# Evaluation of Public Safety at Run-of-River Dams

An Illinois Statewide Program

Submitted To:

Capital Development Board of Illinois

July 20, 2007

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CDB Project # 102-000-061

## Section 1 - General

### 1.1 *Authority*

The Illinois Department of Natural Resources (IDNR) is authorized to carry out inspections of any dam within the State, and to establish standards and issue permits for the safe construction of new dams and the reconstruction, repair, operation and maintenance of all existing dams as stated in Section 23a of the Rivers, Lakes and Streams Act (615 ILCS 5/23a).

### 1.2 *Purpose*

Run-of-river dams have historically posed a public safety hazard throughout the country and within Illinois. These dams span the entire width of a river and are considered low-head, meaning their hydraulic height is less than 25 ft. As a result, water flows freely over the entire dam during normal flow conditions. This poses two significant safety hazards, 1) the dam is not clearly identifiable to an individual traveling towards the dam from upstream, and 2) the dam may produce dangerous flow conditions downstream, known as a submerged hydraulic jump with a reverse roller.

In an effort to increase public safety at run-of-river dams, the State of Illinois has commissioned this report to document and evaluate existing public safety measures at the 25 run-of-river dams listed in Table 1.3.1-1 (further referred to as either "run-of-river dams" or simply "dams"). In addition, this report considers further public safety measures and presents dam removal or dam modification options that would eliminate or lessen the public safety hazards posed by run-of-river dams.

### 1.3 *Scope of the Study*

The scope of the study was as follows:

1. Document the location, type and condition of all public safety measures (e.g., buoys, fences, lighting, railing, signage, etc.) that currently exist at each dam;
2. Document the existing condition of each dam and any appurtenant works (e.g., intakes, gates, fish ladders, bypass channels, tailraces, etc.);
3. Document existing access to each dam and assess the adequacy of such access for maintenance and emergency services use;
4. Document existing boat portage and launching facilities, if any, at each dam site and assess the adequacy of these facilities to provide safe boat passage;
5. Document through limited field survey, existing dam structures and channel riverbed conditions within 500 ft upstream and 500 ft downstream of each dam;
6. Assess how hydropower operations impact public safety at each of those dams where hydropower operations exist;

7. Develop non-structure options for public safety measures at run-of-river dams in conjunction with the ongoing efforts of IDNR's Office of Water Resources (OWR);
8. Develop temporary structure options, associated cost opinions, and anticipated life expectancies to improve public safety at each dam, including the prevention of hydraulic rollers, if present;
9. Develop permanent structure options, associated cost opinions, and anticipated life expectancies to improve public safety at each dam, including the prevention of hydraulic rollers, if present;
10. Present temporary and permanent public safety options at each dam in a consolidated report.

*Table 1.3-1 – Run-of-River Dams Assessed*

| River | Dam | County | City |
|---|---|---|---|
| Kankakee | Momence | Kankakee | Momence |
| | Kankakee | Kankakee | Kankakee |
| | Wilmington | Will | Wilmington |
| | Wilmington Millrace | Will | Wilmington |
| Rock | Oregon | Ogle | Oregon |
| | Sinnissippi | Whiteside | Rock Falls |
| | Lower Sterling | Whiteside | Rock Falls |
| | Sears | Rock Island | Milan |
| | Steel | Rock Island | Milan |
| Fox | McHenry (Stratton L&D) | McHenry | McHenry |
| | Algonquin | McHenry | Algonquin |
| | Carpentersville | Kane | Carpentersville |
| | Elgin Kimball Street | Kane | Elgin |
| | South Elgin | Kane | Elgin |
| | St. Charles | Kane | St. Charles |
| | Geneva | Kane | Geneva |
| | Batavia | Kane | Batavia |
| | North Aurora | Kane | North Aurora |
| | Aurora East | Kane | Aurora |
| | Montgomery | Kane | Montgomery |
| | Yorkville | Kendall | Yorkville |
| Des Plaines | Hofmann | Cook | Riverside |
| Vermilion | Danville | Vermillion | Danville |
| Sangamon | Riverside Park | Sangamon | Springfield |
| | Petersberg | Menard | Petersberg |

The scope of this study included the review of all existing documentation assembled and provided by IDNR at each dam. Data included existing field survey, inspection reports prepared by others, aerial photography topography, dam as-built information, channel information, hydraulic information, river flow data, adjacent property information, ownership information, the age of each dam, historic safety information, recreational use information, lock usage information, hydroelectric generation information, and other pertinent information. Due to the wide range of data that was collected, not all information was available at all dams.

A visual reconnaissance and assessment was also performed at the beginning of the project. Based upon the results of the initial assessments, an additional assessment was made to determine the feasibility of dam removal. Reconnaissance teams thoroughly documented each dam site using a standard checklist specifically developed for this project. The check list included site features such as the type of dam crest, the condition and visibility of all public safety measures at the dam, the existing condition of the dam and any appurtenant works, including visually obvious deficiencies, existing access to each dam, the adequacy of such access for maintenance and emergency services use, existing boat portage and launching facilities, if any, and the adequacy of these facilities to provide safe boat passage. No structural or geotechnical analysis was performed as part of the assessment. Approximate locations of all visible features were recorded on topographic maps and aerial photos. GPS units were used to record the approximate horizontal location of key features. Digital photos and video recordings were taken of all major site features. A field reconnaissance report was prepared for each site and appears in Appendix I. Initial concepts to improve safety were developed and documented during the inspection along with the basic field data. A survey questionnaire was also developed and forwarded to municipal emergency responders (EMS) to solicit feedback on public safety issues. The survey included questions about existing emergency plans, known incidents at the dams, and public education measures. Responders were also allowed to suggest potential ideas to improve safety. A small percentage of responses were received by mail and additional responses were received through follow up phone calls.

Of the 25 dams that were investigated, 17 required additional field surveys to identify bed elevations and cross-section data (see Table 1.3-2 below).

*Table 1.3-2 - Dams where Additional Field Surveys were Performed*

| River | Dam | County | City |
|---|---|---|---|
| Kankakee | Momence | Kankakee | Momence |
| | Kankakee | Kankakee | Kankakee |
| | Wilmington Millrace | Will | Wilmington |
| Rock | Oregon | Ogle | Oregon |
| | Sinnissippi | Whiteside | Rock Falls |
| | Lower Sterling | Whiteside | Rock Falls |
| | Sears | Rock Island | Milan |
| | Steel | Rock Island | Milan |
| Fox | Carpentersville | Kane | Carpentersville |
| | Elgin Kimball Street | Kane | Elgin |
| | St. Charles | Kane | St. Charles |
| | Geneva | Kane | Geneva |
| | Aurora East | Kane | Aurora |
| Des Plaines | Hofmann | Cook | Riverside |
| Vermilion | Danville | Vermillion | Danville |
| Sangamon | Riverside Park | Sangamon | Springfield |
| | Petersberg | Menard | Petersberg |

The field survey was conducted as follows using a 2 to 4 person crew with a small, out-board motorboat.

- One channel cross-section was surveyed 200 to 500 ft upstream of the dam using a single beam fathometer or at-the-point-of-depth-sounder. The exact location of the cross-section was determined in the field based on an assessment of the safety conditions to be surveyed. Cross-sections were limited to 3 to 5 survey points depending upon channel configuration and the type of device used.

- Estimates of the depth of soft sediment and a visual characterization of the sediment (e.g., fine, gravelly, etc.) were taken at three locations upstream of each dam using a probe. Actual locations of sediment probes along the cross-section were determined in the field. The probe was manually pushed into the sediment to the point of refusal. The intent was to estimate the thickness of the soft sediment in the channel to produce a gross estimate of soft sediments in the pool upstream of the dam. No sediment samples were collected for analysis.

- Three channel cross-sections were surveyed downstream of the dam location. One section was surveyed as close as possible to the downstream face of the dam based as an assessment of the safety conditions. Due to the high flows that occurred during the field survey, crews were unable to access either the dam crest or the area immediately downstream of the dam face where potential scour holes could be expected. As a result, limited cross-section data is available to estimate the elevation of the river bed immediately downstream of the dam. Two additional sections were performed approximately 100 and 200 ft downstream of the first cross-section. The exact location of these cross-sections was determined in the field based on an assessment of the safety conditions.

Surveys were referenced to a temporary local bench mark (TBM) or local benchmarks that had been established and provided by IDNR. Level circuits and traverses to tie TBM's into established benchmarks was not part of the scope of this investigation.

## 1.4 *Definitions*

The following is a list of definitions of technical terms that appear throughout the report.

**1 to 100 year Flow Rate;** A flow rate that has a certain percentage chance of occurring each year. For example, a 100 year flow rate would have a 1% chance of occurring each year, calculated as shown in Equation 1.4-1. A 1, 2, 5 and 50 year flow rate would have a 99%, 50%, 20%, and 2% chance of occurring each year, respectively.

$$\frac{1}{100} = 0.01 = 1\% \qquad \text{Eq. 1.4-1}$$

**Bypass Channel;** An engineered channel that diverts the flow around a run-of-river dam. An in-stream bypass channel diverts flow around the dam along one of the abutments within the river channel, while a full bypass channel diverts flow around the dam outside of the abutments.

**Computation Fluid Dynamic Model (CFD);** A computer model that simulates the flow of water

**Critical Flow;** A type of flow that occurs when the velocity of a wave on the surface of the water is equal to the average velocity of the river. This type of flow occurs during the transition between sub-critical and super-critical flow.

**Cross-section;** The ground elevation profile of a section of the river spanning from bank to bank, perpendicular to the river flow.

**Dam Crest;** The top of the dam face.

**Dam Face;** The downstream side of the dam. Several types of dam faces were observed during this investigation, including vertical, sloping, ogee, and stepped.

**Dam Width;** Length of the dam from left abutment to right abutment.

**Discharge;** The volume of water flowing over the dam per period of time.

**Downstream;** The direction corresponding to the direction of the average flow of the river.

**Entrainment;** In the case of run-of-river dams, entrainment is the process of air being added to the water when it flows over the dam crest. If the entrainment is significant, the density of the water may decrease such that a person is no longer buoyant.

**Exclusion Zone;** A region of the river intended to limit access and increase awareness of the hazard posed by a nearby run-of-river dam.

**FEMA;** Federal Emergency Management Agency. http://www.fema.gov/

**FERC;** Federal Energy Regulatory Commission. http://www.ferc.gov/

**Flood Insurance Study (FIS);** A study, commissioned by FEMA, that provides information on the flows and water surface elevations of a river during storm events.

**Headwater;** The water surface elevation upstream of a run-of-river dam.

**Hydraulic Jump;** A hydraulic condition where water transitions from a super-critical to a sub-critical flow type and which creates turbulent flow patterns.

**NGVD;** The National Geodetic Vertical Datum

**Normal Depth;** Depth of uniform flow in a channel or culvert

**Physical Model;** A scale model constructed out of physical materials (i.e. concrete, wood, masonry, etc.) used to represent the flow of water.

**Portage;** A location on the banks of a river that provides canoeists and kayakers the opportunity to exit and / or enter the river and transport their canoe or kayak.

**Reverse Roller;** Flow characterized by circulating eddies or jets that can reverse the direction of flow in a localized area. A reverse roller often occurs within a submerged hydraulic jump.

**Riffle Pool Rock Ramp;** A series of boulders placed in the river channel that form a series of riffles (i.e. short drops with mildly turbulent water) and pools (i.e. calm, low-velocity regions).

**River Bed;** The section of river that is normally covered by water.

**River Channel;** The section of river from bank to bank.

**River Invert;** The lowest elevation of the river bottom.

**Run-of-river Dam;** A dam spanning the width of a river with a height less than 25 ft which during normal flow conditions typically experiences flow over the entire dam.

**Scour Hole**; The area below a run-of-river dam that becomes eroded due to the high velocities of the water flowing over the dam face.

**Spillway;** The portion of a dam that conveys flow. In the case of a run-of-river dam, the entire dam crest typically acts as the spillway.

**Standard Hydraulic Jump;** A hydraulic jump where the transition from super-critical to sub-critical flow is not submerged due to low tailwater conditions. The turbulent transition region is visible and clearly identifiable, and a reverse roller is not typically present.

**Standing Wave;** A type of wave in which there is no forward progression and the surface oscillates between fixed nodes, such that the crest at one moment becomes and the trough at the next.

**Stepped Face;** A series of concrete or natural stone steps placed against the dam face and extending downstream. A stepped face can be used to reduce or eliminate a reverse roller downstream of a run-of-river dam. A stepped face could be designed to allow for a small amount of water to trickle over the face at all times if reducing or eliminating the flow results in undesirable environmental impacts.

**Sub-critical Flow;** A type of flow that occurs when the velocity of a wave on the surface of the water is greater than the average velocity of the river. Sub-critical flow is generally characterized by deeper, slower moving water when compared to super-critical flow.

**Super-critical Flow;** A type of flow that occurs when the velocity of a wave on the surface of the water is less than the average velocity of the river. Super-critical flow is generally characterized by shallower, faster moving water when compared to sub-critical flow.

**Submerged Hydraulic Jump**; A hydraulic jump where the transition from super-critical to sub-critical flow is submerged due to high tailwater conditions. The submerged, turbulent region may result in a reverse current depending on the flow, river geometry, and dam characteristics.

**Tailwater;** The water surface elevation downstream of a run-of-river dam.

**Turbulence;** Flow characterized by irregular and apparently random, non-uniform flow patterns with the potential for eddies or jets.

**Upstream;** The direction opposite to the direction of the average flow of the river.

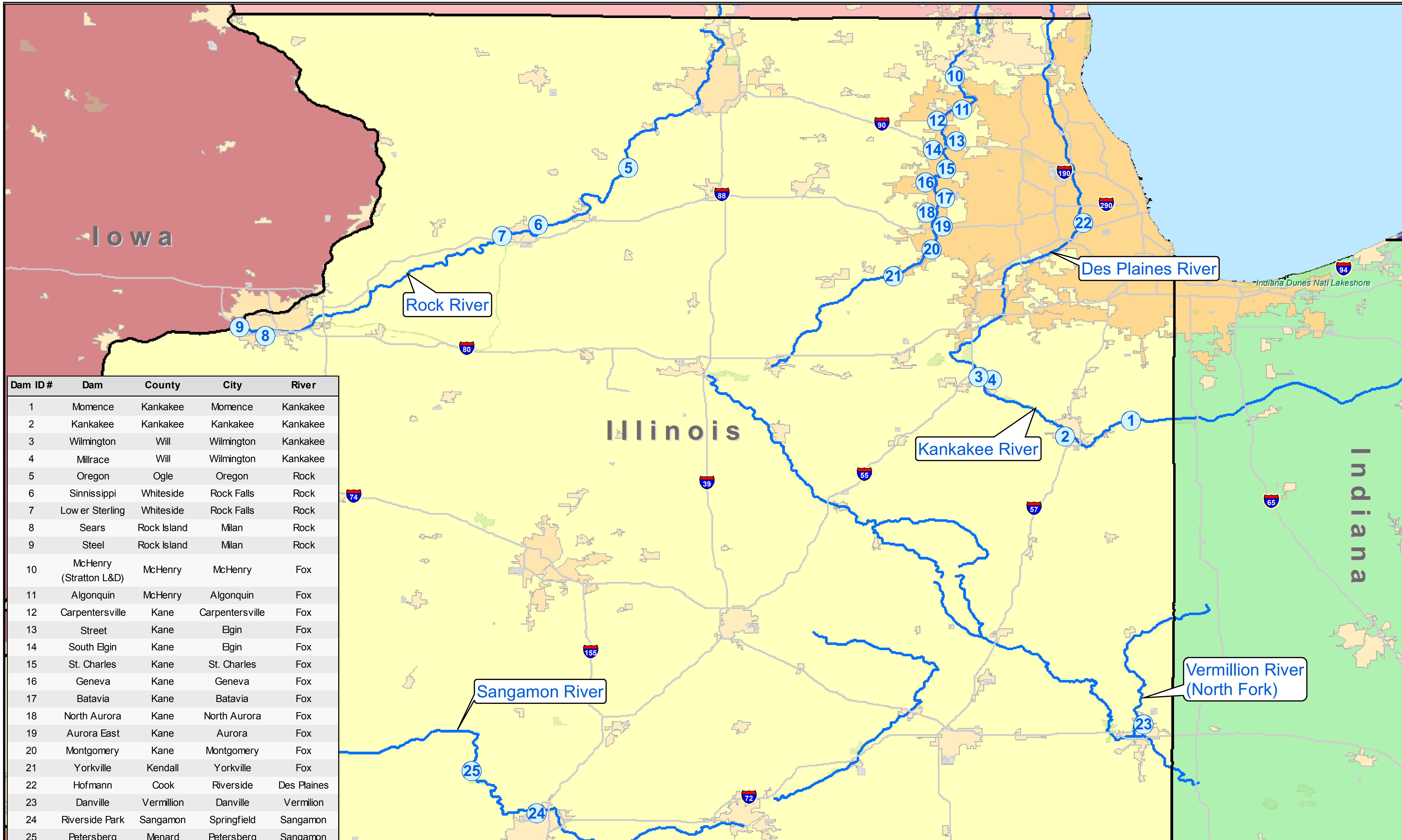
**Weir;** A hydraulic structure placed perpendicular to the flow of a channel, commonly used to measure the rate of flow or act as a spillway to pass flow while providing an increase in the upstream water surface elevation run-of-river dam crests act as weirs.

## 1.5 *Report Organization*

This report is organized initially by river and secondly by dam. Section 2 presents general signage guidelines intended to address public safety, and a recommended signage plan for each dam. Section 3 outlines a general public awareness program, a non-structural option for enhancing public safety. Section 4 presents a general discussion of the temporary and permanent structural options considered, an assessment of those options that were found to be feasible for each dam, and a summary of opinions of cost. Appendix I contains the field reconnaissance notes and EMS inquiry responses. Appendix II contains the photo and video documentation of the field reconnaissance, and Appendix III contains a detailed breakdown of the opinions of cost summarized in Section 4. Appendix IV contains the detailed calculations and a discussion of the temporary rock fill options at each dam. Appendix V contains dam ownership information, a brief history and a listing of available documents reviewed during this report, the visual reconnaissance summary made by the dam reconnaissance teams, the field survey, a summary of responses from EMS responders, and an assessment of field conditions.

## 1.6 *Dams and Locations*

Each dam and their location is presented in Figure 1.6-1 on the following page.




| Dam ID # | Dam | County | City | River |
|---|---|---|---|---|
| 1 | Momence | Kankakee | Momence | Kankakee |
| 2 | Kankakee | Kankakee | Kankakee | Kankakee |
| 3 | Wilmington | Will | Wilmington | Kankakee |
| 4 | Millrace | Will | Wilmington | Kankakee |
| 5 | Oregon | Ogle | Oregon | Rock |
| 6 | Sinnissippi | Whiteside | Rock Falls | Rock |
| 7 | Lower Sterling | Whiteside | Rock Falls | Rock |
| 8 | Sears | Rock Island | Milan | Rock |
| 9 | Steel | Rock Island | Milan | Rock |
| 10 | McHenry (Stratton L&D) | McHenry | McHenry | Fox |
| 11 | Algonquin | McHenry | Algonquin | Fox |
| 12 | Carpentersville | Kane | Carpentersville | Fox |
| 13 | Street | Kane | Elgin | Fox |
| 14 | South Elgin | Kane | Elgin | Fox |
| 15 | St. Charles | Kane | St. Charles | Fox |
| 16 | Geneva | Kane | Geneva | Fox |
| 17 | Batavia | Kane | Batavia | Fox |
| 18 | North Aurora | Kane | North Aurora | Fox |
| 19 | Aurora East | Kane | Aurora | Fox |
| 20 | Montgomery | Kane | Montgomery | Fox |
| 21 | Yorkville | Kendall | Yorkville | Fox |
| 22 | Hofmann | Cook | Riverside | Des Plaines |
| 23 | Danville | Vermillion | Danville | Vermilion |
| 24 | Riverside Park | Sangamon | Springfield | Sangamon |
| 25 | Petersberg | Menard | Petersberg | Sangamon |

Figure 1.6-1

*Dams and Locations*

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Figure 1.6-1

*Dams and Locations*

## 1.7 *Overview of Public Safety Issues at Run-of-River Dams*

As discussed, run-of-river dams span the entire width of a river channel, and water continuously flows over the crest of the dam. The drop over the dam crest, and often the turbulent reverse currents downstream, contribute to dangerous conditions for river users and pedestrians. These currents may challenge even the best swimmer, canoeist, or kayaker, as seen in past incidents across the country. Air may also become entrained in the turbulent water, decreasing water density and buoyancy, and making it more difficult to stay afloat. Even if an individual is wearing a safety vest, they may be forced and held under the water. Run-of-river dams and their surrounding areas are often considered attractive to fishermen, canoeists, kayakers, and children; however, river users and pedestrians may be unaware of the risks associated with these dams. Would be rescuers frequently underestimate the power of the water and become victims themselves. During the summer of 2006 alone, there were several incidents at run-of-river dams in Illinois, resulting in drowning deaths. While there are no official, historical statistics on incidents in Illinois, anecdotal evidence suggests that during an average summer, several accidents and / or deaths related to run-of-river dams occur.

One of the phenomena that can occur at a run-of-river dam and that may pose a hazard to recreational users is the formation of a submerged hydraulic jump at the base of the dam. These jumps form as a result of three types of flow conditions that often occur in a river. In order to provide a clear understanding of how dangerous currents occur at run-of-river dams, a brief description of the engineering science behind these three flow types is provided.

A river may experience super-critical, critical, and sub-critical flow at the same time along different reaches due to changes in the characteristics of the river. Sub-critical flow occurs when the average velocity of the stream is less than the velocity of a wave traveling on the surface of the water, known as the wave velocity. For example, if a stone is dropped in a flowing river and it produces waves in all directions, the flow is sub-critical. For a given stream, sub-critical flow corresponds to larger water depths and / or lower velocities. When the average velocity of the stream equals the wave velocity, the flow becomes critical. Once the average stream velocity becomes greater than the wave velocity, the flow is considered to be super-critical. In the case of super-critical flow, a stone dropped in the river will produce waves only in the downstream direction. Super-critical flow is characteristic of higher velocities and shallower depths when compared to sub-critical flow. Run-of-river dams present unique situations where the flow over the crest of the dam is typically super-critical and the flow downstream of the dam is generally sub-critical. When each of these types of flow occur in a river or at a dam, a transition takes place at the location where they meet. This transition is called a hydraulic jump, and it results in turbulent water and strong currents.

Several types of hydraulic jumps may occur depending on a river's channel geometry and flow rate; however, this report we will only consider two types, a standard hydraulic jump and a submerged hydraulic jump with a reverse roller. A standard hydraulic jump is characterized by Figure 1.7-1, where the jump forms uninterrupted due to low tailwater conditions. A standard hydraulic jump results in fast moving, turbulent flow with a standing wave at the downstream face of a run-of-river dam. These flow patterns do not have a strong circulatory nature, but may pose a safety hazard due to the degree of turbulence.

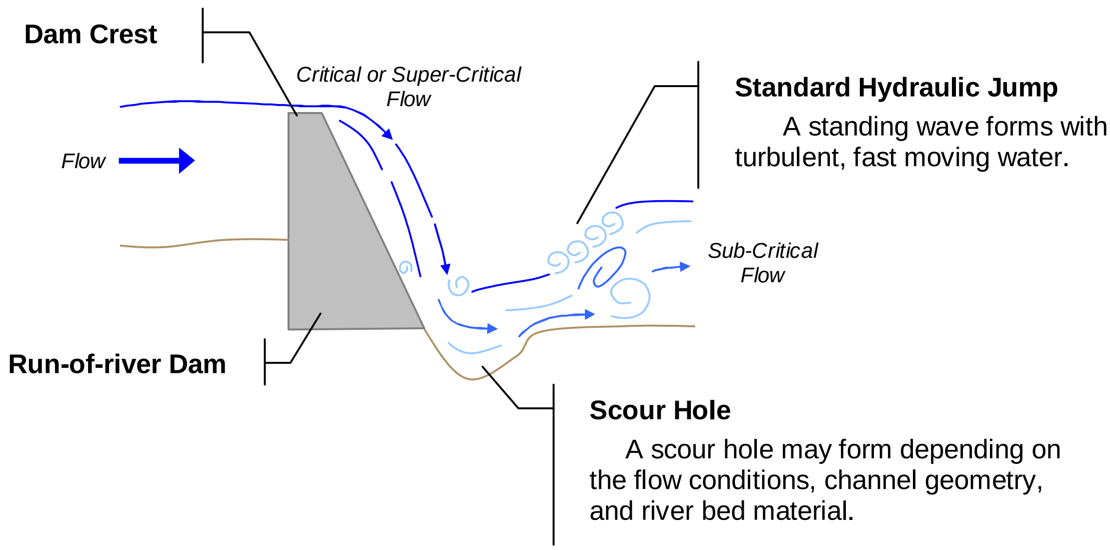



*Figure 1.7-1 - Standard Hydraulic Jump*

A submerged hydraulic jump with a reverse roller occurs when the tailwater increases such that the turbulent, transitional area of the jump becomes submerged as characterized in Figure 1.7-2. In the case of a hydraulic jump located downstream of a run-of-river dam, as the water plunges over the dam, it is forced upwards, producing a circular flow resulting in a reverse roller.

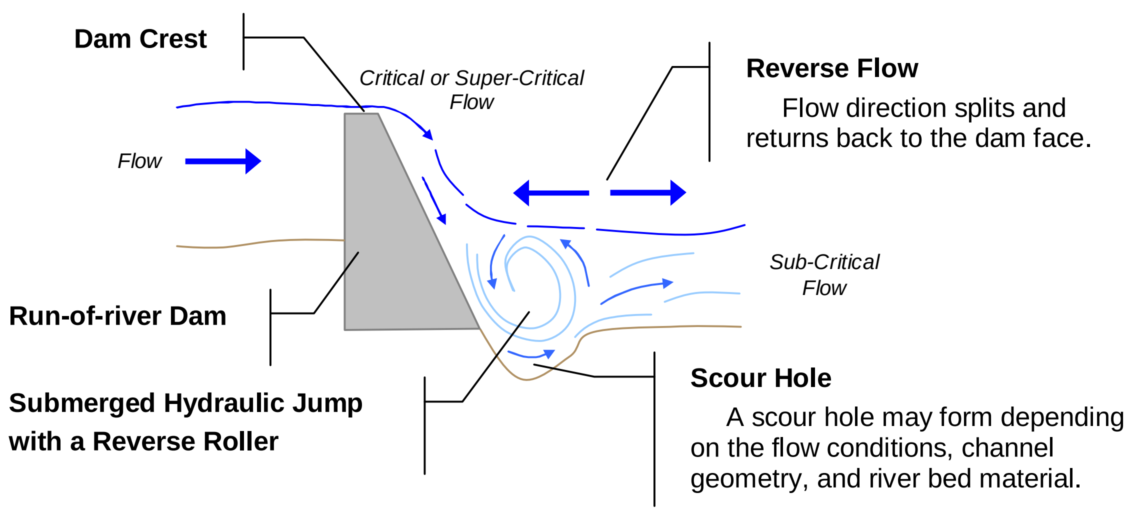
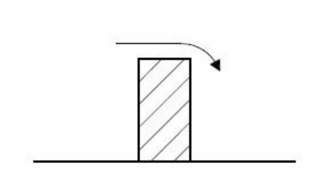
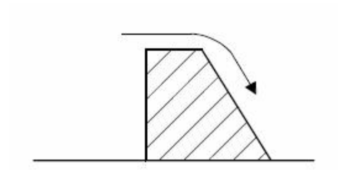



*Figure 1.7-2 - Submerged Hydraulic Jump*

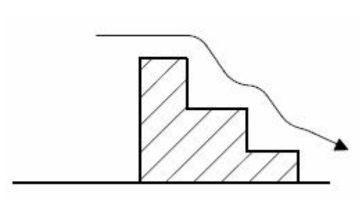
The presence and types of hydraulic jumps and turbulent currents downstream of dams are influenced by the shape and size of the dam crest and face, or spillway, discharge rate, downstream flow regime, and river geometry. Several types of run-of-river dam spillways were observed during this investigation, as shown in Figure 1.7-3, below. Floating debris also represents a danger to swimmers, especially if it becomes trapped in the hydraulic jump. This was not addressed specifically in this report due to the planning level scope, and because the amount of debris and its impact on safety is often difficult to quantify.



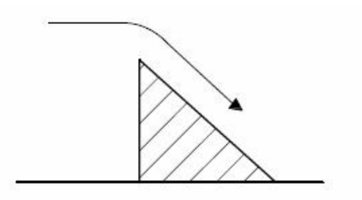
*Broad Crest with Vertical Face*



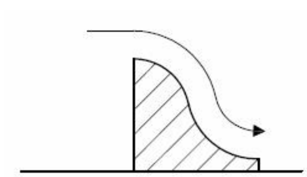
*Broad Crest with Sloping Face*



*Broad Crest with Stepped Face*



*Narrow Crest with Sloping Face*



*Rounded Ogee Crest with Sloping Face*

*Figure 1.7-3 – Dam Spillway Types*



# Evaluation of Public Safety at Run-of-River Dams

Section 2 - Signage

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# Section 2 - Signage

## 2.1 *Limitations and Future Needs*

Signage for both recreational river users and shoreline users can serve as an effective tool in the effort to enhance public safety at run-of-river dams. The signage guidelines presented in Section 2.3 of this report should be viewed as a basis for developing statewide standards. The guidelines developed for this study were based upon warning Illinois river and shoreline users while incorporating standard signage guidelines from other state and federal agencies.  The minimum viewing distance of each sign, specifically for river users, was a large consideration in the development of the signage guidelines.  This must be given careful consideration since river users must have sufficient time to react after viewing a sign.  Unfortunately, this can result in large, unwieldy signs requiring excessive initial cost and maintenance, particularly if river user signage is limited to the river banks.  There are additional considerations that should be taken into account in the development of signage standards that were beyond the scope of this study, including, but not limited to:

- Right-of-way (ROW) acquisition;
- Access for construction;
- Maintenance and replacement;
- Placement of permanent signs on sign signage piers located in the river;
- Alternative signage for unique specific situations;
- Community acceptance.

The general guidelines and individual dam signage plans presented in this study did not take ROW acquisition within private property into account. While it is understood that the state has restrictions with regard to private property, this should be addressed in the final implementation of the signage plans presented herein and in the future development of signage standards.

Limited consideration was given to construction access issues in this study. Regardless of ROW acquisition, construction access should be considered particularly when private property, urban amenities, dedicated parks, forests, and other environmentally sensitive areas are involved.

Maintenance and sign replacement should also be factored into future signage standards. While access for initial construction may prove problematic, access for maintenance and replacement may prove more so. The frequency of vandalism of signage is an unknown and most probably will vary from site to site.

Alternatives such as placing smaller signs located on permanent, dedicated piers in the river, as opposed to or in addition to placing signs on the banks, should be considered. This would significantly reduce the size required for bank signage. Additionally, site specific signage will need to be considered depending on the configuration of each dam, its appurtences, and the upstream and downstream river geometry. Community acceptance of warning signage should be addressed when developing individual site plans. It is expected that this will vary from community to community as well.

Installation of fencing should also be considered at dam abutments and along the shore immediately upstream and downstream of the dam. Temporary fencing, such as chain link, could be installed at a 6 ft height at the dam abutments and at a 4 ft height 50 ft upstream and downstream of the dam. A 6 ft high, 6 gage galvanized steel chain link fence would cost approximately $28 per linear ft based on material costs and labor. A similar 4 ft high fence would cost approximately $24 per linear ft based on material costs and labor. A permanent fence made from ornamental aluminum or decorative wrought iron would cost $174 and $375 per linear ft, respectively.

Signage at the dam face and abutments should be lit if possible. Lighting could be supplied by a standard lighting housing using halide or sodium bulbs. The cost of lighting, if electricity were available, would include materials (e.g., electrical, lighting fixture, bulb, etc.), installation (e.g., fixture installation and electrical hook-up), and future maintenance. In general, for highway transportation purposes, halide and sodium bulbs would require replacement every 2 and 5 years, respectively. If electricity were not available on-site, a comprehensive solar lighting system could be installed. Solar lighting systems are available from a wide range of manufacturers, who often times provide installation. A pole mounted solar lighting system would cost approximately $2,000 to $3,000 depending on the size, capacity, and manufacturer. An example solar system could include a solar generator, a 50 watt solar panel, and an energy efficient Light Emitting Diode (LED) flood light, providing 150 LUX during the 10 year life span of the bulb. Advantages to solar lighting include reductions in maintenance costs due to a longer bulb life expectancy, ability to place units without having to consider an electrical supply, no wiring costs from electrical hook-ups to the fixture, and a reduction in power usage.

Lastly, the continued development of signage standards should include performance based criteria to handle the variety of individual circumstances that will occur at each dam. A performance based criterion states the desired outcome of a design or project without calling out specific details or establishing universal rules. There are four groups of signage guidelines referred to for this report, which list specific guidelines related to each group. At the beginning of each group, a performance criterion is provided. For example, under the group “Dam Abutments,” the performance based criterion reads “Signage should be present at the dam abutment to identify the dam crest and inform river users and pedestrians of the dangers posed by the dam.” Performance based criteria allow for maximum flexibility in achieving a design or implementing a project, and should be considered when developing signage standards.

## 2.2 *General Approach*

Signage guidelines and individual signage plans were developed such that recreational river and shore users could clearly view the signs in order to be adequately informed of and be provided with enough time to avoid the hazard posed by each dam. Signage fonts, sizes and materials were developed based on the Illinois Department of Transportation's standards, the Federal Energy Regulation Committee's Guidelines for Public Safety at Hydropower Projects, the U.S. Army Corps of Engineer's EP 310-1-6a Sign Standards Manual, the Pennsylvania Fish & Boat Commission's standards, and engineering judgment.

## 2.3 *Signage Guidelines*

The following pages contain proposed guidelines for new signage installation at public, state-owned run-of-river dams in Illinois. The signage guidelines are placed into four groups:

- Exclusion Zones;
- River Banks within Exclusion Zones in Public Areas (For Pedestrian Users);
- Dam Abutment;
- Portages.

Each group includes specific signage guidelines, and a performance based criterion is stated for each group. These guidelines are shown in Table 2.3-1, while Table 2.3-2, presents signage graphics, descriptions, and guideline references for sizing and placement of each sign. It should be noted that a number of the signs are presented as optional, since they contain additional warnings which might not be deemed necessary at every dam. A general signage plan depicting the guidelines discussed in Table 2.3-1 is included as Figure 2.3-1.

The signage guidelines presented in this report are intended for use under optimal conditions (e.g. acquired right of ways, adequate access for posting, etc.); therefore, variations may be required due to site specific conditions not addressed herein. While it is recommended that these guidelines should be implemented whenever possible, a variance from these guidelines is preferable to no signage at all. Short-term signage should be placed where feasible, in close proximity to the location referred to in the guidelines. Long-term signage solutions should conform to the guidelines listed in Table 2.3-1. Individual signage plans were developed for each dam and are presented in Section 2.4. These plans were developed assuming optimal conditions and did not account for land ownership, access, or other site features that might impact placement of signage. A summary of sign sizes is presented in Table 2.3-3. It should be noted the signage plans and sizes include the optional signs called out in Table 2.3-3.

Opinions of signage cost are presented in Table 2.3-4. To give an approximate price range, Table 2.3-4 presents opinions of cost with and without optional signs. These opinions of cost are based on installation at a relatively flat, easily accessible area, and do not include potential increases to costs resulting from difficult installations. Costs for portage trails and portage installations beyond an entrance sign are not included in the opinion of cost.

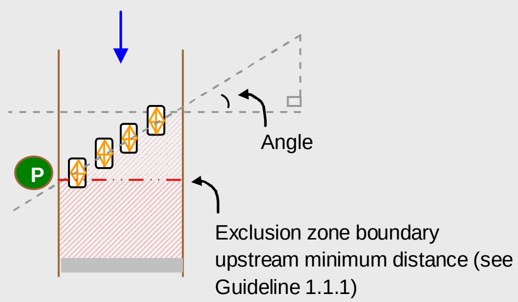
*Table 2.3-1 - Signage Guidelines*

| Guideline | Sign (see Table 2.3-2) |
|---|---|
| **1.0 Exclusion Zone** | |
| *An exclusion zone should be present upstream and downstream of the dam to limit access and increase awareness of the hazard posed by the dam. Exclusion zones may be adjusted if necessary to address right-of-way or other physical features such as portages, boat ramps, docks, etc. that would otherwise be located within the zone. Exclusion zones should be defined with the intent to prohibit access to areas considered hazardous for river users.* | |
| 1.1 - ***Exclusion Zone Boundary*** | |
| 1.1.1 - Upstream minimum distance of 300 ft from dam face or a minimum of 300 ft plus ½ * (*dam width* - 300 ft) if the dam is greater than 300 ft wide. The exclusion zone layout may be adjusted to allow for passage to *approved* river bank features (e.g. approved portages) that would have been located within the exclusion zone. Modification of the exclusion zone layout should be addressed on a case by case basis in accordance with Guidelines 1.2.5, with the safety of river users placed foremost. | |
| 1.1.2 - Downstream minimum distance of 50 ft from the dam face or a minimum of 25 ft downstream of the downstream edge of the hydraulic jump, roller, or turbulance during a 1 year flow event, whichever is the greater distance from the dam. | |
| 1.2 - ***Buoys*** | |
| 1.2.1 - Standard floatable "can" buoys with or without a collar should be installed along the upstream exclusion zone. Can buoys are floatable markers with printed symbols and / or words denoting a warning message to river users. A collar is a ballast at the bottom of the buoy which provides added stability over a standard can buoy. | ***Sign B1*** |
| 1.2.2 - There should be a minimum of two (2) buoys spaced uniformly. | |
| 1.2.3 - Buoys should have a maximum spacing of 100 ft. | |
| 1.2.4 - Buoys should be marked with the standard marine symbol for "restricted zone", i.e.  and the words "Danger Dam". | |

*Table 2.3-1 - Signage Guidelines (continued)*

| Guideline | Sign (see Table 2.3-2) |
|---|---|
| 1.2.5 - When approved portages are located at the edge of the exclusion zone, buoy lines should be angled to direct river users to the portage (see Figure 1.2.5-1). The angle should be large enough to be apparent to upstream river users (approximately 10 to 30 degrees). | |

Figure 1.2.5-1



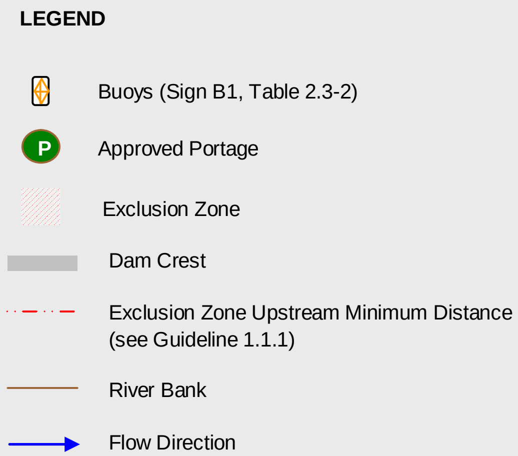



*Table 2.3-1 - Signage Guidelines (continued)*

| Guideline | Sign (see Table 2.3-2) |
|---|---|
| 1.2.6 - When approved portages and / or other approved river features are located within what would have been a standard exclusion zone, the exclusion zone may be adjusted such that buoys define approved access areas and direct river users to the approved river feature (see Figure 1.2.6-1, *Option A* and *B*). | |
| 1.2.7 - A minimum of one (1) solar-powered beacon light should be installed on the center-most buoy within the buoy line. | |
| 1.3 - ***River Bank Signage (For River Users)*** | |
| 1.3.1 - Signage should be installed on the upstream banks, at the edge of the exclusion zone facing upstream for river users. <u>Note:</u> Portions of this sign are optional and should be used where deemed appropriate and necessary. | ***Sign 1*** |
| 1.3.2 - When passage is allowed to portages or other approved uses within the exclusion zone upstream minimum distance, an alternate sign should be used warning of the dam and the distance ahead, but not prohibiting access. <u>Note:</u> Portions of this sign are optional and should be used where deemed appropriate and necessary. | ***Sign 1A*** |
| 1.3.3 - Signage should be installed on the downstream banks at the edge of the exclusion zone facing downstream for river users. | ***Sign 6*** |

Figure 1.2.6-1

***Option A*** ***Option B***

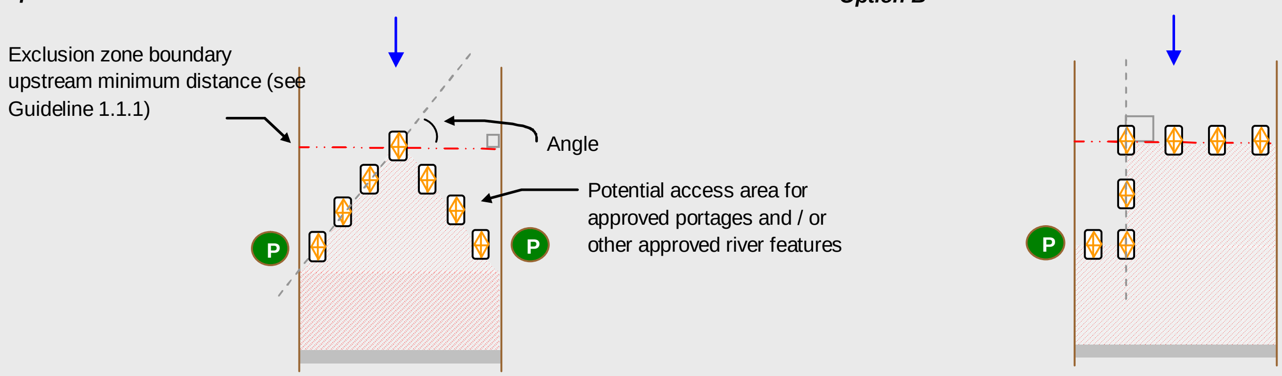


*Table 2.3-1 - Signage Guidelines (continued)*

<table>
<tr><th>Guideline</th><th>Sign<br>(see Table 2.3-2)</th></tr>
<tr><td>

1.3.4 - Signage Guidelines 1.3.1 and 1.3.2 should be designed such that the *largest text* (i.e., 1.5*A) is readable from the center of the river at a minimum distance V (see Figure 1.3.4-1 and Eq. 1.3.4-1).

When the immediate river width (dimension W) is less than or equal 300 ft, a minimum value of 300 ft should be used for dimension d.

When the immediate river width is greater than 300 ft, a minimum value of 300 ft plus ½ * (river width - 300 ft) should be used for dimension d.

Color palletes and font styles equivalent to Sign Standards Manual[1] should be used.

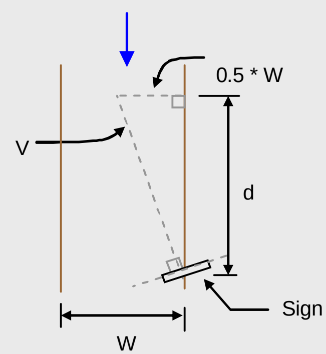
If the signage is not visible from the required distance due to an obstruction in the river, additional signage should be placed upstream of the obstruction.

Equations 1.3.4-1 and 1.3.4-2 are used to determine font and sign size, where *A, the font size,* is in inches.[1]

Eq. 1.3.4-1 $$V = \sqrt{d^2 + (0.5 * W)^2}$$

Eq. 1.3.4-2 $$A = \left(\frac{V}{28}\right) * \frac{1}{1.5}$$

Figure 1.3.4-1




</td><td></td></tr>
</table>

*Table 2.3-1 - Signage Guidelines (continued)*

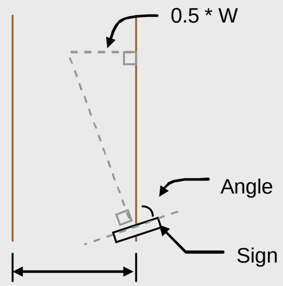
| Guideline | Sign (see Table 2.3-2) |
|---|---|
| 1.3.5 - Signage Guideline 1.3.3 should be designed using Equations 1.3.4-1 and 1.3.4-2 such that the largest text is readable from the center of the river at a distance V where d equals 100 ft downstream. Color palletes and font styles equivalent to *Sign Standards Manual*[1] should be used. | |
| 1.3.6 - Signage Guidelines 1.3.1 and 1.3.2 should be placed at an angle to the river bank such that river users in the center of the stream maintain a perpendicular, direct line-of-sight. For rivers with an immediate upstream river width of 300 ft or less, the angle shown in Figure 1.3.6-1 can be computed using Equation 1.3.6-1, where W is the width of the river at the sign in feet.<br>Eq. 1.3.6-1 $\text{Angle} = 90 - \tan^{-1}\left(\frac{0.5 * W}{300 + 0.5 * (W - 300)}\right)$<br>For rivers greater than 300 ft wide, the angle to the river bank in degrees can be computed using Equation 1.3.6-2.<br>Eq. 1.3.6-2 $\text{Angle} = 90 - \tan^{-1}\left(\frac{0.5 * W}{300}\right)$<br><br><br>Figure 1.3.6-1 | |
| 1.3.7 - Signage Guideline 1.3.3 shall be placed at an angle to the river bank such that river users in the center of the stream maintain a perpendicular, direct line-of-sight. The angle shown in Figure 1.3.6-1 can be computed using Equation 1.3.7-1, where W is the width of the river at the sign in feet.<br>Eq. 1.3.7-1 $\text{Angle} = 90 - \tan^{-1}\left(\frac{0.5 * W}{100}\right)$ | |

*Table 2.3-1 - Signage Guidelines (continued)*

| Guideline | Sign (see Table 2.3-2) |
|---|---|
| 1.3.8 - Signage Guidelines 1.3.1, 1.3.2 and 1.3.3 should be permanent and maintained year-round such that signs are clearly visible to river users. | |
| 1.3.9 - When feasible, Signage Guidelines 1.3.1, 1.3.2 and 1.3.3 should be lighted during low-light hours using an automated sensor. | |
| 1.3.10 - Signage Guidelines 1.3.1 and 1.3.2 (Signs 1 & 1A) that read: "Danger! Dam XXX ft Ahead" should be made of IDOT Type AA highly reflective material[2]. | |
| 1.4 - **Sign Size Limitations** | |
| 1.4.1 - Single Signs should generally be limited to a height no greater than 24 ft (not including clearance height) or a width no greater than 40 ft, unless special structural support is to be implemented.<br>Signs larger than this size may be dealt with by an alternate signage configuration such as:<br>● replacement of a single sign with multiple signs along the bank, providing an equivalent viewing coverage<br>● placement of signs on in-stream piers, reducing the required size of a single sign<br>● reduction of the sign size to within the 24 ft by 40 ft, if safety does not appear significantly compromised. | |
| 1.4.2 - Multiple Signs should generally be limited to a height no greater than 24 feet (not including clearance height) or a width no greater than 40 feet, unless special structural support is to be implemented.<br>Signs larger than this size may be dealt with by:<br>● placing signs on separate sign posts adjacent to one another but visually distinguishible<br>● elimination of optional signs<br>● reduction of the individual sign size to within the 24 ft by 40 ft, if safety does not appear significantly compromised. | |
| 1.5 - ***Miscellaneous*** | |
| 1.5.1 - Bridges located within 2,000 ft upstream of the dam should have signage installed on at least one (1) pier indicating the distance to the dam. | *Sign 7* |
| 1.5.2 - Signs addressing Signage Guideline 1.5.1 should be designed such that the *largest text* is readable from the center of the river 100 ft upstream. Font and sign size should be determined using Equations 1.3.4-1 and 1.3.4-2.<br>The signs should be made of IDOT Type AA highly reflective material[2].<br>Color palletes and font styles equivalent to Sign Standards Manual[1] should be used. | |
| 1.5.3 - Bridges near an exclusion zone boundary should be considered for use as the exclusion zone boundary if they meet Guidelines 1.1.1 and 1.1.2. | |

*Table 2.3-1 - Signage Guidelines (continued)*

| Guideline | Sign (see Table 2.3-2) |
|---|---|
| **2.0 River Banks within the Exclusion Zones in Public Areas (For Pedestrian Users)** | |
| *Signage should be present along river banks within exclusion zones to insure that pedestrians are aware of the dam and the hazard posed by it.* | |
| 2.1 - ***Signage*** | |
| 2.1.1 - There should be a minimum of one (1) upstream and one (1) downstream general warning sign between the dam abutments and the exclusion zone boundaries. Additional signage should be considered for bank areas in excess of 100 ft in length where river access is possible and a single sign fails to adequately warn pedestrians. | ***Sign 4*** |
| 2.1.2 - When portage or other passage is allowed within the exclusion zone minimum distance, a sign warning of the danger of the dam but not limiting access should be placed on the banks in the same manner as indicated in Guideline 2.1.1 | ***Sign 4A*** |
| 2.1.3 - There should be a minimum of one (1) upstream and one (1) downstream fishermen warning sign between the dam abutments and the exclusion zone minimum distance. Additional signage should be considered for bank areas in excess of 100 ft in length where river access is possible and a single sign fails to adequately warn fishermen. | ***Sign 5*** |
| 2.1.4 - Signage Guidelines 2.1.1, 2.1.2 and 2.1.3 should be directed towards pedestrians. A font style of Helvetica Medium[1] or equivalent should be used. | |
| **3.0 Dam Abutment** | |
| *Signage should be present at the dam abutment to identify the dam crest and inform river users and pedestrians of the hazards posed by the dam.* | |
| 3.1 - ***Signage*** | |
| 3.1.1 - One (1) diamond shaped warning sign should be installed at each abutment and directed toward upstream river users. | ***Sign 2*** |
| 3.1.2 - Signage Guideline 3.1.1 should be directed to the river users at an angle to be determined similarly as in Guideline 1.3.6. | |
| 3.1.3 - Signage Guideline 3.1.1 should be sized such that the *smallest text* (i.e., A) is readable from the upstream exclusion zone, as specified in Guideline 1.1.1. Font and sign sizing should be determined using Equations 1.3.4-1 and 3.1.3-1 (below) If an upstream bridge is located within the exclusion zone and obstructs the view of Signage Guideline 3.1.1, Guideline 3.1.1 \may be sized to be viewable from the downstream face of the bridge. Additional signage, such as Guidelines 3.1.1 or 1.5.1, must be placed at the bridge and must be sized according to the specific guideline.<br>Eq. 3.1.3-1 $A = \left(\frac{V}{28}\right)$ | |
| 3.1.4 - Signage Guidelines 3.1.1 should be made of IDOT Type AA highly reflective material[2]. Color palletes and font styles equivalent to Sign Standards Manual[1] should be used. | |

*Table 2.3-1 - Signage Guidelines (continued)*

| Guideline | Sign (see Table 2.3-2) |
|---|---|
| 3.1.5 - Detailed warning signs should be installed at the dam abutments directed towards pedestrians. Color palletes and font styles equivalent to Sign Standards Manual[1] should be used. | ***Sign 3*** |
| **4.0 Portages** | |
| *Signage should be present at approved portages to warn upstream river users of the dam ahead and to direct them to exit the river.* | |
| 4.1 - ***Signage*** | |
| 4.1.1 - Signage should be installed on each river bank between 300 ft and 1/4 mile upstream of the upstream portage. The signage should indicate the distance to the portage ahead, the location of the portage (i.e. left or right bank), and should notify river users of the dam ahead. | ***Sign P1*** |
| 4.1.2 - Signage Guideline 4.1.1 should be designed such that the largest text is readable from the center of the river 100 ft upstream. Font and sign sizes should be determined using Equations 1.3.4-1 and 1.3.4-2, where W is the width of river at the sign. Color palletes and font styles equivalent to Sign Standards Manual[1] should be used. | |
| 4.1.3 - Signage Guidelines 4.1.1 should be placed at an angle to the river bank such that river users in the center of the stream maintain a perpendicular, direct line-of-sight. The angle shown in Figure 1.3.6-1 can be computed using Equation 1.3.7-1, where W is the width of the river at the sign in feet. | |
| 4.1.4 - Portages upstream and downstream should have open-gated, ranch-style entrances with appropriate informational signage. | ***Sign P2*** |
| 4.1.5 - Signage should be installed a minimum of every 500 ft along a portage trail indicating the trail's direction. | ***Sign P3*** |

References:

[1] *Sign Standards Manual*, EP 310-1-6a , USACE, June 1, 2006

[2] *Standard Specification for Road and Bridge Construction*, IDOT, 2006

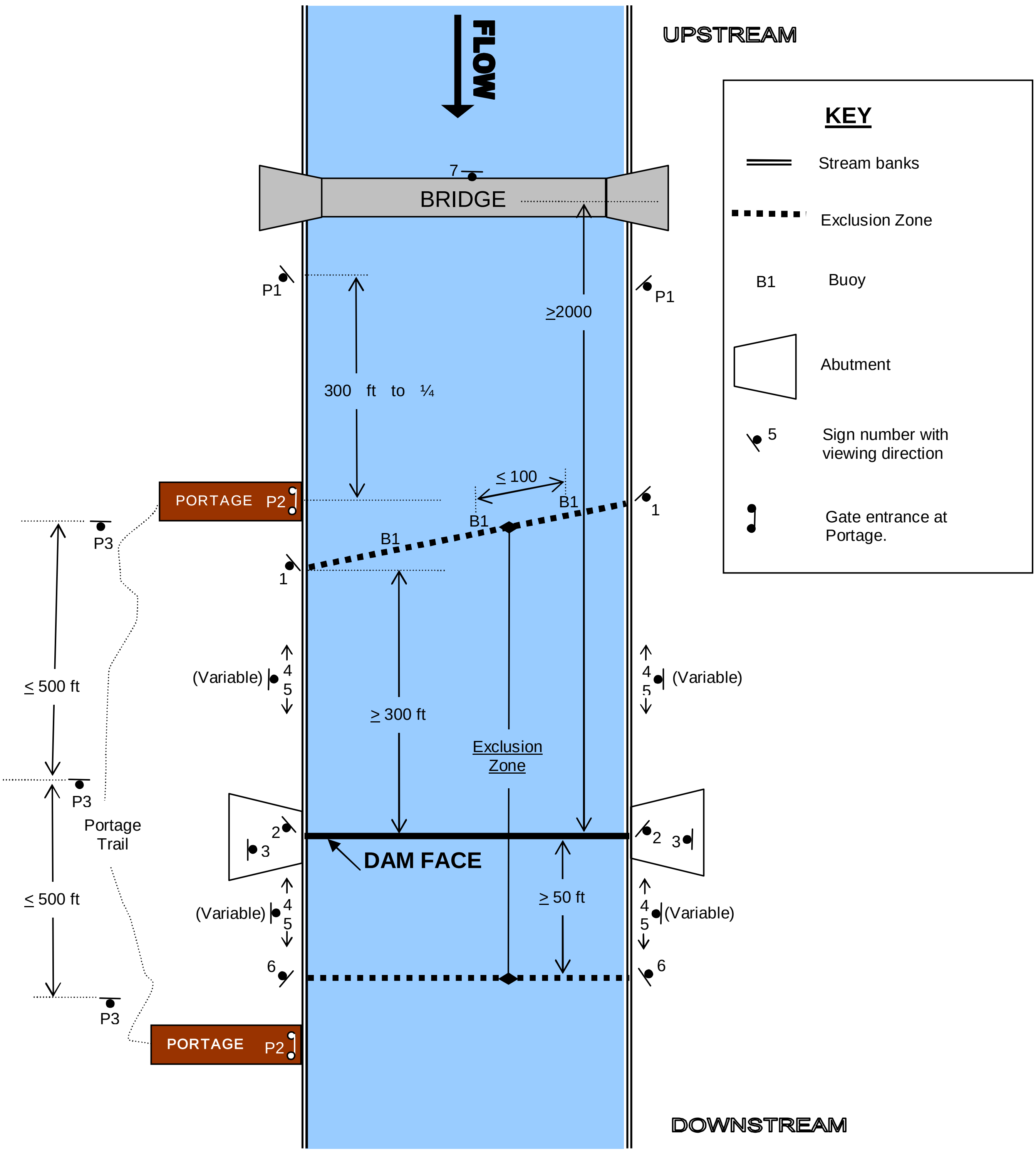



*Figure 2.3-1 - General Signage Schematic*

*Table 2.3-2 – Proposed Signage*

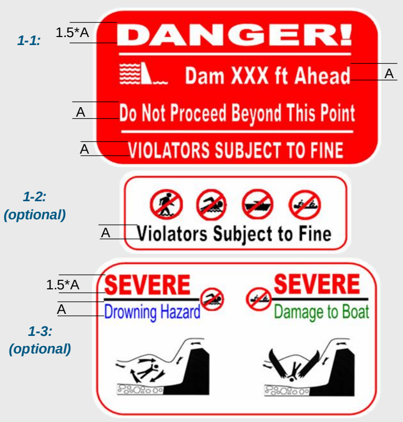
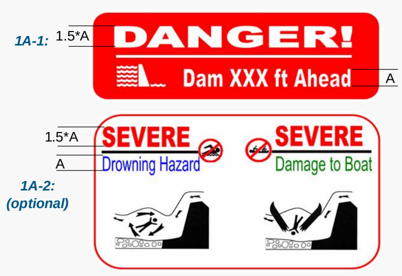
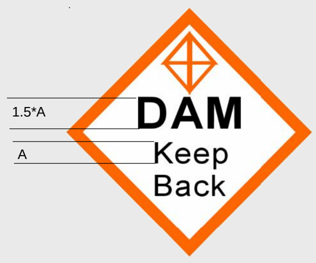
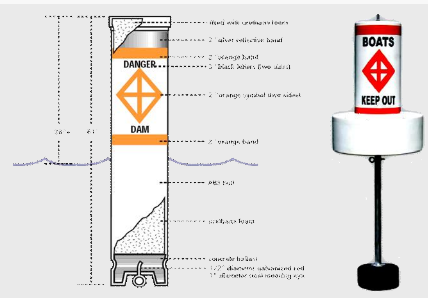
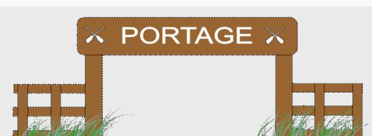
| Graphic | Description | Guideline Reference | Graphic | Description | Guideline Reference |
|---|---|---|---|---|---|
|  Sign 1 | • Warns ***upstream*** river users that they are approaching the dam and notifies them that they are not allowed to proceed past the exclusion zone.<br><br>• Should be placed on the banks at the ***upstream exclusion zone boundary*** and angled towards approaching river users.<br><br>• Signs 1-2 and 1-3 are optional precautions. | *1.3.1* |  Sign 6 | • Sign warns ***downstream*** river users that they are approaching the dam and notifies them that they are not allowed to proceed past the exclusion zone.<br><br>• Sign should be placed on the banks at the ***downstream exclusion zone boundary*** and angled towards river users. | *1.3.3* |
|  Sign 1A | • Warns ***upstream*** river users that they are approaching the dam.<br><br>• Should be placed on the banks at the calculated upstream exclusion zone boundary when the exclusion zone boundary is decreased to allow for portage or other approved access and angled towards approaching river users.<br><br>• Sign 1A-2 is an optional precaution. | *1.3.2* |  Sign 7 | • Sign warns ***upstream*** river users that they are approaching the dam.<br><br>• Sign should be ***placed on bridge piers*** upstream of and within the upstream exclusion zone and angled towards approaching river users. | *1.5.1* |
|  Sign 2 | • Sign notifies ***upstream*** river users of the location of the dam crest.<br><br>• Sign shall be placed at the ***dam abutment*** and angled towards approaching river users. | *3.1.1* |  Can Buoy Collared Buoy<br>Sign B1 | • Buoys warn ***upstream*** river users that they are approaching the dam and notifies them that they are not allowed to proceed past the exclusion zone.<br><br>• Buoys shall be placed uniformly with a maximum spacing of 100 ft. | *1.2.1* |
|  Sign 3 | • Sign notifies river bank ***pedestrians*** of the hazards posed by the dam.<br><br>• Sign instructs river bank pedestrians not to enter the water.<br><br>• Sign should be placed at the ***dam abutment*** and angled towards river bank pedestrians. | *3.1.5* |  Sign P1 | • Sign warns ***upstream*** river users that they are approaching the dam and notifies them that they are approaching a portage.<br><br>• Sign should be placed on the banks ***upstream of the upstream exclusion zone*** boundary and angled towards approaching river users. | *4.1.1* |
|  Sign 4<br> Sign 4A | • Sign notifies river bank ***pedestrians*** of the hazards posed by the dam.<br><br>• Sign instructs river bank pedestrians not to enter the water.<br><br>• Sign 4 should be placed ***along exclusion zones*** and angled towards river bank pedestrians.<br><br>• Sign 4A should be placed along other hazardous areas and angled towards river bank pedestrians. | *2.1.1 & 2.1.2* |  Sign P2<br> Sign P3 | • Portage entrance should include a ***"ranch" style gate*** that directs river users to the portage location and trail.<br><br>• Sign should be placed along the ***portage trail*** to direct river users between the upstream "take-out" and downstream "put-in" locations. | *4.1.4 & 4.1.5* |
|  Sign 5 | • Sign notifies river bank ***fishermen*** of the hazards posed by the dam.<br><br>• Sign instructs river bank fishermen not to enter the water and to fish only from the shore.<br><br>• Sign should be placed ***along exclusion zones*** and angled towards river bank fishermen. | *2.1.3* | | | |

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*Table 2.3-3* – Proposed Signage Sizing (Including Optional Signs)

| | | | Sign 1 | | | Sign 1A | | | Sign 2 | | | Sign 6 | | | Sign 7 | | | Sign P1 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Dam | Dam Width (ft) | U/S Exclusion Zone Minimum Distance (ft) | Largest Letter Height (1.5*A)[3] (inches) | Total Height[1] (ft) | Max. Width (ft) | Largest Letter Height (1.5*A)[3] (inches) | Total Height[1] (ft) | Max. Width (ft) | Largest Letter Height (1.5*A)[3] (inches) | Height (ft) | Width (ft) | Largest Letter Height (1.5*A)[3] (inches) | Total Height (ft) | Max. Width (ft) | Largest Letter Height (1.5*A)[3] (inches) | Height (ft) | Max. Width (ft) | Largest Letter Height (1.5*A)[3] (inches) | Total Height (ft) | Max. Width (ft) |
| Momence | 141 | 300 | 12.0 | 17 | 13 | | | | 16.5 | 12 | 11 | 4.5 | 6 | 3 | 6.0 | 3 | 9 | 12.0 | 10 | 13 |
| Kankakee | 440 | 370 | 16.5 | 24 | 18 | | | | 12.0 | 9 | 8 | 9.0 | 11 | 7 | 16.5 | 6 | 17 | 13.5 | 11 | 15 |
| Wilmington | 645 | 473 | 34.5 | 49 | 38 | | | | 31.5 | 23 | 20 | 12.0 | 15 | 9 | | | | 24.0 | 20 | 27 |
| Wilmington Millrace | 70 | 300 | 10.5 | 15 | 11 | | | | 16.5 | 12 | 11 | 4.5 | 6 | 3 | 4.5 | 2 | 5 | 12.0 | 10 | 13 |
| Oregon | 976 | 638 | 30.0 | 43 | 33 | 30.0 | 30 | 34 | 27.0 | 19 | 17 | 18.0 | 22 | 13 | | | | 15.0 | 13 | 17 |
| Sinnissippi | 1165 | 733 | 36.0 [2] | 52 | 39 | 34.5 [2] | 34 | 39 | 33.0 [2] | 24 | 21 | 21.0 | 26 | 16 | | | | 25.5 | 21 | 28 |
| Lower Sterling | 963 | 632 | 12.0 | 17 | 13 | 12.0 | 12 | 14 | 33.0 [2] | 24 | 21 | 18.0 | 22 | 13 | | | | 19.5 | 16 | 22 |
| Sears | 504 | 402 | 13.5 | 19 | 15 | | | | 25.5 | 18 | 16 | 9.0 | 11 | 7 | | | | 12.0 | 10 | 13 |
| Steel | 781 | 541 | 36.0 [2] | 52 | 39 | | | | 33.0 [2] | 24 | 21 | 15.0 | 18 | 11 | | | | 30.0 | 25 | 33 |
| McHenry (Stratton L&D) - Main Dam | 274 | 300 | 13.5 | 19 | 15 | | | | 19.5 | 14 | 13 | 7.5 | 9 | 6 | | | | 10.5 | 9 | 12 |
| McHenry (Stratton L&D) - Gates | 121 | 300 | 10.5 | 15 | 11 | | | | 16.5 | 12 | 11 | | | | | | | | | |
| Algonquin | 243 | 300 | 12.0 | 17 | 13 | | | | 7.5 | 5 | 5 | 6.0 | 7 | 4 | 6.0 | 2 | 6 | 15.0 | 13 | 17 |
| Carpentersville | 373 | 337 | 16.5 | 24 | 18 | 16.5 | 16 | 19 | 21.0 | 15 | 13 | 7.5 | 9 | 6 | | | | 13.5 | 11 | 15 |
| Elgin Kimball Street | 312 | 306 | 15.0 | 21 | 16 | | | | 10.5 | 8 | 7 | 6.0 | 7 | 4 | 9.0 | 3 | 9 | 13.5 | 11 | 15 |
| South Elgin | 361 | 331 | 24.0 | 34 | 26 | | | | 19.5 | 14 | 13 | 7.5 | 9 | 6 | | | | 18.0 | 15 | 20 |
| St. Charles | 289 | 300 | 15.0 | 21 | 16 | | | | 18.0 | 13 | 12 | 6.0 | 7 | 4 | 9.0 | 3 | 9 | 13.5 | 11 | 15 |
| Geneva | 450 | 375 | 18.0 | 26 | 20 | 18.0 | 18 | 20 | 24.0 | 17 | 15 | 9.0 | 11 | 7 | | | | 15.0 | 13 | 17 |
| Batavia | 372 | 336 | 22.5 | 32 | 24 | 22.5 | 22 | 25 | 21.0 | 15 | 13 | 7.5 | 9 | 6 | | | | 16.5 | 14 | 18 |
| North Aurora | 377 | 339 | 21.0 | 30 | 23 | 21.0 | 21 | 24 | 21.0 | 15 | 13 | 7.5 | 9 | 6 | | | | 16.5 | 14 | 18 |
| Aurora East | 167 | 300 | 18.0 | 26 | 20 | | | | 15.0 | 11 | 10 | 4.5 | 6 | 3 | 10.5 | 4 | 11 | 15.0 | 13 | 17 |
| Montgomery | 327 | 314 | 12.0 | 17 | 13 | | | | 19.5 | 14 | 13 | 7.5 | 9 | 6 | 6.0 | 2 | 6 | 12.0 | 10 | 13 |
| Yorkville | 519 | 410 | 18.0 | 26 | 20 | | | | 25.5 | 18 | 16 | 10.5 | 13 | 8 | | | | 15.0 | 13 | 17 |
| Hofmann | 264 | 300 | 12.0 | 17 | 13 | | | | 18.0 | 13 | 12 | 6.0 | 7 | 4 | | | | 12.0 | 10 | 13 |
| Danville | 220 | 300 | 10.5 | 15 | 11 | | | | 22.5 | 16 | 14 | 6.0 | 7 | 4 | 4.5 | 2 | 5 | 12.0 | 10 | 13 |
| Riverside Park | 441 | 371 | 12.0 | 17 | 13 | 12.0 | 12 | 14 | 22.5 | 16 | 14 | 9.0 | 11 | 7 | | | | | | |
| Petersberg | 116 | 300 | 10.5 | 15 | 11 | | | | 16.5 | 12 | 11 | 10.5 | 13 | 8 | | | | 10.5 | 9 | 12 |

***Standard Sizing for Other Signs***

| Sign | Largest Letter Size (inches) | Height (ft) | Max Width (ft) |
|---|---|---|---|
| Sign 3 | 3.0 | 6 | 3 |
| Sign 4 | 3.0 | 5 | 3 |
| Sign 4A | 3.0 | 3 | 3 |
| Sign 5 | 1.0 | 1 | 2 |
| Sign P3 | 2.0 | 2 | 1 |

Notes

[1] Multi-part signs greater than 24 feet high will be assumed to be mounted separately.
[2] Font size reduced to prevent individual sign parts from exceeding 24 x 40 feet dimensions
[3] See Table 2.3-2 for dimension 1.5*A

*Table 2.3-4* – Signage Opinions of Cost

| Dam | Opinion of Cost [1] Without Optional Signs | | Opinion of Cost [1] With Optional Signs | |
|---|---|---|---|---|
| Momence | $ | 55,000 | $ | 62,000 |
| Kankakee | $ | 76,000 | $ | 92,000 |
| Wilmington | $ | 195,000 | $ | 266,000 |
| Wilmington Millrace | $ | 50,000 | $ | 56,000 |
| Oregon | $ | 177,000 | $ | 300,000 |
| Sinnissippi | $ | 277,000 | $ | 416,000 |
| Lower Sterling | $ | 152,000 | $ | 170,000 |
| Sears | $ | 77,000 | $ | 86,000 |
| Steel | $ | 242,000 | $ | 320,000 |
| McHenry (Stratton L&D) | $ | 73,000 | $ | 88,000 |
| Algonquin | $ | 63,000 | $ | 71,000 |
| Carpentersville | $ | 86,000 | $ | 108,000 |
| Elgin Kimball Street | $ | 72,000 | $ | 92,000 |
| South Elgin | $ | 108,000 | $ | 142,000 |
| St. Charles | $ | 70,000 | $ | 83,000 |
| Geneva | $ | 94,000 | $ | 121,000 |
| Batavia | $ | 103,000 | $ | 146,000 |
| North Aurora | $ | 106,000 | $ | 144,000 |
| Aurora East | $ | 78,000 | $ | 97,000 |
| Montgomery | $ | 71,000 | $ | 79,000 |
| Yorkville | $ | 99,000 | $ | 118,000 |
| Hofmann | $ | 57,000 | $ | 65,000 |
| Danville | $ | 62,000 | $ | 68,000 |
| Riverside Park | $ | 24,000 | $ | 30,000 |
| Petersberg | $ | 57,000 | $ | 63,000 |
| **Total Cost** | **$** | **2,524,000** | **$** | **3,283,000** |

[1] These costs are based on limited information and are for planning purposes only. They do not include costs associated with land acquisition, maintenance, or difficult installation.

## 2.4 *Kankakee River Dams Signage Plans*

The following pages include signage plans for Kankakee River dams.



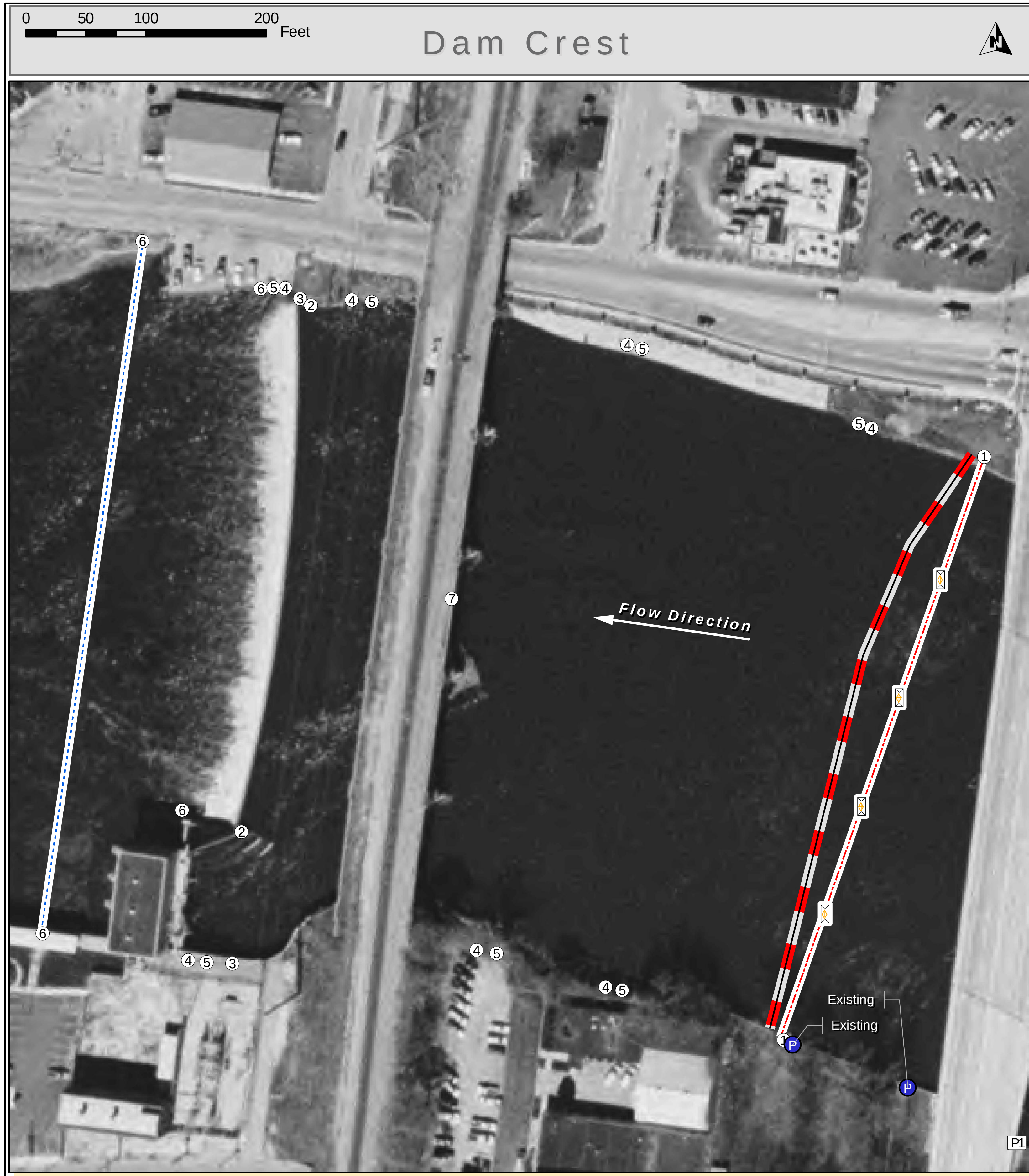
***Signage Plan***

Momence Dam - Kankakee River

*Momence, Illinois*

CTE | AECOM

Figure 2.4.1
Momence





**Signage Plan**

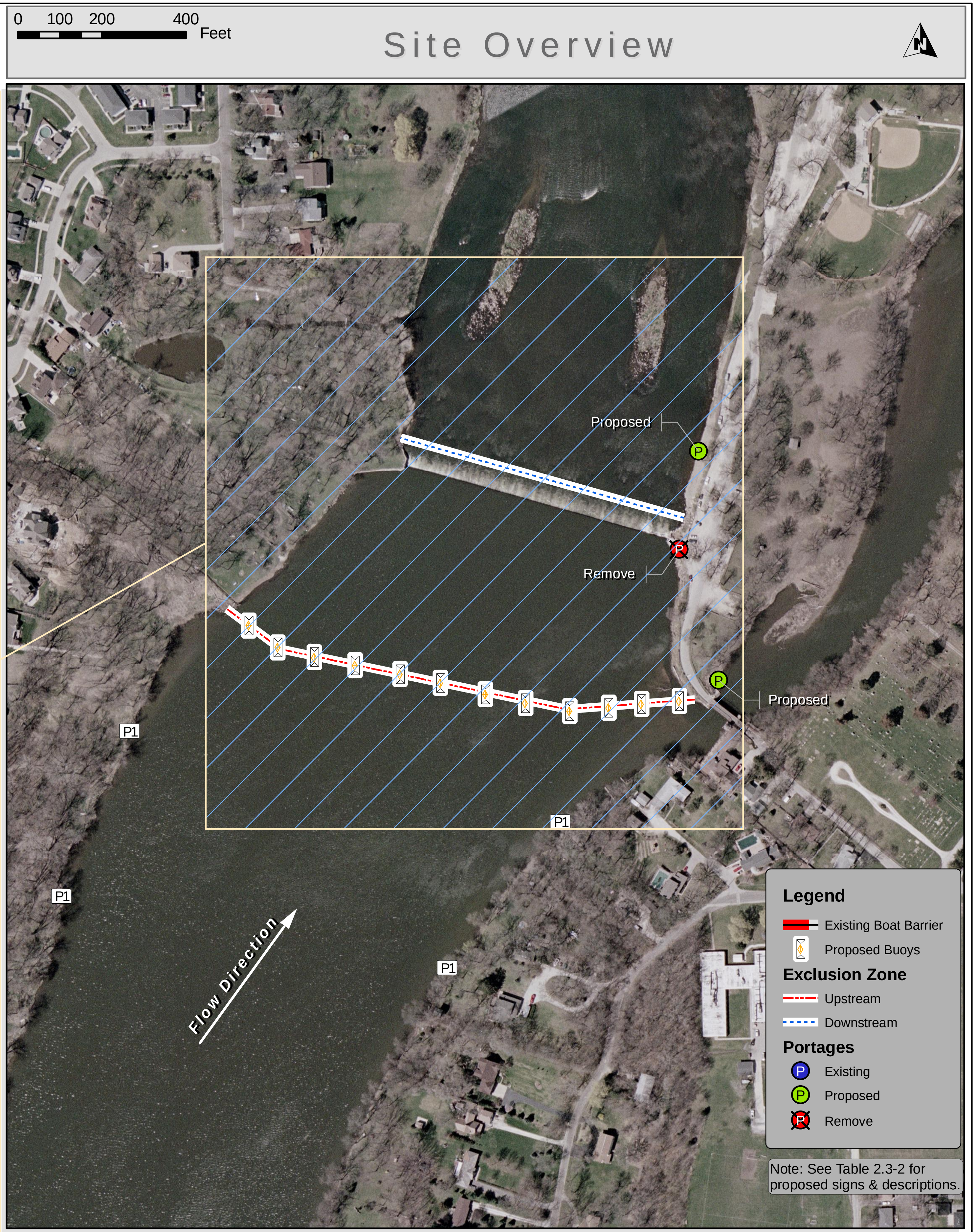
Kankakee Dam - Kankakee River

*Kankakee, Illinois*

CTE | AECOM

Figure 2.4.2

Kankakee


***Signage Plan***

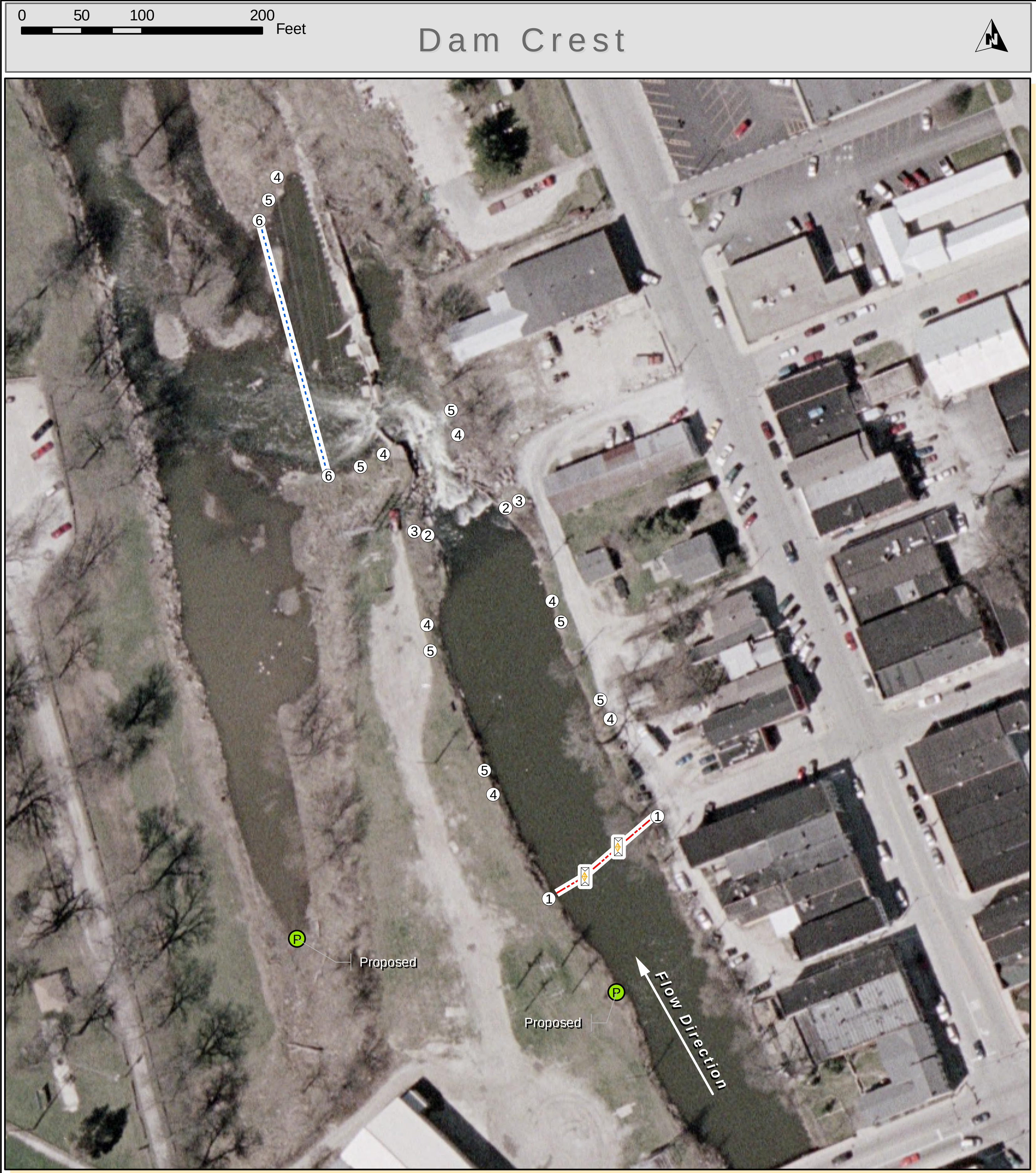
Wilmington Dam - Kankakee River

*Wilmington, Illinois*

CTE | AECOM

Figure 2.4.3

Wilmington





**Signage Plan**

Wilmington Millrace Dam - Kankakee River

*Wilmington, Illinois*

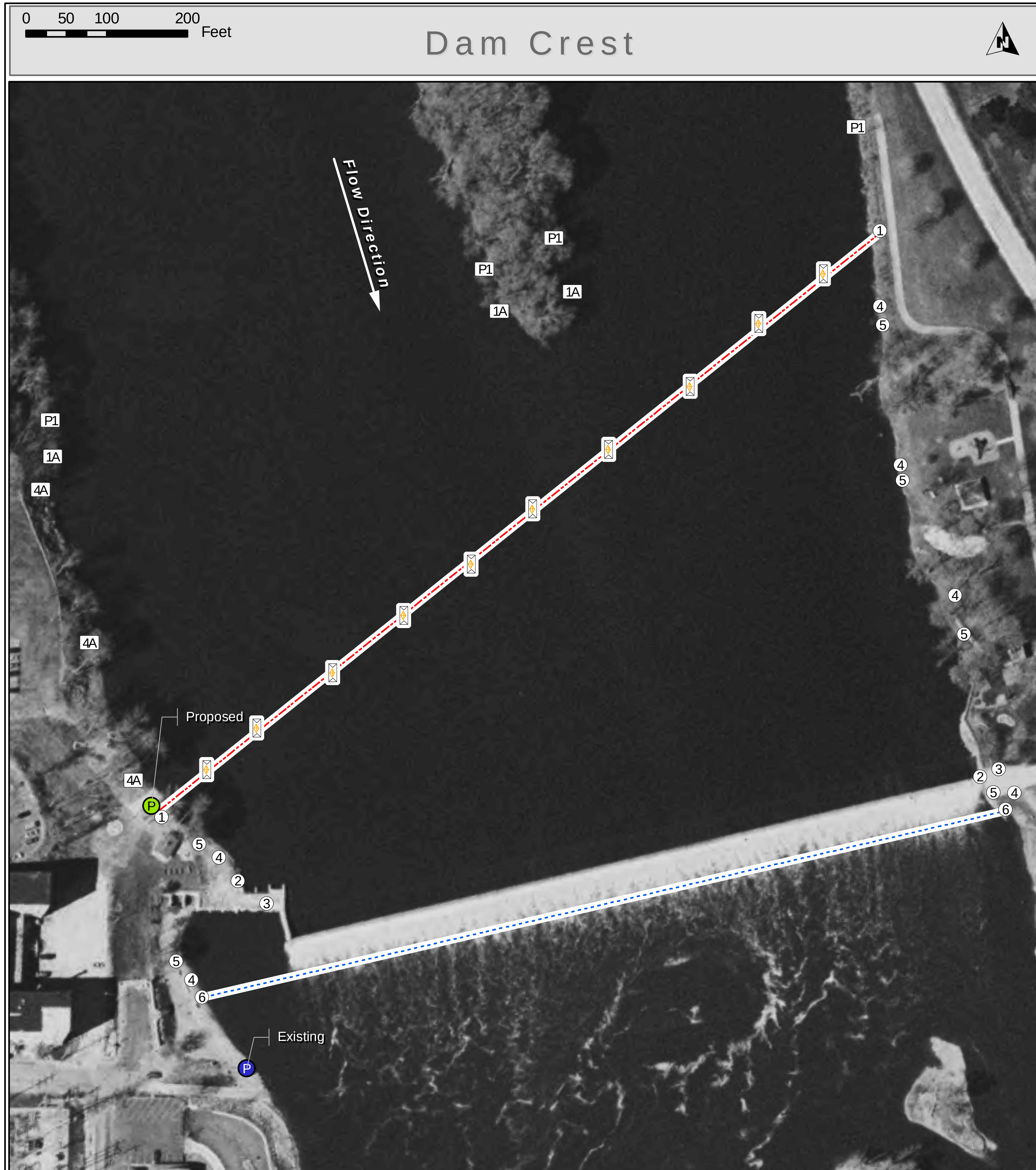
CTE | AECOM

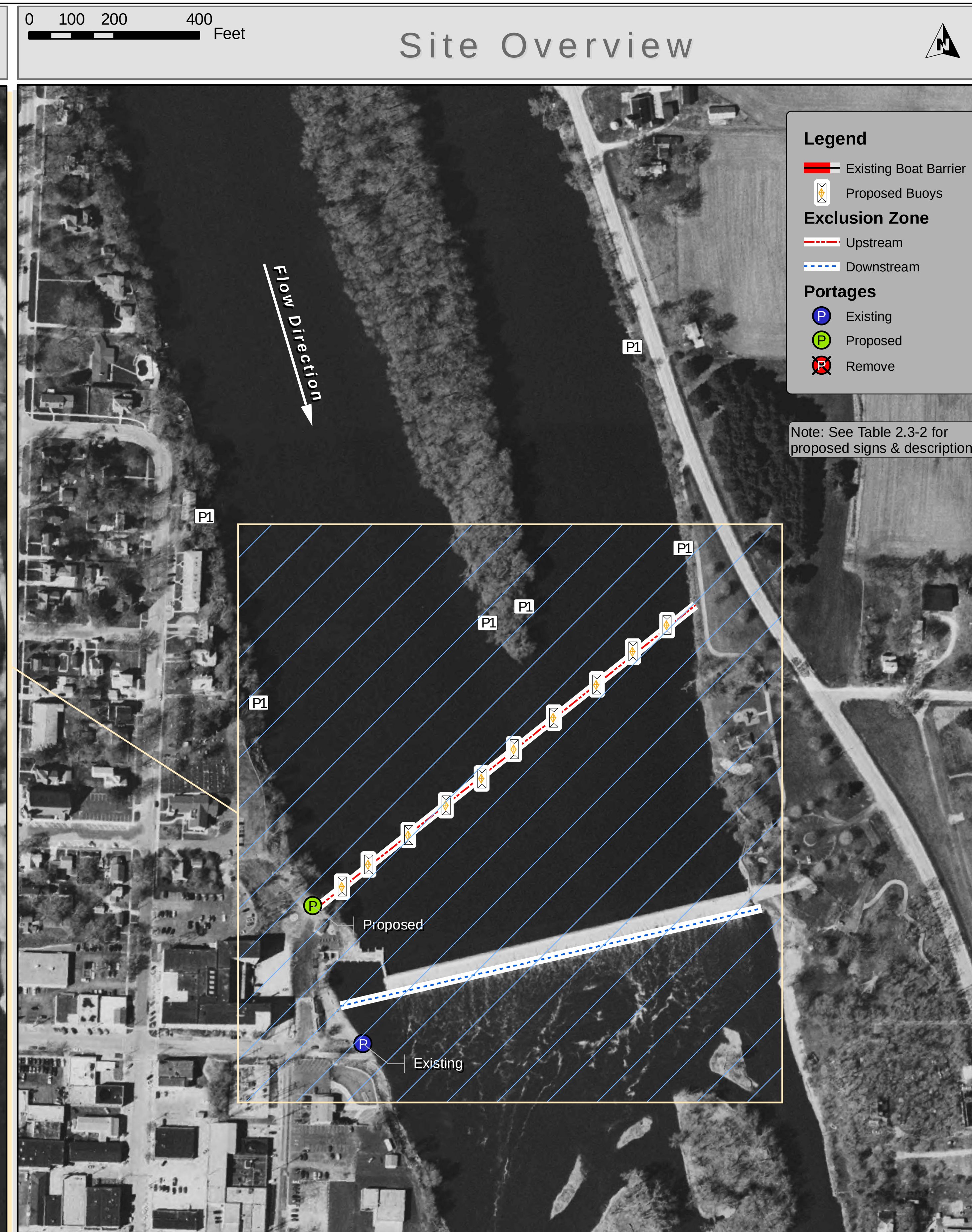
Figure 2.4.4

Wilmington Millrace

## 2.5 *Rock River Dams Signage Plans*

The following pages include signage plans for Rock River dams.




***Signage Plan***

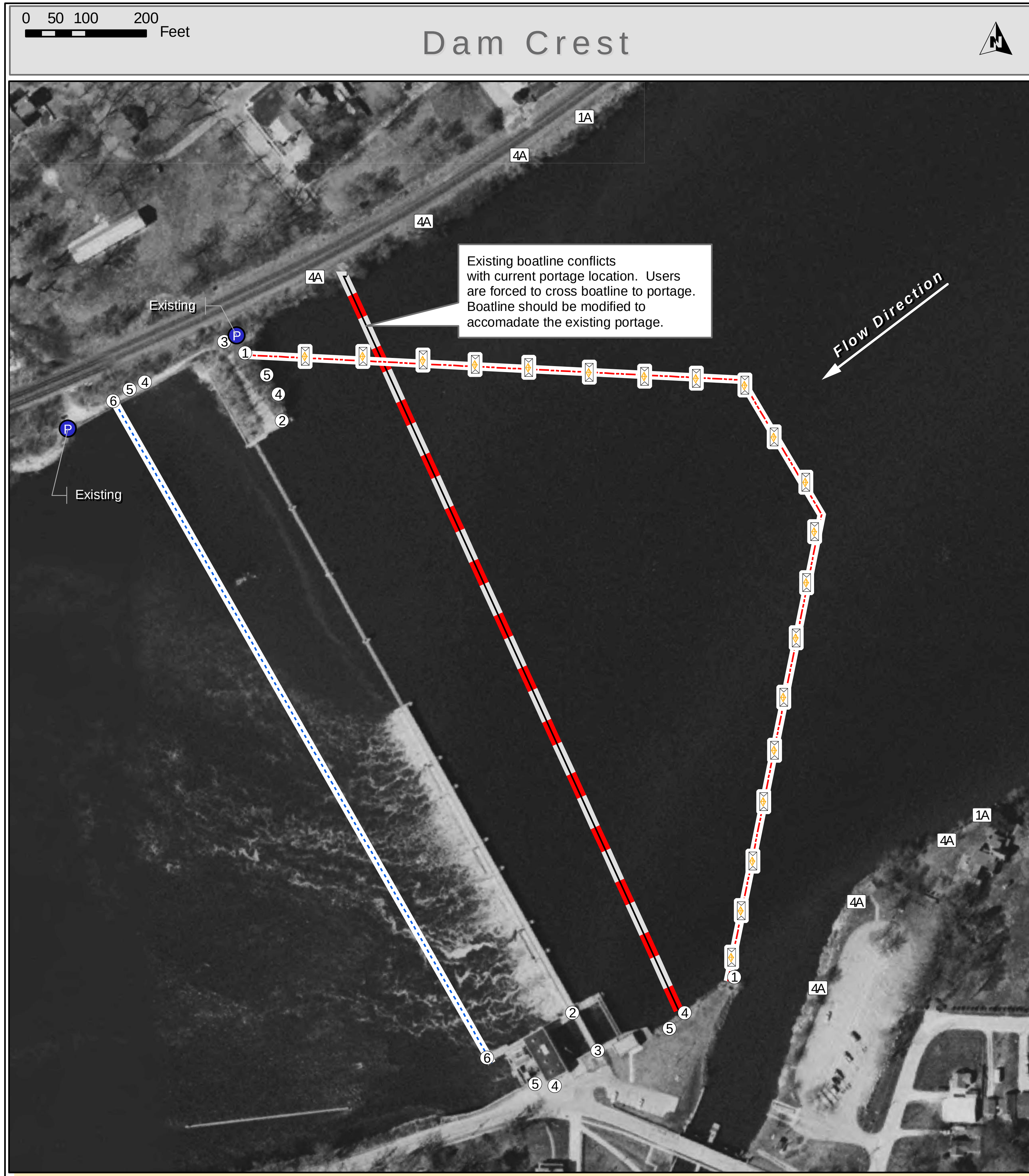
Oregon Dam - Rock River

*Oregon, Illinois*

CTE | AECOM

Figure 2.5.1

Oregon





***Signage Plan***

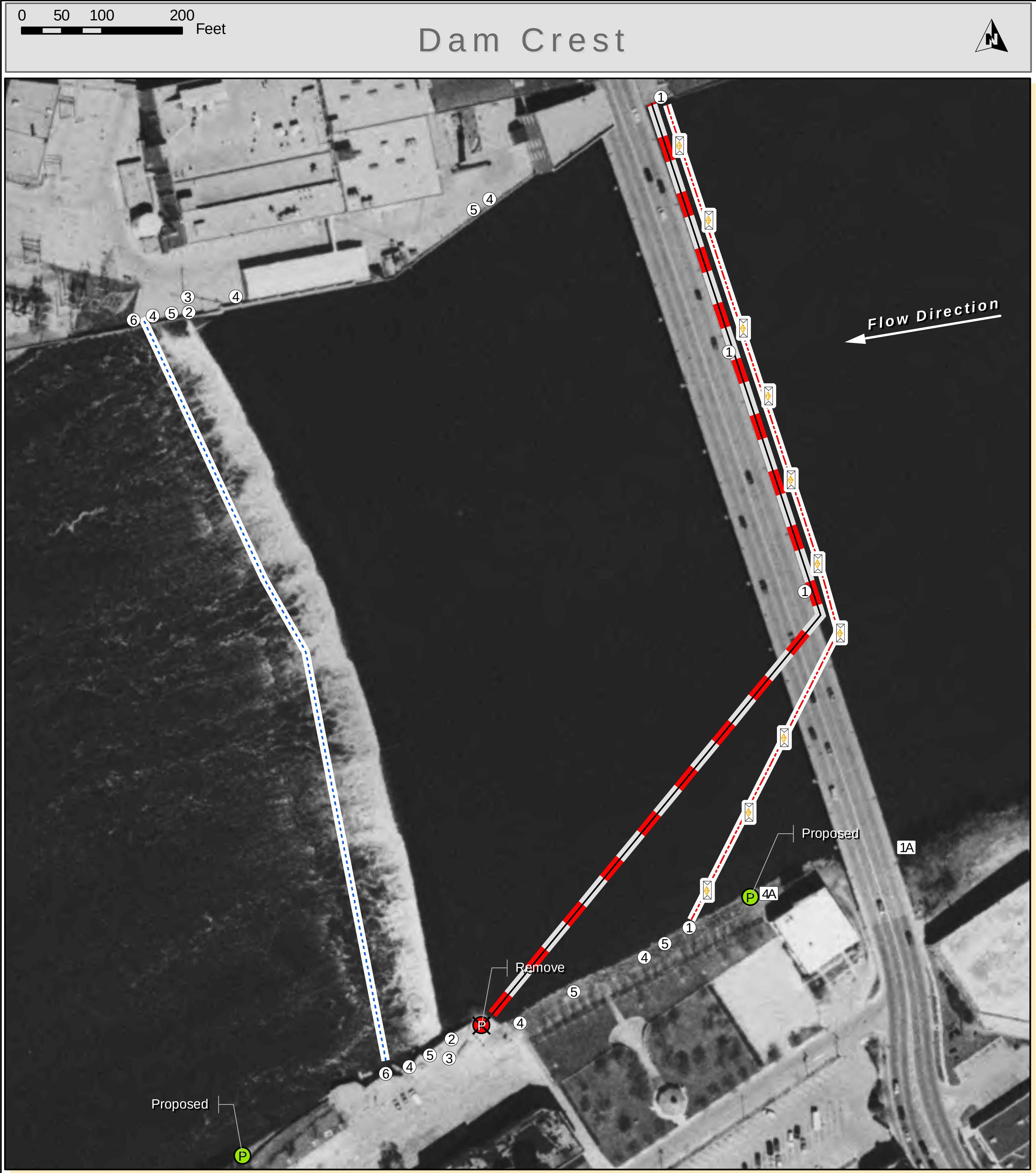
Sinnissippi Dam - Rock River

*Rockfalls, Illinois*

CTE | AECOM

Figure 2.5.2

Sinnissippi





**Signage Plan**

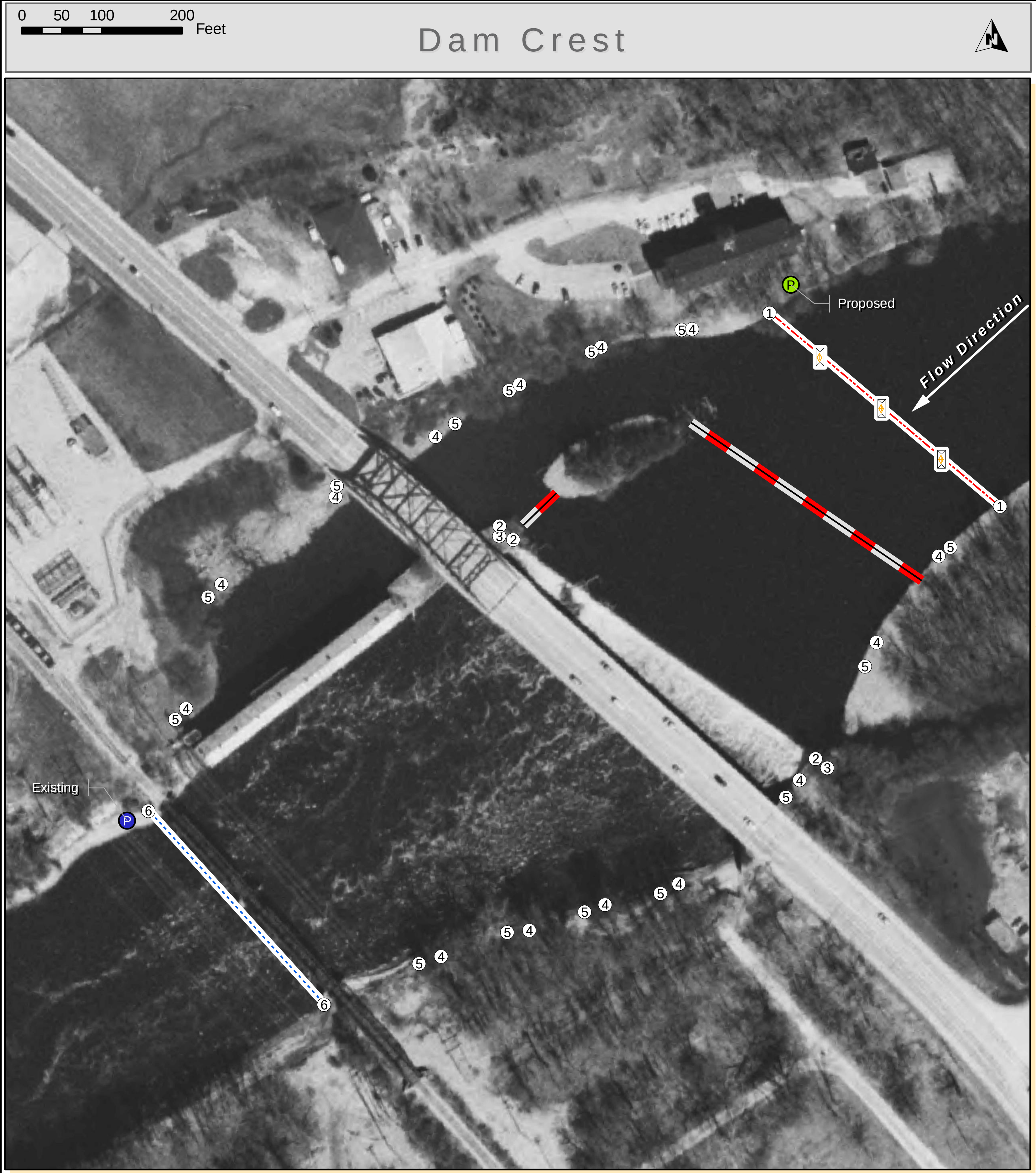
Lower Sterling / Lower Rock Falls Dam - Rock River

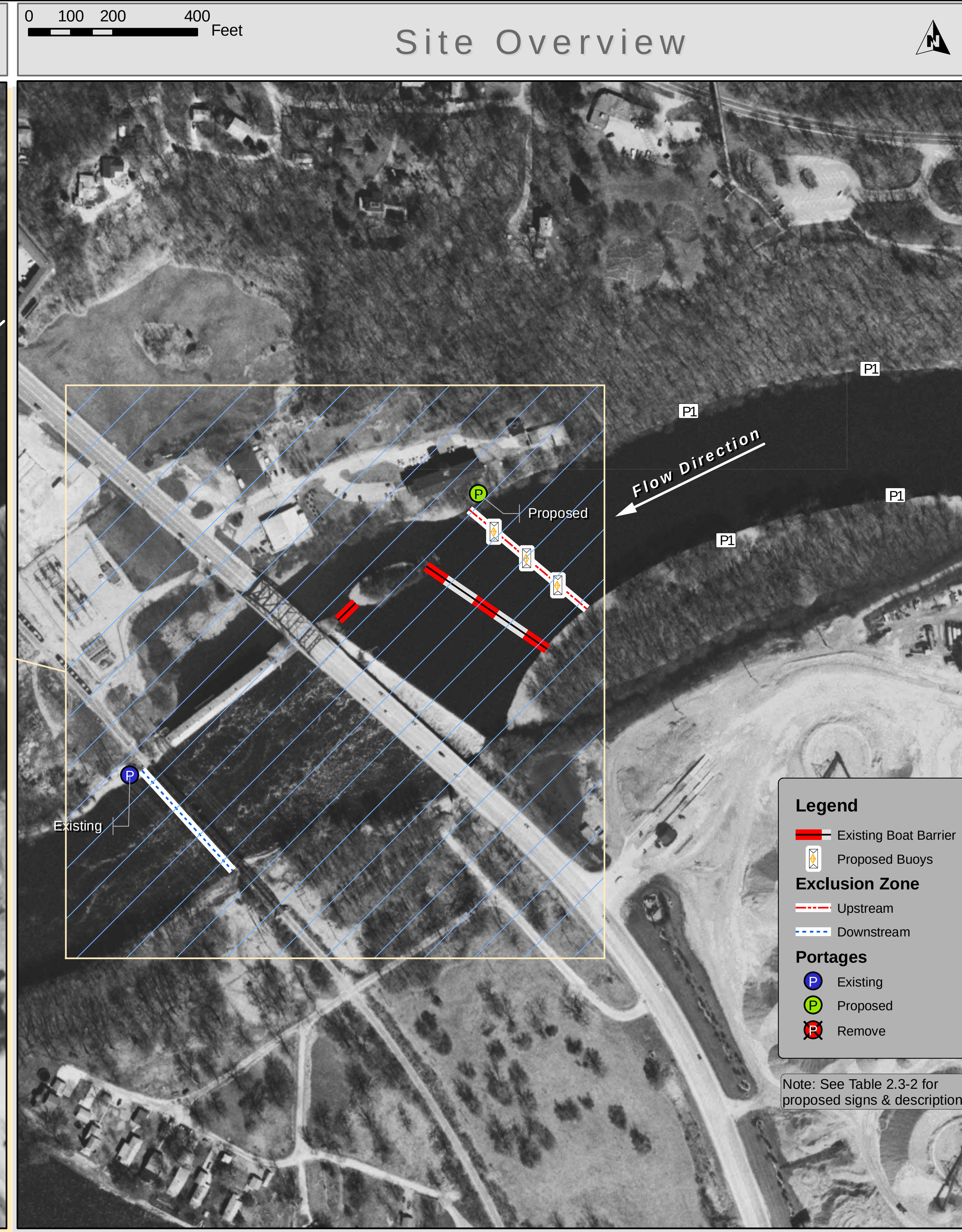
*Rock Falls, Illinois*

CTE | AECOM

Figure 2.5.3

Lower Sterling




***Signage Plan***

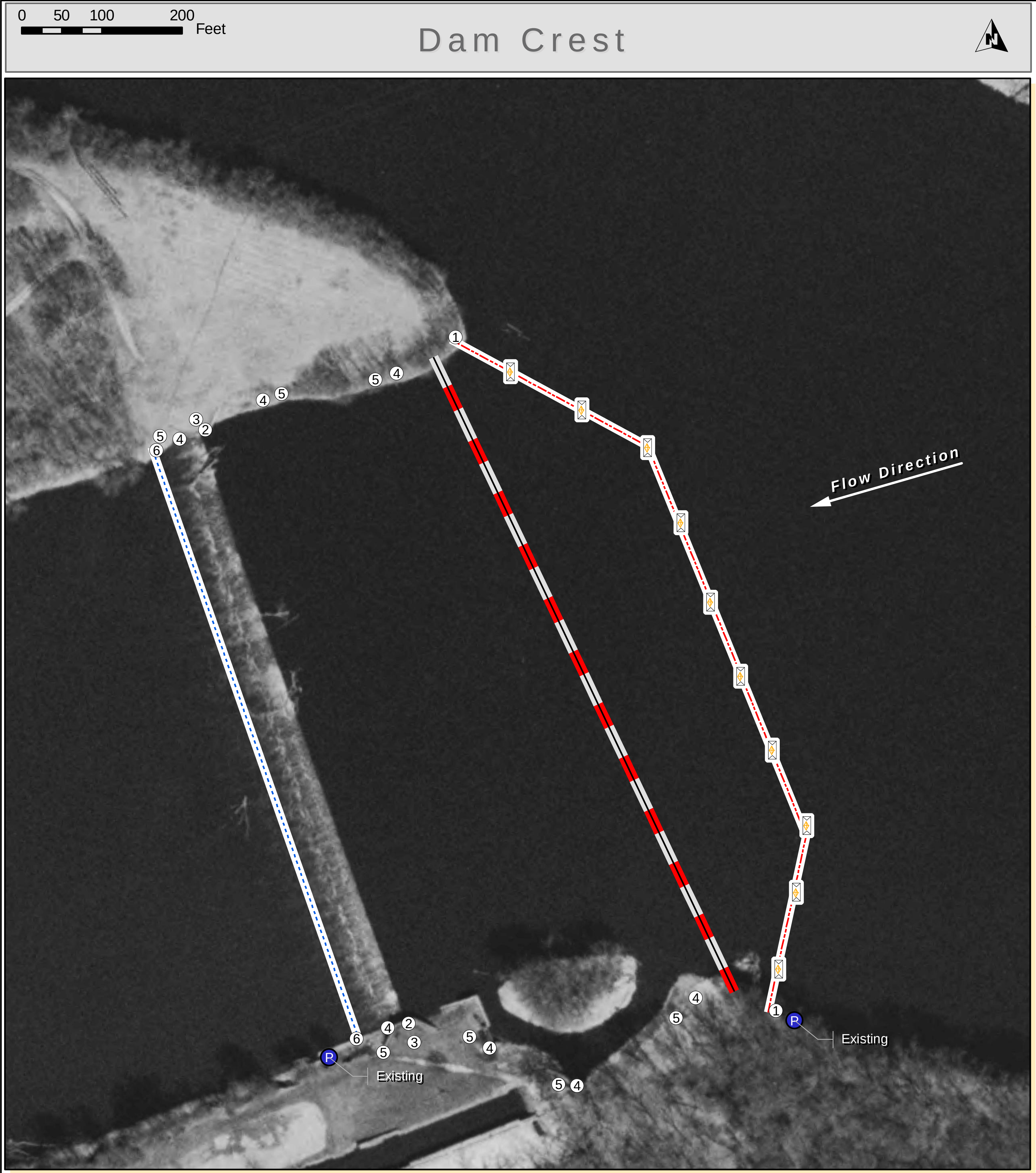
Sears Dam - Rock River

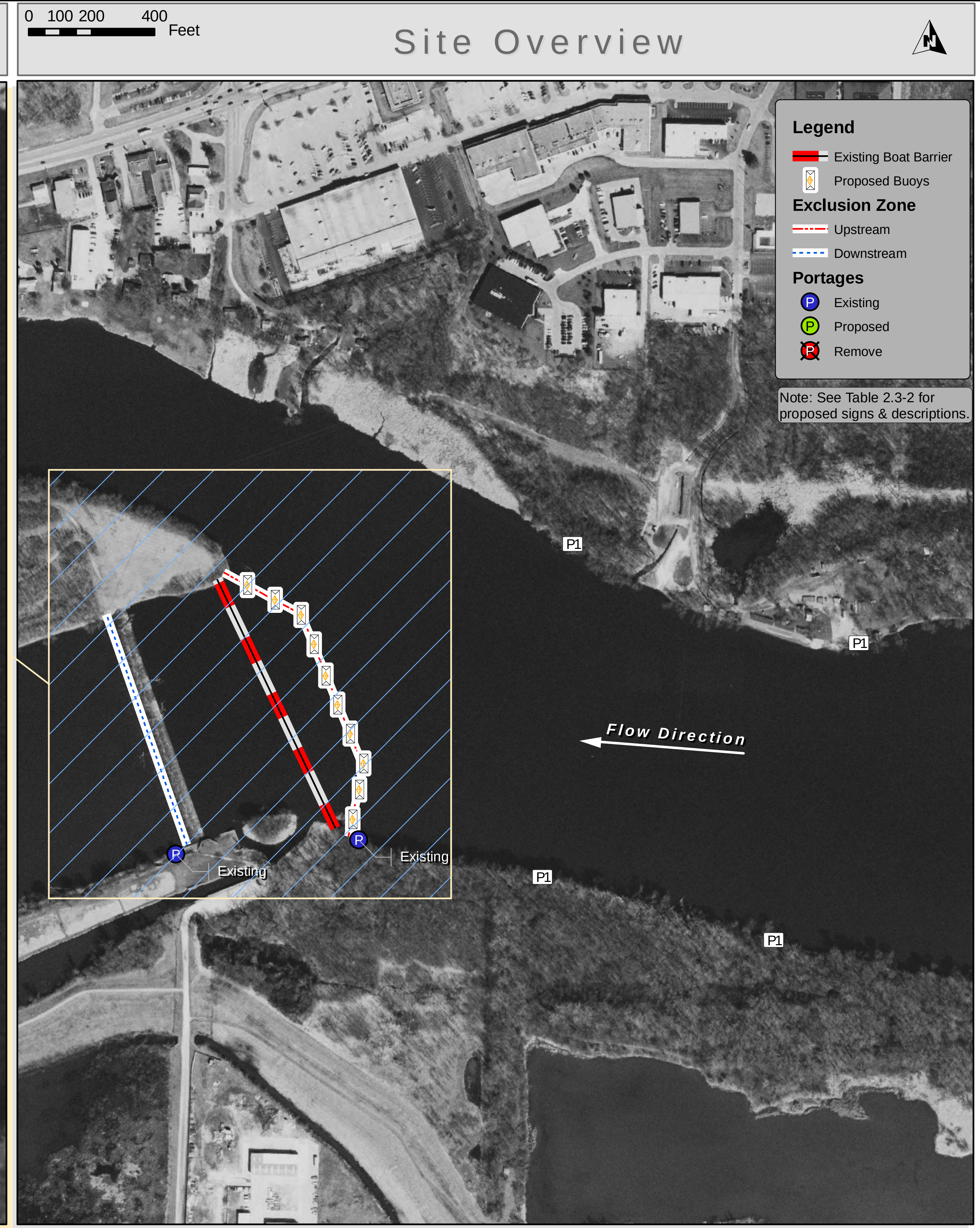
*Milan, Illinois*

CTE | AECOM

Figure 2.5.4

Sears




***Signage Plan***

Steel Dam - Rock River

*Milan, Illinois*

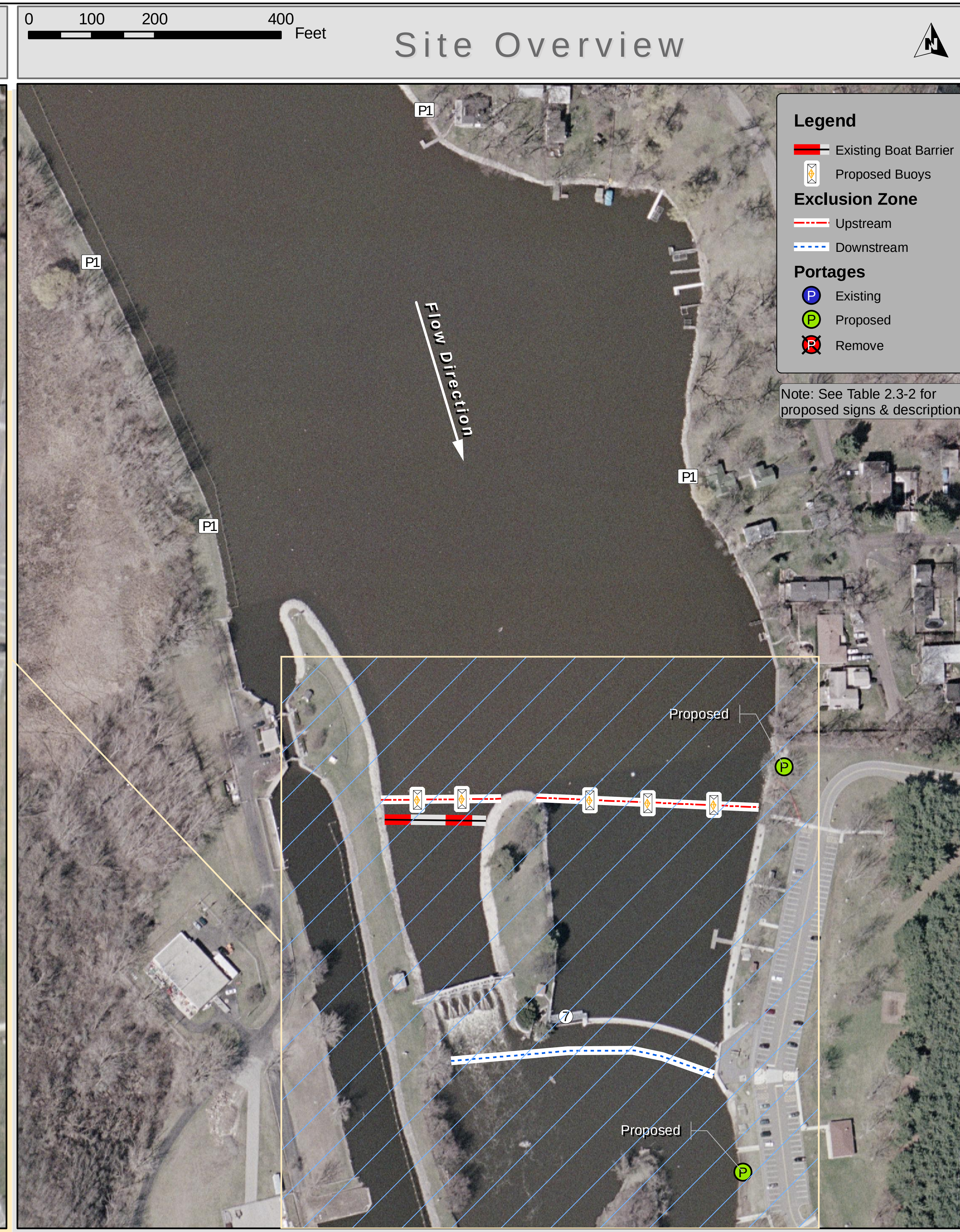
CTE | AECOM

Figure 2.5.5

Steel

## 2.6 *Fox River Dams Signage Plans*

The following pages include signage plans for Fox River dams.


**Signage Plan**

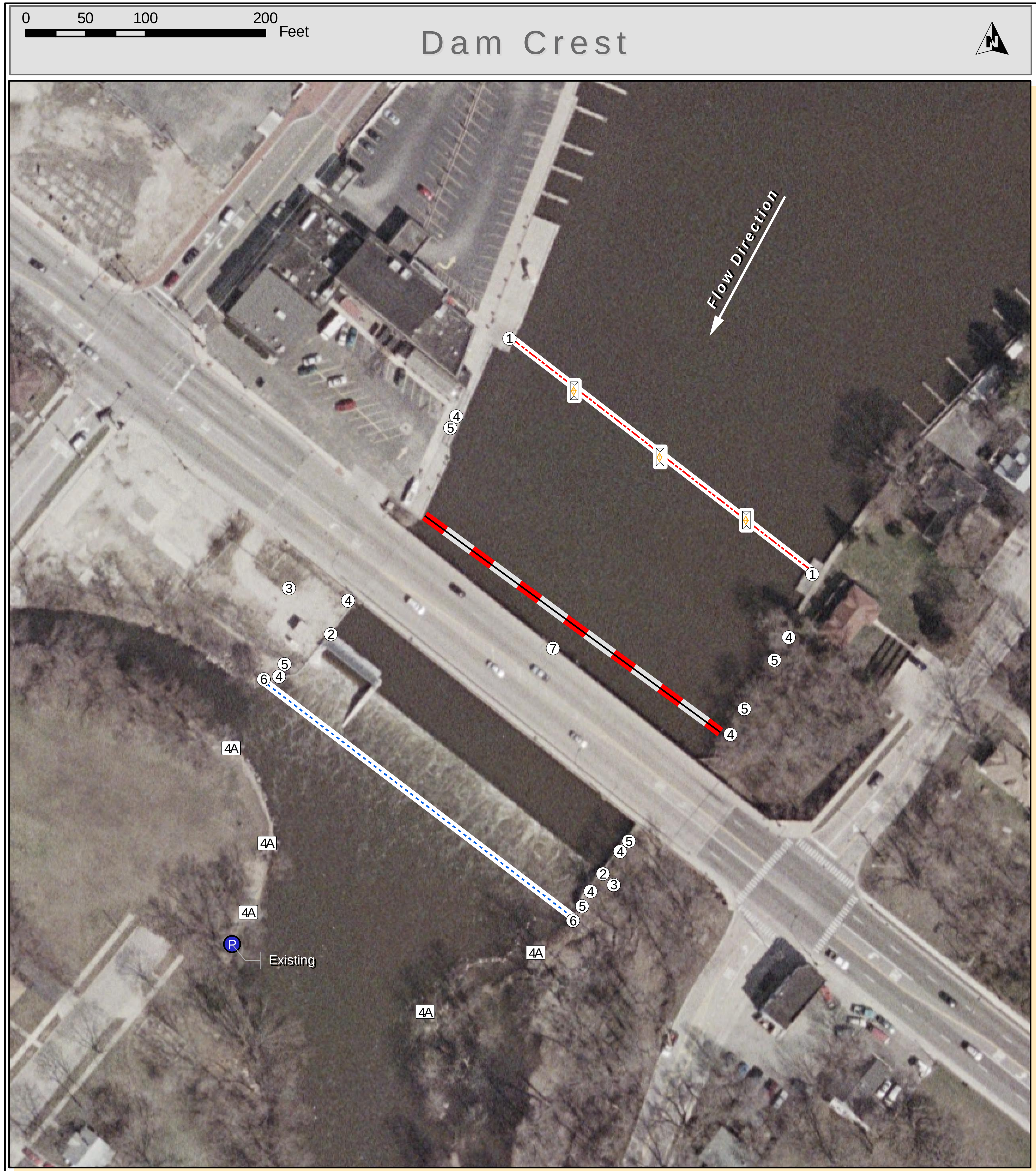
Mc Henry (Stratton L and D) Dam - Fox River

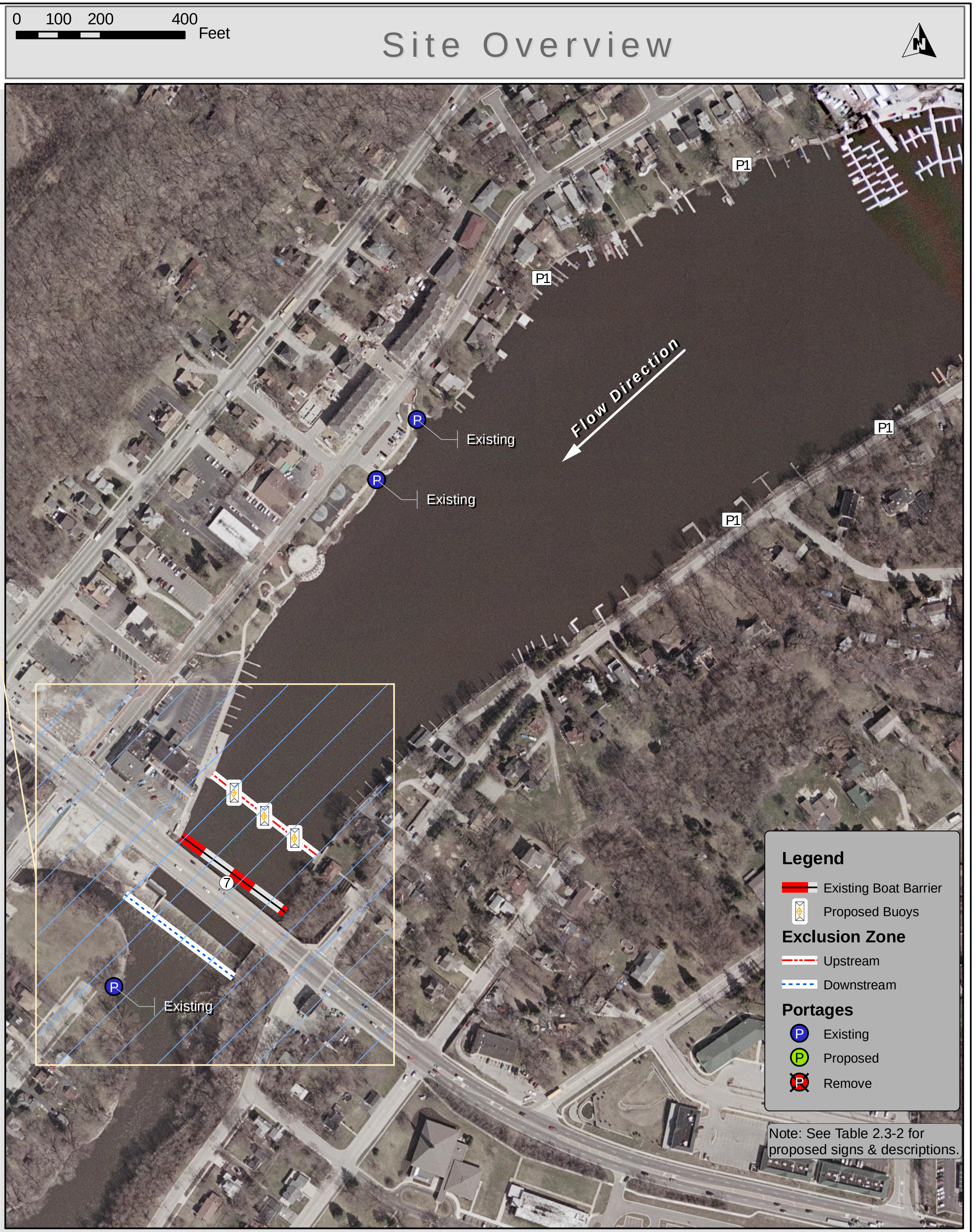
*Mc Henry, Illinois*

CTE | AECOM

Figure 2.6.1

McHenry/Stratton



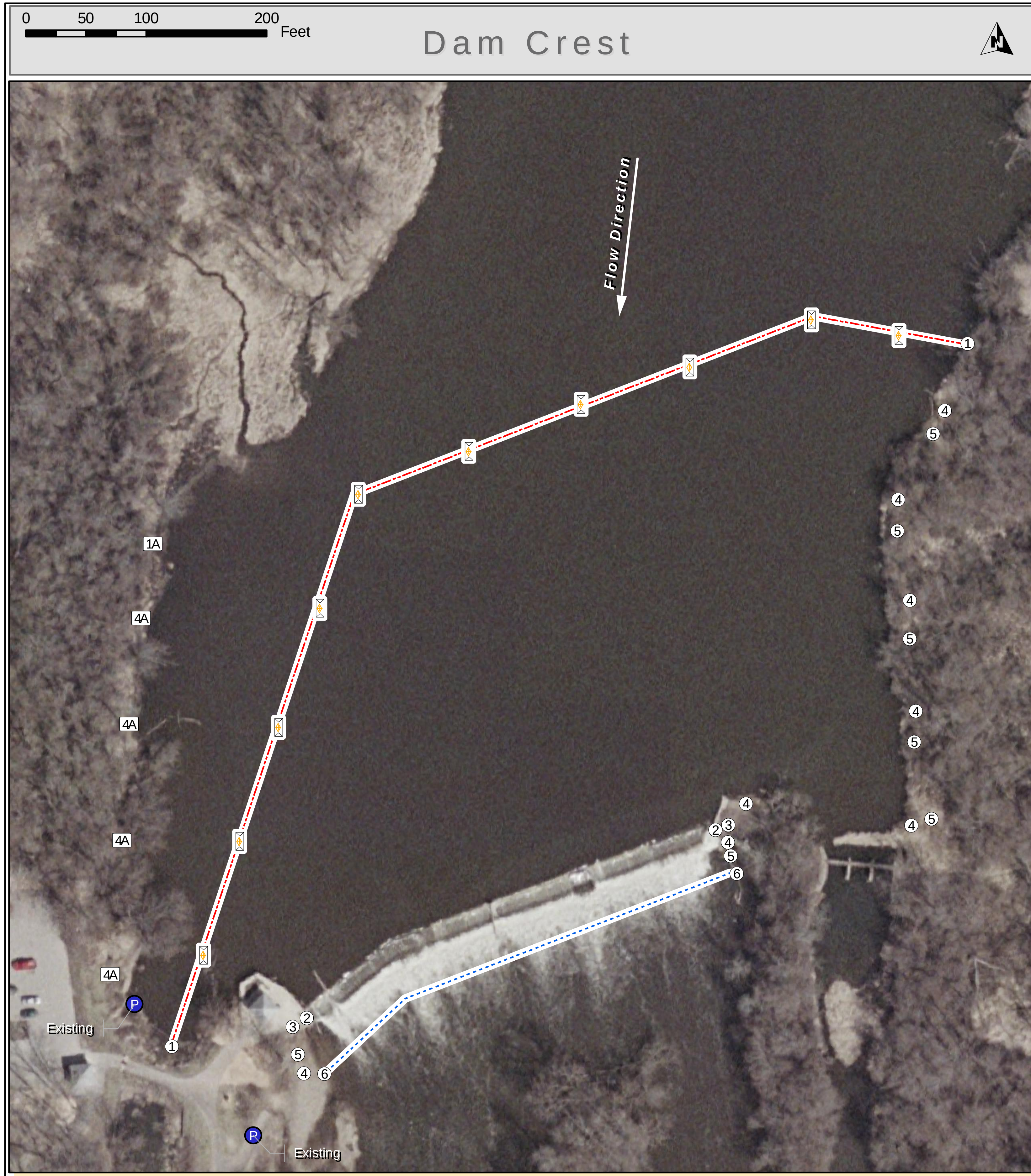

**Signage Plan**

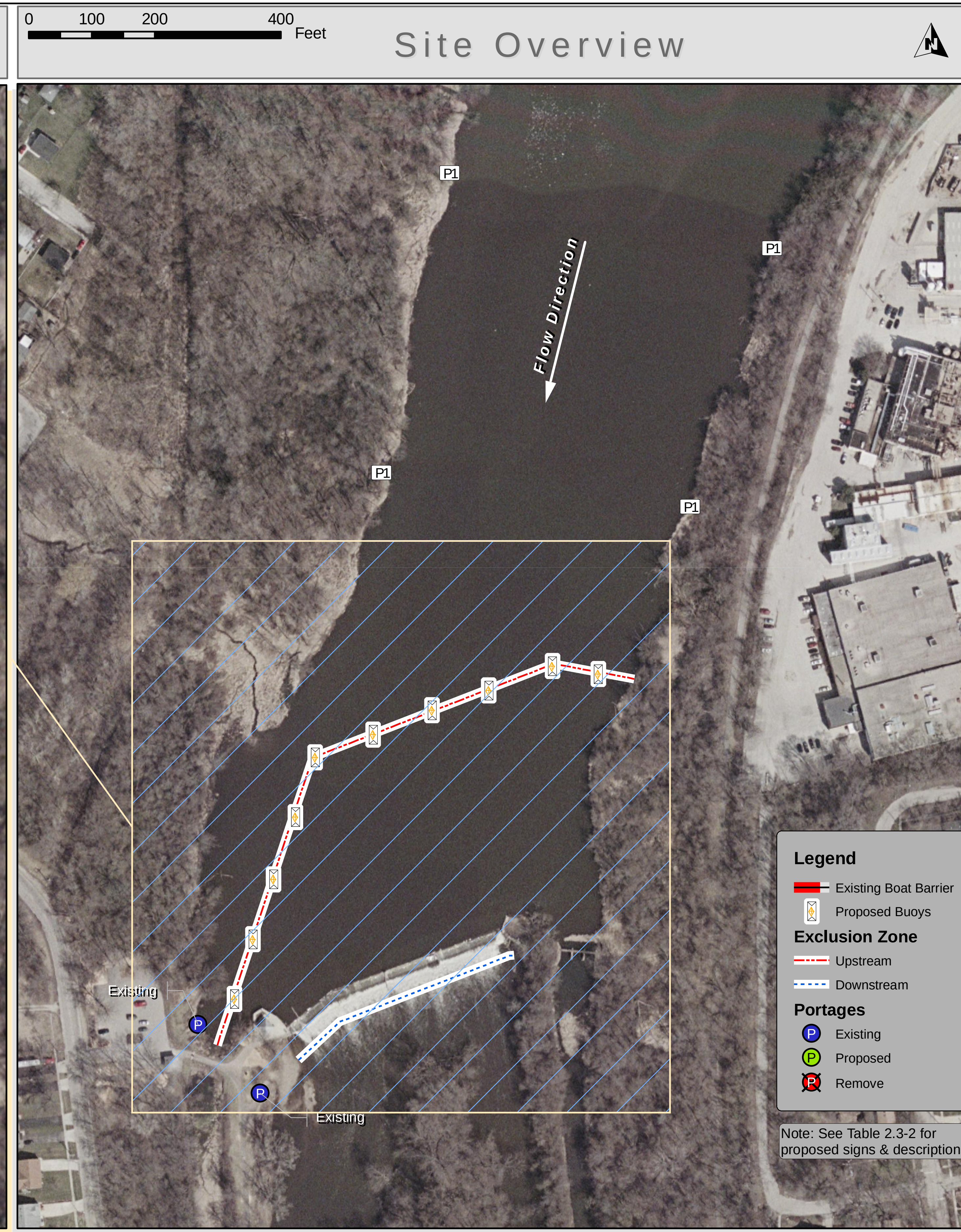
Algonquin Dam - Fox River

*Algonquin, Illinois*

CTE | AECOM

Figure 2.6.2
Algonquin



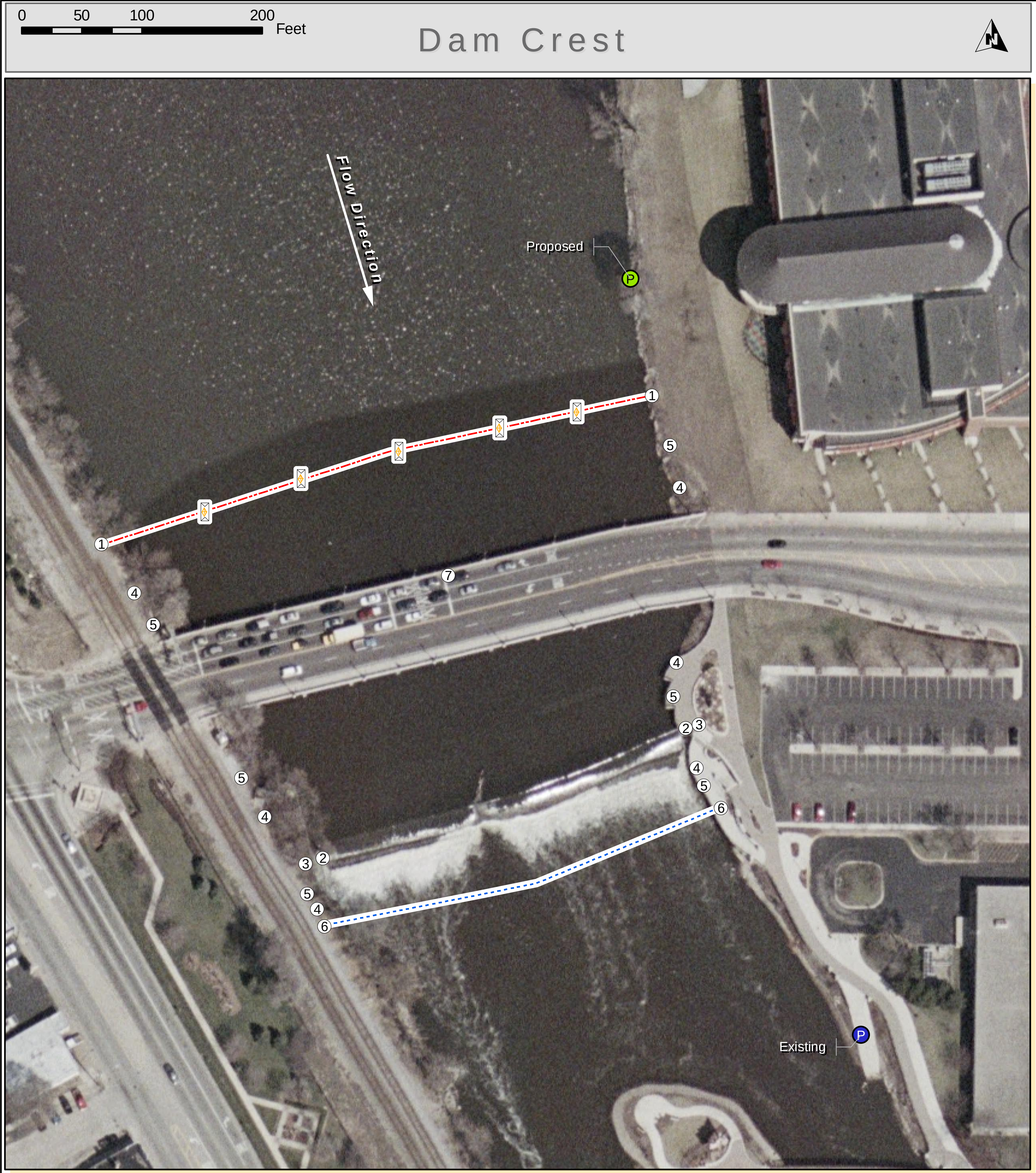

**_Signage Plan_**

Carpentersville Dam - Fox River

*Carpentersville, Illinois*

CTE | AECOM

Figure 2.6.3
Carpentersville





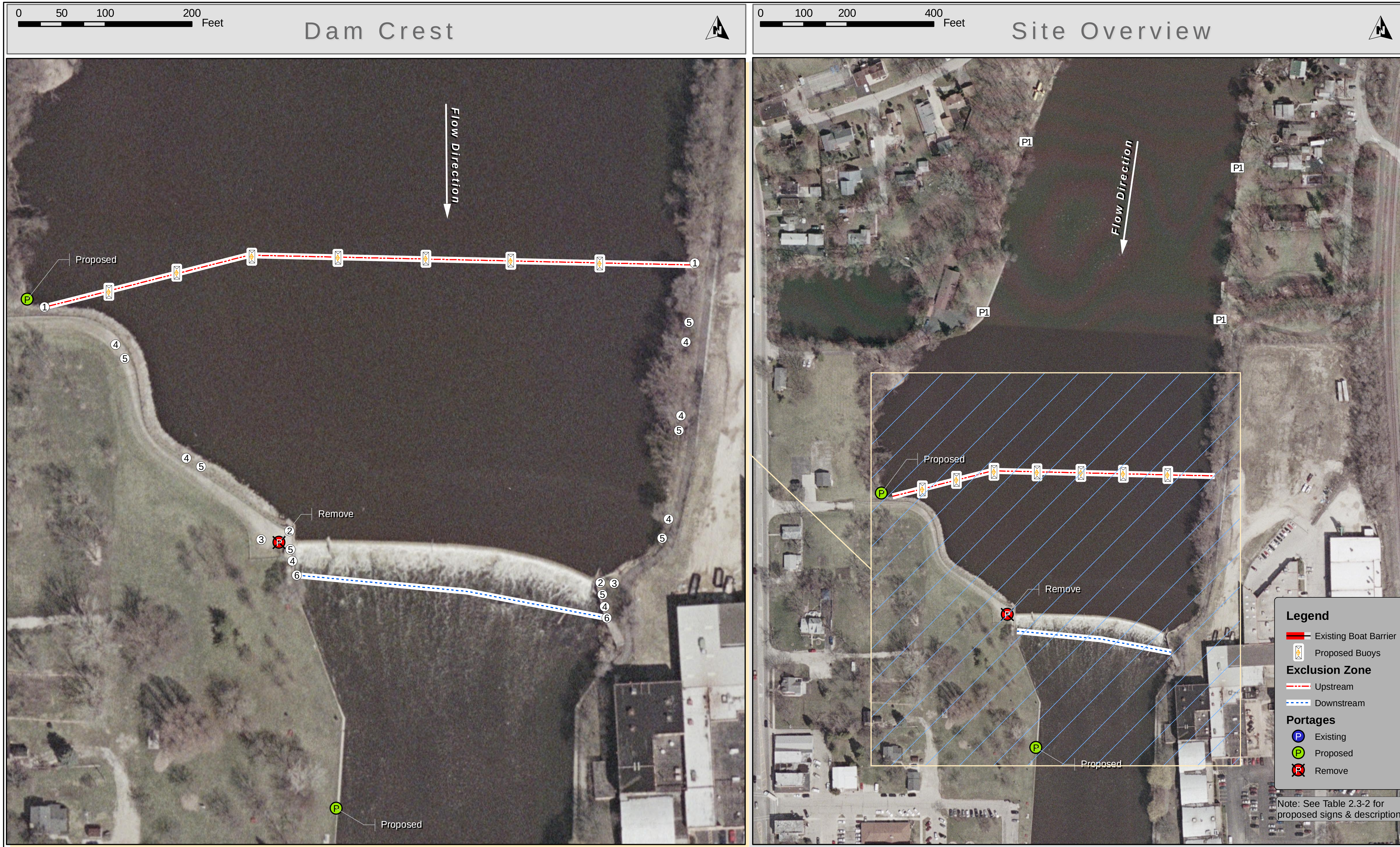
***Signage Plan***

Elgin Kimball Street Dam - Fox River

*Elgin, Illinois*

CTE | AECOM

Figure 2.6.4
Elgin Kimball




***Signage Plan***

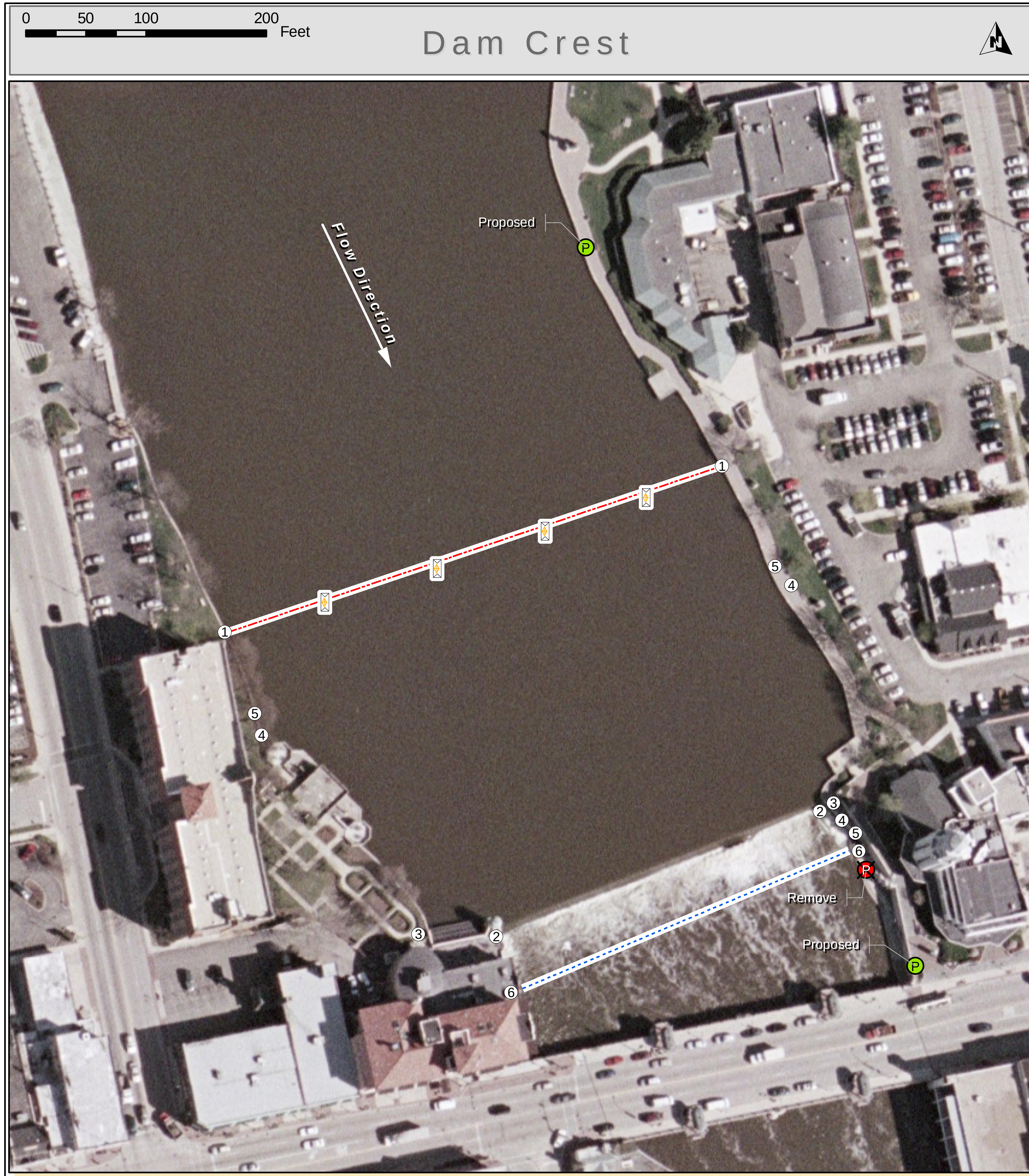
South Elgin Dam - Fox River

*Elgin, Illinois*

CTE | AECOM

Figure 2.6.5

South Elgin





**_Signage Plan_**

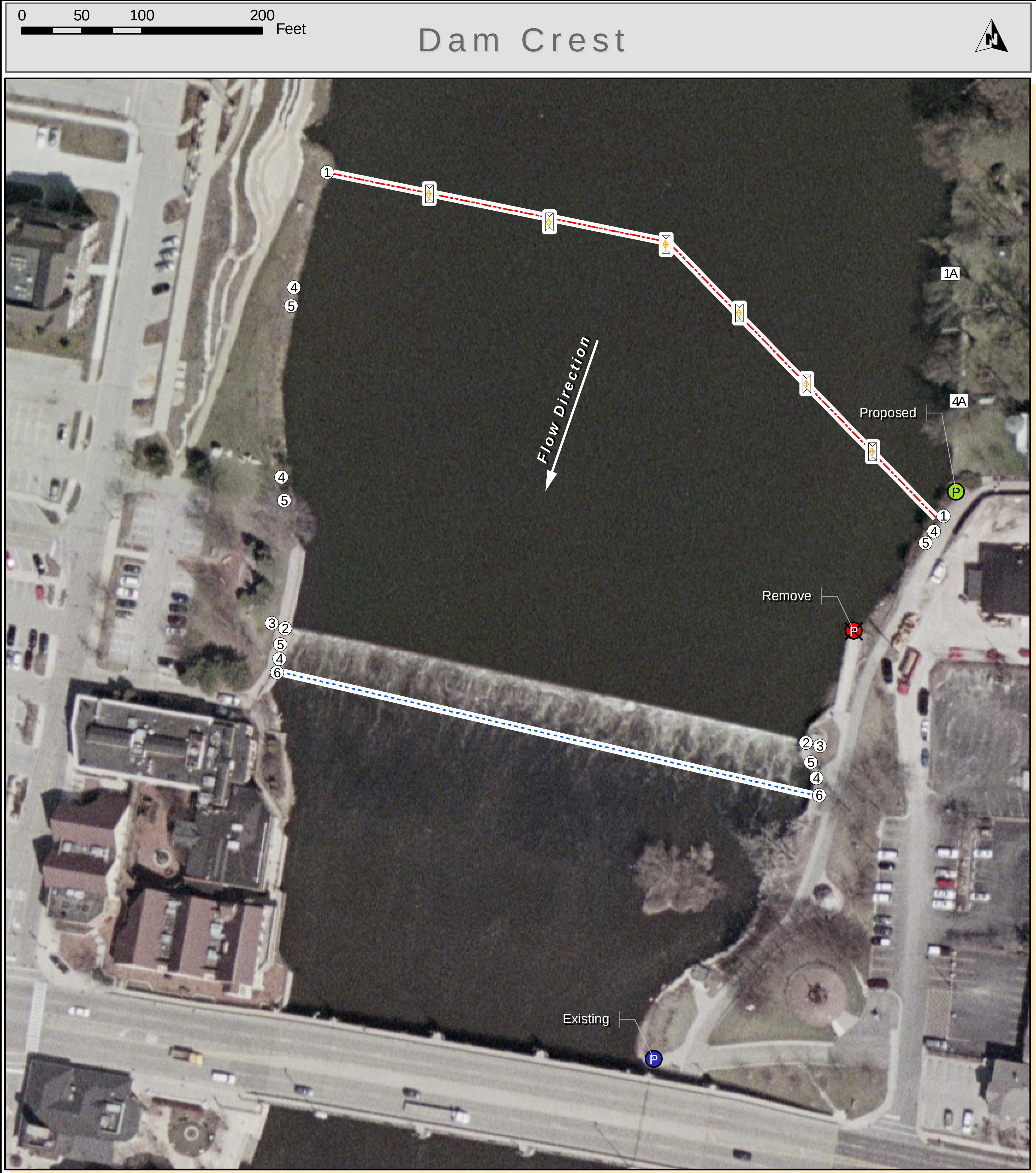
St. Charles Dam - Fox River

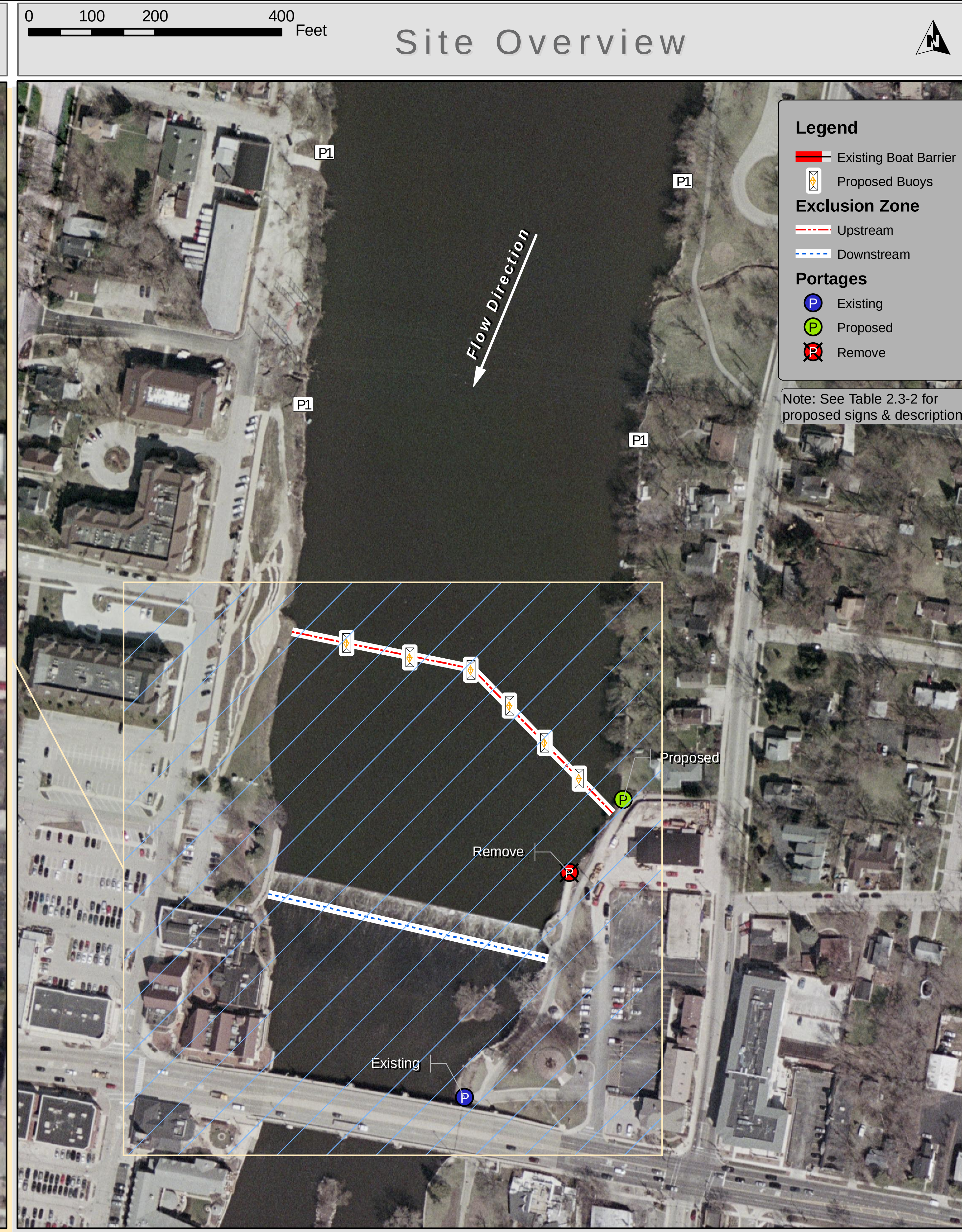
*St. Charles, Illinois*

CTE | AECOM

Figure 2.6.6

St. Charles



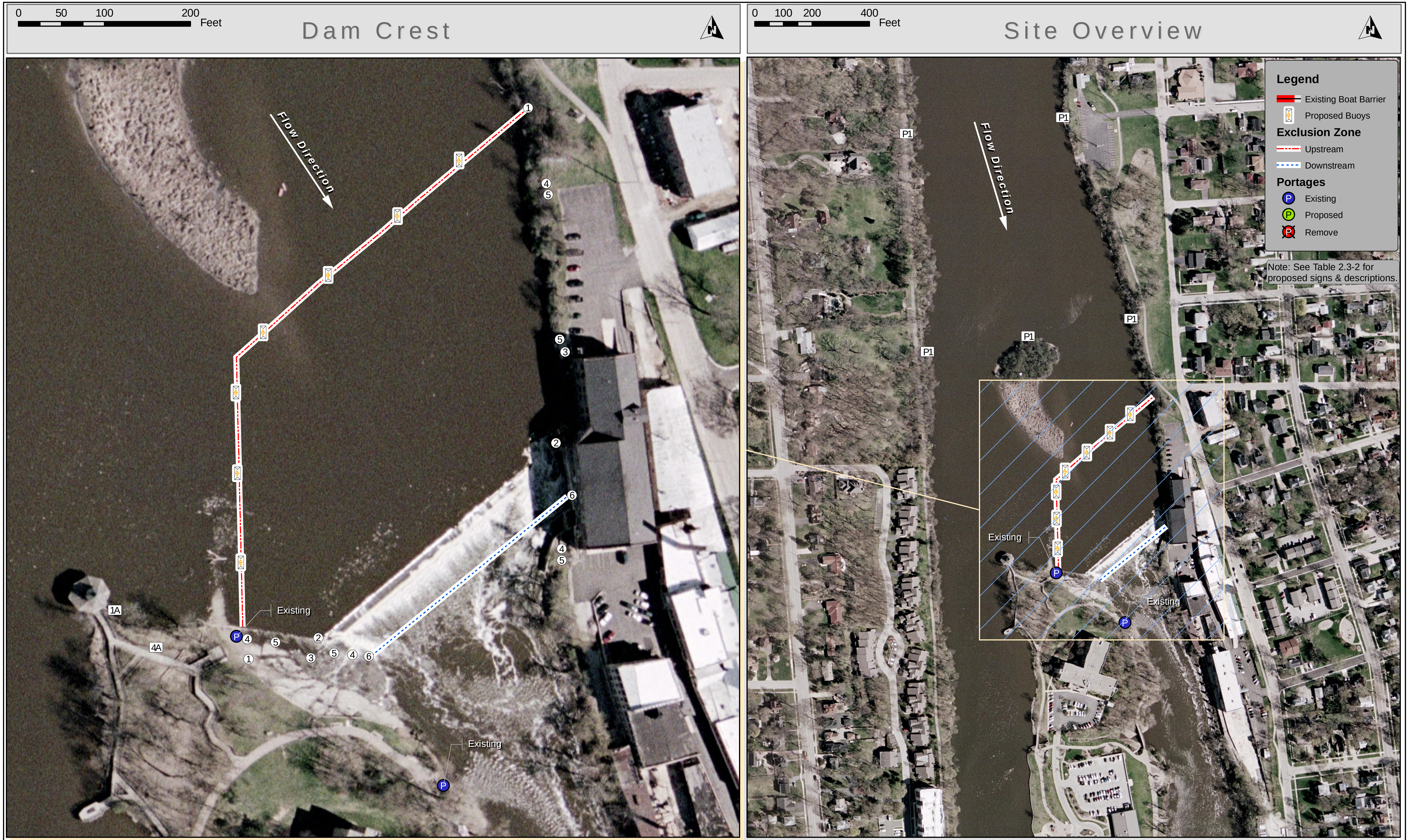

**Signage Plan**

Geneva Dam - Fox River

*Geneva, Illinois*

CTE | AECOM

Figure 2.6.7
Geneva




**_Signage Plan_**

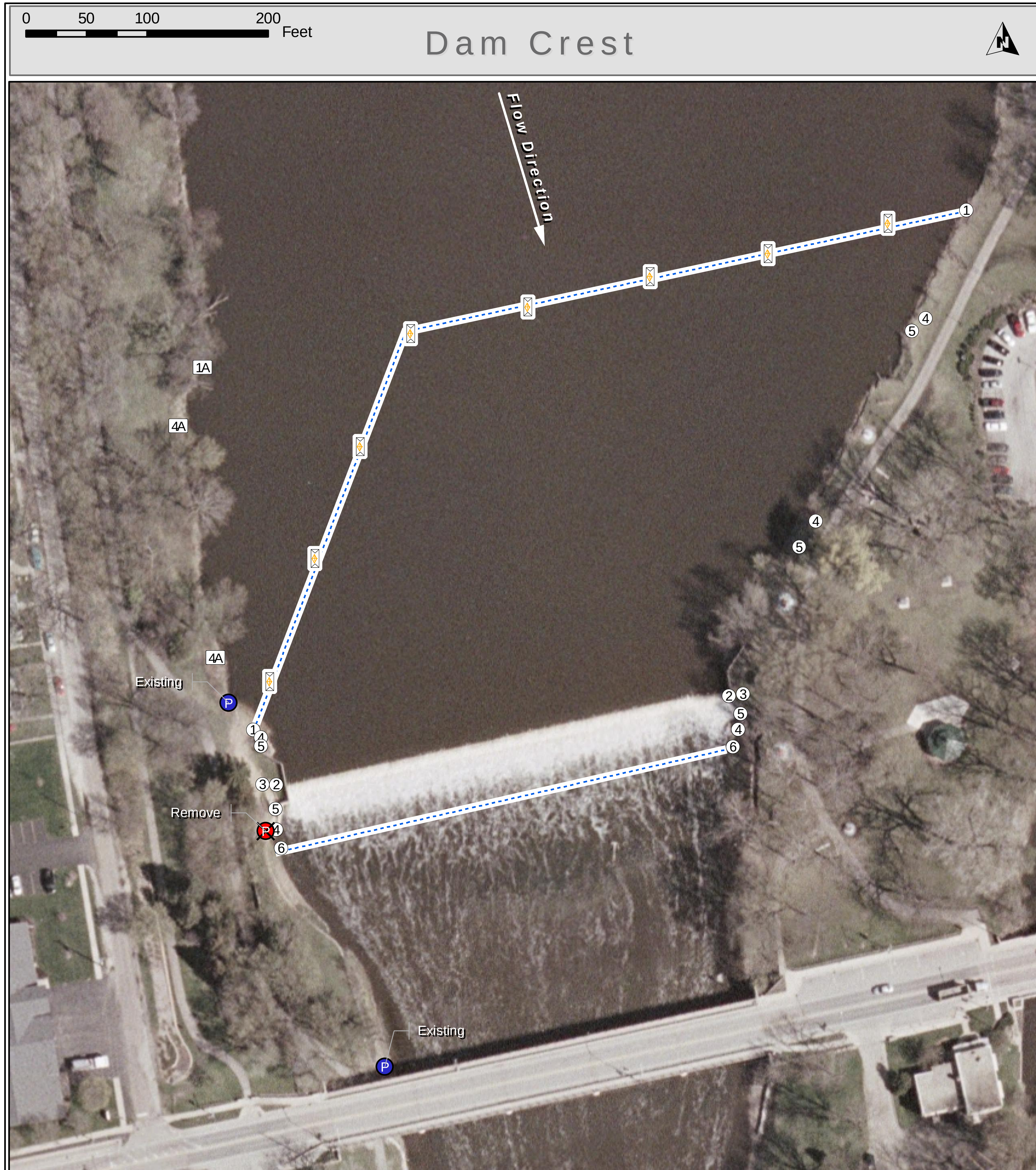
Batavia Dam - Fox River

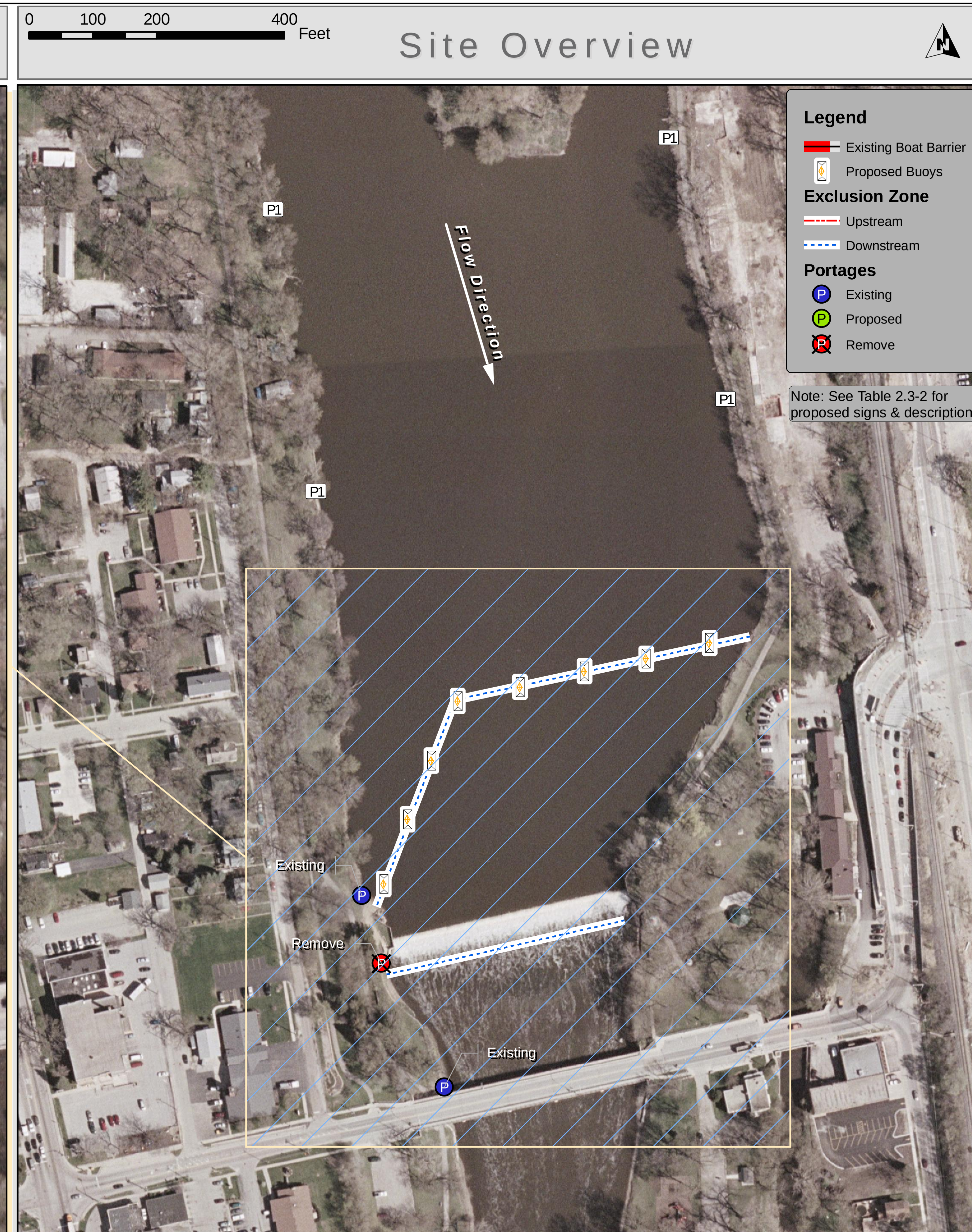
*Batavia, Illinois*

CTE | AECOM

Figure 2.6.8

Batavia




**Signage Plan**

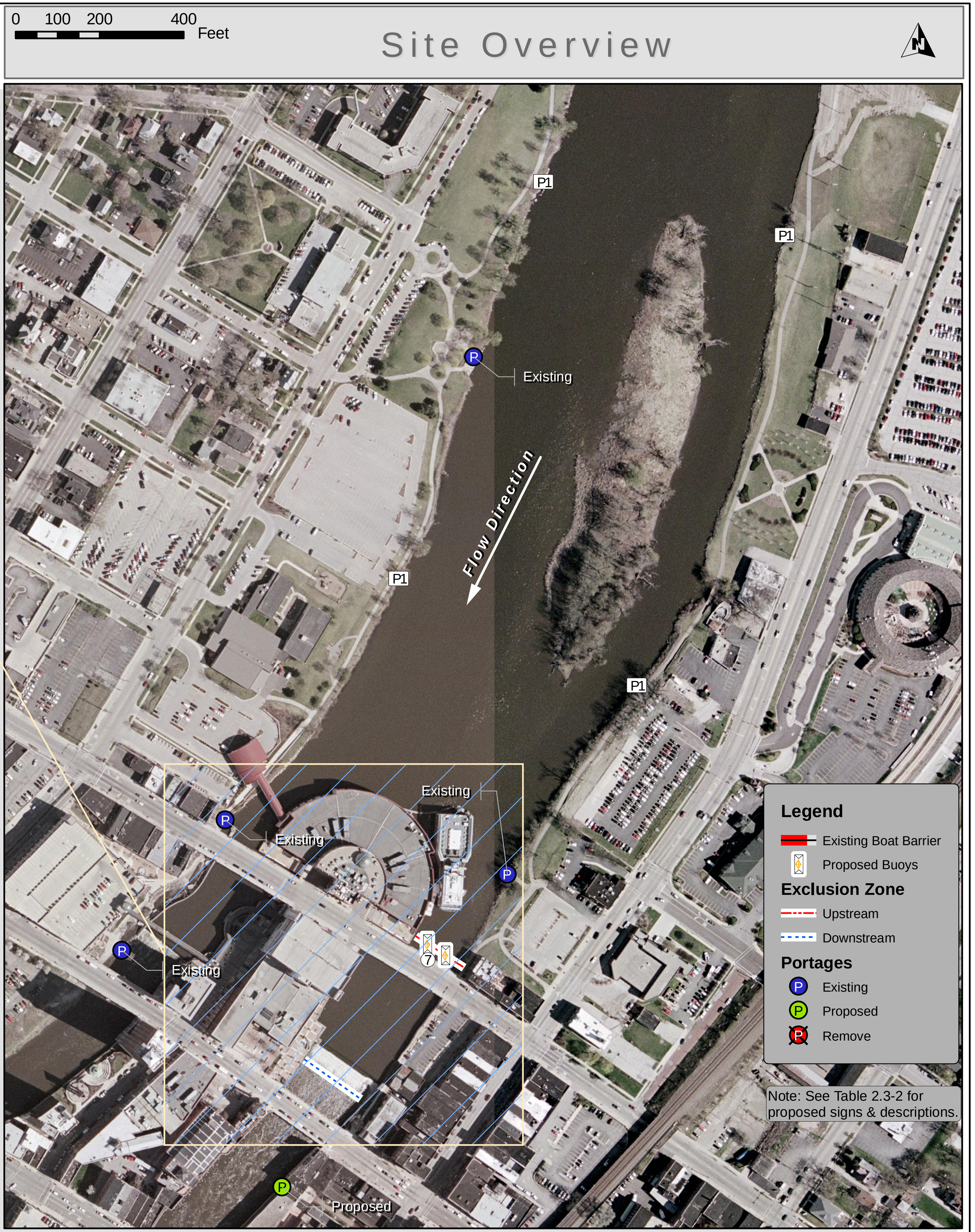
North Aurora Dam - Fox River

*North Aurora, Illinois*

CTE | AECOM

2.6.9

North Aurora


**Signage Plan**

Aurora East Dam - Fox River

*Aurora East, Illinois*

CTE | AECOM

Figure 2.6.10

Aurora East



***Signage Plan***

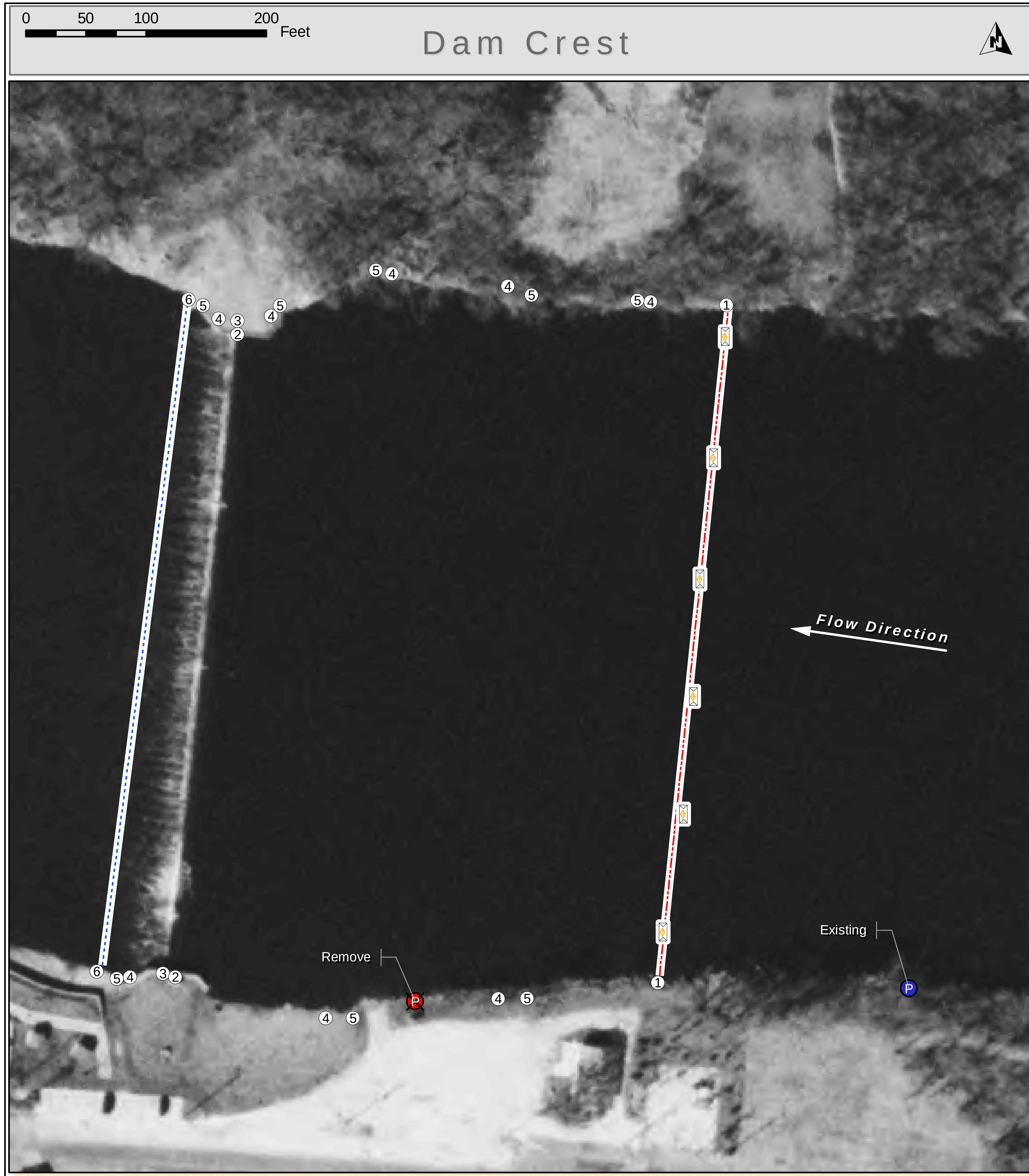
Montgomery Dam - Fox River

*Montgomery, Illinois*

CTE | AECOM

Figure 2.6.11

Montgomery





***Signage Plan***

Yorkville Dam - Fox River

*Yorkville, Illinois*

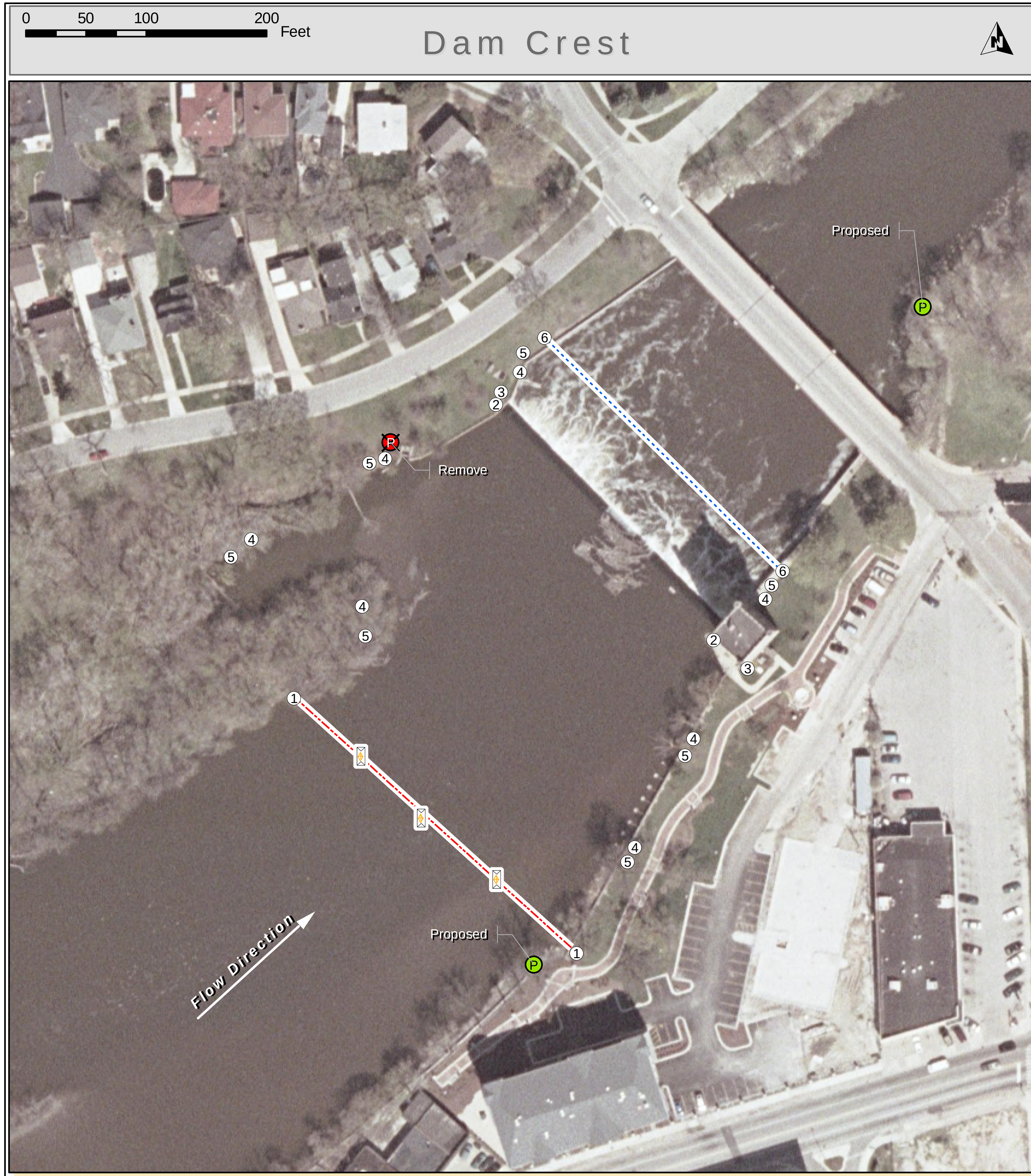
CTE | AECOM

Figure 2.6.12

Yorkville

## 2.7 *Des Plaines River Dam Signage Plan*

The following page includes a signage plan for the Hoffman Dam on the Des Plaines River.





***Signage Plan***

Hofmann Dam - Des Plaines River

*Riverside, Illinois*

CTE | AECOM

Figure 2.7.1

Hofmann

## 2.8 *Vermillion River Dam Signage Plan*

The following page includes a signage plan for the Danville Dam on the Vermillion River.

## Dam Crest


## Site Overview


***Signage Plan***

Danville Dam - Vermillion River

*Danville, Illinois*

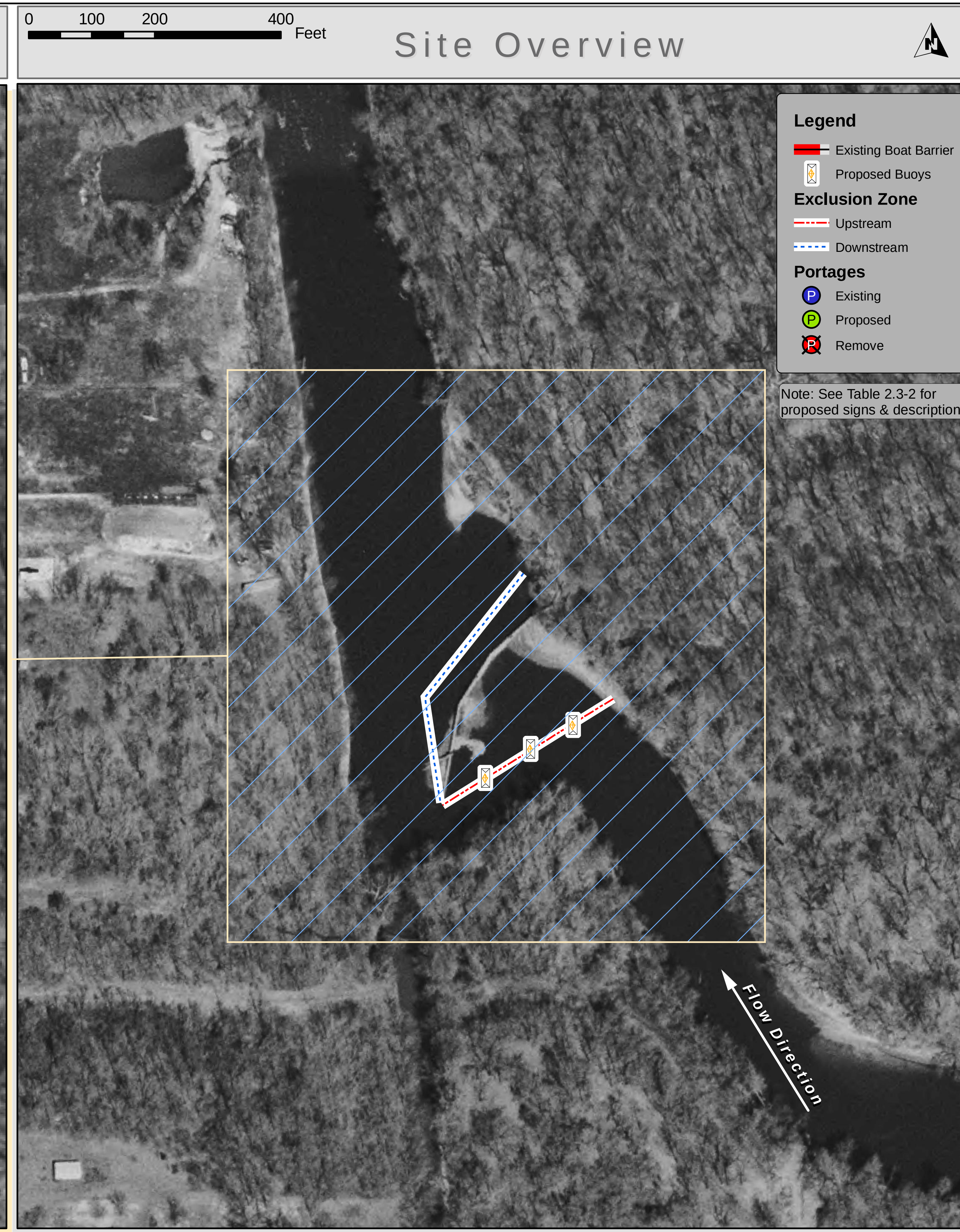
CTE | AECOM

Figure 2.8.1

Danville

## 2.9 *Sangamon River Dam Signage Plans*

The following pages include signage plans for Sangamon River dams.


**_Signage Plan_**

Riverside (Carpenter) Park Dam - Sangamon River

*Springfield, Illinois*

CTE | AECOM

Figure 2.9.1

Riverside



***Signage Plan***

Petersburg Dam - SangamonRiver

*Petersburg, Illinois*

CTE | AECOM

Figure 2.9.2

Petersburg

## Section 3 - Public Awareness Program

A public awareness campaign should be implemented to inform recreational river users and pedestrians of the hazard posed by run-of-river dams. The objective of the campaign would be to reach those who might be put at risk through their interaction with run-of-river dams. The message should be that there are real, specific, non-obvious dangers associated with run-of-river dams, both upstream and downstream of the dam, and that the only safe action is to stay out of the river near these dams.

The target audience for the public awareness program should include river and pedestrian (i.e. shoreline) users, owners, and emergency responders. This includes boaters, fishermen, dam and shoreline visitors, and in some cases, adjacent land owners. Boaters and fisherman may be able to be reached through state licensing and registration programs. Dam or shoreline visitors can be educated, initially, through educational signage and exhibits or kiosks at the dam. Emergency responders should be provided public awareness information directly, and should be encouraged to share information among EMS personnel throughout the state. Adjacent land owners, including local units of government, such as municipalities, park districts and forest preserves and private citizens, should also be contacted directly to receive public awareness information.

The public awareness campaign should explain the hidden dangers of run-of-river dams, specifically the dangerous currents associated with the downstream hydraulic jump and reverse roller. The campaign should communicate a clear message: the only safe action is to stay out of the water near a run-of-river dam. This message could be communicated verbally through broadcasting and public meetings, pictorially through brochures, pamphlets and the newspaper, and, if possible, physically using a physical scale model of a run-of-river dam. Clearly, the public needs to be educated about the hazards posed by these dams, and they should understand why they must not approach or go over the crest of a dam nor enter the roller by foot or boat from downstream. The campaign should also explain why they should not attempt to rescue individuals caught in the roller, and instead what emergency actions could be taken if they witness someone caught in or near it.

Emergency responders have indicated they often participate in school presentations, which would be an ideal opportunity to inform children at all age levels. Educational materials could be prepared to assist in making those presentations, and these could then be taken home to be shared with parents. Canoeing, boating, hunting and fishing clubs, as well as safety groups such as the U.S. Coast Guard, present another opportunity to distribute educational material to river users and pedestrians. Educational pamphlets could also be distributed with fishing and boating licenses each year, directly targeting those individuals who will be using the river.

## Section 4 - Structural Options

This section presents the general methodology and design limitations for the structural options developed in this report.

### 4.1 *Limitations and Future Design Needs*

The structural options presented for each dam were determined using basic hydraulics and limited physical data. The engineering methods used to estimate the layout of each option may not fully account for the complex flow patterns that can occur at each dam since a detailed design was beyond the scope of this study. Therefore, it is suggested that a detailed hydraulic analysis, detailed design and additional survey be completed prior to implementing each option. While additional analysis may delay construction, it will provide a needed level of confidence. Specifically,

- Surveys should be performed to determine upstream and downstream bed elevations, bank elevations, and dam face characteristics, and
- A detailed computational or physical hydraulic analysis should be completed for each option, including those dams and options that exhibit complex features, such as varying widths, lack of abutments, downstream confluences, etc., and
- Additional engineering design should be performed to provide site specific plans and specifications for each option, and
- An analysis of available real estate rights and potential acquisition of temporary and permanent easements for construction should be conducted, and
- Existing utilities in the river and shoreline areas should be identified and assessed, and
- River bed material should be analyzed to determine proper foundation and bedding requirements, and
- Physical sediment properties should be analyzed to develop plans for sediment management, and
- Sediment should be analyzed to determine its quality from an environmental standpoint. This may include sediment probing, physical analysis (both In-Situ and laboratory testing) and chemical analysis, and
- Potential water quality, biological, and ecological impacts of each option should be assessed, and
- Environmental documents may be developed and submitted for public review as needed, and
- The state should coordinate with regulatory agencies, primarily the Illinios Environmental Protection Agency (IEPA), US Army Corps of Engineers (USACE), Federal Emergency Management Agency (FEMA), Federal Energy Regulatory Commission (FERC), and the US Fish & Wildlife Service (USFWS), and

- The state should coordinate with affected communities and stakeholder groups.

## 4.2 General Approach

The physical data (i.e., river bed elevation, flow rates, dam crest elevation, etc.) used to size the options presented in this report were obtained from either the Federal Emergency Management Agency's (FEMA) effective Flood Insurance Study (FIS), *Personal Safety at Run-of-River Dams on Public Waters in Illinois (*IDNR 2006), additional survey data provided by IDNR, and the limited survey data collected as part of this report. The majority of FIS's were developed in the 1970's and 1980's and may not reflect current flood levels at each dam. Survey data, although conducted fairly recently, does not adequately show the extent of scour or the current bed profile at many dams. Representative elevations were selected based on engineering judgment to provide a reasonable estimate of cost suitable for planning purposes. The shape and dimensions of most upstream dam faces is unknown and therefore assumed to be vertical for all dams. Where dam faces have been obviously modified from their original condition, an attempt was made to approximate their dimensions or at the least, make note in the report. There is also limited information on the physical properties of the sediments in the vicinity of each dam.


# Evaluation of Public Safety at Run-of-River Dams

## 4.3 Temporary Rock Fill Structural Option

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## 4.3 *Temporary Rock Fill Structural Option*

This section discusses the methodology used to size the temporary rock fill option for each dam, as shown in Figure 4.3-1. A temporary rock fill option consists of placing large rock on the downstream side of the dam face that would extend downstream at a mild slope. The rock fill would reduce the drop at the dam crest and may reduce the hydraulic jump and reverse roller. Rock fill provides a structural alternative that can be implemented in a shorter time frame and, in general, at a lower cost than permanent structural options, while addressing the public safety hazard at each dam. Placement of rock fill downstream of the dam face is a temporary structural option with a limited capacity to reduce the public hazard at run-of-river dams. Although the solution has been sized to be stable up to a 50 year flow event, the method of placement (i.e. simply dropping stone in the river), coupled with the variability of flow, human activity, and river debris, may require that maintenance be performed as frequently as 2 to 5 years to maintain the desired slope.

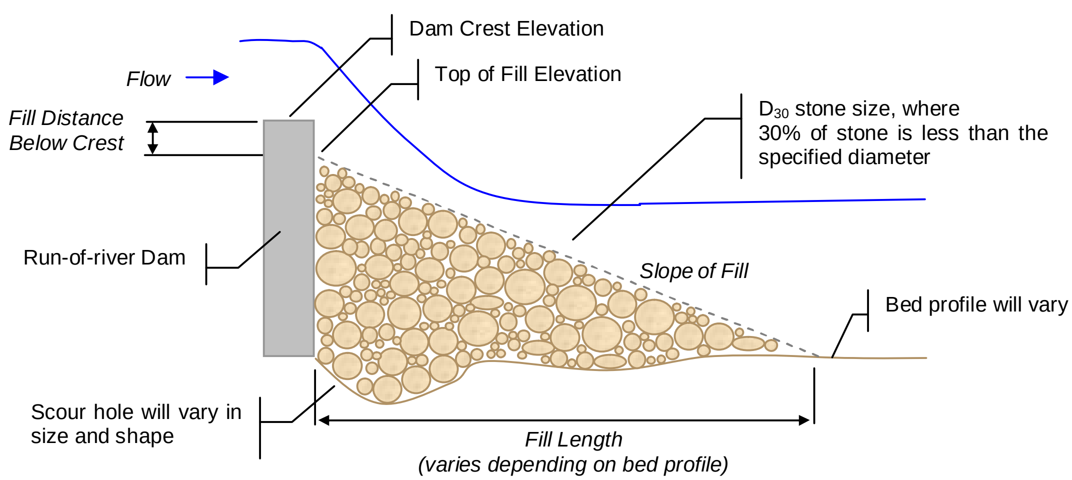


*Figure 4.3-1 - Rock Fill Concept Schematic*

USACE guidelines were used to set the diameter of the rock fill, however, the USACE methodology applies to placement in dry conditions without tailwater impacts. The rock fill options provided in this report should be placed during low-flow periods. Regardless placement will occur in wet conditions. Additionally, they will continually be impacted by a significant tailwater. How these conditions influence the sizing of stone is unknown, although the current sizing may prove reasonable given the large diameters and weights that were computed.

Additional design should also be conducted to appropriately size the filter, toe, and abutment protection. Filters are required due to the slope and size of stone, while toe protection may be required beyond simply burying the rock. Protection at the dam abutments may also be needed to reduce scour at the abutments due to increased water surfaces during low frequency events. Each of these issues reinforce the need for additional analysis and design.

The criteria below were used to layout and size each rock fill option. During final design, these criteria should be reviewed to establish a consensus on the performance required.

1. Rock fill must not increase the upstream or downstream 100 year maximum water surface, and
2. Sub-critical flow must be maintained over the rock fill for the 1, 2, and 5 year flow events, eliminating a hydraulic jump at the downstream toe, and
3. Rock fill must prevent a submerged hydraulic jump from forming at the dam crest for the 1, 2, and 5 year flow events, and
4. Rock fill must be stable for events up to the 50 year design storm.

*Criterion 1 - Must not increase the upstream and downstream 100-year maximum water surface*

To determine whether the rock fill would impact the upstream 100 year water surface, the U.S. Department of Agriculture (USDA) Soil Conservation Service *Drop Spillways* weir submergence criteria was used. However this methodology has several limitations. The USDA submergence criteria were developed based on tests conducted on a wide range of weirs and earth embankments, and specify that "precise results should not be expected…" In addition, the placement of rock fill below the dam will alter the characteristics of the dam such that it no longer performs as an idealized weir. Therefore, the submergence criteria have been used here to obtain a reasonable expectation of a maximum rock fill elevation such that the upstream 100 year water surface is not impacted.

Figure 4.3-2 depicts a generalized weir or dam crest with a crest elevation, upstream water surface elevation and downstream water surface elevation (i.e. tailwater) above the weir crest. $H_1$ is the difference between the headwater elevation and weir dam crest elevation. $H_2$ is the difference between the tailwater elevation and the weir dam crest. As the tailwater of a run-of-river dam rises above the elevation of the dam crest, the flow over the dam may be reduced, resulting in increased upstream water surfaces. The dam crest will experience partial submergence when the submergence ratio, $H_2 / H_1$, exceeds 0.38. When $H_2$, equals $H_1$, the weir will be entirely submerged. For ratios between 0.38 and 1, the dam crest will be partially submerged and may experience a reduction in flow. As the flow over the dam crest is reduced, upstream water surfaces may increase. Based on the 100 year normal depth of flow over the rock fill, an $H_2 / H_1$ ratio of less than 0.38 was maintained for the options developed in this report in order to prevent increases in the upstream 100-year water surface. The rock fill was also designed such that 100 year normal depth of flow over the rock fill would not be greater than the existing downstream 100 year water surface.

It should be understood that for higher frequency events, (e.g., 1 yr, 2 yr, and 5 yr) water surfaces along the rock fill will be greater than water surfaces under existing conditions. If flooding increases due to these increases in the water surface, an abutment along the rock fill may be necessary to mitigate the increased flooding.

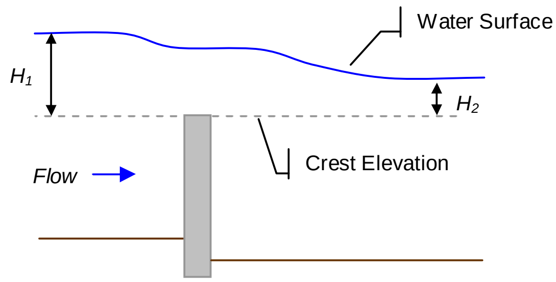


*Figure 4.3-2 - Weir Flow Submergence Criteria*

Based on the above methodology, a top of fill elevation was calculated. A safety factor of 1 ft was then applied such that the rock fill was at least 1 ft below the elevation calculated from the submergence criteria. For construction purposes, the top of fill elevation was set such that it was at least 1 ft below the elevation of the dam crest.

*Criterion 2 - Must maintain sub-critical flow over the rock fill for the 1, 2 and 5 year flow events, therefore eliminating a hydraulic jump at the downstream toe*

Due to the high tailwater conditions at each dam, the majority of rock fill designs would experience submerged hydraulic jumps at the downstream toe during most flow events. As discussed previously, submerged hydraulic jumps are capable of producing reverse rollers depending on the degree of submergence; therefore, it was determined that during the 1, 2, and 5 year flow events, the rock fill should be designed to prevent a hydraulic jump from forming at all. To prevent a hydraulic jump from forming downstream of the toe, the slope of the rock fill configuration was selected to produce sub-critical flow. During flow events greater than the 5 year frequency, a hydraulic jump with a reverse roller may still occur.

*Criterion 3 - Rock fill must prevent a submerged hydraulic jump from forming at the dam crest for the 1, 2, and 5 year flow events, and*

Since the flow over the dam will either be critical or super-critical during the 1, 2, and 5 year flow events, and the flow over the rock fill will be sub-critical, a hydraulic jump will occur immediately downstream of the dam crest. As discussed in Section 1.7, a hydraulic jump forms because the water flowing over the crest is super-critical, while the flow along the rock fill is sub-critical. If the tailwater is high enough, the jump may become submerged, and may produce a reverse roller. In most cases, these jumps are not submerged; however, for dams with high tailwater elevations, the top of fill elevation was raised to within 1 ft of the crest.

Although the design methodology for rock spillways is well known, design methods directed at improvements to public safety at run-of river dams are not. The final design of each rock fill option aimed at reducing the hydraulic jump and associated downstream reverse roller at each dam, will likely require a two-dimensional or three-dimensional computational fluid dynamic model or physical model. A one-dimensional model, such as the USACE HECRAS, could be used to determine upstream and downstream water surfaces and flow rates, while multi-dimensional models such as the U.S. Army Corps of Engineer's, Adaptive Hydraulics, Flow Science's FLOW-3D or equivalent would provide a more reliable determination of when a dangerous reverse roller would occur. A multi-dimensional model would also provide additional insight into velocities, scour, and non-intuitive flow patterns. Table 4.3.1 presents the concept parameters and opinions of cost for each temporary rock fill option considered.

*Table 4.3-1 - Temporary Rock Fill Option Parameters and Opinions of Cost*

| Dam | Crest Elevation (NGVD - ft) | Top of Fill Elevation (NGVD - ft) | Fill Distance Below Crest (ft) | Slope of Fill | Fill Length (ft) | Fill Volume ($yd^3$) | $D_{30}$ (ft) | Opinion of Cost † |
|---|---|---|---|---|---|---|---|---|
| Momence | 618.00 | 617.00 | 1.00 | 5.0% | 95.00 | 1,243 | 2.0 | $ 470,000 |
| Kankakee | | | *Rock fill not suggested due to hydropower operations* | | | | | |
| Wilmington | 531.00 | 530.00 | 1.00 | 5.0% | 327 | 7,812 | 2.8 | $ 2,170,000 |
| Wilmington Millrace | | | *Rock fill not suggested due to the existing fill* | | | | | |
| Oregon | 670.20 | 664.56 | 5.64 | 5.5% | 309 | 105,408 | 2.3 | $ 38,130,000 |
| Sinnissippi [1] | 636.00 | 627.50 | 8.50 | 6.0% | 84 | 12,297 | 2.0 | $ 4,510,000 |
| Lower Sterling | 627.00 | 625.13 | 1.87 | 5.5% | 229 | 57,423 | 2.2 | $ 18,030,000 |
| Sears | 557.84 | 552.98 | 4.86 | 6.0% | 154 | 12,544 | 2.0 | $ 3,790,000 |
| Steel | 558.20 | 557.20 | 1.00 | 6.0% | 70 | 4,252 | 2.1 | $ 1,670,000 |
| McHenry (Stratton L&D) [2] | 736.50 | 735.50 | 1.00 | 12.0% | 49 | 1,707 | 1.2 | $ 720,000 |
| Algonquin [3] | N/A | N/A | N/A | N/A | 130 | 4419 | 1.6 | $ 1,460,000 |
| Carpentersville | 720.70 | 719.24 | 1.46 | 7.0% | 111 | 5,982 | 1.3 | $ 1,640,000 |
| Elgin Kimball Street | 708.40 | 707.24 | 1.16 | 7.0% | 169 | 10,862 | 1.5 | $ 2,860,000 |
| South Elgin | 700.00 | 696.48 | 3.52 | 7.0% | 56 | 2,233 | 1.5 | $ 660,000 |
| St. Charles | 684.60 | 683.09 | 1.51 | 6.5% | 124 | 5,363 | 1.8 | $ 1,820,000 |
| Geneva | 675.40 | 671.74 | 3.66 | 7.0% | 61 | 2,133 | 1.4 | $ 860,000 |
| Batavia | 665.10 | 661.06 | 4.04 | 6.5% | 133 | 12,634 | 1.5 | $ 3,210,000 |
| North Aurora | 646.00 | 641.63 | 4.37 | 6.5% | 84 | 1,829 | 1.6 | $ 850,000 |
| Aurora East | 628.40 | 624.44 | 3.96 | 6.0% | 84 | 1,311 | 2.1 | $ 490,000 |
| Montgomery | 614.00 | 613.00 | 1.00 | 6.0% | 108 | 4,857 | 2.1 | $ 1,360,000 |
| Yorkville | | | *Rock fill not suggested due to the existing stepped face* | | | | | |
| Hofmann | 603.50 | 599.94 | 3.56 | 6.5% | 90 | 2,562 | 1.4 | $ 900,000 |
| Danville | 519.00 | 518.00 | 1.00 | 5.0% | 140 | 6,991 | 4.2 | $ 2,190,000 |
| Riverside Park | | | *Rock fill not suggested since the dam has failed* | | | | | |
| Petersberg | 477.50 | 476.50 | 1.00 | 5.0% | 220 | 2,586 | 2.0 | $ 1,020,000 |
| | | | | | | | **Total Cost** | **$ 88,810,000** |

† These costs are based on limited information and are for planning purposes only. They do not include costs associated with land acquisition, final engineering design, and permitting. Additional survey data at Oregon and Sinnissippi dam resulted in a reduced cost of as much as 38%. If additional data is gathered at each dam, the total opinion of cost could range from $55,060,000 to $88,810,000.

1 Sinnissippi Dam already maintains a sloped dam face, which may reduce the 1 yr, 2 yr, and / or 5 yr roller; however, a cost has been provided for a rock fill sized according to the guidelines discussed in this report.

2 A traditional rock fill may not prevent a submerged hydraulic jump at the dam crest during the 5 yr flow; however, a cost has been provided for planning purposes.

3 A traditional rock fill may not prevent a submerged hydraulic jump at the dam crest during the 1, 2, and 5 yr flow; therefore, a rock fill is not suggested, and a cost has been provided for filling the scour hole only.

*References*

The following references were used when considering the temporary rock fill option. Many of these references were also used to assess the other structural options.

Cotton, George K., *Hazard Rating for Low Drops,* Water Resources Engineering, 1995, Vol. 2, p. 1111-1115.

Garcia, Anizka, Cole Marr, Anita Rogacs, Michael Robinson, *Drowning Machines – Low-Head Dam Hydraulics and Hazard Remediation Options,* Prepared for the Indiana Department of Natural Resources, Engineering Forensics Research Institute, July, 2005.

Hotchkiss, Rollin and Max Comstock, Discussion of: *Drownproofing of Low Overflow Structures,* Journal of Hydraulic Engineering, 1992, Vol. 118, Issue 11, p. 1586-1589.

Leutheusser, Hans and Warren M. Birk, *Drownproofing of Low Overflow Structures,* Journal of Hydraulic Engineering, 1991, Vol. 117, Issue 2, p. 205-213.

Leutheusser, Hans and Jerry Fan, *Backward Flow Velocities of Submerged Hydraulic Jumps,* Journal of Hydraulic Engineering, June, 2001, Vol. 129, Issue C, p. 514-517.

*Personal Safety at Run-of-River dams on Public Waters in Illinois,* Illinois Department of Natural Resources (IDNR) Office of Water Resources, September 2006.

Robinson, Michael, Ph.D.; Robert Houghtalen, Ph.D.; Cole Marr; Anita Rogacs; and Anizka Garcia. *Dangerous dams, removal or retrofitting improves public safety at low-head dams,* CE NEWS, February 2007, Pages 24-29.

U.S. Army Corps of Engineers, *Hydraulic Design of Spillways*, Engineer Manual 1110-2-1603, 16 January 1990.

U.S. Army Corps of Engineers, *Channel Stability Assessment for Flood Control Projects,* Engineer Manual 1110-2-1418, 31 October 1994.U.S. Department of Agriculture, Soil Conservation Service, *Drop Spillways*, Engineering Handbook, Section 11.


Source: Paddeln.de

# Evaluation of Public Safety at Run-of-River Dams

## 4.4 *Permanent Structural Options*

### 4.4.1 Full Bypass Channel

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## 4.4 *Permanent Structural Options*

The objective of this section is to discuss and provide permanent solutions that specifically and primarily address public safety at each dam. The following subsections provide a description of the four permanent structural options proposed in this report, including a full bypass channel; a riffle pool rock ramp, an in-stream bypass channel, and a dam face modification. These options are listed in a general sequence of benefit with regard to improving public safety at the dam site. Dam removal, presented in Section 4.5, would entirely eliminate the hazard and would provide a substantial benefit to public safety; however, the permanent structural options would reduce the hazard. In addition, permanent structural options may provide positive recreational, cost, and environmental benefits.

### 4.4.1 Full Bypass Channel

A full bypass channel is an engineered channel which diverts flow around a dam and prevents a dam from overtopping up to a certain level of flow (e.g., the 5-year flow). The slope and width of the bypass channel are designed to convey the design flow before the dam overtops. This is meant to eliminate a reverse roller downstream of a dam for flows up to the design flow. Additionally, the bypass could be designed to serve as a canoe chute and to also be passable by fish.

General selection and sizing criteria have been developed for the full bypass channel based on case histories of bypass and whitewater channels as well as previous studies regarding the design of riffle pools and canoe chutes. The criteria address the flow rates experienced at each dam, the availability of land at river banks, and the need to provide safe passage for canoeists and kayakers. These criteria have been selected for use in developing preliminary layouts of proposed full bypass channels adequate for and planning purposes only. Detailed hydraulic analyses of proposed full bypass channels is needed to develop a design suited to the characteristics of each site, which may result in designs that differ significantly from those presented here.

The majority of bypass and whitewater channels constructed to date throughout the country convey flows between 50 to 3,000 cfs. This is a result of the size of the rivers on which bypass channels have typically been built. Within Illinois, the Fox River experiences a range of 5 year flows between 3,700 to 11,000 cfs, while the Kankakee, Rock, and Vermillion experience a range of 13,500 to 38,000 cfs. In order to reduce the public safety hazard during the majority of flow events, the 5 year flow has been selected as the bank-full flow event for this report. This would prevent overtopping of the dams during the 1, 2, and 5 year flow event. Eliminating flow over the dam may result in negative environmental impacts downstream from the dam face to the confluence of the bypass channel. During a detailed design, solutions should be investigated in order to reduce any potential environmental impacts. In addition, the dry dam face may prove inviting for shoreline users or fishermen an issue which should be addressed during a detailed design.

Three design criteria dictated the selection and sizing of the full bypass channel, and are discussed in detail on the following pages.

1. Channel Invert Elevation
2. Channel Width
3. Channel Slope

*Criterion 1 - Channel Invert Elevation*

Typical profiles of a full bypass channel are depicted in Figures 4.4.1-1 and 4.4.1-2. A plan view of a full bypass channel is depicted in Figure 4.4.1-3. The channel invert elevation is defined as the bed elevation of the channel at the upstream end. As the invert is lowered, the conveyance capacity of the bypass increases; however, the channel slope and number of riffles / steps will decrease. In general, if the invert elevation is less than 3 ft above the existing bed elevation, the bypass was considered infeasible since the available vertical drop would be insufficient to provide a reasonable slope through the bypass.

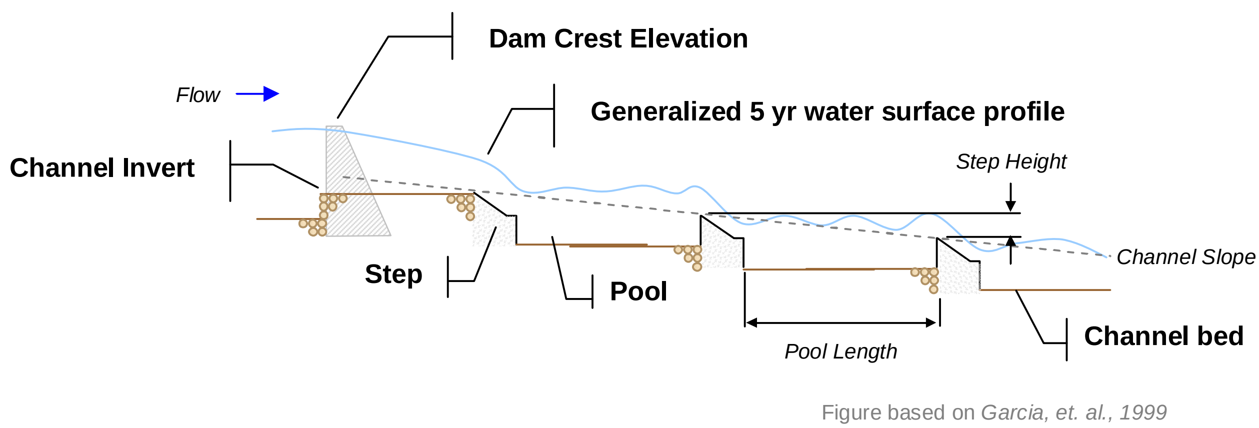


*Figure 4.4.1-1 - Profile view of generic full bypass channel with concrete steps*

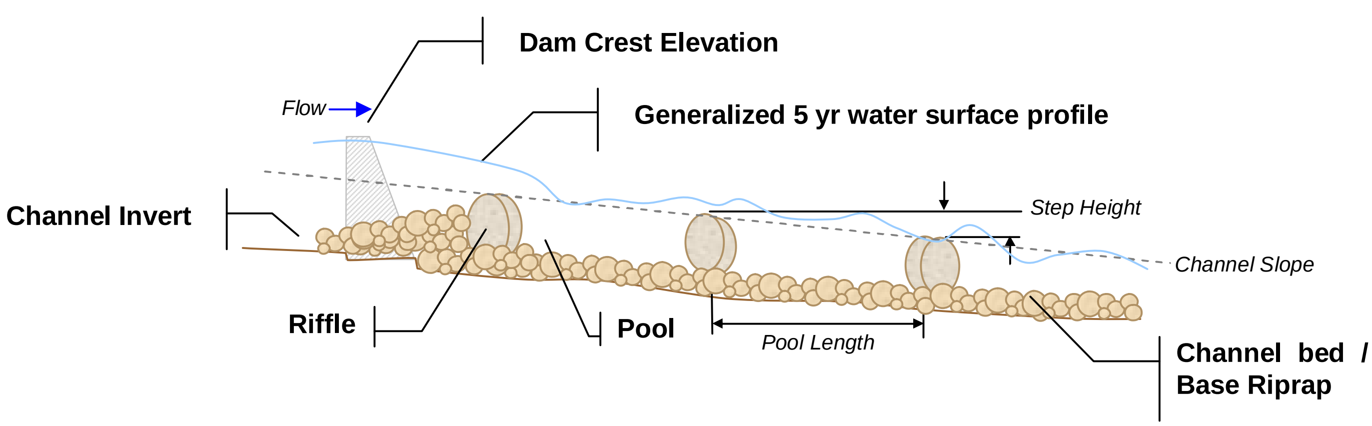


*Figure 4.4.1-2 - Profile view of generic full bypass channel with natural rock*

*Criterion 2 - Channel Width*

As the width of a full bypass is increased, the conveyance capacity of the bypass increases; however, the width is limited by river bank development and realistic expectations of how wide a bypass should be. In general, if a bypass had to be more than 200 ft wide to pass the 5 year flow at the maximum allowable slope, the bypass was considered infeasible. A minimum width of 20 ft was set based on the length of a typical canoe and a review of historic bypass designs.

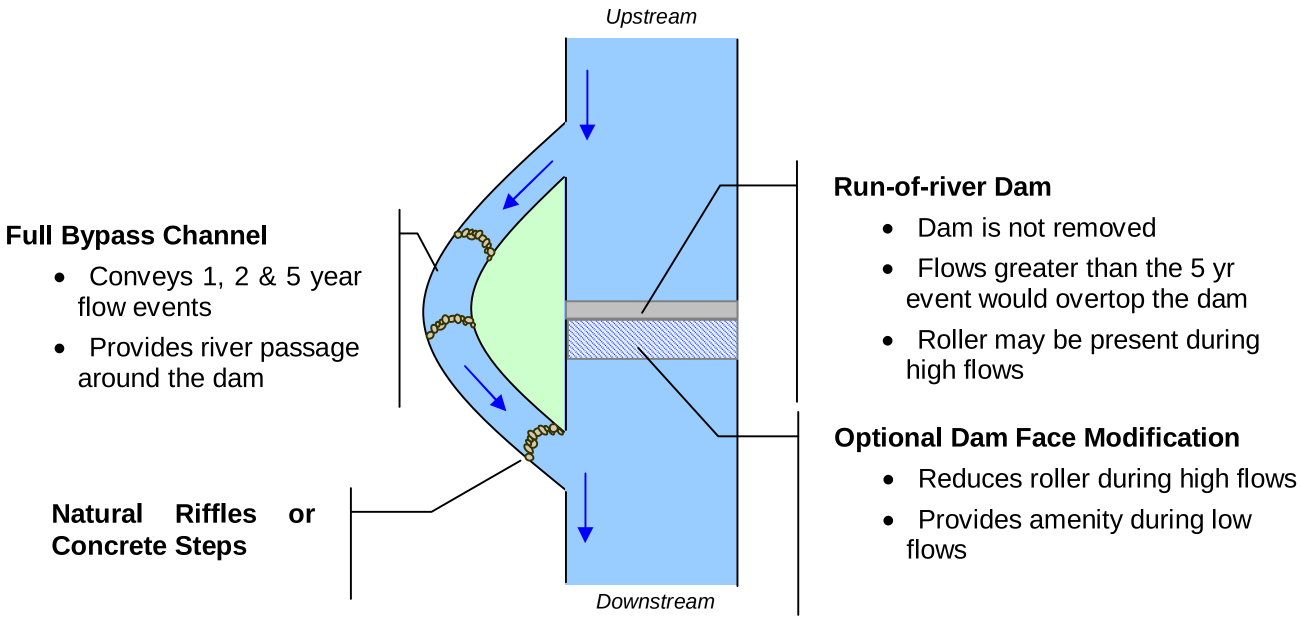


*Figure 4.4.1-3 – Plan view of full bypass option with riffle boulders*

*Criterion 3 - Channel Slope*

As the slope of a full bypass is increased, the conveyance capacity of the bypass increases; however, the slope is limited by a need for safe passage by canoeists and kayakers. The slope is controlled by the invert elevation of the bypass, the existing downstream bed elevation, the height / drop of each riffle / step (see *Step Height*, shown in Figures 4.4.1-1 and 4.4.1-2), and the length of each pool (see *Pool Length*, shown in Figures 4.4.1-1 and 4.4.1-2). An ideal slope of 1% was selected to provide safe passage for canoeists and kayakers while also providing a reasonable grade. If the slope of a bypass had to be more than 2.5%, it was considered infeasible. In addition, the maximum height / drop of each step was set at 1 ft for safe passage of canoes and kayaks. The pool length has been allowed to vary based on the step height and slope, but was set to a minimum length of 40 ft, or approximately the length of two canoes. This was done to provide adequate space for maneuvering canoes and kayaks.

Table 4.4.1-1 on the following page presents the concept parameters and opinions of cost for each full bypass channel at dams where the option was considered feasible. Detailed descriptions of each full bypass channel are given in Sections 4.6 through 4.11.

*Table 4.4.1-1 - Full Bypass Channel Parameters and Opinions of Cost*

| Dam | Dam Crest Elevation (NGVD - ft) | Dam Height (ft) | Channel Invert (NGVD - ft) | Channel Design Slope | Channel Width (ft) | Number of Riffles | $D_{30}$[1] Size (ft) | Base Riprap Volume ($yd^3$) | Riffles ($yd^3$) | Bank Riprap ($yd^3$) | Opinion of Cost† |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Steel * | 658.20 | 6.00 | 658.20 | 0.5% | 45 | 5 | 0.5 | 556 | 75 | 528 | $ 3,470,000 |
| Carpentersville | 720.70 | 8.00 | 715.17 | 1.5% | 100 | 5 | 1.0 | 3,424 | 166 | 275 | $ 5,250,000 |
| South Elgin | 700.00 | 8.31 | 696.32 | 2.5% | 130 | 4 | 1.3 | 5,042 | 175 | 323 | $ 7,790,000 |

* This option does not convey the 5-year flow, but is only meant to serve as a canoe chute.

[1] The $D_{30}$ value represents a specific range of stone riprap diameters where 30% of the stone is sized at or below the specified diameter.

† These costs are based on limited information and are for planning purposes only. They do not include costs associated with land acquisition, final engineering design, permitting, or environmental considerations such as sediment quality.

*References*

The following references were used when considering the full bypass channel option. References listed in the other structural option sections were also used.

Gaboury, M.N., R.W. Newbury, C.M. Erickson, *Pool and Riffle Fishways for Small Dams,* Manitoba Natural Resources, Fisheries Branch, 1995.

Holmquist-Johnson, Chris, Joseph Mercure, David Mooney, Rock Weir Design Guidelines, U.S. Department of the Interior, Bureau of Reclamation, Downloaded in April 2007, http://www.usbr.gov/pmts/sediment/projects/RStructs.

Wildman, Laura, Piotr Parasiewicz, Christos Katopodis, Ulrich Dumont, *An Illustrative Handbook on Nature-Like Fishways – Summarized Version,* 2007.


# Evaluation of Public Safety at Run-of-River Dams

4.4 *Permanent Structural Options*

4.4.2 Riffle Pool Rock Ramp

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### 4.4.2 Riffle Pool Rock Ramp

The criteria used to develop the riffle pool rock ramp layouts vary from the full bypass in that the width of the riffle pool is equal to the width of the river channel. Riffle pool rock ramps use a series of boulder "weirs" to provide the transition from the dam crest to the downstream river bed elevation. Figures 4.4.2-1 through 4.4.2-3 show a riffle pool rock ramp typical profile, plan, and section. The first boulder weir should be placed at an elevation equal to the dam crest. Additional hydraulic analysis is required to ensure the 100 year water surface profile is not impacted, although at least one study by the USACE Waterways Experiment Station has shown that the 100 year water surface elevation may not be raised when riffles are placed downstream of a dam (Fuller & Bernard, 2007). Additional riffle pools are constructed downstream of the dam as depicted in each figure. To decrease the likelihood of creating rapids greater than Class II under most flow conditions, a 1% slope was used whenever possible. The maximum slope of the riffle pools was set at 2.5% for this report. Since rapids ratings are generally subjective and vary depending on flow rate a detailed design will be required prior to implementation. The stone both for the riffles and the underlying riprap, as well as the protective riprap on the banks was sized to be stable up to a 50 year flow event with proper maintenance riffle pool rocks ramps could have a life expectancy comparable to the design flow event. This was done to provide stability with minimal maintenance for this report. However, *Newbury, et. al., 1998* suggests that the diameter of the riffles should be sized to withstand the bankfull event, and that for larger events maintenance should be expected. This would result in sizing the stone to be stable for events such as the 5 or 10 year. For the purposes of this report, the stone was sized for the 50 year storm event, resulting in large stone sizes and high construction costs. The feasibility of sizing for the 50 year storm event should be addressed if detailed designs are conducted. In addition, if a temporary rock fill has been implemented, that stone may possibly be used to reduce the cost of purchasing new material.

To prevent negatively impacting bridges, riffle pools were placed no closer than 100 ft upstream or downstream of existing bridges. Further analysis is required to determine an exact distance at each bridge. In addition, each riffle pool option maintains a final riffle 2 to 3 ft above the channel bed. A detailed hydraulic study is required to determine the exact height of the final riffle.

Table 4.4.2-1 presents the concept parameters and opinions of cost for riffle pool rock ramps at dams where this option was deemed feasible. Detailed descriptions of these options are given in Sections 4.6 through 4.11.

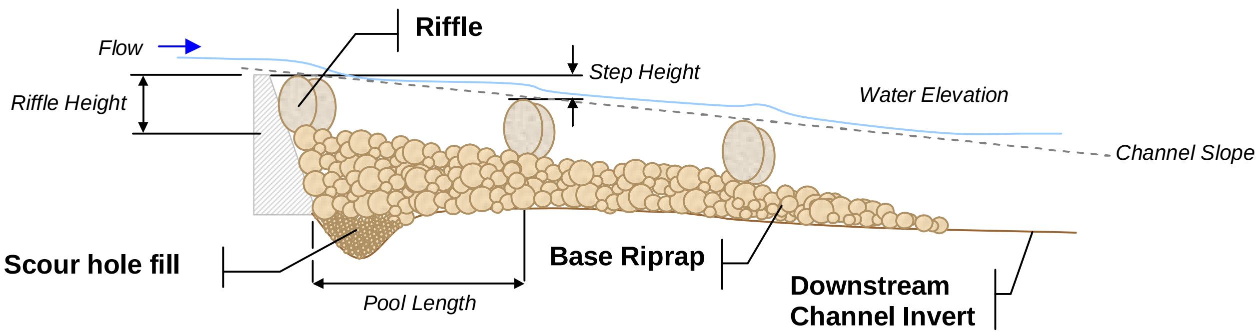


*Figure 4.4.2.-1 - Profile view of riffle pool rock ramp*
*(Parallel to flow)*

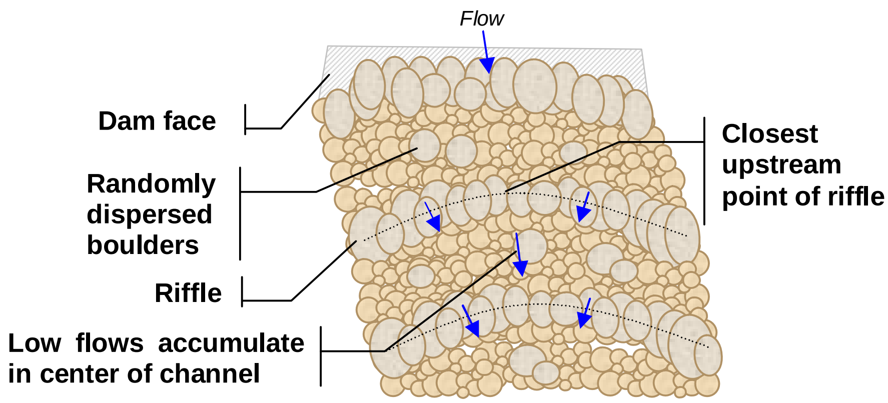


*Figure 4.4.2-2 - Plan view of riffle pool rock ramp*

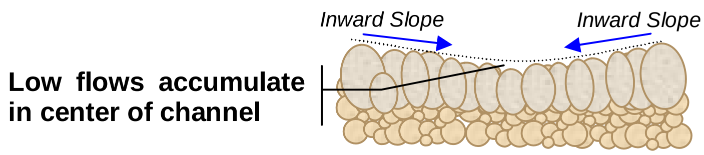


*Figure 4.4.2.-3 - Section of riffle pool rock ramp*
*(Perpendicular to flow)*

*Table 4.4.2-1 - Riffle Pool Rock Ramp Option Parameters and Opinions of Cost*

| Dam | Crest Elevation (NGVD - ft) | Riffle Pool Slope | Number of Riffles | Riffle Spacing (ft) | $D_{30}$[1] (ft) | Base Riprap Volume ($yd^3$) | Riffle Volume ($yd^3$) | Bank Riprap ($yd^3$) | Scour Fill ($yd^3$) | Opinion of Cost[†] |
|---|---|---|---|---|---|---|---|---|---|---|
| Momence | 618.00 | 1.5% | 3 | 67 | 0.8 | 1,604 | 145 | 96 | 92.6 | $ 690,000 |
| Wilmington | 531.00 | 1.0% | 5 | 100 | 1.1 | 20,988 | 1,164 | 508 | 1,794.4 | $ 5,270,000 |
| Wilmington Millrace | 525.00 | 1.0% | 6 | 100 | 0.8 | 3,242 | 265 | 535 | 0 | $ 1,450,000 |
| Steel | 658.20 | 1.0% | 5 | 100 | 0.8 | 17,071 | 1,281 | 355 | 1,069.4 | $ 4,770,000 |
| Carpentersville | 720.70 | 1.0% | 9 | 100 | 0.4 | 20,673 | 755 | 449 | 350.0 | $ 6,620,000 |
| South Elgin | 700.00 | 1.5% | 6 | 67 | 0.6 | 14,814 | 607 | 287 | 495.8 | $ 3,940,000 |
| Montgomery | 614.00 | 1.0% | 8 | 100 | 0.8 | 28,622 | 870 | 695 | 300.9 | $ 7,530,000 |
| Danville | 519.00 | 1.0% | 7 | 100 | 1.7 | 27,772 | 567 | 1,802 | 203.7 | $ 7,220,000 |

[1]The $D_{30}$ value represents a specific range of stone riprap diameters where 30% of the stone is sized at or below the specified diameter.

[†] These costs are based on limited information and are for planning purposes only. They do not include costs associated with land acquisition, final engineering design, permitting, or environmental considerations such as sediment quality.

*References*

The following references were used when considering the riffle pool rock ramp option. References listed in the other structural option sections were also used.

Aadland, Luther, Personal communication and powerpoint presentation supplied in April 2007 discussing the Minnesota Department of Natural Resource's design methodology for riffle pools.

Fuller, Dwayne and Robert S. Bernard, *Numerical Investigations of Changes in Water-Surface Elevation Induced by Modification of Channel Geometry Fargo South Dam,* Coastal and Hydraulics Laboratory, U.S. Army Engineer Research and Development Center, Vicksburg, Mississippi, Supplied by Luther Aadland, Minnesota Department of Natural Resources in April 2007.

U.S. Army Corps of Engineers Waterways Experiment Station (WES), *The WES Stream Investigation and Streambank Stabilization Handbook*, Vicksburg, Mississippi.

Source: AmericanWhitewater.org

# Evaluation of Public Safety at Run-of-River Dams

## 4.4 *Permanent Structural Options*

### 4.4.3 In-Stream Bypass Channel

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### 4.4.3 In-Stream Bypass Channel

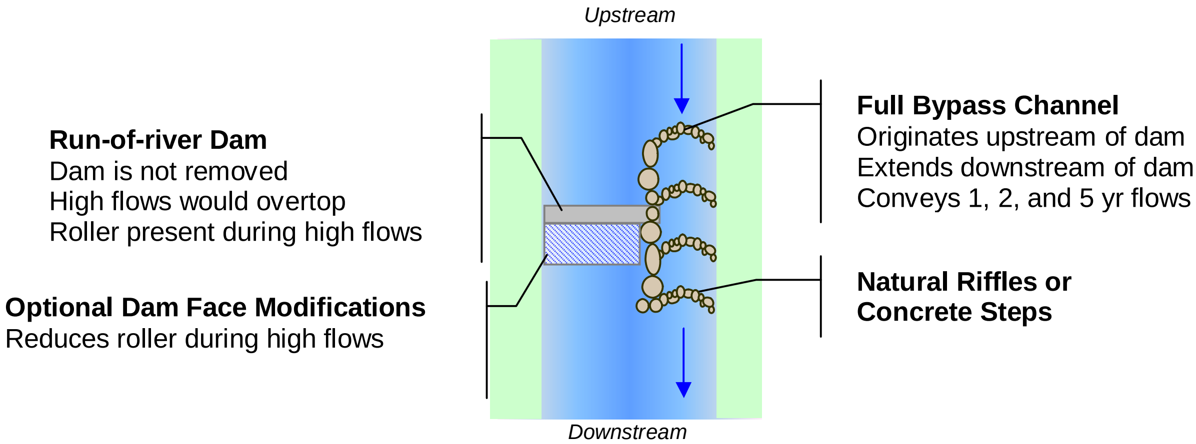
An in-stream bypass channel is a flow passage which avoids a dam crest by removing a portion of the dam (i.e., notching the dam). The entire stream flow is able to pass through the "notch" to a selected flow (i.e., the 5-year flow for this report), preventing any flow going over the dam crest. This is meant to eliminate a reverse roller downstream of the dam for flows up to the design flow. Additionally, the bypass can be designed to serve as a canoe and / or kayak chute and fish passage.

An in-stream bypass channel would require that a dam be notched preferably at an abutment. As shown in Figure 4.4.3-1 on the following page, the bypass may originate upstream and end at the dam, requiring partial removal or notching of the dam to the riverbed. Alternatively, the channel may originate upstream of the dam and end downstream, requiring partial removal or notching of the dam to a point above the bed elevation. As a third alternative the channel may originate at the dam face and extend downstream, again requiring partial removal of the dam. The width of the dam that is removed in each scenario is equal to the width of the bypass, while the height of the dam that is removed will vary depending on the channel slope, riffle / step height, and channel invert elevation.

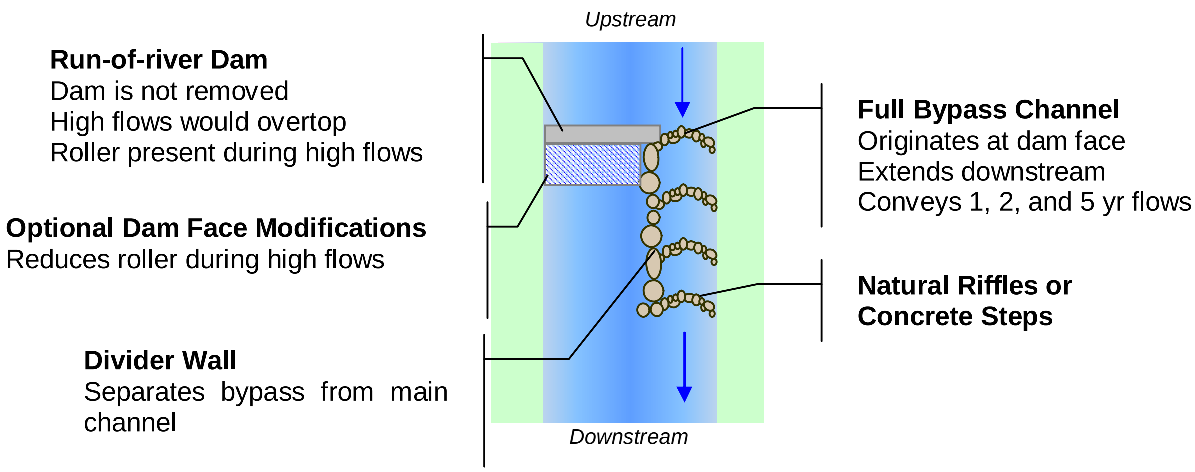
An in-stream bypass channel follows sizing criteria similar to that used for the full bypass channel. In-stream bypass channels were sized for a 5 year flow with the slope and invert elevation based upon the dam height and existing channel bed elevation.

Table 4.4.3-1 presents concept parameters and opinions of cost for in-stream bypass channels at dams where the option was deemed feasible. Detailed descriptions of these options are given in Sections 4.6 through 4.11.

*Bypass with downstream riffle pools*




*Bypass with downstream riffle pools*




*Bypass with upstream riffle pools*

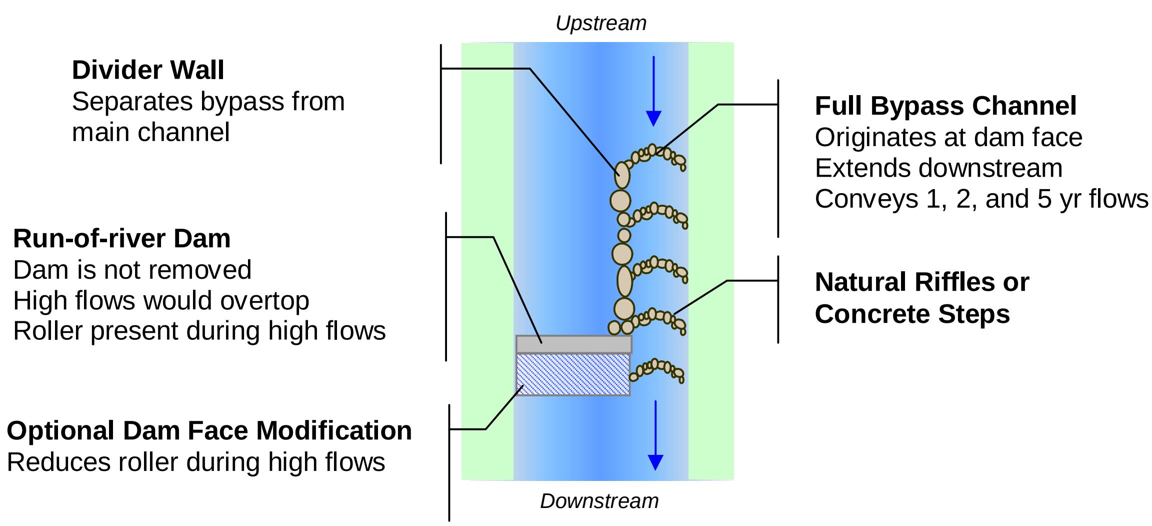



*Figure 4.4.3-1 - In-Stream Bypass Channel Generic Layout*

*Table 4.4.3-1 - In-Stream Bypass Channel Option Parameters and Opinions of Cost*

| Dam | Dam Crest Elevation (NGVD - ft) | Channel Slope | Channel Invert (NGVD -ft) | Number of Riffles | $D_{30}$[1] Size (ft) | Dam Notch Height (ft) | Dam Notch Width (ft) | Wall Length (ft) | Channel Excavation ($yd^3$) | Channel Riprap ($yd^3$) | Riffle Boulders ($yd^3$) | Bank Riprap ($yd^3$) | Opinion of Cost[†] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Lower Sterling | 627.00 | 2.5% | 617.56 | 5 | 2.2 | 8.0 | 300.0 | 190 | 0 | 2,408 | 400 | 161 | $ 1,960,000 |
| Elgin Kimball Street | 708.40 | 2.5% | 703.39 | 7 | 1.6 | 5.0 | 80.0 | 249 | 0 | 1,526 | 187 | 370 | $ 1,430,000 |
| South Elgin | 700.00 | 2.5% | 696.84 | 3 | 1.3 | 3.7 | 150.0 | 90 | 0 | 644 | 105 | 57 | $ 480,000 |
| Geneva | 675.40 | 2.5% | 671.62 | 3 | 1.3 | 5.8 | 150.0 | 140 | 595 | 864 | 158 | - | $ 780,000 |
| North Aurora | 646.00 | 2.5% | 641.79 | 5 | 1.5 | 6.0 | 150.0 | 217 | 1,656 | 2,588 | 280 | 121 | $ 1,610,000 |
| Hofmann | 603.50 | 2.5% | 599.18 | 5 | 1.4 | 8.4 | 100.0 | 348 | 3,450 | 829 | 173 | 80 | $ 1,290,000 |

[1]The $D_{30}$ value represents a specific range of stone riprap diameters where 30% of the stone is sized at or below the specified diameter.

[†] These costs are based on limited information and are for planning purposes only. They do not include costs associated with land acquisition, final engineering design, permitting, or environmental considerations such as sediment quality.

*References*

References used when considering the in-stream bypass channel option were the same as those used for the temporary rock fill, full bypass channel, and riffle pool rock ramp options.

Source: CanoeColorado.com

# Evaluation of Public Safety at Run-of-River Dams

4.4 *Permanent Structural Options*

4.4.4 Dam Face Modification

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#### 4.4.4 Dam Face Modification

Due to the limited scope of this report the dam face modification options considered were limited to adding a stepped face to the downstream dam face. The height and length of each stepped face considered was based on the current design of the Yorkville Dam improvements since detailed physical analyses had been conducted. These heights and lengths may require adjustment during the detailed design to account for the differences in dam faces, flow rates, and downstream bed elevations when compared to Yorkville. A series of steps was designed from the dam crest to the downstream bed elevation, based on a comparison of the existing FIS bed profile, IDNR survey data, and CTE survey data. Additional survey should be conducted prior to a final design to determine the extent of scour and the current bed profile. The initial step was set at an elevation equal to the top of fill elevation calculated for the temporary rock fill structural option to prevent increases to the upstream 100 year water surface. The first step length was set at 4.78 ft, and each subsequent step length was set at 5.625 ft. Each step height was set to 1.125 ft, while the final step height was determined based on the downstream bed elevation. Steps originate at the dam face and continue downstream to the existing bed elevation, where a riprap apron then extends 40 ft at a depth of 3 ft. The length of the apron was selected for cost estimating only, based on the range of apron lengths used at the Yorkville Dam. A stone $D_{30}$ diameter was computed for the apron such that it would be stable for the 50 year flow event at an assumed river bed slope of 0.5%. Volumes of concrete and riprap size A3 were computed for the steps based on the bed elevation at the dam face. It was assumed that 70% of the volume under steps would be concrete and 30% of the volume would consist of riprap A3 fill. Table 4.4.4-1 shows the elevation of the first step, the total number of steps, the total length of the steps, the $D_{30}$ diameter and volume of stone required for the apron, and the volume of concrete and riprap A3 computed for the steps. It should also be noted that anchoring of the steps to the dam face will be required but is not included in this cost estimate due to the variety of types and conditions of the dam faces. Additionally, other materials such as natural stone may be used instead of concrete, but these have not been considered in this planning level study.

Table 4.4 presents concept parameters and opinions of cost for stepped dam face modifications at dams where this option was deemed feasible. Detailed descriptions of these options are given in Sections 4.6 through 4.11.

*Table 4.4.4-1 – Stepped Face Design Parameters and Opinions of Cost*

| **Dam** | **Elevation of First Step** (NGVD - ft) | **Number of steps** | **Step Length** (ft) | **$D_{30}$[1] Size** (ft) | **Step Volume *Concrete*** ($yd^3$) | **Step Volume *Riprap A3*** ($yd^3$) | **Apron Volume** ($yd^3$) | **Opinion of Cost**[†] |
|---|---|---|---|---|---|---|---|---|
| Wilmington | 530.00 | 4.00 | 21.66 | 0.8 | 1,408 | 604 | 2,822 | $ 3,890,000 |
| Sears | 552.98 | 8.00 | 44.16 | 0.5 | 3,168 | 1,358 | 2,240 | $ 5,690,000 |
| Lower Sterling | 625.13 | 8.00 | 44.16 | 0.6 | 9,663 | 4,141 | 4,280 | $ 14,980,000 |
| McHenry (Stratton L&D) | 735.70 | 4.00 | 21.66 | 0.4 | 416 | 178 | 982 | $ 1,260,000 |
| Algonquin | 729.30 | 5.00 | 27.28 | 0.4 | 1,229 | 527 | 1,080 | $ 2,300,000 |
| Elgin Kimball Street | 707.24 | 10.00 | 55.41 | 0.4 | 2,564 | 1,099 | 1,387 | $ 4,360,000 |
| St. Charles | 683.09 | 7.00 | 38.53 | 0.4 | 1,240 | 531 | 1,284 | $ 3,100,000 |
| Geneva | 671.74 | 3.00 | 16.03 | 0.4 | 328 | 140 | 1,653 | $ 1,410,000 |
| North Aurora | 641.63 | 5.00 | 27.28 | 0.4 | 2,110 | 904 | 1,676 | $ 3,860,000 |
| Aurora East | 624.44 | 4.00 | 21.66 | 0.5 | 255 | 109 | 742 | $ 1,560,000 |
| Hofmann | 599.94 | 5.00 | 27.28 | 0.3 | 573 | 246 | 1,173 | $ 2,820,000 |
| Danville | 518.00 | 8.00 | 44.16 | 1.2 | 1,410 | 604 | 978 | $ 2,520,000 |

[1] The $D_{30}$ value represents a specific range of stone riprap diameters where 30% of the stone is sized at or below the specified diameter.

[†] These costs are based on limited information and are for planning purposes only. They do not include costs associated with land acquisition, final engineering design, permitting, or environmental considerations such as sediment quality.

*References*

The following references were used when considering the stepped dam face modification option. References listed in the other structural option sections were also used.

Freeman, Jeffrey W. and Marcelo H. Garcia, Hydraulic Model Study for the Drown Proofing of Yorkville Dam, Illinois, May 1996, Prepared for Illinois Department of Natural Resources Office of Water Resources, Hydrosystems Laboratory, Department of Civil Engineering, University of Illinois at Urbana – Champaign, Urbana, Illinois.


Source: Emily Stanley, University of Wisconsin

# Evaluation of Public Safety at Run-of-River Dams

4.5 *Dam Removal*

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## 4.5 *Dam Removal*

Fifteen dams were identified for potential removal, since dams with significant hydropower, cooling water impoundment, or major upstream recreation were not considered viable for removal. Field investigations were conducted at each dam, which documented the condition of the dam channel type, surrounding land uses, construction, and unique properties of the dam, impoundment conditions and stream channel characteristics. The investigation along with available information and FEMA data was used to provide an evaluation of alternatives for dam removal. For each dam, a concept layout to show how removal could be accomplished, and an order-of-magnitude opinion of cost were developed.

Many of the dams assessed in this study are located in urban areas with social and/or historic interest. Run-of-river dams, typically built in the 19$^{th}$ and early 20$^{th}$ centuries, were generally constructed to provide mechanical water power for mills. Dam builders often preferred river sites with shallow bedrock that provide a solid foundation, and sites at the upstream end of a riffle or rapids to maximize the available head. River sites with firm substrate and shallow riffles were also excellent fords for road crossings, and later for highway bridges. As a result, many communities developed at the dams and associated mills. The dams at Rockport, Elgin, Carpentersville, St. Charles, Geneva, Batavia, Aurora, and Montgomery all fall into this category.

### 4.5.1 General Approach

Due to the variations in type and condition of each dam and its surrounding land uses, a variety issues and, subsequently a variety of removal concepts, arise. Specific site issues control which type of removal technique is most appropriate, taking into account the operation and maintenance needs required for each option. Removal of each dam raises numerous special concerns as noted on the following page. Only some of these concerns apply to specific dams; however, in general, a final removal plan will need to mitigate any adverse impacts that are predicted to occur.

Construction Access

In order to evaluate the difficulty of dam removal, construction access to the dam must be evaluated. Access must allow both personnel and vehicles safe and stable access to the dam at vital locations relative to the removal process. Access points can be at the left or right abutment, along the downstream channel, or along the upstream channel for cofferdam installation. Consideration must also be given to assessing usable contractor staging areas which will pose minimal disturbance to adjacent land uses.

Erosion

Modification or removal of the dams must not create or encourage excessive riverbed scour. Removing or breaching a dam will alter upstream flow velocities and could potentially result in upstream bank erosion if not properly addressed in the design process. Accordingly, hydraulic modeling results of proposed conditions should always be utilized to evaluate future velocities in the channel over a range of river flows. Where erosion is predicted to occur, the upstream banks must be properly stabilized to minimize erosion.

Fish Passage and Habitat

River channels must have appropriate velocities, width, and depth over a range of flows to allow for fish passage and to sustain a viable aquatic habitat.

Flooding

Any proposed modifications to the dams and surrounding areas must not significantly raise upstream or downstream floodwater levels. Exacerbating existing flood problems would not be acceptable to the surrounding communities. Additionally, FEMA regulations preclude raising flood levels without first revising the community's flood insurance study.

Land Use

Several of the dams are located in urban areas where communities have developed formal walkways and bikeways near the dams, as well as scenic overlooks and parks. Modifying or removing these dams may result in extensive infrastructure disturbance and post removal repair.

Pool Usage

Dams often times create a large upstream impoundment area or pool that has multiple uses. As part of a dam removal assessment, this pool must be taken into account. If a dam is removed, existing recreational and water supply uses must be evaluated.

<u>Sediment Control</u>

Various options are available for addressing sediment control. The "do nothing" option can be applied where sediment is present and but when analysis indicates that natural erosion will not pose a hazard to the health of the watercourse or habitat. Cofferdams can be installed that allow for a more gradual erosion of material, but these add to the cost of the removal. In cases where large sediment deposits could pose a threat if released upon dam removal, partial or full dredging of the impoundment area may be warranted. The excavated sediment must be properly disposed of. The final option is to stabilize the impounded sediment in place, and gradually draw down the water upstream of the dam.

<u>Sediment Considerations</u>

The status and disposition of sediment deposits in the pools upstream of each dam is a key element when considering crest reduction, partial dam breach, or full dam removal. Specific concerns include potential mobilization of pollutants, creation of downstream-suspended sediment loads and increased turbidity, downstream sediment deposition that buries the native substrate, and unstable incised channels in the pool areas. Understanding sediment processes requires knowledge of geology, soil mechanics, fluid mechanics and hydraulics. The erosion, transport, and deposition of sediments are functions of the hydrologic conditions and the fluid in which they are immersed. The movement of sediment particles is also dependent upon particle size, shape, and densities, as well as the cohesion, shear strength, chemistry, and density roughness of the sediment deposit.

*Sediment Transport*

Dams and their low velocity pools act as settling basins that trap sediments carried by the river. In some cases, entire impoundments can be filled with sediment, complicating dam removal. The movement of sediments in a river occurs when the flow of water exerts sufficient force on the individual particles to initiate movement and then transport them. Lighter particles are carried in suspension while heavier particles are dragged, rolled, or slide along the riverbed as “bed load.” The driving force is due to the drag caused by the flow and related turbulent lift forces; the resistance forces are the weight of the particle, friction, and cohesion. Numerous methods are available with which to evaluate the long-term potential stability of the sediments impounded behind the dams as a guide to whether they should be left in place, stabilized in place, or removed. These methods include, but are not limited to, threshold velocity methods, critical shear stress methods, regime theory, geomorphic relations, stream power theory, sediment transport (steady state), and sediment transport (continuous simulation).

Fortunately, most of the run-of-river Illinois dams that were surveyed appear to have limited sediment. However, much more detailed investigations will be needed at a later date to confirm sediment volumes, stability, and contaminants.

*Sediment Management*

Several common approaches are available for handling sediments within a riverbed. The simplest and least expensive option is to simply leave the sediments in place, with the understanding that some will be washed downstream during and shortly after the initial dam breach and possibly during flood events. Another alternative is to relocate the sediment within the channel to a less erosion prone area to avoid having to remove it from the site. A third approach to sediment management is to stabilize it in place with an erosion resistant cover material (i.e., rock riprap) or vegetation. The final option is to dredge the sediment from the impoundment prior to breaching the dam.

Dam impoundments often have significant sediment loads that settle on the upstream side of the dam. The volume of sediment accumulated is attributed to many factors including flow velocity and type of channel. When a dam is removed, there is a possibility that this accumulated sediment will wash downstream since it is no longer being held in place by the impounded water and spillway. Large sediment loads can have serious impacts on water quality as well as downstream channel stability. The IEPA has previously expressed concern with regard to sediment management related to other dam removal projects. Among the options for sediment management are to allow sediments to erode naturally, stabilize sediments in place upstream of the dam, relocate sediment on site, and / or remove sediment and dispose of it off site.

<u>Water Control</u>

All dam removal operations require that a certain amount of water be controlled or diverted in order to complete the removal. The method used depends on engineered flow control devices, flow velocities and volumes, construction issues, safety issues and other channel characteristics. Example situations and potential solutions include the following:

- Existing low level outlet or gate, with capacity, and is operable
  - Solution – Drain channel and then breach dam
- Existing raceways with gates
  - Solution - Drain channel through the raceway
- Bypass channel at abutment
  - Solution – Drain channel and then breach dam
- Breach embankment
  - Solution - Live breach of the dam with armored channel
- Notch spillway
  - Solution - Live breach of dam in several phases

Dam Removal Complexity

The list of parameters shown in Table 4.5-1 help identify the degree of effort and complexity that may be anticipated at each dam, keeping in mind that some parameters are more important than others. These parameters have been used to determine the complexity of dam removal at each site, and subsequently support the opinions of cost provided in this report.

*Table 4.5-1 - Dam Removal Complexity*

| **Site Parameter** | **Parameter Complexity** | |
|---|---|---|
| | *Simple* | *Complex* |
| Owner Cooperation | Good | Poor |
| Construction Plans or NID Report | Yes | No |
| Dam Height | <6 feet | >25 feet |
| Dam Length | <50 feet | >100 feet |
| NID[1] Hazard Class | Low | High |
| Mean August Flow | <20 cfs | >100 cfs |
| Ratio Dam Length to River Width | <3 | >6 |
| Industrial/Hazardous Waste Pollution Potential | None | Possible |
| Sediment Grain Size, Impoundment | Gravel | Silt, clay |
| Impoundment Sediment Layer Thickness | <3 feet | >6 feet |
| Alluvial River Channel | No | Yes |
| Downstream Channel Gradient | Steep | Mild |
| Upstream Valley Width | Narrow | Broad |
| Rare or Endangered Species | No | Yes |
| Dam/Pond Historic Value | Low | High |
| Pond Habitat Value | Low | High |
| Pond Recreational Value | Low | High |
| Available Flood Insurance Study | Yes | No |
| Hydroelectric Power | No | Yes |
| Water Supply Shed | No | Yes |
| Water Quality | Good | Poor |
| Intensive Shoreline Development | No | Yes |
| Public Utilities Present | No | Yes |
| Adequate Construction Access | Yes | No |
| Percent of Annual Runoff Stored | <2% | >10% |
| Percent of Annual Sediment Load Stored | <50% | >100% |
| Adequate Funding | Yes | No |
| Physical Condition of Dam | Good | Poor |

[1] NID – National Inventory of Dams

### 4.5.2 Dam Removal Options

While the following paragraphs describe various options for dam removal, not all may be viable options for each dam. Each option is discussed in greater detail in subsequent sections as they are applied to each dam.

No Action

Under a no action option, the dam would remain in place. This alternative does not address the existing public safety concern. When a dam is left in place, it requires maintenance to prevent deterioration and ultimate failure. Sediment continues to build up behind the dam, increasing upstream channel bed and water surface elevations. Potential debris jams at dams can also increase upstream flooding during critical periods of high flow. Run-of-river dams raise upstream flood levels, while having little or no reduction in downstream flood levels.

Full Dam Removal

Complete dam removal is often the most desirable alternative for a variety of reasons. The benefits of full dam removal are numerous and include public safety, re-establishment of historic flow conditions, elimination of channel snags and debris associated with impoundments, avoidance of future potential dam breaks, and recreational boat passage.

Full dam removal can become expensive if large amounts of contaminated sediment have accumulated in the impounded area and must be removed and disposed of off site. The environmental impacts of removing a dam in the case of contaminated sediments can sometimes be prohibitive. The fate of impounded sediments and the associated environmental impacts must be addressed when developing a final removal plan. Numerous methods of handling impounded sediments are available, including conventional excavation, mechanical dredging, hydraulic dredging, and relocation of sediments on site, partial removal of the sediments with erosion protection, staged breaching, and removal through natural erosion.

In certain situations, simple removal may not be feasible due to the possibility of undermining upstream structures. Additionally, the capacity of downstream structures and the quality of downstream habitat can be diminished if significant volumes of sediment are released and deposited downstream during and after dam removal.

Partial Dam Removal (e.g., breaching or lowering)

Partial dam removal can be a viable alternative where complete dam removal is not feasible or necessary. Partial dam removal alternatives can be accomplished by breaching a dam along a portion of its length or by lowering the dam crest. Partial removal is not feasible if the remaining portion of the structure is unstable, since removing a portion of the dam could cause a failure of the entire structure. For example, removing the top course of large stones from a masonry dam exposes smaller internal rock to higher stress and erosion, potentially leading to structural failure.

Partial dam removal may be less costly than full removal and may provide similar benefits. However, the remaining portion of the dam can be unsightly and may collect debris and backwater flood flows. In addition, when the sediment behind a dam is contaminated, lowering the dam crest to the level of the accumulated sediments may provide a more cost effective solution as compared to a complete dam removal. Lowering a dam can sometimes be accomplished in combination with other remedies, such as a riffle pool rock ramp, in-stream options bypass channel, or stepped face dam modification.

### 4.5.3 Cost Items Associated with Dam Removal

Due to the varying dam removal options, each project will incur different costs. A multitude of cost factors exist, depending on the specific site restrictions, dam, spillway type, and flow. Detailed opinions of cost are provided in Appendix III, while a summary is provided in Table 4.5-2 on the following page. These cost opinions are based on the removal option, variable construction rates, and estimates of time and material expenses. A comprehensive list of construction activities that could be required as part of a dam renovation is listed below. Not all items will apply in all cases, and in cases where extensive restoration of private property or highly used public recreation areas is required, "loss of use" costs are often difficult to quantify. The general cost items are as follows:

- Mobilize, set up staging area
- Open gates or headrace
- Access breach, bank area
- Create partial breach, drawdown
- Access haul road, bank area
- Extend haul road, cofferdam along dam
- Progressive demolition of dam
- Monitor, stabilize
- Restore impoundment bed
- Bridge scour protection
- Utility protection
- Clear and grub
- Bike trail repair
- Urban park property repair
- Fish relocation
- Earth excavation

- Sediment stabilization, relocation and monitoring
- Addressing special issues of sediments

In order to access the dam with construction equipment, a significant portion of the water must be removed from the work area. This can be achieved by installing a temporary cofferdam. There are many options for the type of cofferdam that can be used, such as earthen cofferdams, Portadam, or Geotubes. The type used must be decided on a case by case basis during a final design.

In many cases, the dams are located upstream of major bridge crossings. Dams serve to dissipate the flow energy of the river waters by impounding large volumes of water and creating a hydraulic jump. With the removal of the dam, this energy dissipation is also removed. River water can become more forceful and potentially dangerous for bridge piers and structures within the river. Removing or breaching dams will reduce upstream water levels in the former pool area, resulting in higher velocities. These velocities may create scour along the banks and at upstream bridge piers or abutments. In cases where the dam alteration may introduce increased concern for upstream and downstream scour, additional protection may need to be implemented.

Many of the subject dams are located in highly urbanized areas, and as such, have been incorporated into downtown destination areas. Some areas surrounding the dams have been renovated to include gardens, bike trails, scenic overlooks and parks. With the installation of construction access roads and the presence of large construction equipment, dam removal may have the potential to disturb these areas. If this is the case, the primary concern may not just be in estimating the cost of dam removal, but rather estimating the cost of the post-removal repair costs.

*Table 4.5-2 - Dam Removal Opinions of Cost*

| Dam | Opinion of Cost[1] | |
|---|---|---|
| Momence | $ | 380,000 |
| Lower Sterling | $ | 8,290,000 |
| Carpentersville | $ | 940,000 |
| Elgin Kimball Street | $ | 3,290,000 |
| South Elgin | $ | 720,000 |
| St. Charles | $ | 2,250,000 |
| Geneva | $ | 2,380,000 |
| Batavia | $ | 2,030,000 |
| North Aurora | $ | 1,550,000 |
| Aurora East | $ | 2,900,000 |
| Montgomery | $ | 670,000 |
| Hofmann | $ | 1,850,000 |
| Danville | $ | 2,050,000 |
| Riverside | $ | 270,000 |
| Petersburg | $ | 290,000 |

[1] These costs are based on limited information and are for planning purposes only. More detailed information on sediment may result in a substantial increase in dam removal costs.

*References*

The following references were used when considering the dam removal option. References listed in the other structural option sections may also have been used.

*Dam Removal Costs,* American Rivers, Downloaded 2007, www.americanrivers.org.

ICF Consulting, *A Summary of Existing Research on Low-Head Dam Removal Projects*, Prepared for the American Association of State Highway and Transportation Officials, September, 2005.

McLaughlin Water Engineers, Ltd., Patrick Engineering, Inc., John Anderson, Chinook Engineering, *Alternative Evaluation Report, Replacement of Upper Batavia Dam, Kane County, Illinois,* Prepared for Illinois Department of Natural Resources, December, 2000.

*Dam Removal Science and Decision Making,* The H. John Heinz III Center for Science, Economics and the Environment, 2002.

U.S. Army Corps of Engineers and Illinois Department of Natural Resources, *Hofmann Dam Section 206 Ecosystem Restoration Detailed Project Report*, December 2006.

U.S. Army Corps of Engineers, *Section 206 DRAFT Preliminary Restoration Plan for the 5th Avenue Dam Removal / Modification Ecosystem Restoration Project,* January 2004.

# Evaluation of Public Safety at Run-of-River Dams

## 4.6 *Kankakee River Dams*

### 4.6.1 Momence Dam

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## 4.6 *Kankakee River Dams*

The following section includes a description of the existing conditions of and proposed structural options for dams located on the Kankakee River.

### 4.6.1 Momence Dam

The following section includes a description of the existing conditions of and proposed structural options for Momence Dam.

#### 4.6.1.1 Existing Conditions Overview

Momence Dam was visited on January 22, 2007 and March 19, 2007. Observations made on these occasions and a review of IDNR summaries form the basis for the existing conditions summary.

*Dam Structure and Channel Condition*

The Momence Dam is a run-of-river, dam with a length of 152 ft and a water elevation differential of approximately 3 ft. It is located between the river's right bank and a mid channel island, immediately behind City Hall. The owner of the dam is currently unknown. There is no equivalent dam structure on the left side of the island, where the left channel free flows at 1 to 2 ft per second under normal flow conditions. The spillway is located 380 ft downstream of a modern three-span bridge with steel beams and 2 concrete piers. The weir has a broad crest approximately 8 ft wide and is constructed of stone masonry. The total dam height was not visible, but survey suggests it could be up to 6 ft.

The left abutment is located on a private island and is comprised of loose cut stones and is in poor condition, with a bank failure just downstream of the dam. The downstream end of the island hosts one residence and several small buildings, with a broad low level grass field behind the abutment. The right abutment is composed of limestone masonry with mortar, in poor condition. The sidewalk along the right bank upstream of the dam has been undermined and is dangerous to use. The vertical abutments are concrete. The downstream right bank has concrete poured over rock masonry and earth, probably a scour repair. June 20, 2006 photographs by IDNR indicate that the bank has been undermined by scour. Two large trees are lodged on the dam crest and one is caught in the roller.

An interview with a local angler suggests that the area provides good fishing for bass, walleye, and muskie. They report that the left side of the islands channel is located on bedrock and the right channel above the dam is 4 to 5 ft deep with 1 ft of silt/sand over bedrock. Local police report that despite the low banks, they do not know of any overbank flooding. Both dam abutments and adjacent banks are in poor condition.

*Channel Type/Flow Regime*

During inspection, the dam head (i.e., depth of flow over the dam crest) was approximately 0.5 ft, with a distinguishing drawdown in the approach flow. The dam has a strong reverse roller due to plunging flow at a 45 degree angle. The roller extends approximately 6 to 10 ft downstream of the face of the dam.

*Surrounding Land Uses*

The left bank upstream of the dam is a mid-river island containing one residential house. The right bank upstream contains the City Hall and an associated parking lot. The left bank downstream of the dam is a grass field located on the same, mid-river island.

*Portages*

There do not appear to be any established portages. Portages on the left bank would be ineffective since the bank is an island and all private property. The right bank could support a portage upstream, although the bank would have to be regraded. The downstream right bank seems to be entirely private property, limiting access for a portage.

*Warning/Information Signage*

No signage was observed with regard to dam safety on either bank upstream or downstream. Information signs noted were "Attention Anglers" (104) for fish identification, located along the upstream right bank, and an "Adopt a River" and (119), "No Trespassing" (121) sign of the right bank.

*Construction Access*

The right abutment and bank is directly accessible from the City Hall and police station parking lot, but contractors would need to avoid blocking the police department garage doors. A 30 ft wide level grass lawn separates the parking lot from the dam abutment, requiring a gravel pad if used for construction access. A short length of fencing runs along the right upstream bank, immediately downstream of the vehicle bridge. The fence extends a short distance to the bank and does not continue along the river walk. Access from the right abutment would require removal of this fence.

The left abutment is located adjacent to private, residential land on the island, therefore access along the left bank is distinctly restrictive. However, there is potential for access farther upstream at a local park.

#### 4.6.1.2 Options Assessment

##### 4.6.1.2.1 Temporary Rock Fill

A consistent lack of information for Momence Dam requires that a detailed hydraulic analysis be conducted along with additional survey data. The crest elevation of the dam is unknown and is not clearly defined on the FIS profile. At the least, the crest elevation of the dam is needed. Additionally, the downstream water surfaces are likely impacted by the confluence of the main channel directly downstream. The only flow data available are the 10, 50, 100 and 500 year flows downstream in the main channel. To accurately estimate the percentage of flow that occurs in the north channel where the dam is located requires a detailed hydraulic analysis. For the purposes of this report, a cost estimate is provided based on a rock fill layout comparable to the other run-of-river dams. The design characteristics of the rock fill have been select in an attempt to provide a conservative cost estimate. The following characteristics of been selected for the temporary rock fill at Momence Dam:

- A rock fill with a slope of 5% has been assumed.
- A crest elevation of 618 ft NGVD was assumed, which appears to be the highest elevation on the FIS profiles. A top of fill elevation of 617 ft NGVD has been assumed.
- A $D_{30}$ value of 2.0 ft was assumed.

##### 4.6.1.2.2 Dam Removal

_Removal Feasibility_

The Momence Dam is a small structure with easy access to both ends, enabling a straight-forward demolition process. However, the water is 5 ft deep on the downstream side and cannot be waded by conventional heavy equipment.

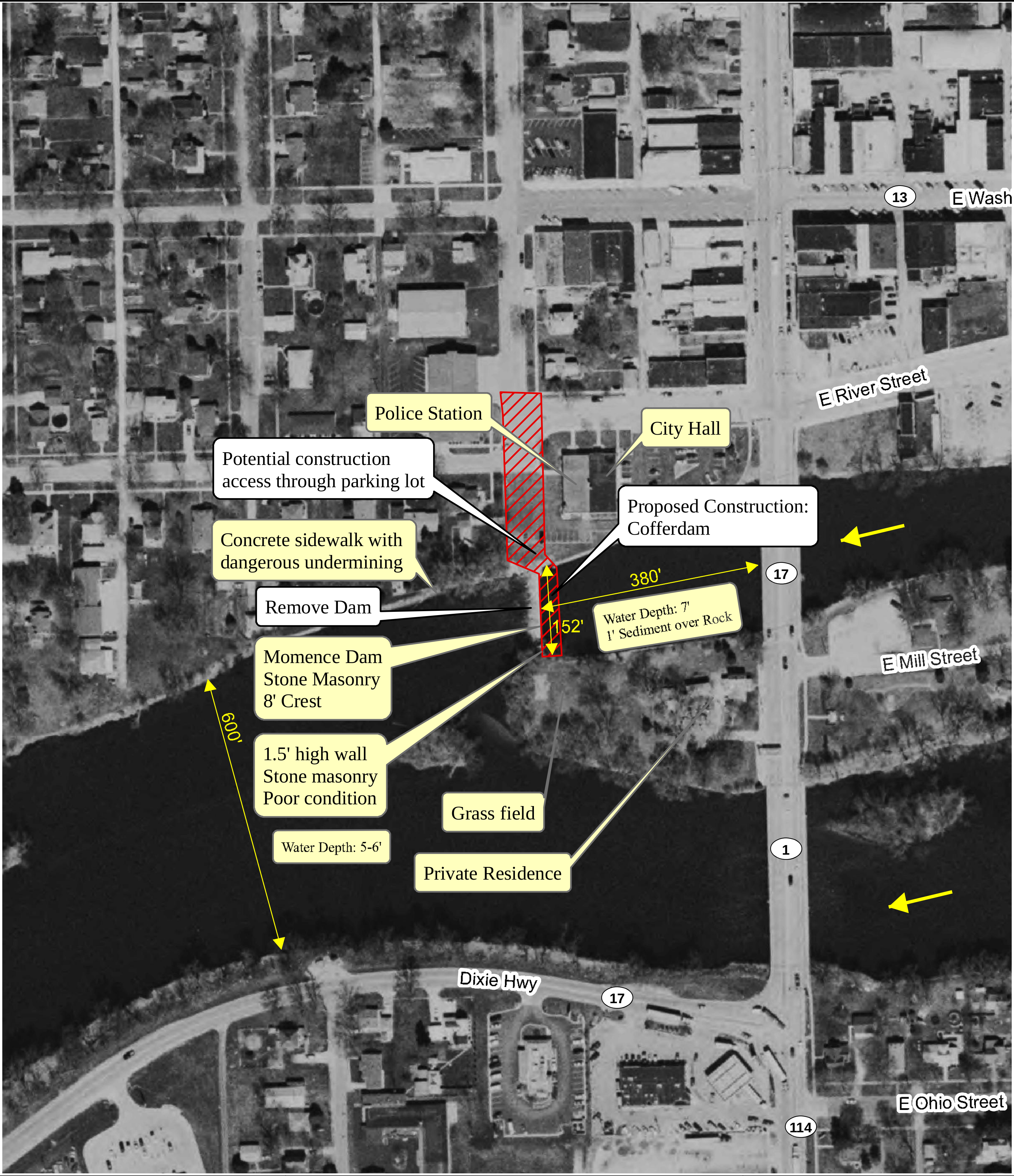
The spillway could be breached and partially removed with use of long boom backhoes (60 ft each) or cranes operating from shore, without use of cofferdams. Operating from shore and through the water would create a significant breach, likely providing boat passage without a hydraulic jump. Shore access will disturb the City Hall parking lot and river bank sidewalk. The various layers of spillway rock masonry should readily pull apart with proper heavy equipment.

Full removal of the dam will require use of a temporary cofferdam to dewater the site and provide a haul road across the river. Simply diverting water to the left side of the island will not provide adequate access due to high tailwater against the downstream side of the dam. The exact cofferdam and access road locations would depend on detailed bathymetric surveys. For budget purposes, it is assumed that rock fill is placed across the river against the dam, to both control water and provide access.

*Special Issues*

Breaching or totally removing the dam will alter the flow patterns around the island and could impact normal water surface elevations. It is anticipated that there would be less flow on the left side, more flow on the right side, unless secondary controls are implemented. Nevertheless water levels in the right channel behind City Hall would decline by approximately 2 ft. Remaining waters would likely still fill the full channel width. Initial evaluation indicates that there is minimal sediment behind the dam and is not expected to be an issue.

The dam removal design should evaluate future flow velocities along the banks of the right channel and assess bank stability, as some bank work is anticipated. The Dixie Highway Bridge over the right channel should also be checked for potential scour and its probable foundation on bedrock should be verified.




| | | | |
|---|---|---|---|
| *Engineering, Landscape Architecture and Environmental Science*<br>**MILONE & MACBROOM®** | **IL Dam Removal Project**<br>**Momence Dam - Kankakee River** | | **LOCATION:**<br>**Momence, IL** |
| **99 Realty Drive**<br>**Cheshire, Connecticut 06410**<br>**(203) 271-1773 Fax: (203) 272-9733**<br>**www.miloneandmacbroom.com** | **MMI#:** 3131-01<br>**MXD:** H:Momence.mxd<br>**SOURCE:** | N<br>**Existing Conditions and Proposed Removal** | **DATE:** 04/10/07<br>**SCALE:** 1":200' |
| | | | **SHEET:**<br>**Figure 4.6.1.2.2** |

### 4.6.1.2.3 Full Bypass Channel

The full bypass channel option was not considered due to the apparent lack of public land on either bank. Additionally, a bypass exists through the south branch of the Kankakee River. However, based on interviews with emergency personnel, depths may be insufficient to allow passage during low flows. This should be further investigated during detailed designs.

### 4.6.1.2.4 Riffle Pool Rock Ramp

A riffle pool rock ramp was developed for this reach (See Figure 4.6.1.2.4). A riffle pool rock ramp could be placed, beginning at the dam crest and continue downstream with a 1% slope. Consideration must be given for the potential of downstream flooding, especially on the right bank. Installation of retaining walls on one or both banks could be considered to remedy this problem.

### 4.6.1.2.5 In-Stream Bypass Channel

The in-stream bypass channel option was not considered for this reach due to the narrow width and low height of the dam, as well as the poor dam condition. An in-stream bypass would likely require a large width due to the need to convey flows from both the north and south branches of the Kankakee River in order to lower stages to desired levels. Lowering of stages could also result in an undesirable decrease in pool depth, potentially drying the channel.

### 4.6.1.2.6 Dam Face Modification

Dam face modification was not considered for this reach due to the poor condition of the dam. Momence Dam has experienced extensive spalling and appears to have partially failed along portions of the dam crest.

### 4.6.1.2.7 Summary of Cost

The costs for each option vary due to the type of material used, required access to the site, and other considerations that are discussed in the detailed cost section. The opinions of cost for the temporary rock fill and riffle pool options are $470,000 and $690,000, respectively. The opinion of cost for dam removal is $380,000.


RIFFLE POOL
ROCK RAMP

Momence Dam - Kankakee River

*Momence, Illinois*

CTE | AECOM

Figure 4.6.1.2.4

Momence


# Evaluation of Public Safety at Run-of-River Dams

## 4.6 *Kankakee River Dams*

## 4.6.2 Kankakee Dam

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### 4.6.2 Kankakee Dam

The following section includes a description of the existing conditions of and proposed structural options for Kankakee Dam.

#### 4.6.2.1 Existing Conditions Overview

Kankakee Dam was visited on January 22, 2007. Observations made on these occasions and a review of IDNR summaries form the basis for the existing conditions summary.

*General Dam & River Bank Condition*

The dam crest appears to be in good condition and has been modified to include an inflatable "bladder" system with a steel plate fixed to the upstream face of the bladder. The bladder is used to raise the water surface for hydro-power operations. A section of the dam, roughly 1/3 of the total dam length, beginning from the left embankment, was visible discolored, though the cause was not identifiable during the visual inspection.

There are minor surface cracks at both abutments, although in general the left abutment appears to be in good condition, while the right abutment does exhibit some spalling and debris at the dam crest.

*Evidence of Roller*

The reverse roller was roughly 20 to 30 ft wide along the entire length of the dam, and extremely turbulent, especially at the right bank. It should be noted that the downstream right bank experienced a significant eddy and return flow with a roller length of about 50 ft, possibly due to the alignment of the left embankment wall.

*Portages*

There are two portages on the upstream left bank between the traffic bridges immediately upstream. There is also one portage / boat ramp on the right bank downstream of the bridge immediately downstream of the dam. Neither location had signage stating the existence of the portage or of the dam. The trek between portages would be around 0.75 miles, under the rail road bridge and over a major roadway intersection. Portages were in good condition and included 1 ft concrete steps. The furthest upstream portage also included a pulley and wench system.

*Warning/Information Signage*

The only warning signs present were related to the hydropower facility. One sign was located on the hydropower facility downstream wall that read “Hazardous Conditions” and “Turbulent Water Horn Will Sound”, both readable from 15 yards. The operators place a boat line across the river during boating season; otherwise there are no warning signs for the dam on either of the two upstream bridges. In addition, there were no warning signs against fishing, swimming, etc. There was a warning sign at the downstream left embankment park stating that the stairs leading to the water were restricted; however, the sign was broken and laying on its side, not visible. Additional warning signs such as no trespassing and “danger keep out” were located on the right bank upstream of the structure.

*Hydropower Assessment*

The Kankakee Dam provides electricity to the local municipal water treatment plant. The hydropower operations include an inflatable bladder mounted on the crest of the dam that can be raised to increase the upstream pool elevation. The operations facility for the intake and discharge structures and the intake and discharge points are located at the left abutment and are accessible to river users. The discharge point is located at the left abutment approximately 100 ft downstream of the dam face and is signed and lighted. The intake point is also signed and lighted. The facility is well fenced, gated, and lighted, and includes signage. Signage at the dam should be in accordance with FERC guidelines; however, it was beyond the scope of this study to verify that. Access is not limited to the intake and discharge by boaters, although there is one sign warning boaters to stay away and the hydropower facility is located along the left bank of the river.

The facilities are operated using an automated computer control system, known as a SCADA system, but it was not determined under what flow conditions the hydraulic bladder and the hydropower facilities operate at. The height of the bladder and the height of the dam crest are also unknown.

Dam modification that would impact the upstream and downstream water surface elevations may negatively impact the hydropower operations by changing the amount of available head. The bladder operation could be adversely affected as well, limiting the ability of the facility to generate electricity.

### 4.6.2.2 Options Assessment

#### 4.6.2.2.1 Temporary Rock Fill

A temporary rock fill was not sized for the Kankakee dam due to several issues. The foremost issue is the lack of available data on the hydropower plant's inflatable hydraulic bladder and the diversion of flows for hydropower operations. The lack of data prevents the sizing of height, slope, and stone size. Were the dimensions of the bladder and the flow diversions known, a detailed hydraulic analysis would still be required to assess the impact of the rock fill on the bladder and the hydropower discharge pipes. While a rock fill solution may be feasible at the dam, a detailed hydraulic analysis is necessary to determine its impacts on the hydropower operations.

#### 4.6.2.2.2 Dam Removal

Dam removal was not considered due to the significant hydropower operations.

#### 4.6.2.2.3 Full Bypass Channel

A full bypass channel was not considered due to the decrease in stage which would result, and which could negatively impact hydropower operations. Additionally, sufficient public land does not appear available on either bank.

#### 4.6.2.2.4 Riffle Pool

A riffle pool option was not considered feasible due to the large height of the dam and its variable height mechanism. It would not be feasible to design a riffle pool to function properly under variable height settings. Additionally, a riffle pool would increase low flow water surface elevations and likely impact hydropower operations.

#### 4.6.2.2.5 In-Stream Bypass Channel

An in-stream bypass channel option was not considered for this reach due to the significant hydropower operations at this dam. An in-stream bypass would conflict directly with the variable dam height mechanism and decrease the upstream pool.

#### 4.6.2.2.6 Dam Face Modification

A dam face modification option was not considered for this reach due to the large height of the dam and depth of the channel. A stepped face option was preliminarily sited but resulted in a stepped face that was approximately 120 ft long with 21 steps. The cost for this face was ruled prohibitive.

#### 4.6.2.2.7 Summary of Cost

No options were considered feasible for Kankakee Dam due to the existing hydropower facilities.

# Evaluation of Public Safety at Run-of-River Dams

## 4.6 *Kankakee River Dams*

### 4.6.3 Wilmington Dam

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## 4.6.3 Wilmington Dam

The following section includes a description of the existing conditions of and proposed structural options for Wilmington Dam.

### 4.6.3.1 Existing Conditions Overview

Wilmington Dam was visited on January 22, 2007. Observations made on these occasions and a review of IDNR summaries form the basis for the existing conditions summary. Final designs of those options discussed here should be made with the consideration of the Wilmington Millrace Dam.

*General Dam & River Bank Condition*

The right abutment is composed of an old stone/granite wall with some cracks. This abutment is in need of periodic inspection due to its age. The upstream side of abutment has a steep slope that was quite slippery and dangerous due to snow and ice. The left abutment is concrete with some deterioration and cracking and is in need of periodic inspection as well.

*Evidence of Roller*

A reverse roller was evident that extends 20 to 25 ft. Large tree branches were observed cycling in roller, and it was clear that the roller is very strong, especially near the left bank.

*Portages*

No marked portages were found on either the right or left banks. A portage could be made at the right abutment; however, it is very close to the dam. River users might attempt to bypass the dam using the right millrace and come upon the dangerous millrace dam. The left bank is private property with no portages, and a “No Trespassing” sign was observed just downstream of dam.

*Warning/Information Signage*

Multiple large, colorful warning signs on the right bank and throughout the park were noted. Precautions to general public as well as to fisherman and boaters are given. There is a large sign positioned for upstream river users, titled “Keep Back, Dangerous Dam”. The left bank has no signage, except a “No Trespassing” sign facing the river. Both abutments have the word “DAM” spray-painted on them. All signs are visible from about 30 yards.

No portage signage was observed.

*Hydropower Assessment*

The Wilmington Dam appears to provide an upstream pool for the Braidwood Nuclear Power Plant, the largest nuclear plant in Illinois containing two pressurized light water reactors. The owner and operator of the power plant, Exelon Corporation, has not yet responded whether the nuclear plant would be able to maintain day-to-day operations as a result of a dam modification or removal. Even if the power plant could maintain day-to-day operations with a decrease in stage, the pool may be necessary during emergency operations. For the purposes of this report, it has been assumed that the dam does provide necessary stages for the power plant.

#### 4.6.3.2 Options Assessment

##### 4.6.3.2.1 Temporary Rock Fill

A rock fill layout was designed for Wilmington Dam based on limited data. The only FIS profile that was available was the 100 year; therefore, the remaining profiles were computed using a normal depth calculation with a bed elevation of 525.1 ft, a water surface slope of 0.0025, and a Manning's n of 0.035. The rock fill parameters are as follows:

- A top of fill elevation of 530 ft, 1 ft below the dam crest
- Slope of 5%
- Sub-critical flow may occur during the 10 year event as well as the 1 – 5 year, eliminating a hydraulic jump at the toe and a reverse roller at the dam crest.

##### 4.6.3.2.2 Dam Removal

A dam removal option was not completed for Wilmington Dam due to a lack of information. It has yet to be determined whether the dam maintains the upstream pool for cooling operations for the Braidwood Power Plant. An inquiry was sent to the owners of the power plant, but a reply was not received prior to the submittal of this report.

##### 4.6.3.2.3 Full Bypass Channel

A full bypass channel option was considered for this reach but deemed infeasible due to the high level of flows and the apparent lack of available public land for such a bypass. The Wilmington Millrace currently serves to bypass flows from Wilmington Dam, and during low flows, the millrace conveys all of the flow. Attempts to increase the Millrace width and depth to increase conveyance up to the 5 year flow proved infeasible due to limited space.

Although determined infeasible in this report, during final design special consideration should be given to modifying the millrace to provide a passable bypass channel. The option would allow avoidance of Wilmington dam altogether and would improve the safety of the Millrace Dam as well.

#### 4.6.3.2.4 Riffle Pool

A riffle pool concept was developed for this reach (See Figure 4.6.3.2.4). A riffle pool placed at the dam crest and continuing downstream of the dam with a slope of 1% is presented.

Special consideration must be given for the potential of downstream flooding. Additionally, further studies are needed to assist impacts on the tributary entering on the left bank, approximately 300 ft downstream.

#### 4.6.3.2.5 In-Stream Bypass Channel

An in-stream bypass channel option was not considered feasible for this reach. An in-stream bypass channel would need to be more than half the width of Wilmington Dam to be effective. This would be nearly equivalent to dam removal.

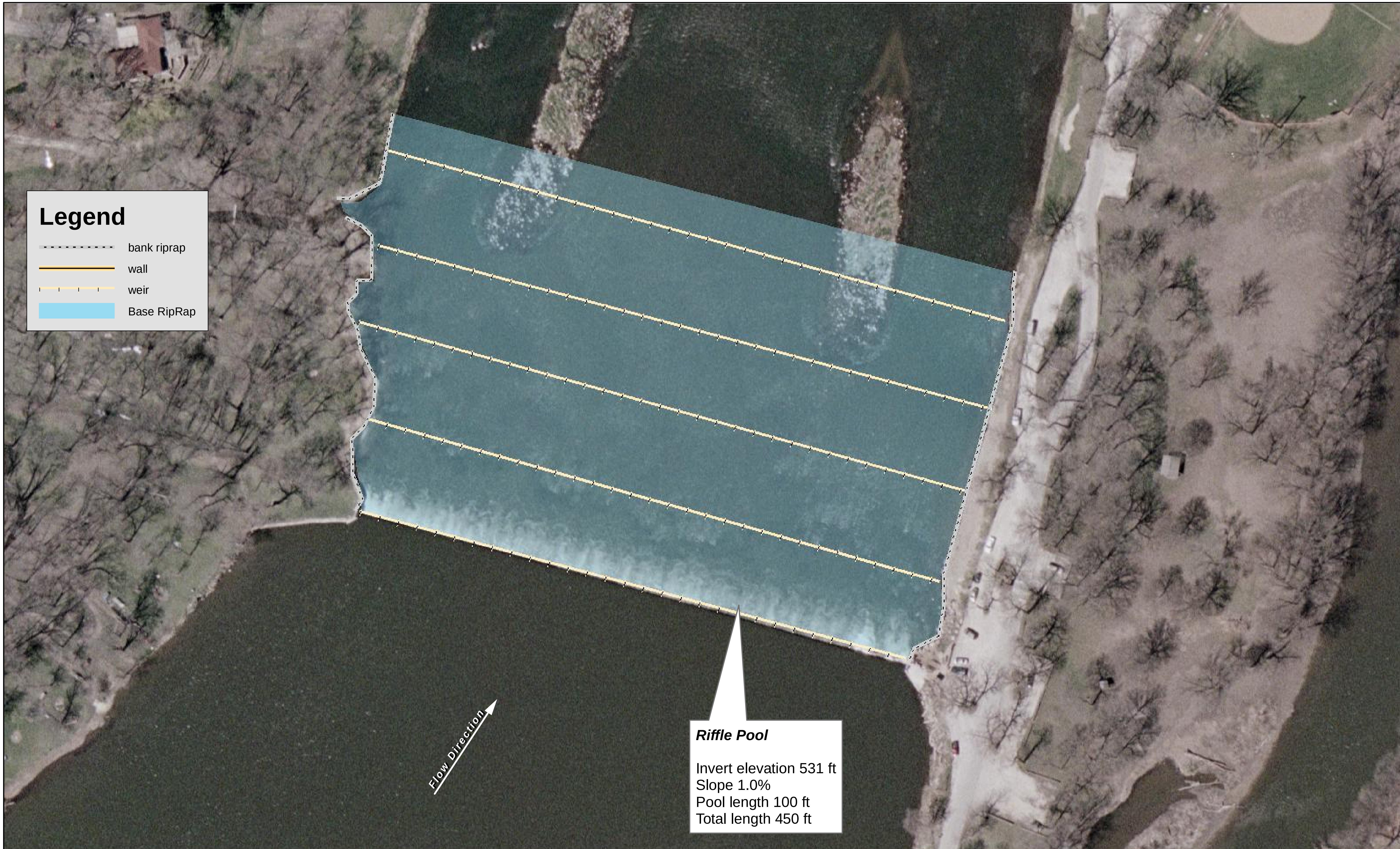
Additionally, it is yet to be determined whether Wilmington Dam maintains the pool for the Braidwood Power Plant. An in-stream bypass could significantly lower the pool during low flow events and negatively impact the Braidwood Plant.

#### 4.6.3.2.6 Dam Face Modification

A dam face modification concept was considered for this reach (See Figure 4.6.3.2.6). Although this dam is quite wide, considering the danger posed by this dam, a stepped face may be a desirable safety feature, even though it would result in a high cost. Hydraulic studies and/or physical modeling of the proposed structure would be required to determine a final design and the effectiveness of the stepped face.

#### 4.6.3.2.7 Summary of Cost

The costs for each option vary due to the type of material used, required access to the site, and other considerations that are discussed in the detailed cost section. The opinions of cost for the temporary rock fill, riffle pool, and dam face modification options are $2,170,000, $5,270,000, and $3,890,000, respectively.




0 50 100 Feet

RIFFLE POOL
ROCK RAMP

Wilmington Dam - Kankakee River

*Wilmington, Illinois*

CTE | AECOM

Figure 4.6.3.2.4

Wilmington


0 50 100 Feet

DAM FACE MODIFICATION

Wilmington Dam - Kankakee River

*Wilmington, Illinois*

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Figure 4.6.3.2.6

Wilmington

# Evaluation of Public Safety at Run-of-River Dams

## 4.6 *Kankakee River Dams*

### 4.6.4 Wilmington Millrace Dam

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### 4.6.4 Wilmington Millrace Dam

The following section includes a description of the existing conditions of and proposed structural options for the Wilmington Millrace Dam.

#### 4.6.4.1 Existing Conditions Overview

The Wilmington Millrace Dam was visited on January 22, 2007. Observations made on these occasions and a review of IDNR summaries form the basis for the existing conditions summary.

Option selection should be made with consideration of Wilmington Dam Options, since these act as one system.

*General Dam & River Bank Condition*

The dam appears to have been constructed from large concrete blocks and has partially failed near the left abutment. The partial failure has resulted in a series of drops over 60 ft totaling approximately 10 ft. The right abutment is concrete with some deterioration and cracking and is in need of inspection. The condition of the left abutment appears to be breached, resulting in a partial failure.

*Evidence of Roller*

A series of heavy whitewater rapids occurs over the 60 ft by 10 ft rock fill. This does not appear to be safety navigatable by boat.

*Portages*

There are no marked portages on the right or left banks upstream or downstream. A canoe/boat launch was observed on the upstream of the dam, immediately upstream of the bridge.

*Warning/Information Signage*

There are 2 large, colorful warning signs on the left bank, near the abutment, and an additional warning sign at the nearby park (see photo 3.2.4.2-2). These signs are precautions to the general public as well as to fisherman and boaters. The right bank and abutment has no warning signs and there are no warning signs visible to upstream river users to warn them of the dam.

There was no portage signage observed.

### 4.6.4.2 Options Assessment

#### 4.6.4.2.1 Temporary Rock Fill

A rock fill concept was not completed due to the existing concrete fill.

#### 4.6.4.2.2 Dam Removal

It is yet to be determined whether the Wilmington Millrace Dam maintains the upstream pool for cooling operations for the Braidwood Power Plant. The current state of the dam is a concrete fill which prevents erosion over a steep reach. Removal would result in increased erosion for this reach.

#### 4.6.4.2.3 Full Bypass Channel

A full bypass channel option was not considered for this reach. Due to width and existing condition of the dam, creating a full bypass channel would not be a practical solution. Altering the existing channel into a passable reach would likely be the most cost effective solution.

#### 4.6.4.2.4 Riffle Pool Rock Ramp

A riffle pool rock ramp concept was developed for this reach (See Figure 4.6.4.2.4). Regrading the rock fill at this location to create a riffle pool rock ramp would create a passable reach that would greatly reduce the safety concerns currently present and would also allow river users to completely bypass the Wilmington Dam. This option requires large scale regrading and also removal of the remaining portions of the old Wilmington Millrace Dam. Land acquisition might be required.

#### 4.6.4.2.5 In-Stream Bypass Channel

An in-stream bypass option was not considered for this reach. Since no dam is present, except for a temporary rock fill, this option was considered infeasible.

#### 4.6.4.2.6 Dam Face Modification

A dam face modification option was not considered for this reach since the dam consists of temporary rock fill and is not structurally competent.

#### 4.6.4.2.7 Summary of Cost

The only option that was determined to be feasible was a riffle pool rock ramp, with an opinion of cost of $1,450,000.


RIFFLE POOL
ROCK RAMP

Wilmington Millrace Dam - Kankakee River

*Wilmington, Illinois*

CTE | AECOM

Figure 4.6.4.2.4
Wilmington Millrace


# Evaluation of Public Safety at Run-of-River Dams

4.7 *Rock River Dams*

4.7.1 Oregon Dam

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## 4.7 *Rock River Dams*

The following section includes a description of the existing conditions of and proposed structural options for dams located on the Rock River.

### 4.7.1 Oregon Dam

The following section includes a description of the existing conditions of and proposed structural options for Oregon Dam.

#### 4.7.1.1 Existing Conditions Overview

Oregon Dam was visited on January 29, 2007. Observations made on these occasions and a review of IDNR summaries form the basis for the existing conditions summary.

*General Dam & River Bank Condition*

In general, the dam is in great condition. There is an abandoned dam works on right bank but it is well fenced while the left abutment shows some structural degradation.

*Evidence of Roller*

There is a reverse roller extending approximately 5-15 ft from the dam.

*Portages*

There is an unmarked portage on the right bank.

*Warning/Information Signage*

There are detailed warning signs on both downstream bank, “no swimming” signs on the upstream left bank and right abutment, and skull and crossbones graffiti on the abandoned dam works on the right abutment. IDNR noted six (6) seasonal warning during their summer inspection.

*Hydropower Assessment*

The Oregon dam provides an upstream pool for the Byron Nuclear Power Station in order to allow the plant to intake sufficient water for normal operations. The owner and operator of the power plant, Exelon Corporation, has indicated that the Byron Nuclear Power Station design and licensing documentation contains information on the Oregon dam and its function in supporting the operation of the nuclear power plant. The removal of the Oregon dam or lowering of the Oregon dam pool would affect pump operations during normal summer low water conditions and therefore affect the ability of the power plant to function as currently designed.

Exelon Corporation notes that the intake for emergency cooling water pumps would be able to operate should the Oregon dam fail. The pumps needed for normal plant operations draw water from the river at a higher elevation than the pumps required for emergency cooling water.

#### 4.7.1.2 Options Assessment

##### 4.7.1.2.1 Temporary Rock Fill

A rock fill layout was developed for the Oregon Dam based on limited information. The dam face has been modified to include a series of steps. The FEMA FIS water surface profiles were not available and do not appear to have been completed. A cost estimate and general layout has been provided for the purposes of this report; however, the complex dynamics of the varying slopes and the lack of available hydraulic information require that a detailed hydraulic analysis be completed to determine the feasibility of a rock fill option

- The downstream water surface slope was estimated based on the slopes experienced at the other Rock River dams.
- Downstream water surfaces were computed using a bed elevation of 654.8 ft and adjusted based on a water surface slope of 0.0003. A Manning's n value of 0.035 was used.
- The upstream 100 year water surface elevation was computed using the broad crested weir equation.
- Based on the design characteristics, sub-critical flow may occur during the 10 year event as well as the 1 to 5 year events, eliminating a hydraulic jump at the toe and a reverse roller at the dam crest.

##### 4.7.1.2.2 Dam Removal

Dam removal was not considered for this reach since Oregon maintains the cooling pool for the Byron Power Plant.

##### 4.7.1.2.3 Full Bypass Channel

A full bypass channel option was not considered for this reach since it would reduce the impact the upstream pool and negatively affect the operations of the Byron Power Plant.

##### 4.7.1.2.4 Riffle Pool

A riffle pool option was not considered for this reach due to the large width of the channel (870 ft) and the depth of the downstream reach, approximately 25 ft below the dam crest.

#### 4.7.1.2.5 In-Stream Bypass Channel

A full bypass channel option was not considered for this reach since it would reduce the impact the upstream pool and negatively affect the operations of the Byron Power Plant.

#### 4.7.1.2.6 Dam Face Modification

A dam face modification option was not considered for this reach due to the large width of the channel (870 ft) and the depth of the downstream reach, approximately 25 ft below the dam crest.

#### 4.7.1.2.7 Summary of Cost

The costs for each option vary due to the type of material used, required access to the site, and other considerations that are discussed in the detailed cost section. The opinion of cost for the temporary rock fill is $38,130,000.

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# Evaluation of Public Safety at Run-of-River Dams

## 4.7 *Rock River Dams*

## 4.7.2 Sinnissippi Dam

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### 4.7.2 Sinnissippi Dam

The following section includes a description of the existing conditions of and proposed structural options for Sinnissippi Dam.

#### 4.7.2.1 Existing Conditions Overview

Sinnissippi Dam was visited on January 28, 2007. Observations made on these occasions and a review of IDNR summaries form the basis for the existing conditions summary.

*General Dam & River Bank Condition*

Dam spillways and appurtenances are well marked. Only issue is trees need to be cut down on upstream earthen embankment on right bank and vandalized signage for right bank portage needs to be replaced.

*Evidence of Roller*

Yes, reverse roller at discharge gates 5' – 10' out. Gates' position individually controlled by inflatable "bladders".

*Portages*

Yes, on right bank.

*Warning/Information Signage*

Sign on left bank fence describing dam roller. Sign reads "Danger Hazardous Recirculating currents below this dam can trap and drown a victim". Warning signs facing upstream on both banks, warning signs normally on dam were removed during construction. Left abutment fencing had warning sign depicting roller effect; informational signs on right bank identifying portage, with some defaced. There is a defaced portage sign on the upstream right bank. There is a portage trail sign at portage located at the downstream right bank. Obsolete sign warning of lock operation; however the lock is now defunct and is used as the discharge bay for hydroelectric plant.

Seven (7) seasonal buoys are present upstream of the dam between May and September.

*Hydropower Assessment*

The Sinnissippi dam is a hydroelectric facility operated by the City of Rock Falls. The hydropower operations include 500 ft of pneumatically operated hinge-leaf gates. There is an automated gate operating system for 24 hour monitoring and discharge regulation to adjust to varying headwater conditions. The intake and discharge structures and the intake and discharge points are located at the left abutment and are fenced to reduce accessibility to river users, though fisherman commonly use the walkway system. The