ERDC Geotechnical and Structures Laboratory (GSL), Geosciences and Structures Division (GSD), Structural Engineering Branch (StEB), and the ERDC Coastal and Hydraulics Laboratory (CHL), Flood and Storm Protection Division, during the period April 2009–January 2010. The work was performed under the supervision of Dr. Gordon W. McMahon, ERDC Program Manager for CIPR research projects, and Dr. Michael K. Sharp, ERDC Technical Director for Water Resources Infrastructure.

Dr. Joseph A. Padula was the Principal Investigator and team leader for this study. This report was prepared by Dr. Padula and Kevin L. Haskins, GSL, along with Dr. David D. Abraham, River Engineering Branch, CHL. Dr. Donald L. Ward, CHL, assisted with development and testing of the water-filled fabric structure.

During this investigation, Terry R. Stanton was Chief, StEB; Bartley P. Durst was Chief, GSD; Dr. William P. Grogan was Deputy Director, GSL; and Dr. David W. Pittman was Director, GSL.

COL Gary E. Johnston was Commander and Executive Director of ERDC. Dr. Jeffery P. Holland was Director.
## Unit Conversion Factors

<table>
<thead>
<tr>
<th>Multiply</th>
<th>By</th>
<th>To Obtain</th>
</tr>
</thead>
<tbody>
<tr>
<td>cubic feet</td>
<td>0.02831685</td>
<td>cubic meters</td>
</tr>
<tr>
<td>degrees (angle)</td>
<td>0.01745329</td>
<td>radians</td>
</tr>
<tr>
<td>feet</td>
<td>0.3048</td>
<td>meters</td>
</tr>
<tr>
<td>kips (force)</td>
<td>4.448222</td>
<td>kilonewtons</td>
</tr>
<tr>
<td>pounds (force) per square foot</td>
<td>47.88026</td>
<td>pascals</td>
</tr>
<tr>
<td>pounds (mass)</td>
<td>0.45359237</td>
<td>kilograms</td>
</tr>
<tr>
<td>square feet</td>
<td>0.09290304</td>
<td>square meters</td>
</tr>
</tbody>
</table>
1 Introduction

Problem

The U.S. Army Corps of Engineers (USACE) maintains and operates over 200 locks and dams on the nation’s navigable waterways and over 400 reservoir dams. As the result of a series of accident-related failures and the tragedies of September 11, 2001, there has been increased concern for maintaining the integrity of water resource, flood control, and navigation projects. Methods for reducing the threat of structural damage have been investigated, and some of the identified mitigation measures have been implemented. However, because of the random nature of accidents, operational mishaps, and terrorist actions, it is essentially impossible to eliminate the possibility of catastrophic damage from such varied underlying causes, even if unlimited funding were available.

Both USACE and other interested parties in the dams sector have identified the critical need for measures to facilitate rapid recovery to minimize the consequences of structural damage to lock and dam projects. Typically, lock and spillway gates are relatively vulnerable features on a project. Gate structures can suffer significant damage from accidents, inadequate or improper maintenance, terrorist attacks, and natural events, resulting in loss of pool and/or shutdown of navigation with severe economic and life safety consequences. Destruction of lock or dam gates could result in economic losses to an entire region if repairs cannot be performed in a reasonable amount of time, regardless of the underlying cause of the damage. In addition, the USACE (2006) guidance provided in Engineer Manual (EM) 1110-2-1604 notes that the consequences of losing the upper pool to areas downstream of a project may include blocked or hazardous navigation, rapid flooding of riverfront property, and interference with operation of boat docks. The impacts of pool loss to upstream areas include loss of water supply to communities, bank instability, loss of power-generating capability for projects with hydropower, and severe and adverse impacts on fish and wildlife. The capability to stop flow through failed spillway or lock gates would reduce these impacts.

Not every project owned by the Corps of Engineers needs emergency closure capability. Indeed, some Districts have concluded that loss of pool is an acceptable option. Other projects already have the capability for
emergency closure. For many reservoir dams, the gates control only the amount of water in the reservoir, and the most substantial portion of the reservoir would be retained even if flow over the spillway were no longer controlled. Some navigation projects on the Mississippi and Ohio Rivers have emergency bulkheads that are stored at the project and are placed with a crane mounted on the project's service bridge or have auxiliary lift gates that can be used for emergency closures.

Performing rapid emergency closures at a catastrophic gate failure is problematic. To perform repairs to lock and dam gates in a timely manner, methods for providing temporary closure must be developed that can be deployed under conditions of flow. Most projects managed and maintained by the Corps of Engineers have bulkheads or stoplogs that can be used to provide a temporary damming surface during maintenance activities. The majority of these bulkheads are fabricated steel structures that slide into slots within the concrete structures and are lifted into place by cranes. Figure 1 shows maintenance bulkheads installed at the downstream end of Greenup lock on the Ohio River (note the floating plant in the background required for placement of the bulkheads). Many bulkheads must be placed under conditions of static head, but a few projects have emergency bulkheads or stoplogs that have been designed for placement under conditions of flow. There is a need to develop alternate methods for damming gate bays, when a catastrophic event occurs and the placement of bulkheads or stoplogs must be performed when flow is present.

**Recent emergency closures**

In recent years, a number of Corps and other projects have experienced lock or spillway gate failures that have required emergency closures of gate bays or lock chambers. Causes of these incidents have included

- Tow or barge accidents
- Improper operational procedures
- Lack of proper maintenance
- Generally aging infrastructure that has exceeded its design life

Other potential causes of gate failures include

- Terrorist attack
- Overload from natural events
Causes of tow or barge accidents may be varied but high water and strong current are common contributing factors.

In January 2005, three of nine loaded barges that broke loose from a twelve barge tow slammed into spillway piers at Belleville Lock and Dam. The spillway gates were rendered inoperable as a result of interference from the barges. Consequently, a loss of pool occurred halting river traffic on a 42-mile stretch of the Ohio River from Belleville to Willow Island Lock and Dam above Parkersburg, WV. Figure 2 shows the barges at the spillway of Bellville Dam after the accident (Hite 2008).

Also in January 2005, barges broke loose from a tow exiting the lock at Montgomery Lock and Dam on the Ohio River. Three barges went over the dam, and three were caught in the dam at Gate 3 and sank. The towboat also struck the dam, went over one of the spillway bays stern first, and partially sunk. Four fatalities occurred. Figure 3 shows the sunken tugboat at the dam after the accident.
Figure 2. Barges in spillway at Bellville Dam.

Figure 3. Sunken towboat after accident at Montgomery Lock and Dam.
In July 1995, the spillway tainter Gate No. 3 at Folsom Dam failed, resulting in a sustained release of water into the Lower American River. A forensic investigation concluded that a bracing member between strut arms was the initial failure point. It was determined through analysis that high friction in the trunnion would result in overstressing the bracing member. Further corroboration of this mode of failure was provided by friction tests on the trunnion of the failed gate. Figure 4 shows the failed tainter gate at Folsom Dam.

In late December 2000, there was a failure of the downstream miter gates in the main lock at Mel Price Locks and Dam on the Mississippi River at Alton, IL. During an attempt to pass ice through the lock chamber, the miter gates were severely damaged when one of the leaves was forced past the miter position. At the initiation of the ice passing operation, the gates were placed in the open position. However, one leaf of the gate was not fully open and not completely in the lock wall recess. As the ice was passed into the chamber, the flow forced the gate to swing into the chamber and ultimately past the miter position. Extensive damage was done to the Illinois side gate leaf. There were many contributing causes to the accident, including not positioning the gates in the fully open position and modifications made to the hydraulic system which operates the gates. The damaged gates are shown in Figure 5. Note that the view is looking upstream and the gate leaf is well past the miter position.

Figure 4. Failed tainter gate at Folsom Dam.
There have been other accidents where miter gates have been forced past the miter position causing damage to the gates. For example, in October 2004, the downstream miter gate in the auxiliary lock at Melvin Price Locks and Dam were forced past miter when the leaves failed to miter properly, resulting in extensive damage to both leaves. Recently, a miter gate failure was averted at The Dalles Lock and Dam on the Columbia River. The lock was closed when there were indications that the gates were not mitering properly, and previously installed structural instrumentation indicated significant changes in the behavior of the gates. Subsequent inspection of the gate revealed extensive cracking at the bottom ribs and quoin post on both gates as well as sheared bolts at one of the pintles.

In January 2010, one of the anchor arms that secures the gudgeon pin at the top of the miter gate at Greenup Locks and Dam on the Ohio River fractured. The miter end of the leaf dropped approximately 18 in. with the bottom girder resting on the lock floor. Although the displacement of the gate leaf is difficult to see in Figure 6, the fractured anchor arm is shown in the inset.
The incidents cited above are just a sampling of recent accidents and failures that have occurred at lock and spillway gates. In most cases, these incidents have been unanticipated and were rapidly occurring events. In the case of the lock gate incidents, navigation was impacted to varying degrees depending on the existence and size of an auxiliary lock and which lock was closed. In many of these cases, locks were closed for extended periods with some closures lasting more than 2 months.

Impacts to navigation resulting from lock closures can be severe in cases where there no auxiliary lock at the dam such, as The Dalles Lock and Dam and others on the Columbia and Snake Rivers. A lock closure there can severely impact the movement of agricultural products to market and vital products such as fuel to inland ports. Even at dams with main and auxiliary locks, the impact to navigation resulting from closure of the main lock can still be significant. Many auxiliary lock chambers are half the length of the main lock chambers (600 ft vs. 1200 ft), and barge tows have to be broken down to pass through the lock. The process of breaking down and remaking the tow adds significantly to the time required to transit a lock.

With spillway gates, the impacts of a loss of a gate(s) combined with the inability to stop flow in a timely manner can be significant. The lack of an emergency closure system that can be placed under flow conditions does
not allow for many, if any, options for maintaining pool and initiating repairs in a timely manner. While the tow/barge accidents cited above occurred during high flow conditions when the spillway gates were open and did not result in unintended flow through the spillway gate bays, the Folsom Dam tainter gate failure did. Nearly 40% of the water in Folsom Lake was passed into the Lower American River. The initial discharge rate was estimated to be 40,000 cu ft/sec.
2 Current Practice for Maintenance Closures

Common maintenance closure systems

Typical lock and dam projects operated by the Corps have maintenance bulkheads for dewatering lock chambers and spillway gate bays. These bulkheads are typically placed in the slots by an overhead crane on the service bridge or floating plant crane. Maintenance closures normally performed under hydrostatic conditions. This requires that gates be closed for the installation of maintenance bulkheads, which precludes their usage in emergencies where gates are damaged and cannot be closed. (For locks, both upstream and downstream gates would have to be open to preclude placement of maintenance bulkheads.) A few projects have emergency bulkheads that are designed to be placed under flow conditions. Other projects have auxiliary upstream lift gates in addition to the upstream miter gate. These provide a degree of redundancy for closing the upstream end of the lock and can facilitate the passing of ice through the lock chamber.

Other maintenance closure systems include Poiree dams and needle dams, which are schematically shown in Figure 7 along with bulkheads and lift gates. Poiree dams and needle dams are generally not recommended because of the need to use divers to install them and the length of time required for the installation. Dewatering boxes are suitable only for wicket dams (USACE 1995b).

For locks, the most common type of maintenance closure consists of stacked bulkheads installed by a floating crane. However, allowable downtimes, traffic, and economics may dictate the use of a more rapid system (USACE 2006). In addition to the traditional maintenance closure systems listed in EM 1110-2-2607, some innovative float-in structures have been developed and used on Corps projects for maintenance closures.

The float-in type closure structures seem to be viable for adaptation to emergency closure systems that may be placed under flow conditions. One of the objectives of this investigation was to identify innovative closure systems that were already in use at existing projects. A literature search was conducted for float-in bulkheads that could be placed under flow. Although the search did not reveal any systems that could be placed in
flow conditions, the variety of floating bulkheads that have been used to date is useful in determining if a system could be adapted for conditions in which flow was present.

Figure 7. Maintenance closure systems.
Floating bulkheads

The result of the searches indicated that the United States has more applications of floating bulkheads in use than the rest of the world. The searches performed were in English, which may have had an impact on the number of applications that were identified in other parts of the world. The search also showed that the Corps of Engineers use floating bulkheads for a number of locations and may have more experience with floating bulkheads than any other federal agency or private entity.

The Corps of Engineers' Portland, Walla Walla, Tulsa, and Nashville Districts have used floating bulkheads for a number of years. The Walla Walla District has had a floating bulkhead at its Ice Harbor project since the early 1960s, and the Portland District has had one at the John Day project since the late 1960s. Other floating bulkheads were constructed in these Districts in the 1970s. The Tulsa District built floating bulkheads as early as the 1950s and 1960s and, more recently, constructed two different sets of floating bulkheads for two different projects. The Nashville District has a floating caisson that is used as a bulkhead at lock projects and is constructing a second one as part of the Kentucky Lock project.

The bulkheads designed and built for the Portland and Walla Walla Districts are large, single unit bulkheads that are stored in the water. In addition, each of these bulkheads was constructed for a single project, although they can and have been used at other projects. The floating bulkhead at the Ice Harbor project in the Walla Walla District is 24 ft high, 10 ft wide, and 88 ft long. A cross section of the bulkhead is shown as Figure 8. This particular bulkhead fits into slots in the lock walls so that the bulkhead can be rotated into place. One slot is a rectangular slot, similar to most bulkhead slots. The other is tapered. Figure 9 shows a plan view of the slots used for the bulkhead. One end of the bulkhead is placed in the rectangular slot at an angle, and then the bulkhead is rotated into the other slot for final positioning and ballasting.

In addition to the floating bulkhead at the Ice Harbor project, the Walla Walla District has floating bulkheads for the Lower Monumental, Little Goose, and McNary Dams. The floating bulkhead for the Lower Monumental Dam was built in 1971 and was then used for the Little Goose Dam in 1975. This bulkhead is 70 ft long and 25 ft high. Two nearly identical floating bulkheads were built for the McNary Dam in 1975. These bulkheads were over 65 ft long and 30 ft high.
Figure 8. Cross section of the floating bulkhead for the Ice Harbor project (Walla Walla District).
The floating bulkhead for closure of the lock chamber at the John Day project in the Portland District is approximately 30 ft high, 14 ft wide, and 100 ft long. An elevation view of this bulkhead is seen in Figure 10. This bulkhead is positioned in the same manner as the bulkhead at the Ice Harbor project, that is, it is floated into position, one end is put into a slot, and then the bulkhead is rotated into its final position.

The floating bulkheads at these projects are valuable because they can be floated into position and then ballasted to their final position without requiring heavy lifting cranes. Push boats are used to move the bulkheads into position prior to the ballasting process. One of the disadvantages to the bulkheads used in these Districts is that they are stored at the project and remain in the water during storage, therefore subject to corrosion problems.
Within the last 10 years, the Tulsa District has constructed two sets of floating bulkheads. In the late 1990s, the Tulsa District developed a patented floating bulkhead system that consisted of individual components for maintenance closure of the tainter gate bays at the Broken Bow Lake Project. Each component is at least 6 ft high, 2.5 ft wide, and 50 ft long. There are four components, and when they are tied together, they form a bulkhead with a height of over 21 ft. The bulkhead lies flat during assembly, as shown by the picture in Figure 11. The floating bulkhead is then pushed to into a position against the piers that are on either side of the gate bay to be dewatered. At this time, ballasting of the bulkhead begins so that the bottom of the bulkhead sinks, and the system tips into place. Figure 12 shows the floating bulkhead near its final position against the dam piers and as the ballasting process is nearing completion. It can also be seen that this particular floating bulkhead can be moved into place with a pair of 16-ft-long fishing boats. When the dewatering process has been completed, the process is reversed. The fact that the floating bulkhead is constructed using separate components allows the bulkhead to be broken down when it is not in use and the components can be stored on land.

Another floating bulkhead designed and built by the Tulsa District was one that was constructed specifically for repairs of the tainter gates at Keystone Lake. This floating bulkhead (shown in Figure 13) is 7 ft wide, 24 ft high, and 48 ft long. The component-system floating bulkhead could not be used at the Keystone Lake project, because a specific geometry was needed to ensure that the bulkhead was watertight when it was placed in the final position.
Figure 11. Floating bulkhead system after assembly (Tulsa District).

Figure 12. Floating bulkhead system being positioned against faces of dam piers (Tulsa District).
In addition to the floating bulkheads that Tulsa District has at the Keystone Lake and Broken Bow Lake projects, there are several other projects within the District with floating bulkheads. The Hugo Lake, Fort Gibson Lake, and Eufala Lake projects all have floating bulkheads that were built for maintenance closures of the tainter gate bays at the time the projects were constructed. All are single unit, floating bulkheads and were built in the 1950s and 1960s.

The Nashville District has a floating caisson that is used to dewater lock chambers and resembles a barge. The floating caisson is over 27 ft high, 10 ft deep, and 133 ft wide. The 27-ft height includes panels that are removable. The floating caisson is used at several projects and is transported in the horizontal position. After arriving at a project, it is partially ballasted so that it rotates into the vertical position, which is similar to the sectional bulkhead used by the Tulsa District. Figure 14 shows a picture of the floating caisson as the process of rotating the caisson from horizontal to vertical begins.

Once the caisson is in the vertical position, it is pushed into slots in the lock wall, and more ballast is added so that it sinks and forms the closure. Figure 15 shows the floating caisson being placed in its final position within the slots. The differential head on the caisson provides the final seal.
Figure 14. Floating caisson as it is being ballasted from horizontal to vertical position (Nashville District).

Figure 15. Floating caisson being placed into final position (Nashville District).
to the caisson. The floating caisson contains a series of ballasting tanks
(some of these are used to correct the trim of the caisson when it is in the
horizontal position) and has 128,000 lb of concrete ballast attached to the
base of the caisson (vertical position). The floating caisson has worked
well, and another one is being constructed as part of the Kentucky Lock
project. Therefore, future lock closures can be accomplished with a
floating caisson at each end of the lock.

In addition to the floating bulkheads in the Portland, Walla Walla, and
Tulsa Districts, the literature review indicates that the Corps of Engineers
has considered the use of floating bulkheads elsewhere. For construction
of flow deflectors at the spillway of the Bonneville Dam on the Columbia
River, one consideration was the construction of a floating bulkhead
system. It was determined that the construction of a floating bulkhead
would be too expensive and too time consuming. Consideration was also
given to modifying a floating bulkhead used on the Columbia and Snake
Rivers, but this was not pursued because of the extent of revisions that
would be necessary and because of the level of corrosion on the existing
bulkheads (USACE 2001).

Official Corps of Engineers guidance also recognizes the application of
floating bulkheads as a mechanism for performing maintenance closures
(i.e., under static, equal head conditions). Engineer Manual 1110-2-2602
(USACE 1995a) lists floating bulkheads as a means to accomplish mainte­
nance closures. In addition, EM 1110-2-2607 (USACE 1995b) devotes an
entire subsection in the manual to the discussion of floating closure
structures and notes that while floating bulkheads have not been used
extensively, they have performed well and are a feasible option.

The Bureau of Reclamation has also utilized a floating bulkhead. Park Dam,
located in western Arizona, has a large, single-unit floating bulkhead that
was designed and constructed in the 1940s. The bulkhead is stored in a dry
dock at the reservoir. When it is used for the dewatering of a dam gate bay,
it is towed in a horizontal position from the dry dock to an area that is
upstream of the spillway. In the upright position, the bulkhead has two
chambers, one each at the top and bottom of the bulkhead. Once the bulk­
head is near the spillway water is allowed to enter the bottom chamber,
which causes the bottom of the bulkhead to sink, resulting in the bulkhead
being in the upright position. Once upright, more water may be added to the
bottom chamber to get the bulkhead to float at the required depth, and then
it is moved into position to allow dewatering of the gate bay. The bulkhead includes seals that bear against the dam piers on the sides and the spillway on the bottom. After completion of the dewatering, the gate bay is rewatered. An onboard pump is then used to remove water from the bottom chamber, allowing the bulkhead to return to a horizontal position so that it may be returned to storage in the dry dock.

A consulting firm based in Eau Claire, WI, Ayres Associates, has designed several floating bulkheads. Several power companies have used the design developed by Ayres Associates for the purpose of gate rehabilitation and maintenance at hydropower dams (Lux and Bakken 1992). The floating bulkhead is ballasted into place in a manner similar to that of a segmented, overhead garage door closing. Figures 16 and 17 are a schematic representation of the bulkhead emplacement operation. Individual segments of the bulkhead are placed into the water and pinned together to form the bulkhead. The segments closest to the dam piers are then ballasted and sunk into position. Modules are ballasted in sequence until the bulkhead is in place. Recovery is in reverse order with the ballast water expelled from the segments with compressed air.

The Ayres' floating bulkhead provides advantages such as being adaptable to several different configurations. It can be transported by truck and large cranes are not required for installation. Also, bulkhead slots are not required. However, a vertical sealing plane is required, and therefore adding, a beam on the pier nose is often needed. A 6-in.-wide beam embedded at the nose of the pier has proved to be effective to accomplish the seal needed. The Ayres' floating bulkhead also provides advantages over traditional dewatering methods, such as drawing down the reservoir or constructing a temporary cofferdam.

Another company in the United States that has been involved in the area of floating bulkheads is Steel-Fab, Inc., of Fitchburg, MA. The company's Web site (www.steel-fab-inc.com) identifies floating bulkheads as one of the types of water control gates that they have fabricated. These bulkheads appear to be very similar in concept to the ones developed by Ayres Associates.
Figure 16. Bulkhead floats into position prior to placement.

Figure 17. Bulkhead placement (a) with ballasted sections and bulkhead (b) in place.
This is the extent of the information available on floating bulkheads within the United States. If there are other floating bulkheads that exist within the United States, they were not identified, because there was insufficient information to document their existence.

**Floating bulkheads outside the United States**

The search for projects where floating bulkheads have been used outside of the United States resulted in fewer applications identified than within the United States. The information that can be presented on the projects outside of the United States is very limited, because contact with individuals who have first-hand knowledge of the projects was not possible.

A story on the Web site for the magazine *International Water Power and Dam Construction* ([www.waterpowermagazine.com](http://www.waterpowermagazine.com)) discussed a project that was built on the Caroni River in Venezuela. The Guri hydropower dam was built in phases over a 23-year period to minimize the initial investment and to match the power supplied with the growing electrical demand. During one of the phases in which the height of the dam was being raised, a spillway crest was raised by using three floating bulkheads to close off flow to that spillway. Unfortunately, no details are provided regarding the size or configuration of the floating bulkheads.

A set of floating stoplogs was identified as part of the project for the Freudenau Hydropower plant near Vienna, Austria. As was the case with the bulkheads used for the Guri hydropower dam, there is no additional information about the size or configuration of the floating stoplogs.

Another project that noted the use of floating bulkheads was the Carraizo Dam in Puerto Rico. In 1999, a new, hinged floating bulkhead was designed for the project. Steel-Fab, Inc., performed the fabrication of this floating bulkhead. Details about the size of the floating bulkhead for the Carraizo Dam were not available, but a photo of the bulkhead is shown in Figure 18.
Figure 18. Floating bulkhead at Carraizo Dam in Puerto Rico.
3 Emergency Closure Systems

As discussed in Chapter 1, emergency closures at navigation projects are commonly required as the result of barges that break loose from a towboat and collide with a gate or as the result of a sudden, unexpected failure of a major gate component that renders gates inoperable. Operational malfunctions or accidents have also resulted in the need for emergency closures, and since September 11, 2001, the possibility of terrorist acts has been a concern.

Some projects incorporate emergency closure systems that can be placed under flow conditions. The decision to incorporate such systems has been made on a project-by-project basis. In general, the benefits of an emergency closure system come from the ability to rapidly stop flow, ultimately providing for earlier initiation and completion of repairs. The following conditions must be considered and evaluated in determining whether or not an emergency closure structure is warranted:

- Likelihood and consequences of loss of pool (such as effects on water intake and outfall structures and docks and towing industry losses)
- Economic losses to shipping interests due to halt of river traffic
- Possible flood damage and danger to people downstream
- Adverse effects on stability of channel banks (shoreline) due to sudden drawdown
- Cost of the emergency closure system

All elements of the emergency closure system, including the handling equipment and machinery and the structure itself, should be ready for use 24 hr a day. Proper maintenance of all elements is necessary, along with periodic practice installation of the closure. Reliability and a fast installation time are required (USACE 1995a; 1995b).

Existing emergency closure systems

Some navigation projects have bulkheads that can be used for emergency closure. Some lock and dam projects on the Ohio River and the Melvin Price Lock and Dam on the Mississippi River have stacked bulkhead arrangements that can be placed in flowing conditions in the case of an emergency (USACE 1995a). These projects also have a large crane on the
project's service bridge that can place these bulkheads. However, the number of projects where this system is available is limited. Figure 19 shows emergency bulkheads for Melvin Price Locks and Dam (note the roller ends to facilitate placement under flow).

The stacked bulkheads used for emergency closure can be used in flowing conditions primarily because of their weight. When lowering a bulkhead during an emergency condition (i.e., with flow present) hydrodynamic forces will be imparted that may cause uplift or downpull, depending on the design (USACE 1995a; 2006). Obviously, it is critical that during lowering that the weight of the bulkhead must be greater than any uplift forces. Conversely, having net resultant forces that would cause the bulkhead to be pulled downward quickly would also be undesirable because of damages that might occur from the impact of the bulkhead striking the sill.

Another system for emergency closures is to provide vertical lift gates at the upstream end of the lock. Figure 20 shows the gate slots and the towers housing the lift mechanism for the lift gate (gate is in the lowered
position) upstream of the open upstream miter gate (gate leaves in the gate recess) at Greenup Locks and Dam on the Ohio River. The lift gate can be used for emergency closure to stop flow and for dewatering.

Figure 20. Lift and miter gates at Greenup Locks and Dam (Huntington District).

Another system is wicket gates although they are not widely used for emergency closures. Wicket gates are most often used as dam gates but at Lock No. 2 on the Lower Monongahela River in the Pittsburgh District wicket gates are used as bulkheads (Figure 21).

A concept closely related to wicket gates are inflatable dams. Primarily used as dam gates, these structures are similar to wicket gates except that they are raised or lowered with air-inflated bladders to control the height of the spillway.

Obviously, each of these systems must be incorporated into the project during planning and design, particularly, the emergency lift gates and wicket gates, because they require significant supporting infrastructure.
Objectives and challenges for emergency closure systems

In addition to the ability to be emplaced under flow conditions and effectively stop flow, emergency closure systems need to meet several other criteria. At a navigation research workshop held in 2006, the following criteria and concerns were identified (Hite et al. 2006):

- Should not be too expensive
- Should be capable of being deployed quickly
- Should be capable of being removed quickly
- Should be readily available
- Should be transportable
- Should be adjustable
- Should not require major modifications for installation of anchor points

In addition, accommodation of staging and storage requirements warrant significant consideration. Although an emergency closure system may not see significant use, maintenance requirements would also be a concern.

The time to emplace an emergency closure system is of primary concern. The system should be designed and arranged to minimize time required to
install the closure. Obviously, in order to be effective, an emergency closure system must have the capability to be emplaced in less time than it would take to draw down the upper pool. The Corps' Louisville District (USACE 2004) published a report summarizing the "LRD Navigation Risk and Recovery Study," which was undertaken to assess vulnerabilities of navigation infrastructure to possible terrorist attacks. The report identified time estimates for loss of navigation pool under uncontrolled flow conditions for selected projects. These times were based on loss of lock gates or one or two spillway gates. The following times were reported:

- Markland Locks and Dam: 18 hr
- Emsworth Locks and Dam: 24 to 48 hr
- Barkley Lock and Dam: 6 to 7 days at low flow

Obviously, the more rapidly an emergency closure system can be installed, the less likely significant consequences will result from drawdown of the upstream pool and the less severe flooding downstream will be. Also, rapid closure allows repairs to be initiated sooner. The above times provide a benchmark for a closure system. Obviously, an effective closure system must be capable of stopping uncontrolled flow in considerably less time than it would take to lose pool.

Some of the other criteria may have trade-offs. For example, a system that is inexpensive and has minimal storage and staging requirements could be practical to have one at each project site thereby offsetting the requirement that it be transportable between projects. Conversely, a system that could be rapidly transported and emplaced on site could offset higher initial costs by having one system that could serve several projects.

The problem of having suitable anchor or reaction points is challenging for emergency closures, particularly in cases where there is extensive damage such as from an explosive attack. Maintenance and emergency bulkheads are placed in slots in the lock walls. These bulkhead slots provide a reaction point for the large horizontal forces resulting from head differential on the bulkhead. Bulkhead slots are typically close (relative to a 1200-ft-long lock chamber) to the lock gates at either end of a lock and could potentially be damaged or blocked with debris from an explosive attack.

The hydrodynamic forces must be fully considered and accounted for in an emergency closure system. For float-in systems, a challenge is that it must
be light enough that it can be floated to a location and heavy enough to overcome hydrodynamic forces that may be present. In addition to the hydraulic forces and structural considerations, the need to develop a methodology for getting the closure system into position is a major engineering problem. Stability of the system and the ability to control and guide it in rapidly flowing water in an emergency situation are significant challenges.

**Preliminary concepts for emergency type closures**

The following sections discuss some possible solutions to the problem of placing a closure structure under emergency conditions. The solutions presented are concepts that will need further investigation prior to making a determination as to whether or not they are truly feasible solutions. In addition, prior to implementing any system, a better definition of the hydrodynamic forces associated with emergency closures will be needed.

During a workshop held at the ERDC, a brainstorming session on emergency closure systems was held (Hite et al. 2006). Listed below are the ideas and concepts that were introduced.

- Empty barges that can be filled with water or sand and can be sunk
- Transportable, stackable bulkheads
- Stockpile rock in accessible area and dump to close off flow
- Emergency stoplogs with cradle system - Whitten Lock in the USACE, Mobile District has a cradle that will lower five stoplogs
- Tension structure
- Inflatable dam
- Louvered bulkheads
- Wicket type structure with hurter box
- Shuttered A-frame and top beam system (the McAlpine 600-ft lock had a system like this)
- Floating caisson

From the list (shown above), the following concepts were identified by Hite et al. (2006) as viable options:

- Loose stone dumped into the flowing water
- Frames lowered into the flow and filled with a device of some sort to shut off the flow
- Inflatable bladders
- A floating platform sunk in a controlled manner
- Tension gate
- Bulkhead units with adaptable ends

Hite (2008) recommended the bulkhead concept as the most functional for an inland navigation project. Some of the other concepts, for example loose stone, have some obvious drawbacks. In particular, stockpiling and removal of material and the time required to deploy.

**Bulkhead type closures**

One of the first concepts developed in this study is a modified bulkhead concept. Bulkheads have a long history of use as maintenance closures and have been used as emergency closures. One design is to incorporate flow through baffles which would help to balance the hydrodynamic forces on the bulkhead to facilitate placement. The baffles would be closed off once the bulkheads are in place. It is envisioned that these bulkheads would be placed by crane and lowered in the bulkhead slots. Figure 22 depicts placement of the baffled bulkhead system.

![Figure 22. Bulkheads with baffles: (a) placement of first bulkhead, (b) bulkhead stack with baffles open, and (c) bulkhead stack with baffles closed.](image-url)
Another concept for a bulkhead stack is to incorporate the three hinge arch concept of a miter gate to gain some structural efficiency. In this case, it was envisioned that the three hinge bulkhead sections could be floated in, and the ends of the bulkheads would be extended into the bulkhead slots by means of an actuating strut that would increase the angle of the miter. This concept is illustrated in Figure 23.

Another bulkhead concept that was reported by Hite is the tension gate as shown in Figure 24 (Hite 2008). The major advantage of the tension gate is that the structural system is primarily in tension and thus more efficient. However, placement of this system in flowing water is likely to be problematic, and as shown, anchorage of the gate would have to be incorporated into the lock walls.

**Frame type closures**

An emergency closure system that incorporates a framed grid structure with a flexible damming surface was considered for both lock chamber and spillway gate bay emergency closures. It is envisioned that the frame would be a float-in system. For spillway gate closures, the frame would bear against the spillway gate piers. For lock chambers, an anchoring
system would need to be developed. As shown in Figure 25, a possible emplacement scenario for a lock chamber is to float the frame in the horizontal position. The trailing end of the frame is then ballasted until it comes to rest on the lock chamber floor. The top of the frame is anchored to appurtenances added to the top of the lock wall or is stayed with a cable system. Once in place, the flexible or articulated damming surface is emplaced over the supporting grid to stop flow. In this scenario, the flexible damming surface is stored on a roller at the bottom of the frame. Once the frame is in place, the damming surface unrolls from the roller as the roller travels up the frame. The roller is controlled with a gear and/or cable system. Preliminary structural member sizing for such a structure is provided in Appendix B. A small-scale test of this concept was conducted to illustrate the concept and test feasibility. Frame captures of the test are shown in Figure 26.
Figure 25. Emplacement of framed closure structure with flexible damming surface.

Figure 26. Small scale test of float-in frame closure system.
It is also possible that this frame concept can be readily applied to spillway gate bays. Emplacement would be similar in that the frame would be floated horizontally into position in front of the gate bay. The trailing edge would then be ballasted so that it would sink and the frame would assume a vertical position. The damming surface would then be positioned over the grid.

A similar concept to the above frame is to have a self-supporting A-type frame where the top member of the grid would be supported by inclined members that transferred the load down to the lock floor, significantly reducing the requirements for supporting the top member of the grid with cables or other appurtenances. This frame could be floated into the chamber and ballasted so that it would rest on the lock floor.

Flexible fabric structures

Another class of emergency closure structures that were conceptualized is a flexible fabric structure, which would be water filled. This concept was initially conceived based on work by Resio et al. (2009). This work demonstrated that water-filled fabric structures can be used to close breaches in levees. These water-filled structures are based on the concept that the structure resists applied forces through invariance of internal volume due to the relative incompressibility of water. For emergency closure structures for locks or spillway gate bays, various configurations were considered. A lock gate concept is illustrated in Figure 27. This concept consists of a cylindrical shaped fabric structure with a buoyant torus attached to the top of the cylinder. The cylinder and torus both have a diameter larger than the lock chamber width. The concept for emplacing begins with the cylinder evacuated and stored below the buoyant torus so that the system can be floated into place at the entrance to the lock chamber. Once in position, water is pumped into the cylinder forming a plug.

A similar concept is to couple the circular plug with a float-in caisson (Figure 28), so that the cylindrical plug could be considerably smaller in diameter. These structures could be made buoyant and floated into the lock chamber. Once in the chamber, they could be sunk into position. Using two or more of these structures, the flow through the lock channel would be cut off by overlapping the structures.
The concept of operations of emplacement of the structure is as follows: A tugboat would lash onto the first structure and float it into the lock chamber. The structure would be guided to a bumper system and secured to the lock wall. The structure would then be ballasted so that it was resting on the lock floor. The tugboat would then unlash itself from the structure and grab the second structure. The second structure would be floated in so that it would be in contact with the lock wall and the
previously installed structure. Once positioned, the structure would also be ballasted so that it would rest on the lock floor.

A concept for a spillway gate bay was an arch shaped tube. In order to accommodate various water depths and gate bay widths, a double stacked arch tube was conceived and small-scale tests were conducted. Figure 29 shows the small-scale test specimen of the double stacked arch tube. The test setup is shown in Figure 30, where the specimen bears against model dam piers and closes the simulated gate bay from the upper pool. As part of the testing, the specimen was floated into position. These small-scale tests were successful with encouraging results. The small-scale double stacked arch tube was stable with virtually full height head differential. The test arch sealed reasonably well against the model dam piers and with some slight modifications, the seal could be improved. A variation of the arch tube concept was also conceived for use on elevated spillways. This concept employs a semicircular geometry instead of the arched tube and is shown in Figure 31.

Figure 29. Small-scale double stacked arch tube.
A cursory evaluation of the preliminary concepts is provided in Table 1. Each concept is rated -, 0, or + for the attributes listed in the table. Emergency bulkheads are used as the baseline for comparison and are rated 0 in each category. A rating of - indicates that for that attribute the
concept is deemed to provide less of a benefit compared to the baseline bulkheads. For a rating of +, the concept is believed to have the potential to provide improvements compared to emergency bulkheads. Obviously, these ratings are crude and approximate and, in some cases, may need to be revised based on further development.

Table 1. Comparison of emergency closure concepts.

<table>
<thead>
<tr>
<th></th>
<th>Cost</th>
<th>Rapid deployment and recovery</th>
<th>Transportable</th>
<th>Additional anchor points required</th>
<th>Staging and storage requirements</th>
<th>Limitations or potential difficulty of emplacement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Emergency bulkheads</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Modified bulkheads</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Fill material</td>
<td>+</td>
<td>-</td>
<td>-</td>
<td>+</td>
<td>-</td>
<td>+</td>
</tr>
<tr>
<td>Tension gate</td>
<td>+</td>
<td>+</td>
<td>0</td>
<td>-</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Frame type</td>
<td>0</td>
<td>+</td>
<td>0</td>
<td>-</td>
<td>+</td>
<td>-</td>
</tr>
<tr>
<td>Flexible fabric structure</td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>0</td>
<td>+</td>
<td>-</td>
</tr>
</tbody>
</table>
4 Summary and Conclusions

On navigation, flood control, and water resource projects, lock and spillway gates are typically the most vulnerable project components. Accidents and overloads from natural events are most likely to inflict damage on gate structures. Maintenance and fabrication quality issues have also resulted in significant damage to gate structures. Lock and/or spillway gates can also be easily be damaged or rendered inoperable by explosives detonated in a terrorist attack. Severe damage to a spillway gate will initiate drawdown of the upper pool. Loss of upstream and downstream lock gates will also result in rapid drawdown of pool from flow through the lock chamber. Depending on the type of lock gates, loss of a single gate while the other gate is open may result in loss of pool. A review of recent accidents at Corps of Engineers projects provides numerous examples of significant accidents and closures within the last 10 years, as documented in Chapter 1. The cost to the navigation industry and affected stakeholders has been significant.

The events of September 11, 2001, have elevated the awareness of terrorists attack and led to numerous government actions to prevent such attacks from occurring in the future. However, because of the random nature of such attacks, it is apparent that there is still a possibility that not every terrorist attack can be prevented; therefore, plans must be made to ensure that the impact of such an attack is limited.

Disabling a lock and dam on a navigation waterway could result in substantial economic and environmental losses. Substantial damage at a reservoir project could also have economic and environmental impacts as well as result in loss of life.

Many Corps projects have bulkheads that are used to isolate a gate bay so that it can be dewatered for the purpose of performing maintenance on the gate, but the number of projects where the bulkheads can be placed under emergency conditions (i.e., conditions of flow) is limited. Therefore, mechanisms that could be used in emergency conditions were investigated. The initial effort was to identify some of the innovative or unconventional systems that are used for maintenance closures to assess their feasibility for application to emergency closure situations. These included modifications
to existing configurations of emergency bulkheads and adaptation of float-in bulkheads.

The investigation showed that there are a number of floating bulkheads at projects in the United States, as well as around the world. Floating bulkheads have been used for maintenance closures at lock and dam projects since the 1950s, and many of the projects that have floating bulkheads are Corps projects. The research also showed that the configuration of floating bulkheads varies considerably from one project to the next. Some bulkheads are large, single units, while others consist of several smaller units that must be connected prior to being used. However, regardless of the configuration, all of the existing floating bulkheads are used for maintenance closures and are set in place under static head conditions.

Despite the fact that floating bulkheads are currently placed only under no-flow conditions, it may be possible to adapt some of the configurations for use under emergency conditions when flow will be present. The use of floating bulkheads for emergency closures will also require the development of a system for moving the bulkhead into position, which may include the use of large towboats in addition to a cable system connected to the lock or dam. Indeed, for any emergency closure system, development of a concept of operations for emplacement and recovery will be a significant challenge.

In addition to identifying concepts for floating bulkheads, there were other concepts for providing emergency closure of gate bays. These included development of a few concepts for water-filled fabric structures or a combination of such structures coupled with a rigid caisson. Of the concepts identified, the two that seem to hold the most promise are the framed grid bulkhead with flexible damming surface and the water-filled fabric arch. The framed grid bulkhead could be used for both locks and spillways.

Based on the research performed, it appears that it would be possible to develop a floating bulkhead that could be used for emergency conditions. However, it is also apparent that significantly more investigation will be needed, primarily into the loads that will be developed in placing an emergency closure structure under flow and in the development of a concept of operations for emplacement and recovery. This may be further complicated by the fact that defining these loads may be site specific with respect to the size of the gate bay and the differential head present.
Another consideration that must be taken into account before moving forward with plans for developing emergency closure systems is the extent to which the systems are needed for a given project. Other projects already have the capability for emergency closure. For many reservoir dams, the gates control only the amount of water in the reservoir that is released over the spillway, and the most substantial portion of the reservoir would be retained even if flow over the spillway were no longer controlled. For navigation projects, there are projects on the Mississippi and Ohio Rivers that already have emergency bulkheads that are stored at the project and are placed with a crane mounted on the project’s service bridge.
References

Berger, R. C. 1997. HIVEL2D v2.0 users manual. Vicksburg, MS: U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory.


Hite, J. E., Jr. 2008. Concept design for emergency closure system for inland navigation structures. ERDC/CHL CHETN-IV-70. Vicksburg, MS: U.S. Army Engineer Research and Development Center.


_____. 2004. LRD navigation risk and recovery study. Louisville, KY: Great Lakes and Ohio River Division.

Appendix A: Hydraulic Forces on Bulkheads

A preliminary analysis was conducted to determine hydraulic forces on emergency lock bulkheads as they are placed to stop flow through a lock chamber. Two scenarios were considered: (a) compute the forces on an articulated bulkhead that is floated into place, and (b) compute the forces on single bulkheads as they are lowered into place by a crane.

One of the two cases will be addressed by a three-dimensional (3-D) numerical analysis of the lock chamber flows in which force computations are examined in the two-dimensional (2-D) longitudinal and vertical plane. This will allow the determination of changing fluid forces that will occur as a result of the accelerating fluid through the diminishing opening as more bulkheads are lowered into place.

Study results are presented in two sections, the first using analytic techniques and the second by way of numerical modeling. Each part contains a discussion of the problem statement, methodology, and results. These two parts are followed by a summary of both methodologies.

Analytical computations

Discussion of the problem statement

The task is to determine the maximum expected forces applied to emergency lock bulkheads after an attack on a lock and dam. The scenario considered is that of a breached lock gate(s). No further details are given. So to assure a worst-case condition, computations are made as though both sets of lock gates (i.e., upper and lower miter gates) are simultaneously removed. This implies a situation that is similar to a dam failure in that a large mass of water is suddenly released from a high upstream pool to a low downstream pool. In this case, the connection between the high and low water levels is the lock chamber. A free-fall of water at either end of the lock chamber, if it were to occur, is not considered. What is considered is the moving wave and resulting water surface slope between the upstream and downstream pools. Without any bulkheads in place, the greatest slope, and thus the highest fluid accelerations and velocities, should occur during the greatest difference in pool elevations. This appeared to be a logical starting point as a worst-case flow condition into which to place the articulated or
single bulkheads. The purpose of this study was not to design the bulkheads but simply to identify and quantify the greatest expected force that might occur given the stated gate removal condition. As just stated, this maximum force could be expected at the time of greatest pool elevation difference when accelerations have reached their maximum, and high velocities are sustained. The high velocities would be sustained as long as the acceleration of the fluid remains at zero after having reached the maximum value. It is beyond the scope of this study to determine the duration of any of the forces. The discussion thus far considered the flow-induced forces without any bulkheads in place. This leads to the question of what happens when bulkheads are placed into the flow. Again, a worst-case condition is sought to determine maximum forces. One might think that the greatest forces on any bulkhead would occur with just one bulkhead in place, because that is when the highest velocities and thus the greatest dynamic forces will be encountered. While this is true with respect only to dynamic forces, experience shows that hydrostatic forces in deep water are extremely large and probably several times larger than the dynamic forces. To make sure that this was the case, several conditions were tried.

**Methodology**

The problem was approached in two stages. The first stage was to determine the hydraulic reaction to the gate removal. The second was to apply standard fluid dynamic principles in computing the force on the inserted bulkheads.

Determination of the maximum flows and velocities was accomplished using the 2-D numerical model, HIVEL2D. HIVEL2D is a depth-averaged model capable of simulating sub- and super-critical flow and the transitions between these regimes (see Berger 1997). The 2-D model provided the flow field approaching the open lock chamber, flow through the chamber, and flow in the tailrace. The HIVEL2D assumes hydrostatic pressure distribution and therefore is not accurate in the immediate vicinity of the bulkheads, but it did provide the drawdown within the chamber as the flow accelerates through the abrupt contraction that is formed between the upper pool and the chamber. The model also computed the additional drawdown as the flow in the chamber exits into the lower pool, which is essentially an abrupt expansion.

Rather than producing a hypothetical case of flow through a lock, the conditions at the Greenup Locks and Dam were simulated. This project is
located on the Ohio River, 341 miles below Pittsburgh, PA, and 5 miles below Greenup, KY. The dimensions of the main lock are nominally 1200 ft long by 110 ft wide. The design lift is 30 ft with an upper pool elevation of 515 ft and a lower pool elevation of 485 ft NGVD. Note that these elevations are for the upstream and downstream water surface elevations of the HIVEL2D model. They are not the water surface elevations in the vicinity of the lock chamber. Figure A1 shows a plan view of the main lock and dam with a portion of the upper and lower pool, and the water surface elevations resulting from the breaching of both sets of lock miter gates. Also shown in Figure A1 is a shortened version of the 2-D computational mesh. The entire mesh reproduced about 3700 ft of the upper pool and about 7600 ft of the lower pool. The upper guard wall was simulated using a pressure field between the cells representing the wall’s depth of penetration.

Figure A1. A portion of the HIVEL2D mesh showing computed water-surface elevations, ft-NGVD.
The results of the HIVEL2D run indicated water surface elevations in the vicinity of the lock chamber as shown in Figure A2. The elevation 510.8 was used as an initial upstream value in the following analysis.

![Water Surface Profile Through Lock Chamber](image)

**Figure A2.** Water-surface profile within the chamber with two bulkheads penetrating the water surface.

The second stage was to compute the forces for various depths of bulkhead closure. The floated-in bulkhead concept can be visualized as a garage door closing from the top. These bulkheads would be filled with water just as they are dropped in the lock wall bulkhead recesses. As each bulkhead is added, the flow is successively diminished. Flow rates were calculated using the discharge equation,

$$Q = G_o C_D B \sqrt{2gy_1}$$  \hspace{1cm} (1)

where \(G_o\) is the gate opening, \(C_D\) is the discharge coefficient, \(B\) is the width of the lock chamber (110 ft), \(g\) = gravitational acceleration, and \(y_1\) is the depth of flow upstream of the bulkhead. The discharge coefficients were selected using those given for flow under a vertical lift gate (Rouse 1949). The equations and coefficients used in computing the flow reductions for various gate openings are shown in Table A1.

The discharge calculations require an iterative solution, because the flow rate is dependent on the upstream depth, and the upstream depth is dependent on the closure. As the bulkheads are deployed, the "gate" opening is reduced, which reduces the flow rate, yet the upstream water surface increases, which increases the flow. Figure A3 shows the reduction in flow through the opening as the gate is closed.
Table A1. Iterative computation of upstream head, flow and resulting forces on bulkheads.

<table>
<thead>
<tr>
<th>E1</th>
<th>Assumed h1</th>
<th>Computed h1</th>
<th>Depth b</th>
<th>%</th>
<th>Cd</th>
<th>q</th>
<th>V1</th>
<th>hv1</th>
<th>V2</th>
<th>Force</th>
<th>Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>ft</td>
<td>ft</td>
<td>ft</td>
<td>ft</td>
<td>ft</td>
<td>ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| 49.1 | 40.8        | 40.8       | 4.0     | 36.8 | 9.8 | 0.5 | 943.2 | 23.1 | 8.3 | 25.6 | 54.9 | 559.6
| 49.1 | 42.1        | 42.2       | 8.0     | 34.1 | 19.0 | 0.5 | 887.8 | 21.1 | 6.9 | 26.0 | 219.6 | 1154.9
| 49.1 | 43.4        | 43.4       | 12.0    | 31.4 | 27.6 | 0.5 | 830.0 | 19.1 | 5.7 | 26.4 | 494.2 | 1786.0
| 49.1 | 44.5        | 44.5       | 16.0    | 28.5 | 36.0 | 0.5 | 762.8 | 17.1 | 4.6 | 26.8 | 878.6 | 2441.9
| 49.1 | 45.5        | 45.5       | 20.0    | 25.5 | 44.0 | 0.5 | 690.2 | 15.2 | 3.6 | 27.1 | 1372.8 | 3121.2
| 49.1 | 46.1        | 46.2       | 24.0    | 22.1 | 52.1 | 0.5 | 626.2 | 13.6 | 2.9 | 28.3 | 1976.8 | 3646.5
| 49.1 | 47.0        | 46.9       | 28.0    | 19.0 | 59.6 | 0.5 | 554.0 | 11.8 | 2.2 | 29.2 | 2690.7 | 4288.6
| 49.1 | 47.6        | 47.6       | 32.0    | 15.6 | 67.2 | 0.6 | 475.0 | 10.0 | 1.5 | 30.5 | 3514.4 | 4865.6
| 49.1 | 48.0        | 48.1       | 36.0    | 12.0 | 75.0 | 0.6 | 387.0 | 8.1  | 1.0 | 32.2 | 4447.9 | 5415.9
| 49.1 | 48.6        | 48.6       | 40.0    | 8.6  | 82.3 | 0.6 | 288.7 | 5.9  | 0.5 | 33.6 | 5491.2 | 6150.5
| 49.1 | 48.9        | 48.9       | 44.0    | 4.9  | 90.0 | 0.6 | 165.0 | 3.4  | 0.2 | 33.7 | 6644.4 | 7057.6
| 49.1 | 49.1        | 49.1       | 48.0    | 1.1  | 97.8 | 0.6 | 37.1  | 0.8  | 0.0 | 33.7 | 7907.3 | 8008.5

Discharge through lock chamber as it is closed by adding bulkheads from the top

![Graph](image)

Figure A3. Reduction in flow rate through the lock chamber as bulkheads are deployed.

The horizontal force acting on the bulkhead was then computed. These forces were calculated in 4-ft increments of the bulkhead, as it is lowered from the top. The momentum equation provides the reaction force on the bulkhead. See Roberson and Crowe (1965) and Henderson (1966).

\[ F = \frac{1}{2} B \rho g \left(y_1^2 - y_2^2\right) + \rho BQ \left(\frac{1}{y_1} - \frac{1}{y_2}\right) \]

(2)
Here, $F$ is the horizontal force applied to the bulkhead, $B$ is the bulkhead width, $\rho$ is the density of water, $g$ is the gravitational constant, $Q$ is the flow through the chamber, $y_1$ is the water surface upstream of the bulkhead, and $y_2$ is the depth of water below the bulkhead. The bulkheads are modeled as a closing sluice gate. As can be seen in Equation 2, this approach requires knowledge of the channel and bulkhead geometry and flow conditions.

**Results**

Using the computational results for water elevation and flow, values of hydrostatic and total forces on the bulkheads were computed at various levels of submergence and are shown in Figure A4.

![Figure A4. Total horizontal force on the bulkhead system as bulkheads are deployed.](image)

Implicit in this figure are the values of the dynamic force due to the moving water. It can be determined by drawing a horizontal line between the red and blue lines for any given percent of bulkhead closure. For instance, at about 52% closure, the static force is 1,977 kips and the dynamic force is 1,669 kips (3,646 kips to 1,977 kips). This data show that the dynamic force is equal to the static force somewhere between about 45 to 50% of the closure. From that point on, the hydrostatic force dominates until at the bottom the entire force is hydrostatic. By inspection of this graph, it is clear that the hydrostatic forces at the bottom of the chamber when all bulkheads are in place will represent the greatest horizontal (downstream) force on any bulkhead. For the problem as stated in the given lock chamber, the lowest bulkhead comes to rest on the floor.
at a depth of 49.1 ft below the water surface. Thus, the computed pressures on it are 2,939 psf for a bulkhead of 4 ft in depth by 110 ft across. This loading represents the expected horizontal maximum for any bulkhead throughout the lowering process. This analysis does not take into account considerations of any impact.

In addition to the horizontal force on the bulkhead “gate,” a downward force due to the reduced pressure in the moving fluid under the bottom bulkhead must also be considered. In order to do so, the conservation of energy principle was used. This allowed energy comparisons at a section upstream and directly under the lowest floating bulkhead. The pressure under the bulkhead was treated as the unknown quantity and solved for. The results of this analysis for various bulkhead closures are tabulated in Table A2. When the bottom bulkhead is 1.1 ft from the bottom of the lock chamber, this force will be about 1,100 psf in the downward direction. For a bulkhead 110 ft long by 14 ft wide, the surface area is 1,540 sq ft. Thus, the total downward force would be about 1,694 kips. This does not include the weight of the metal of which the bulkheads would be constructed. It also assumes that the bulkheads are lowered at a constant rate. Thus, if the bulkheads were to accelerate downward, then the downward force would be even greater.

<table>
<thead>
<tr>
<th>Computed Pressure Under Bulkhead P2 (psf)</th>
<th>Hydrostatic Pressure of Filled Bulkheads (psf)</th>
<th>DP (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>131.0</td>
<td>249.7</td>
<td>118.7</td>
</tr>
<tr>
<td>277.3</td>
<td>505.1</td>
<td>227.8</td>
</tr>
<tr>
<td>427.1</td>
<td>750.1</td>
<td>323.0</td>
</tr>
<tr>
<td>590.8</td>
<td>1000.7</td>
<td>409.9</td>
</tr>
<tr>
<td>762.6</td>
<td>1249.7</td>
<td>487.1</td>
</tr>
<tr>
<td>905.9</td>
<td>1506.0</td>
<td>600.1</td>
</tr>
<tr>
<td>1054.7</td>
<td>1743.6</td>
<td>688.9</td>
</tr>
<tr>
<td>1192.1</td>
<td>1993.9</td>
<td>801.8</td>
</tr>
<tr>
<td>1307.2</td>
<td>2252.1</td>
<td>944.8</td>
</tr>
<tr>
<td>1435.5</td>
<td>2493.0</td>
<td>1057.5</td>
</tr>
<tr>
<td>1659.6</td>
<td>2747.1</td>
<td>1087.5</td>
</tr>
<tr>
<td>1892.2</td>
<td>2994.6</td>
<td>1102.4</td>
</tr>
</tbody>
</table>
**Single bulkheads**

The considerations for single bulkheads lowered by a crane are similar to the floated-in concept but with the following important differences. If the single bulkheads are constructed of an open truss behind the vertical plate, then the downward loads will be small and probably negligible. Thus, only the horizontal force needs to be considered. A conservative estimate of the force applied over the face of one bulkhead can be made using the highest expected velocity for the high flow condition. This velocity is estimated upstream of the bulkheads at 23 ft/sec. Using this number, the pressure is found to be about 1,026 psf in the horizontal direction. This is the pressure produced only by the dynamic force of the water. However, since this “gate” is built from the bottom up to the top, there will still be water on both the upstream and downstream sides of the bulkhead until such a time as the bulkheads begin to act as a weir. It would seem that up to this height, the hydrostatic forces upstream and downstream should nearly balance each other and thus most of the pressure will be due to the dynamic forces. However, this is not entirely true, since on the downstream side of the lower bulkheads, a low pressure is produced by the flow separation at that location. This low pressure reduces the hydrostatic pressure on that side of the bulkhead and, thus, will produce a net pressure gradient greater on the upstream side of the bulkhead. Our preliminary modeling indicates this difference to be about 500 to 700 psf. The greatest dynamic pressure as before will be about 1026 psf, thus producing a maximum of approximately 1,800 to 2,000 psf. As before, this is *not* the maximum pressure to be expected on any bulkhead using the single bulkhead scheme. That pressure would occur when all flow is stopped, and the downstream channel is drained, analogous to the sluice gate being closed in the floating bulkhead concept. At that point, the maximum expected pressure is, as before, hydrostatic at the bottom bulkhead. Thus, for the single bulkhead method, the maximum expected uniform loading is at the location of the center of the bottom bulkhead and is about 2,939 psf.

This does not include any acceleration or impact considerations. Unlike the floating bulkhead concept, the single bulkhead concept with an open truss behind it should not be subject to the same substantial downward force due to the lower pressure in moving fluid beneath it. That is not to say that there would be no such effect. Further analysis and possible modeling would be necessary to quantify if and how large such an effect would be on the open truss.
Numerical model computations

Discussion of the problem statement

As in Section One, the same breaching scenario and worst-case condition were considered. In this case, the single bulkhead concept was modeled but with closed trusses. That is, the internal structure of the bulkhead would be covered with a skin, like in the floating bulkhead idea. This way we could check two computations made in the analytic analysis with just one numerical run. One was the low pressure computed under the floating bulkheads. The other was the backpressure on the downstream side of the single bulkheads when only one or two bulkheads were in place.

Methodology

As in section one, the problem was approached in two stages. The first stage was to determine the hydraulic reaction to the gate removal. The second was to use a three-dimensional (3-D) Navier-Stokes model (ADH, see Berger and Stockstill 1999) (nonhydrostatic) to compute the force on the inserted bulkheads. The 2-D model results from simulating two bulkheads in place on the lock floor and one penetrating the water surface were used as boundary conditions for the 3-D model. A 3-D tetrahedral mesh was generated representing the lock chamber geometry which had an imposed water surface shape computed with the depth-averaged HIVEL2D model. See Figure A5 for a view of the 3-D mesh.

![Figure A5. Computational mesh of lock chamber with two lower and one upper bulkhead in place.](image)

The water surface was treated as a rigid lid, meaning that pressures at the surface were calculated rather than the mesh moving as a free surface. The flow rate was specified as being 31,000 cfs. The 3-D model was used to compute the vertical variations of velocity and pressure as the flow...
accelerates past the bulkheads. So, in essence, this model provided 2-D results along the longitudinal and vertical plane of the lock chamber.

Results

The pressure contours in Figure A6 illustrate that the flow is generally hydrostatic except in the immediate vicinity of the bulkheads. In this vicinity, the vertical accelerations are significant and the pressure responds accordingly. The low pressure on the upstream portion of the upper bulkhead soffit shows that downward forces will exist. An accurate estimate of the magnitude of these forces would require significantly more modeling effort such that the actual bulkhead shapes and supporting framework were considered. The nonhydrostatic model results also show the pressure difference across the two lower bulkheads. The pressure on the upper side is higher as the flow accelerates over the two bulkheads (an adverse pressure gradient). On the downstream side of the two bulkheads, flow separation occurs and produces relatively low pressures on the lea side of the two bulkheads.

Summary

Forces on bulkheads were computed. Two-dimensional, 3-D, and analytical models were used to compute the flow through an example project. The Greenup Locks and Dam were selected as the demonstration project. The conditions assumed in the calculations were that both sets of main-chamber lock miter gates were removed, steady-state flow from the upper pool of elevation 515 was passed through the open main-lock chamber, and the lower pool was set at elevation 485. These pools are project design conditions. The flow profile through the lock chamber was computed, and an elevation of 510.8 used as the elevation just upstream of the inserted bulkheads. Flow rates and associated forces on bulkheads placed in the flow were calculated. The maximum force over the entire closed "gate" was found to be about 8,008 kips. This force occurs when all the bulkheads are in place. In other words, the maximum total force is essentially that caused by the hydrostatic pressure. The force plot of Figure A6 shows that as the bulkheads are initially placed in the flow, the force is predominantly dynamic, but as the lock is closed, it becomes hydrostatic. From the last two rows on the right column of Table A1, the average pressure on the lowest bulkhead can be computed as about 2,161 psi.
Figure A6. Elevation view of pressure contours at the bulkheads.
Appendix B: Structural Sizing of Frame Type Closure

The results presented in Table B1 were determined from an analysis of the vertical members of a frame type emergency bulkhead as described in Chapter 3. The emergency bulkhead would be used to stop water flow in the event that a lock miter gate was breached.

The lock chamber was assumed to be 110 ft wide with an upper pool elevation of 40 ft from the lock floor. Analysis assumed that once the bulkhead was installed, there would be no lower pool height (lock would be drained). This would be a worst-case scenario if the frame closure was used to dewater the lock. The hydrostatic pressures on the bulkhead can be calculated from these heights. The dynamic forces were based on an assumed flow in the lock once breached.

The bulkhead was statically designed as a beam simply supported at the lock floor and at the waterline. The loading to the bulkhead was analyzed also at varying angle to the lock floor (from 0 to 30 deg). Finally, the tributary area was looked at for widths of 3, 5, and 10 ft. The lightest beam size was then selected from Table B1, which shows the results for bulkhead steel member sizes as a function of the incline angle of the frame and the spacing of the vertical members. The steel beams were assumed to have an unbraced length no greater than 10 ft. Also, a 1.6 live load factor was added in to account for dynamic loads.
Table B1. Beam selection table.

**Beam Selection Summary**

Upper pool height = 40 feet
Lower pool height = 0 feet

**Loading summary**

<table>
<thead>
<tr>
<th>Angle (degrees)</th>
<th>L1 (ft)</th>
<th>L2 (ft)</th>
<th>Lmax (ft)</th>
<th>max load (lb)</th>
<th>Rc (kip)</th>
<th>Rt (kip)</th>
<th>Mmax (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>40.00</td>
<td>0.00</td>
<td>40.00</td>
<td>2496</td>
<td>16640</td>
<td>33280</td>
<td>256,190</td>
</tr>
<tr>
<td>10</td>
<td>40.62</td>
<td>0.00</td>
<td>40.62</td>
<td>2496</td>
<td>16896.7</td>
<td>33793.4</td>
<td>264,154</td>
</tr>
<tr>
<td>20</td>
<td>42.57</td>
<td>0.00</td>
<td>42.57</td>
<td>2496</td>
<td>17707.9</td>
<td>35415.8</td>
<td>290,128</td>
</tr>
<tr>
<td>30</td>
<td>46.19</td>
<td>0.00</td>
<td>46.19</td>
<td>2496</td>
<td>19214.2</td>
<td>38426.4</td>
<td>341,566</td>
</tr>
</tbody>
</table>

**Beam Tributary Area Loads to Individual beam**

<table>
<thead>
<tr>
<th>Angle (degrees)</th>
<th>Tributary area (ft)</th>
<th>max load (kip)</th>
<th>Rc (kip)</th>
<th>Rc(x) (kip)</th>
<th>Rc(y) (kip)</th>
<th>Rt (kip)</th>
<th>Rt(x) (kip)</th>
<th>Rt(y) (kip)</th>
<th>Mmax (kip-ft)</th>
<th>Weight (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>3</td>
<td>11.98</td>
<td>79.87</td>
<td>79.87</td>
<td>0.00</td>
<td>159.74</td>
<td>159.74</td>
<td>1,229.71</td>
<td>W90x116</td>
<td>171,689</td>
</tr>
<tr>
<td>5</td>
<td>19.97</td>
<td>83.20</td>
<td>83.20</td>
<td>0.00</td>
<td>266.24</td>
<td>266.24</td>
<td>2,049.52</td>
<td>V96x150</td>
<td>132,000</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>39.94</td>
<td>266.24</td>
<td>266.24</td>
<td>0.00</td>
<td>532.48</td>
<td>532.48</td>
<td>4,098.03</td>
<td>W40x230</td>
<td>101,200</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>3</td>
<td>11.98</td>
<td>81.10</td>
<td>79.87</td>
<td>14.08</td>
<td>162.21</td>
<td>159.74</td>
<td>1,267.94</td>
<td>W90x116</td>
<td>174,328</td>
</tr>
<tr>
<td>5</td>
<td>19.97</td>
<td>135.17</td>
<td>133.12</td>
<td>23.47</td>
<td>270.35</td>
<td>266.24</td>
<td>2,113.23</td>
<td>V96x150</td>
<td>134,036</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>39.94</td>
<td>266.24</td>
<td>266.24</td>
<td>0.00</td>
<td>532.48</td>
<td>532.48</td>
<td>4,226.46</td>
<td>W40x230</td>
<td>130,462</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>3</td>
<td>11.98</td>
<td>85.00</td>
<td>79.87</td>
<td>29.07</td>
<td>106.25</td>
<td>99.84</td>
<td>1,392.61</td>
<td>W93x118</td>
<td>185,848</td>
</tr>
<tr>
<td>5</td>
<td>19.97</td>
<td>141.66</td>
<td>133.12</td>
<td>48.45</td>
<td>263.33</td>
<td>266.24</td>
<td>2,321.02</td>
<td>V96x160</td>
<td>149,836</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>39.94</td>
<td>266.24</td>
<td>266.24</td>
<td>46.95</td>
<td>540.89</td>
<td>532.48</td>
<td>4,642.05</td>
<td>W40x230</td>
<td>140,471</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>3</td>
<td>11.98</td>
<td>92.23</td>
<td>79.87</td>
<td>48.11</td>
<td>184.46</td>
<td>159.74</td>
<td>1,639.61</td>
<td>W93x130</td>
<td>222,164</td>
</tr>
<tr>
<td>5</td>
<td>19.97</td>
<td>153.71</td>
<td>133.12</td>
<td>78.86</td>
<td>307.43</td>
<td>266.24</td>
<td>2,732.69</td>
<td>V96x194</td>
<td>217,130</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>39.94</td>
<td>307.43</td>
<td>266.24</td>
<td>153.71</td>
<td>614.86</td>
<td>532.48</td>
<td>5,465.37</td>
<td>W40x397</td>
<td>201,703</td>
<td></td>
</tr>
</tbody>
</table>

1. Assumed both connection and floor can support loads
2. Grade 50 steel
3. Assume lateral bracing no greater than every 10 ft
4. Increased loads by 1.6 for (to cover live load factor) in beam tributary table

*Total weight of steel beams across 110 ft wide lock channel (approx.)
Lock gates and spillway gates are vulnerable features of navigation, flood control, and water resource projects. These structures are susceptible to damage from accidents; overloads due to natural events, lack of maintenance, or operational errors; and terrorist attack. Damage to lock or spillway gates can result in uncontrolled drawdown of pool, with possible severe impacts both upstream and downstream of the structure. Emergency gap closure systems are needed to rapidly stop flow and maintain pool and to accommodate early initiation of repairs. The literature was reviewed to identify the types of closure structures in existence and to assess their feasibility for use as emergency closures. In addition, some innovative concepts were developed, at least in preliminary form, and are presented herein.