DESIGN BRIEF

October 5, 2005

Client: Park or FU:	Rideau Canal NHS	Client Project No.:	
Project Title:	Kingston Mills Lock 46		
Client - Responsible Officer:	Bill Pratt, P. Eng. Asset Manager	Tel.:	(613) 283-7199 ext 250
CST for Parks Canada Manager:	John Mazhar, P. Eng., Program Manager	Tel.:	(613) 938-5976
CST ONTARIO Project No. :	453177		

1. SCOPE OF DESIGN BRIEF

To estimate the nature and scope of work required at Kingston Mills Lock 46.

The work includes, but is not limited to, the following items:

1) reviewing existing site investigation reports to assess the past condition of the lock and determine the quantity and condition of historical fabric remaining;

2) retaining a geotechnical consultant to assess the current condition of the concrete and stone and collect information which is lacking in the existing investigation report;

3) providing repair options which are in accordance with Cultural Resource Management (CRM) policies;

4) calculating class D estimates for budgetary purposes.

2. BACKGROUND INFORMATION

2.1 <u>Previous Geotechnical Reports</u>

1) *Kingston Mills Waste Weir Evaluation: Report on Soil Conditions*. Site Investigation Services Ltd, March 31, 1978.

Found that bedrock near waste weir is granite, at about el. 85m. Pinkish-grey to light reddish, medium grained.

2) *Borehole Results: Kingston Mills Lockstation*. Golder & Assoc. May 1979.

Describes results of two vertical boreholes, one at the lower right wing wall of Lock 49 near the gate recess, and the other through the upper sill of Lock 49. Bedrock here was also granite, reported as highly fractured grey to pink, at about el. 44m for the lower hole and about el. 72 for the upper hole.

3) *Report on the Foundation Condition of the Swing Bridge at Kingston Mills*. J. D. Lee Engineering Ltd., September 1976.

Considerable information in this report about a movement of the right wall and bridge pier observed in 1976, as well as cross-sections at the bridge. Pertinent sections of this report are reproduced in Appendix 1. Recommended tying the right bridge abutment to bedrock and grouting behind right lock wall.

2.2 Current Geotechnical Report

1) *Geotechnical Investigation for the Kingston Mills Lock No. 46, Kingston, Ontario.* Jacques Whitford, April 2005.

Investigation included 11 boreholes (one into original limestone, the rest into old concrete) . Compressive strength of the old concrete was found to be 41 to 42 MPa in the zone underlying the extensively fractured surface layer. The old concrete was also analyzed petrographically to determine that it was not air-entrained and is subject to Alkali-Aggregate Reaction (AAR). This old concrete, which exists at the upper and lower monoliths and lower wing-walls, both Jacques Whitford engineers and the independent petrographer, Mr. Grattan-Bellew, indicated is in poor condition.

NOTE: I have not summarized this further because I understand the Rideau Canal has a copy of this report already.

2.3 Other Reports

1) *Construction History of the Rideau Canal*. Karen Price. No date.

Clearing and grubbing work was started 1827. By 1829, progress report says excavations about 2/3 done. Originally conceived of as being 4 flight locks, lock 46 was later changed to a detached lock upstream of a turning basin. A surveying error meant the lift of the flight locks had to be increased. Original masonry was limestone. Lift was 11'-8" with 7'-8" of water on the upper sill.

"The excavation was through a species of granite."

The Imperial Government operated the canal until 1856, when it was turned over to the provincial authorities. Unfortunately, all the canal records from the early period were burnt in the Ordnance office in Montreal during the 1849 riots.

2) Our own Heritage Canals records show some concrete refacing done in 1995 and coping repairs at downstream wingwalls in 1998. Lock 46 was not part of the grouting project undertaken on the flight locks in 1995. I cannot find any other records of interventions to this lock in the last 10 to 15 years.

2. MATERIALS CONDITION ASSESSMENT

2. 1 Stone Masonry

Material testing has been done infrequently on original stone fabric, so we took the opportunity to have Jacques Whitford drill one borehole into stone at an area where some of the face had spalled off. The stone was found to be 400 mm thick, and backed with rubble. Although the recovery percentage was 100%, the RQD value of the recovered core was only 47%, which reflects a "poor quality" rock containing a large number of fractures, according to the Canadian Foundation Engineering Manual. However, this particular sample was selected to be taken at a stone known to be in poor condition. The compressive strength of the limestone was found to be 75.7 MPa (=10,979 psi).

2.2 Concrete Blocks

At some point between the original construction and about mid-20th century, repairs to the upper and lower monoliths and wing walls were done with plain (un-reinforced) concrete blocks laid to imitate masonry. Most of the boreholes Jacques Whitford did were into this material, which is spalling badly in the outer 4" to 18" due to freeze-thaw action. Furthermore, I had two petrographic analyses done to investigate the composition of this concrete and its alkali-aggregate reactivity condition.

It turns out that "the coarse aggregates consist mainly of a mixture of Potsdam sandstone, dolostone, and biotite hornblende granite. The fine aggregate, natural sand, contains mono-mineralic particles of quartz and feldspar and assorted rock fragments. The same types of rocks that were observed in the coarse aggregate occur in the fine aggregate. The cement paste is dense and mostly un-carbonated. ... The concrete is not air-entrained, but contains some entrapped air."[1] The lack of air entrainment would certainly explain the freeze-thaw spalling in the outer foot or so.

The investigation also determined that there is indeed some alkali-silicate reactivity going on in this concrete, with the Potsdam sandstone in the aggregate being the most likely culprit. In fact, significant damage has already occurred to this concrete as its mean Damage Rating Index (DSI) is 124---and DSI's of greater than 40 are considered indicative of significant alkali-silicate reactivity. Further, the petrographer estimated that an expansion of 0.23% has already occurred

[1]From report on petrographic analysis of Kingston Mills samples by P. E. Grattan-Bellew, Materials & Petrographic Research G-B Inc., 472 Edison Avenue, Ottawa, ON, K2A 1T9, for Jacques Whitford, March 2005.

to date (assuming concrete is 40 years old). In terms of how much the concrete will continue to expand, the report says that "...assuming this rate [of expansion] will be maintained for the foreseeable future, which is reasonable based on experience with many large dams, the possible expansion for the next 25 years would be ~0.06%." It would, however, depend on the alkali content of the concrete being high enough to sustain that reaction, something which was not determined from this analysis but which could be in the future.

Considering this together, it would seem that a simple refacing of the weathered surface of the concrete blocks would not be sufficient as a long-term repair. Since the concrete blocks are subject to alkali-silicate reactivity (ASR), they will continue to expand and crack underneath any refacing concrete.

Therefore, it would seem that the most thorough repair would involve removing all the old concrete blocks and replacing these with either new concrete or limestone masonry. This would involve the loss of fabric added during the life of the structure---an approach which is questionable when viewed from a CRM perspective. The CRM "Principle of Value" (§1.1) does not normally allow the sacrificing of materials added later in the structure's lifetime. It may be considered in this case only because the material has already failed (freeze-thaw action) or is in the process of failing (ASR). Removing it is justifiable to save the overall lock structure.

There is, however, another alternative which merits looking into which might be more sympathetic to the CRM: it is possible to treat the concrete to arrest further ASR with a number of Lithium-based products:

> "Evidence indicates that lithium forms an alkali-silica gel that is non-expansive. Lithium silicates are less water-soluble and do not absorb or bind water to the degree that sodium or potassium silicates do. While the reaction of lithium with the ASR reaction product appears irreversible, there should be sufficient lithium present in the pore solution to protect against future attack by any alkalies remaining in the concrete mixture....Reduction of expansion early in the process of ASR will lengthen the life of the structure and extend the time before repairs or replacement becomes necessary. By using a lithium-based treatment product, further ASR expansion can be significantly decreased, extending a structure's life."[2]

As mentioned in the quote above, lithium works best when applied early in the ASR process (which cannot happen here) or during the mixing of concrete (which cannot happen hear either). But there is some effectiveness even when applied late. At any rate, this alternative has not been costed, but merits being looked into as part of the design options phase.

3. ENGINEERING CONDITION ASSESSMENT

J. D. Lee Engineering checked the stability of the walls of the lock in 1976 and found a factor of safety against overturning of 1.0. They also found some bulging of the right hand wall. However, they did not recommend pinning the walls back to bedrock if the right hand bridge abutment (concrete) were pinned back and the wall grouted. I do not know if this work was undertaken or not. I have included 4 anchors (two in each of the left and right bridge abutments)

[2]Nicholas Adams & David B. Stokes. *Using Advanced Lithium Technology to Combat ASR in Concrete.* Concrete International, August 2002, pages 52 ff. See also articles at *http://concreteproducts.com/mag/concrete_lithium_admixtures_scale/* and *http://www.spsrepair.com/asr/index.html* For more articles in the same vein.

and grouting of both left and right walls in Options 1 to 3 inclusive.

4. REPAIR OPTIONS

4.1 Description and Discussion of Options

Some work should be done immediately at areas of coping or high on the wall where there is a danger of spalling concrete chunks falling on boaters below. This work should be done as soon as possible . The cost estimate below assumes the work is done by outside forces: some cost savings would occur if the repair was being done in-house.

After that, there are three options for overall repair any of which could be undertaken within the next decade or so.

Ideally, **Option 3 (Full Restoration in Masonry)** would be selected. This would mean the lock would be repaired with materials and techniques most compatible with the original fabric. This is the most sympathetic to the original construction and most aesthetically pleasing. Such a repair acknowledges the locks as originally being masonry structures and maximizes the "Principle of Respect" (CRM §1.4). It would, however, involve the loss of the old concrete blocks---the CRM "Principle of Value" (§1.1) does not normally allow the sacrificing of later materials. But in this case where over 50% of the material has already failed by freeze-thaw action and the other 50% is subject to AAR, this material is already lost. This repair also assumes that 25% of the extant masonry will require adhesive crack repairs and a total of 10 cu. metres of stone will be found to be too fractured and will have to be replaced with new. Option 3 represents a fairly high level of intervention, but it is a sympathetic repair that will address most of the on-going deterioration and which will give a long-lasting result. It is, however, the most expensive option.

Alternatively, **Option 2 (Functional Rehabilitation in Concrete)** could be selected. This would involve removing failed concrete and stone material with new concrete. This would involve removing both the friable surface layer and the AAR-affected concrete. Extant stone in good condition would be retained, raked and pointed. Again, the cost estimate assumes that 25% of the extant masonry will require adhesive crack repairs and a total of 10 cu. metres of stone will be found to be too fractured and will have to be replaced with new The use of large amounts of modern reinforced concrete means the repair that is not as sympathetic to Colonel By's original construction as that of restoration in masonry. Like Option 3, Option 2 represents a fairly high level of intervention. Since the AAR-affected concrete is removed, this option will also give long-lasting results, but not as long-lasting as Option 3 since concrete is not as durable as stone. It is considerably less expensive than Option 3.

Another alternative would be **Option 1 (Stabilization)**. This option involves removing only the friable surface layer of failed concrete (about a foot to 18" deep) , but not the underlying AAR affected concrete. It would produce the same aesthetic result as Option 2 involving large areas of concrete visible at the monoliths and lower wing walls. It will not give as long lasting a result as Option 2 since the underlying concrete will continue to deteriorate and reflection cracks / debonding of the overlay material will occur in a decade or so. It could, if desired, be done to prolong the life of the structure for another 10 to 15 years whilst awaiting funding for Option 3.

All three options as costed include raking and pointing of extant masonry, post-tensioned anchors in the upper and lower sills and bridge abutments, and full grouting of both walls.

4.2 Some Further Discussion of Certain Cost Estimate Items

Repair Extant Masonry: – The current volume of extant masonry in the lock is about 255 cu. metres (340 square metres at an assumed thickness of 2'-6"). For cost estimating purposes, based on the Jacques Whitford results for material condition, I assumed that 25% of the extant masonry area would really require crack repairs by adhesive injection (estimated at $3000 per square metre, as per McCormick-Rankin's Fort Henry estimate). Also, I assumed that 5% of the existing stones (=12.75 m^3) would require removal for either repair or replacement with new dutchmen or as complete new stone units. Such repair work to the extant masonry would cost about $210,000 (not including scaffolding, housing/heating etc.) This cost has been included in Options 2 and 3 described below. Option 1 includes only raking & pointing at $7000 with no repairs or crack injection.

Comparison of Replacement of Old Concrete with New Concrete vs. with Stone: – The volume of old concrete present is about 260 m^3. To replace this volume with concrete would be about $450,000 whereas to replace it in stone is about $1,370,000 for stone and installation thereof. Furthermore, since masonry work is slower than pouring concrete, there is an additional cost for both scaffolding and heating/hoarding over the costs for these items that would occur if using concrete as a replacement material.

Grouting Costs: – The same grouting programme is assumed for all three options. This consists in drilling 24 holes (12 at each wall), each hole assumed 65 feet deep. I assumed a grout take of 0.65 m^3 per metre drilled. The anchors in sills and abutments (2 per location) are included in the grouting programme cost of $861,000.

5. CLASS D CONSTRUCTION COST ESTIMATES IN 2005 DOLLARS

OPTION NAME	CLASS "D" CONSTRUCTION COST ESTIMATE (2005$)
IMMEDIATE - Safety Related Coping Patching - Prevent concrete chunks from falling on boaters below. Detailed breakdown of this estimate included in Appendix. **Priority**: Immediate	$80,000
OPTION 1 - Stabilization - Remove friable concrete everywhere on the structure and reface these areas. Provide new log checks at upstream end. Add post-tensioned anchors in sills and right bridge pier (if the latter has not already been done). Rake and point existing masonry. Grout behind both left and right walls. **Priority**: Immediate to 10 years.	$1,600,000
OPTION 2 - Functional Rehabilitation in Concrete - Remove not only the friable concrete but also the underlying ASR-affected concrete everywhere on the structure and replace with new concrete. Provide new log checks at upstream end. Add post-tensioned anchors in sills and right bridge pier (if the latter has not already been done). Rake and point existing masonry. Grout behind both left and right walls. **Priority**: Immediate to 10 years.	$2,100,000
OPTION 3 - Restoration in Masonry - Remove not only the friable concrete but also the underlying ASR-affected concrete everywhere on the structure and replace with new limestone masonry. Provide new log checks at upstream end. Add post-tensioned anchors in sills and right bridge pier (if the latter has not already been done). Rake and point existing masonry. Grout behind both left and right walls. **Priority**: Immediate to 10 years.	$5,500,000

NOTE 1: Neither *design* nor *construction supervision* costs is included, since I have assumed that these will come from Core SSA with PWGSC Ontario Region CST for Parks Canada. If this is not to be the case, then additional funds would be required for these tasks.

NOTE 2: These costs must be factored up for estimated inflation when putting into future work programme.

Report prepared by:

Kathleen Murphy, P. Eng.
Mechanical Engineer
Heritage Canals & Engineering Works
October 4, 2005

APPENDIX 1 - SUMMARY OF 1976 GEOTECHNICAL REPORT

Report on the Foundation Condition of the Swing Bridge at Kingston Mills. J. D. Lee Engineering Ltd., September 1976.

Scope:

An examination around the pivot and the two lock walls. At that time, they found the east bridge abutment and east lock wall (right hand wall) had moved about 4 inches into the lock. Apparently there had been problems swinging the bridge, and it had even stuck on occasions. They attempted to understand what was happening and outline possible remedial measures.

Right bridge abutment bears in part on the east lock wall. As-built plans at the time showed the bridge abutment foundations to be on rock some 8-1/2 feet below road level.

2. EXISTING CONDITION:

"The Lock walls are constructed of limestone masonry blocks some three to three and a half feet thick. The top courses of these walls were removed at the time of the building of the wing bridge [said elsewhere in report to be in 1956] to accommodate the substructure. ... The west [left bridge] abutment which is remote from the lock is shown as being keyed into the bedrock. The east abutment, on the other hand, is apparently keyed into the masonry of the east [right] lock wall which partially supports it, but not into the adjacent bedrock on which it is also founded. "

"... the pivot pier and the west [left] abutment show no significant departure from the configuration they had at the time of construction. ... On the east [right] side of the lock, however, the following modifications to the assumed 'as constructed' condition are seen to exist:

1. The east abutment has moved horizontally towards the pivot pier, a distance of about 4 inches. There are also indications that the north side may have moved vertically upwards anywhere from 3/4 of an inch to 1 inch, namely the joint between the deck and the abutment and the concrete elevations which are that much higher than the figures shown on the plans. ...

2. There is evidence that some subsidence of the road surface behind the east abutment has taken place. The two stairways behind the abutment show an inclination of about three percent downward from east to west. At the junction of both these stairways to the abutment, a separation at ground level of approximately 1 - 1/4 inches has occurred at the north stairway and 3/4 inches at the south stairway."

3. DISCUSSION

[Re: the right lock wall] "the wall itself was probably capable of sustaining itself in true alignment, nevertheless, the additional forces imposed upon it by the additional forces imposed on it by the abutment were more than the wall could bear. It speaks well of the original construction that the masonry wall has been able to withstand this amount of deformation with such little visible sign of distress. ...

Horizontal movement of both the abutment and the lock wall has taken place, and there is no evidence of significant differential movement between them. ... it is now logical to assume that no positive anchorage exists between the abutment and the rock. The displacement must also have created a cavity between the rock and the lock wall, and the fact that subsidence may have occurred would indicate that this cavity is partially filled with material from behind the abutment. ...

An analysis of the stability of the abutment and the lock wall indicates a factor of safety against overturning about the toe of the lock wall of about 1.0. This calculation is based on a unit width, and considers only the horizontal forces from the fill behind the abutment. At present the lock wall is tending to span between its own abutments, and as it also receives support along its base, is functioning as a slab supported on three sides. As long as the integrity of the wall is maintained, there is no cause for alarm. If, on the other hand, rupture of the masonry were to occur, an unstable situation could exist, and since the indications are that the abutment is still moving horizontally, this state of affairs will in time be reached. Stabilization of the wall is, therefore, imperative.

It is impossible to know the exact circumstances that led to the initial movement of the wall and abutment. It is reasonable to suppose that the lock wall was in a stable conditions prior to the construction of the swing bridge. The action of frost cannot be discounted since freeze-thaw cycles of moisture within a cavity, albeit exceedingly small, gives rise to a progressive situation that tends to enlarge the void. This would have the effect of forcing the abutment and wall away from the rock and possibly moving the abutment vertically as well. The lock wall has a slope on the front face of roughly 1 to 8 and rotation about the toe would create an upward movement of about the magnitude that was measured as the average in the field. What is difficult to explain is why this movement appears to have taken place only at the north side of the abutment.

The condition of the rock at the time of the concrete placement may be a factor in the movement of the abutment. Even if the surface appeared clean and sound it is possible that a plane of horizontal fracture existed beneath the surface and that slippage is now occurring along this plane. It is also possible that the surface of the rock is sloped towards the lock.

It is apparent, therefore, that since the present configuration of the east [right] lock wall has resulted form the movement of the abutment, anchoring only the abutment should be sufficient to stabilize the situation. Tying back the lock wall with rock anchors would be an expensive proposition and would certainly do nothing to enhance the appearance of the wall.

Water seepage must inevitably have carried some of the finer material from the fill behind the abutment into the voids behind the wall. To what extent this has happened is impossible to say without carrying out a comprehensive coring program through the wall. Even then, due to the probable irregular nature of the cavities behind the wall, the usefulness of the results obtained is debatable. In any event, the cavities must be filled to prevent a continuance of frost action and a worsening of the situation.

4. CONCLUSIONS AND RECOMMENDATIONS

... "The main factor that has led to the present situation on the east side of the lock is a lack of adequate attachment of the concrete abutment to sound bedrock. ... the first problem [movement of the abutment] can be solved by installing anchors that will tie the abutment to sound rock. the lock wall should not require similar treatment since stabilization of the abutment should remove the root cause of its distortion. The second problem [void] can be solved by drilling and pressure grouting behind the wall."

ADDENDUM TO REPORT

Subsurface Investigation

Borings were attempted at ten locations at the east abutment. All except the three behind the abutment reached bedrock. The backfill behind the abutment appears to consist in part of rock fill too coarse to auger. Cone probes were driven through this fill, but refusal may not be bedrock.

The results of the soils investigation and the interpreted rock surface contours are shown on Figure No. 3 attached [reproduced as sketch]. The surface of the rock dips to the north-west and is the order of 7 feet deep at the south wing wall [lower right] and 11 feet deep at the north wing wall [upper right].

The bedrock is slightly weathered in the surface 1-1/2 to 2 feet, but is sound and massive below this level.

APPENDIX 2 - Detailed Breakdown for Immediate Work

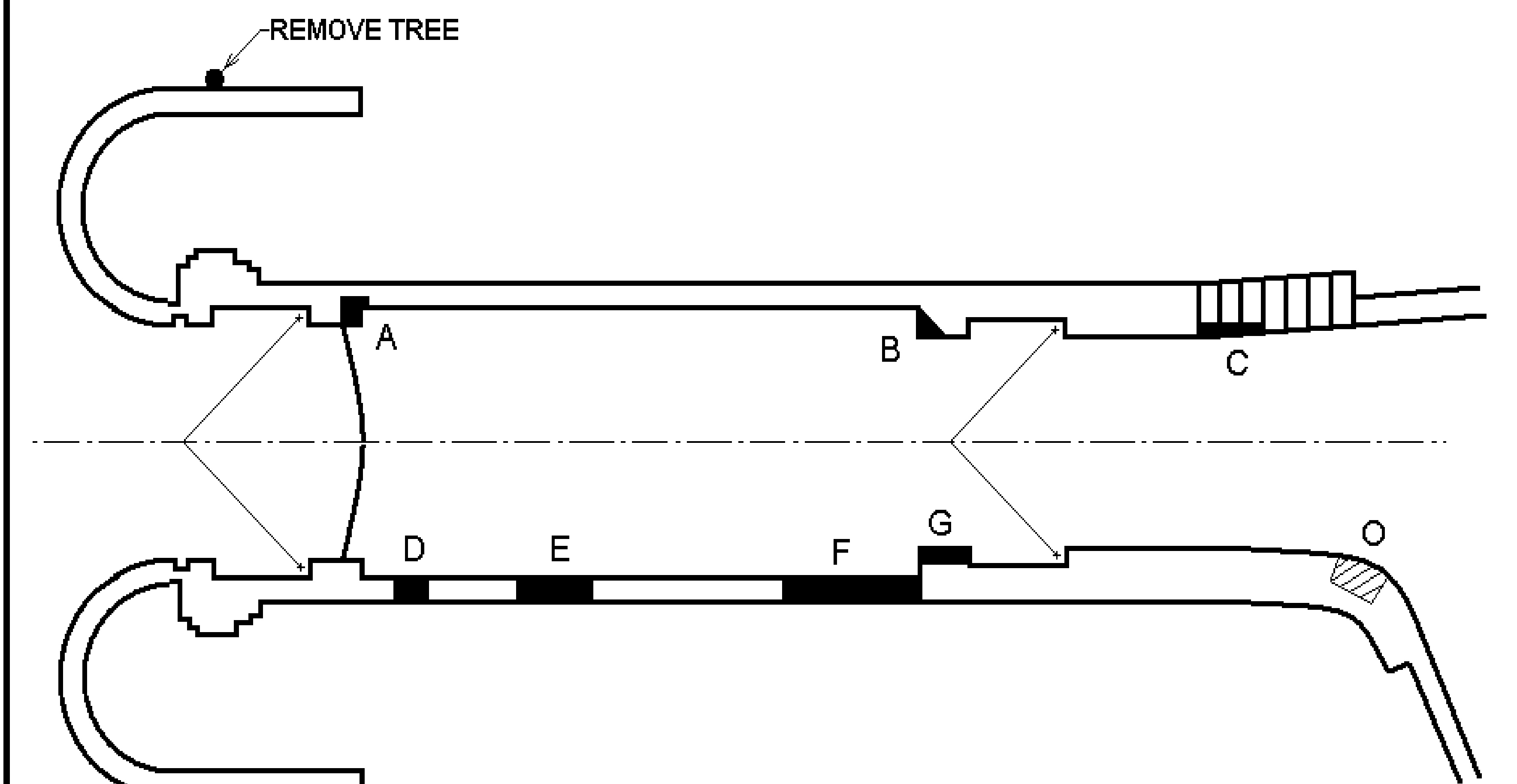
SAFETY RELATED COPING PATCHES - To prevent rocks falling on heads

Item No.	Class of Labour, Plant or Material	Unit of Measure	Estimated Quantity	Estimated Unit Price	Estimated Total	Notes
1	Mob / Demob / Access (say 15% of subtotal)				$8,775	a
2	Sawcuts	lin. metre	50	$70	$3,500	
3	Concrete Removal	cu. Metre	20	$950	$19,000	
4	Reinforcing steel	kg	1500	$4	$6,000	b
5	Concrete	cu. metre	20	$1,500	$30,000	c
				SUBTOTAL	$67,275	

RISK FACTORS

Item No.	Element of Risk (described)	Cost		Additional %	Added Cost	Notes
1	Mob / Demob (say 15% of subtotal)	$8,775		10%	$878	
2	Sawcuts	$3,500		10%	$350	
3	Concrete Removal	$19,000		15%	$2,850	
4	Reinforcing steel	$6,000		15%	$900	
5	Concrete	$30,000		10%	$3,000	
6	Additional Environmental items				$2,000	d
				SUBTOTAL WITH RISK	$77,253	
				ROUND TO	$80,000	

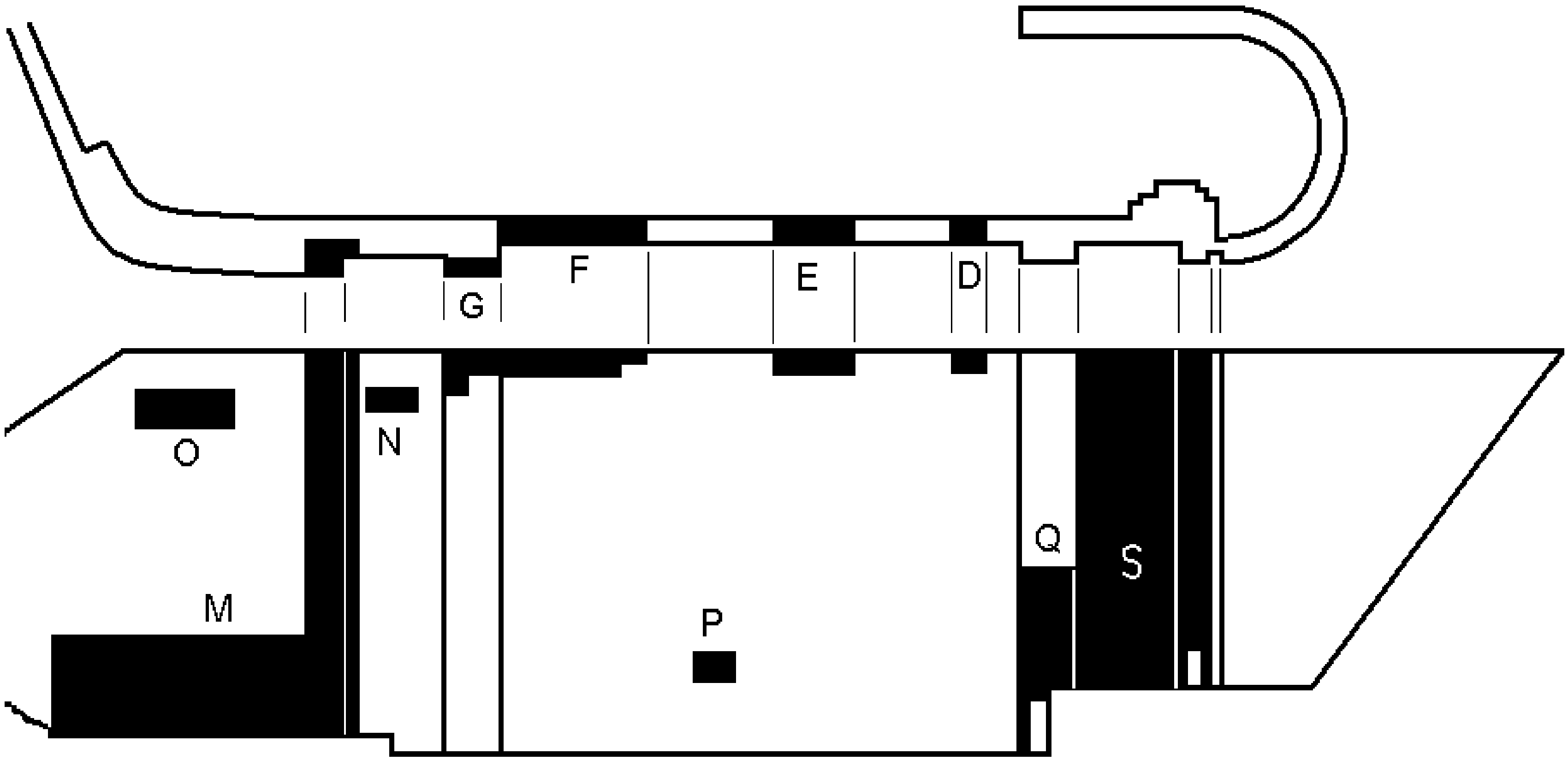
Note	Description
a	Includes ladders, scaffolding required to access coping for formwork. High rate (15%) reflects smaller job.
b	Estimated at 75 kg per cu. Metres of concrete
c	Unit price reflects awkward formwork and small pours. Based on previous jobs where 2nd pour concrete is $1500 per cu. metre
d	For nets or means to capture concrete from falling in water



KMILLS LOCK #46
DESIGN BRIEF

AREAS NEEDING
IMMEDIATE REPAIR

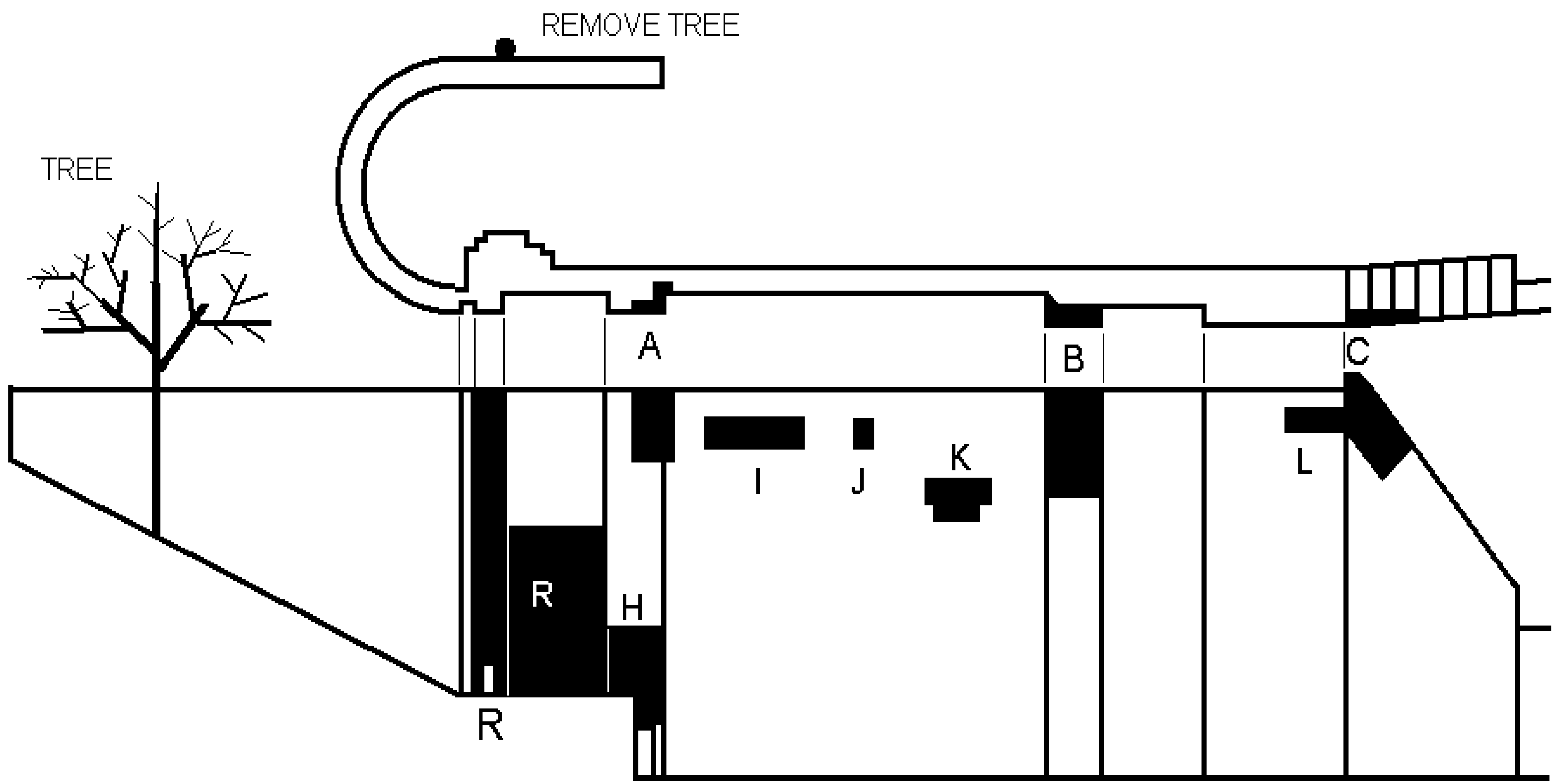
K. Murphy Sept 2005
Not to Scale.



KMILLS LOCK #46
DESIGN BRIEF

OPTION 1 - STABILIZATION
Spalling Areas on L Wall

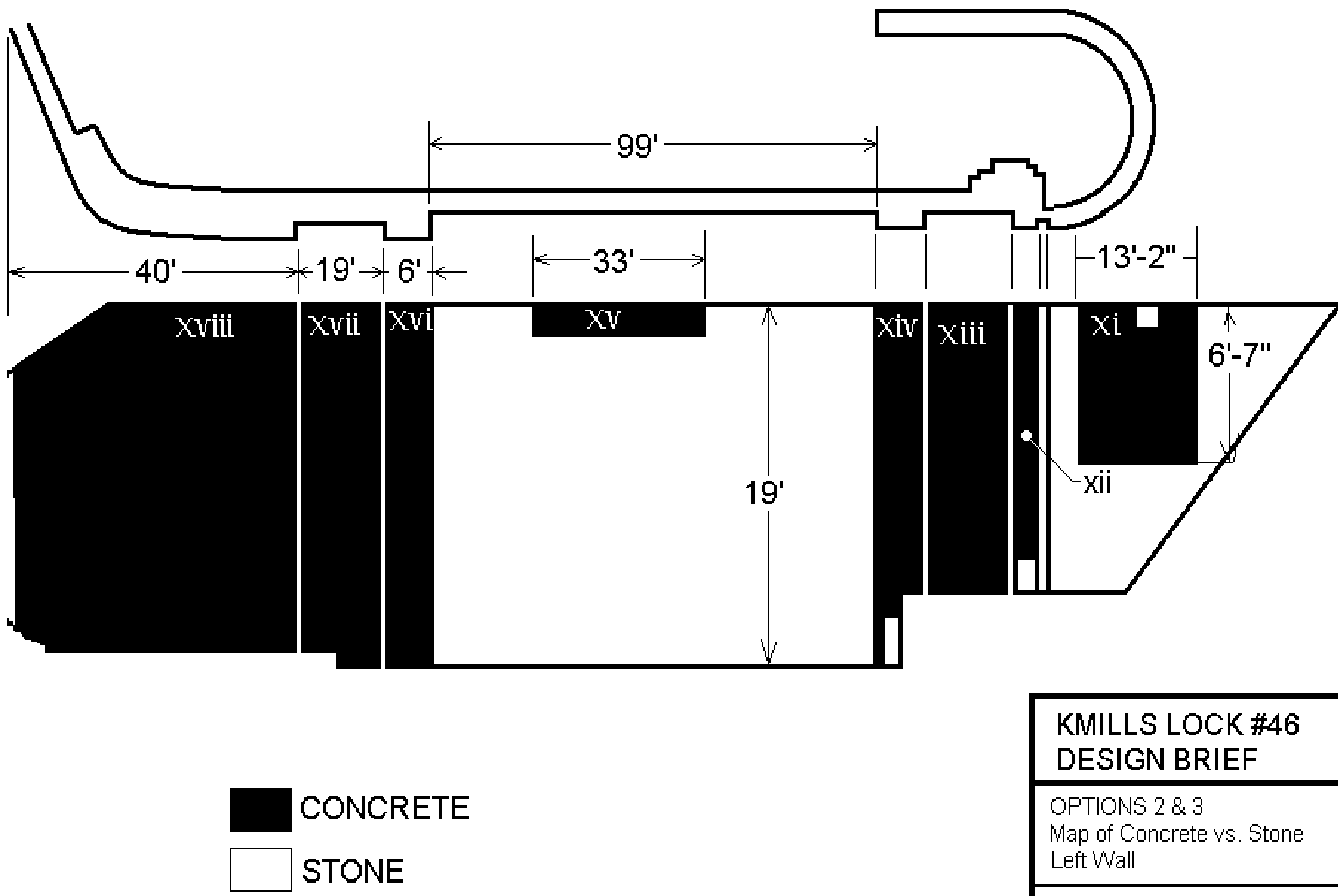
K. Murphy Sept 2005
Not to Scale.



KMILLS LOCK #46
DESIGN BRIEF

OPTION 1 - STABILIZATION
Spalling Areas on R Wall

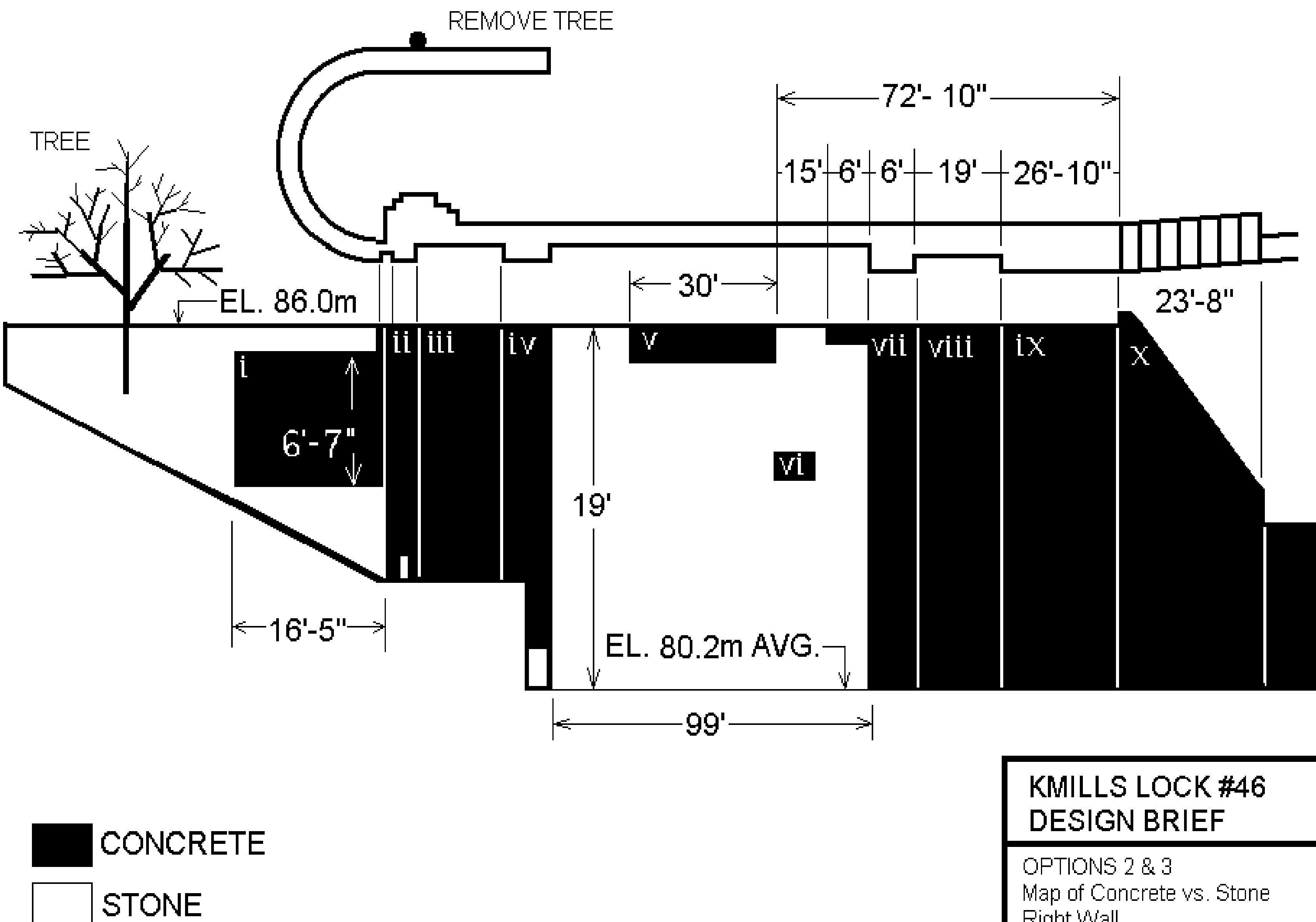
K. Murphy Sept 2005
Not to Scale.



KMILLS LOCK #46
DESIGN BRIEF

OPTIONS 2 & 3
Map of Concrete vs. Stone
Left Wall

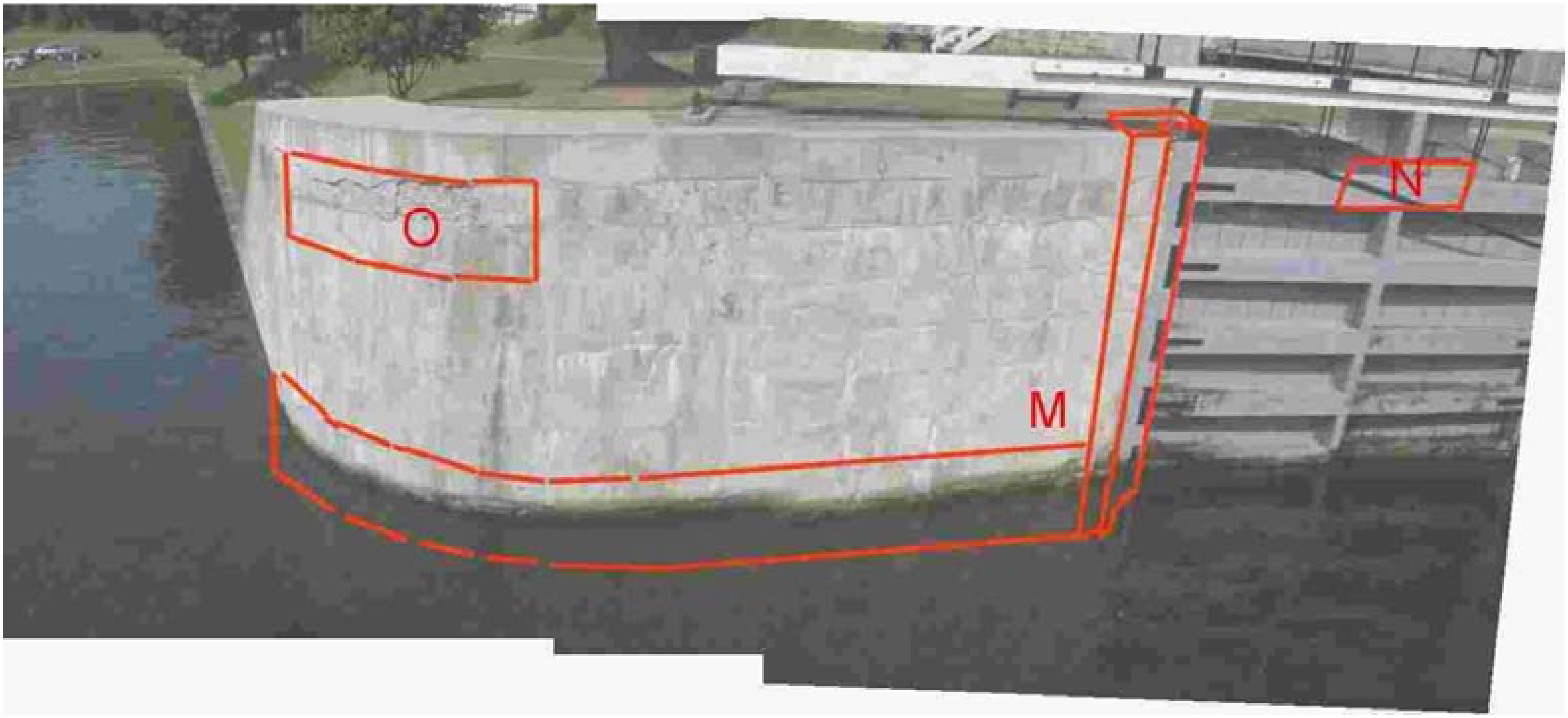
K. Murphy September 2005
Not to scale.



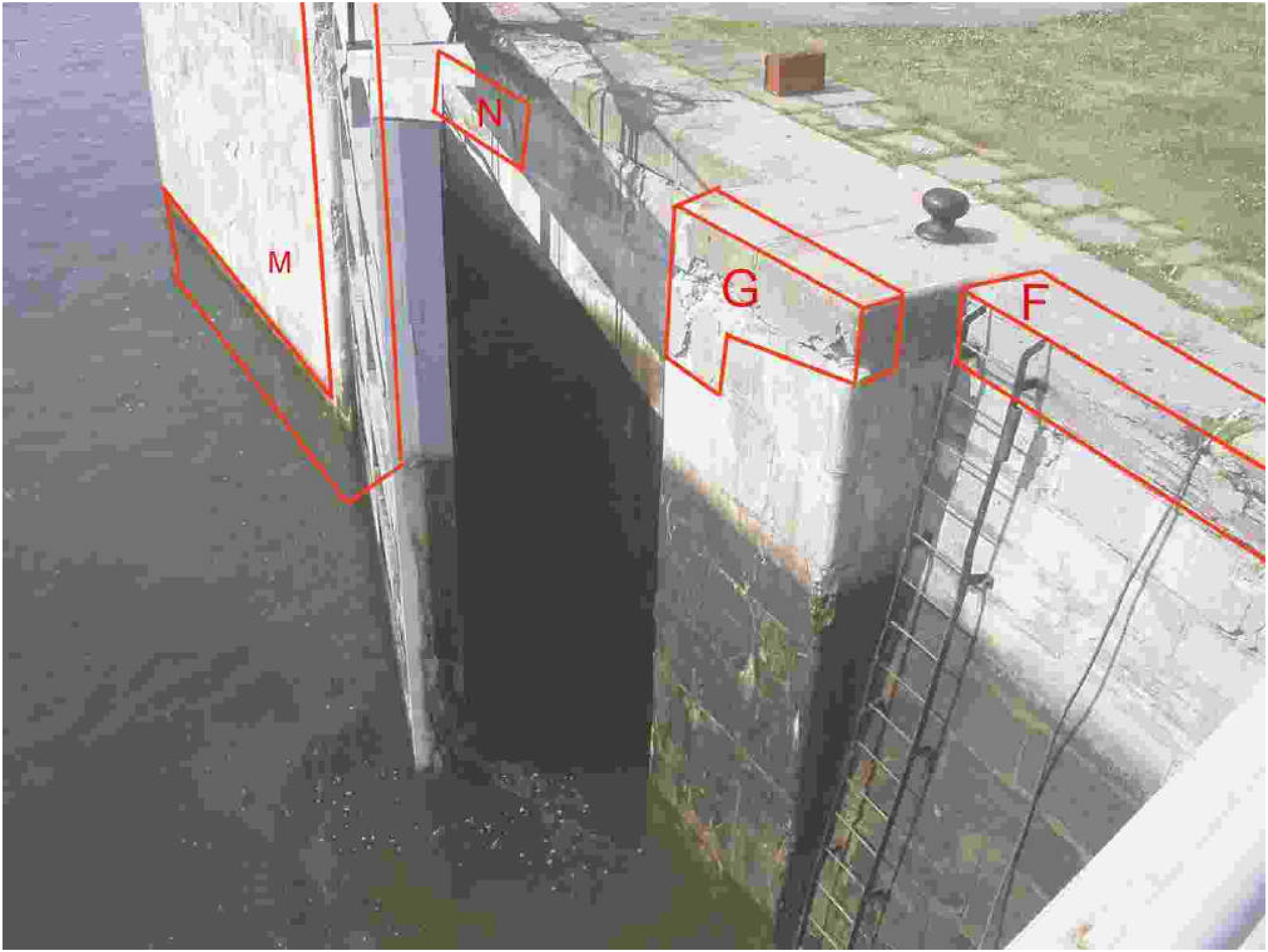
KMILLS LOCK #46
DESIGN BRIEF

OPTIONS 2 & 3
Map of Concrete vs. Stone
Right Wall

K. Murphy September 2005
Not to scale.


O
M
N

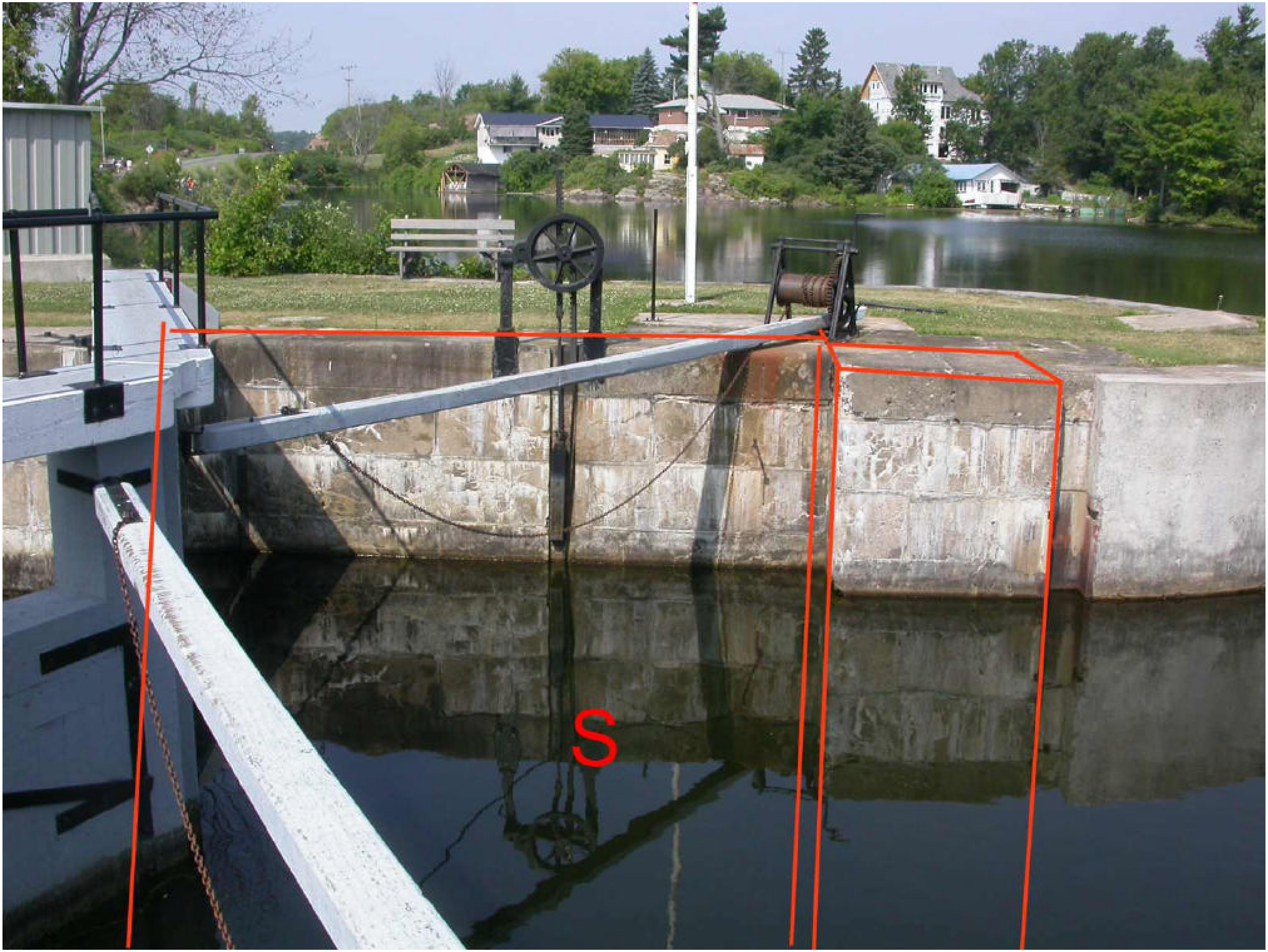

M


N
M
G
F

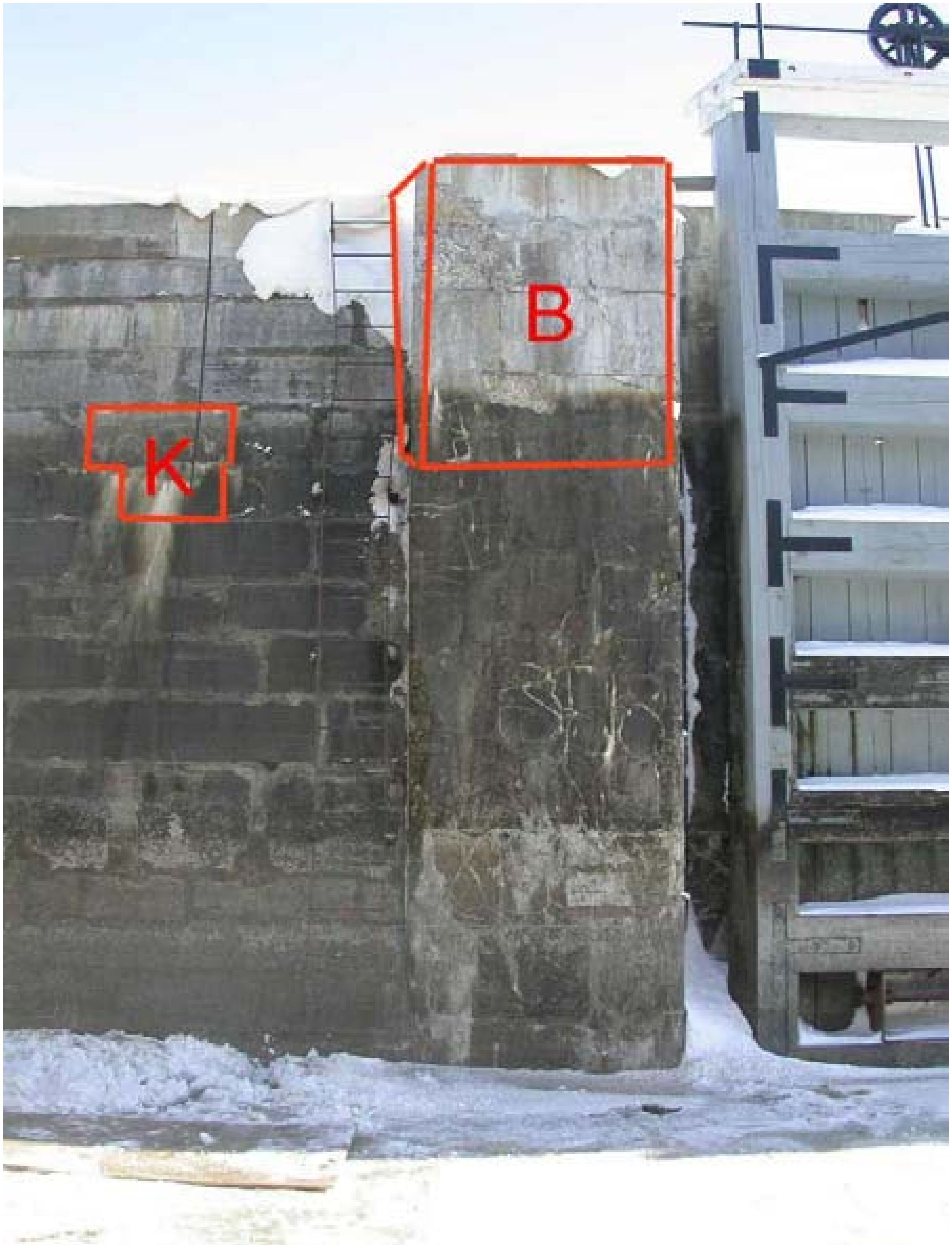

G
F
P


E
D
P

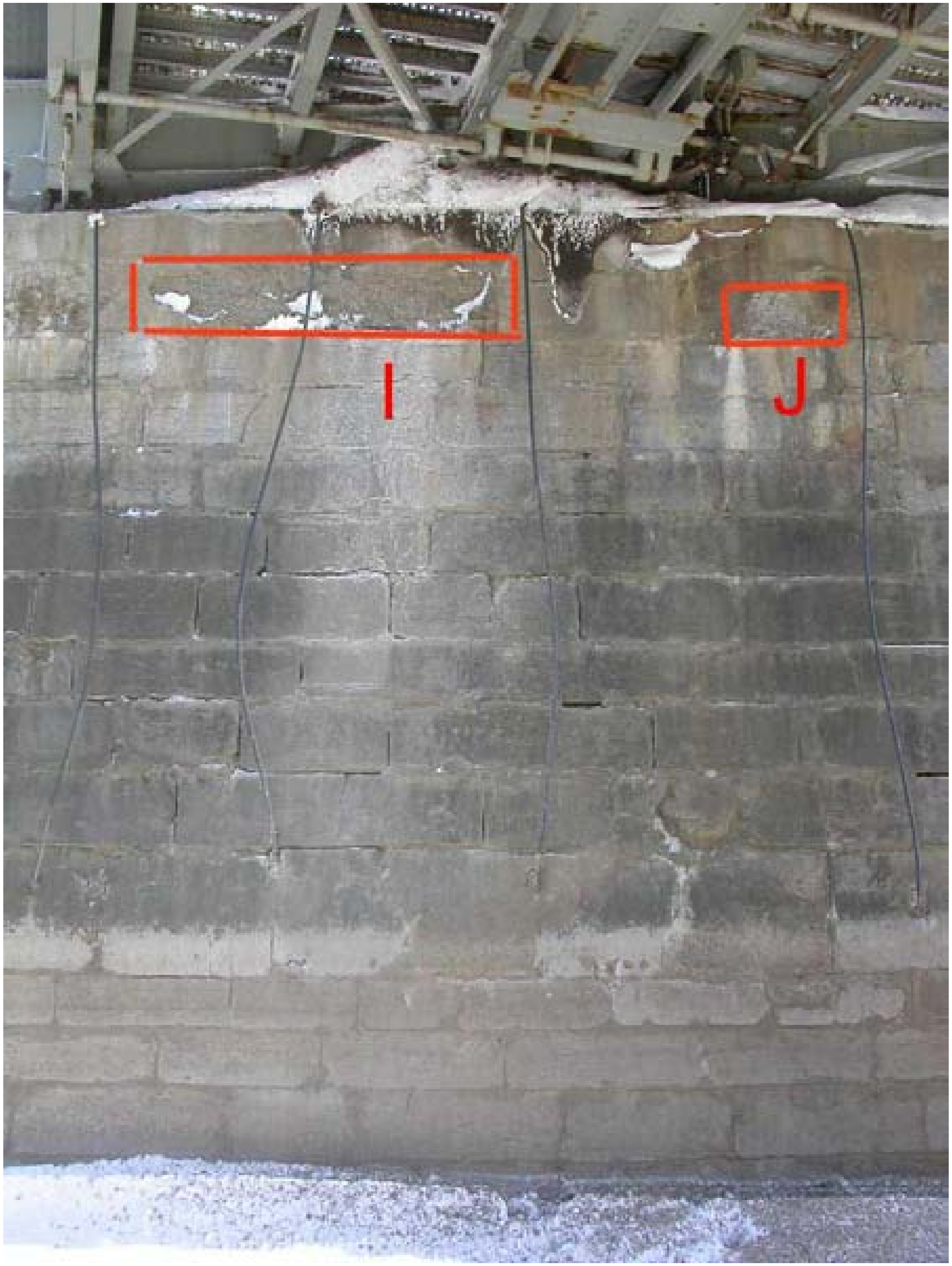
D
E
Q

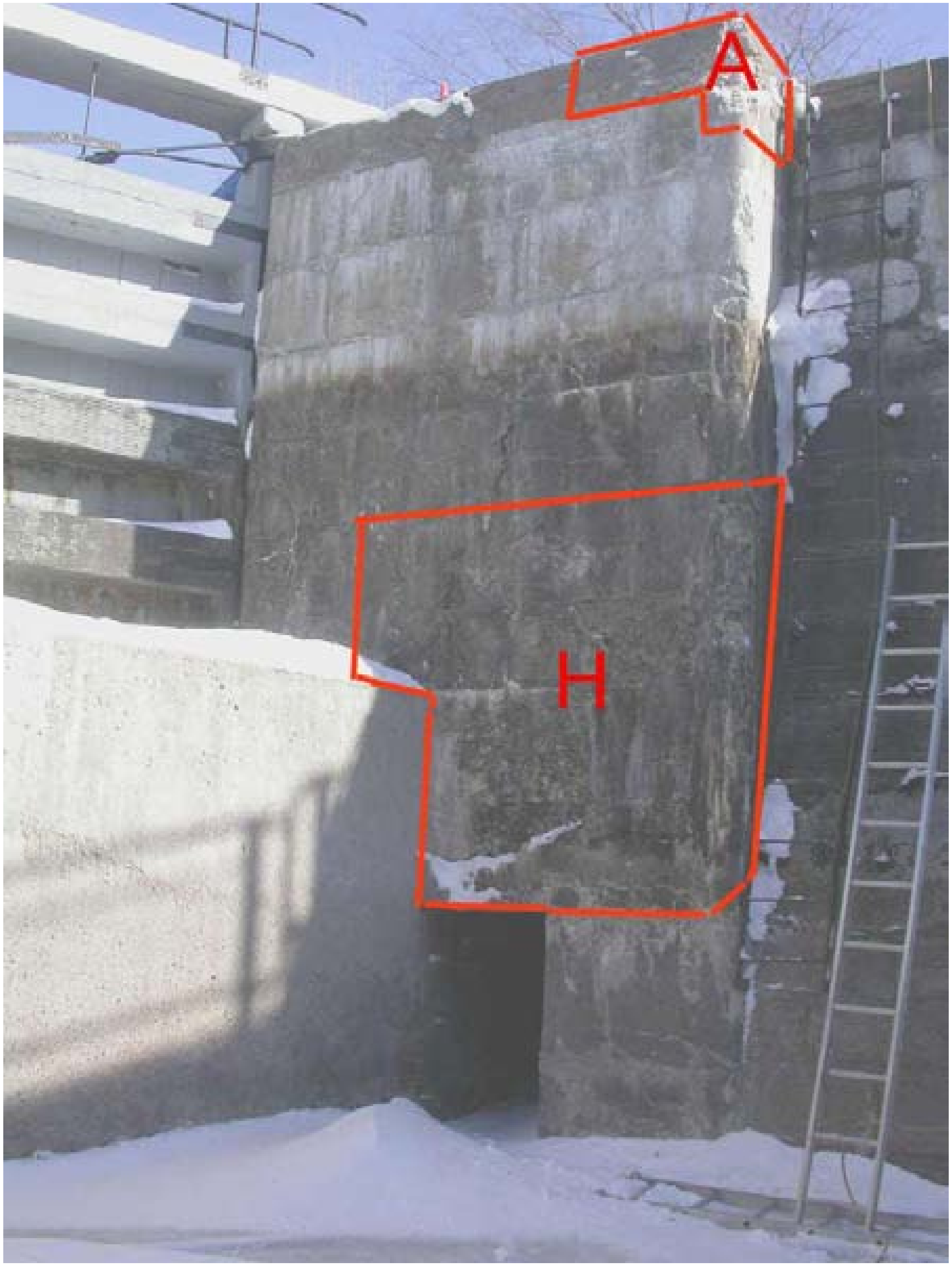

S

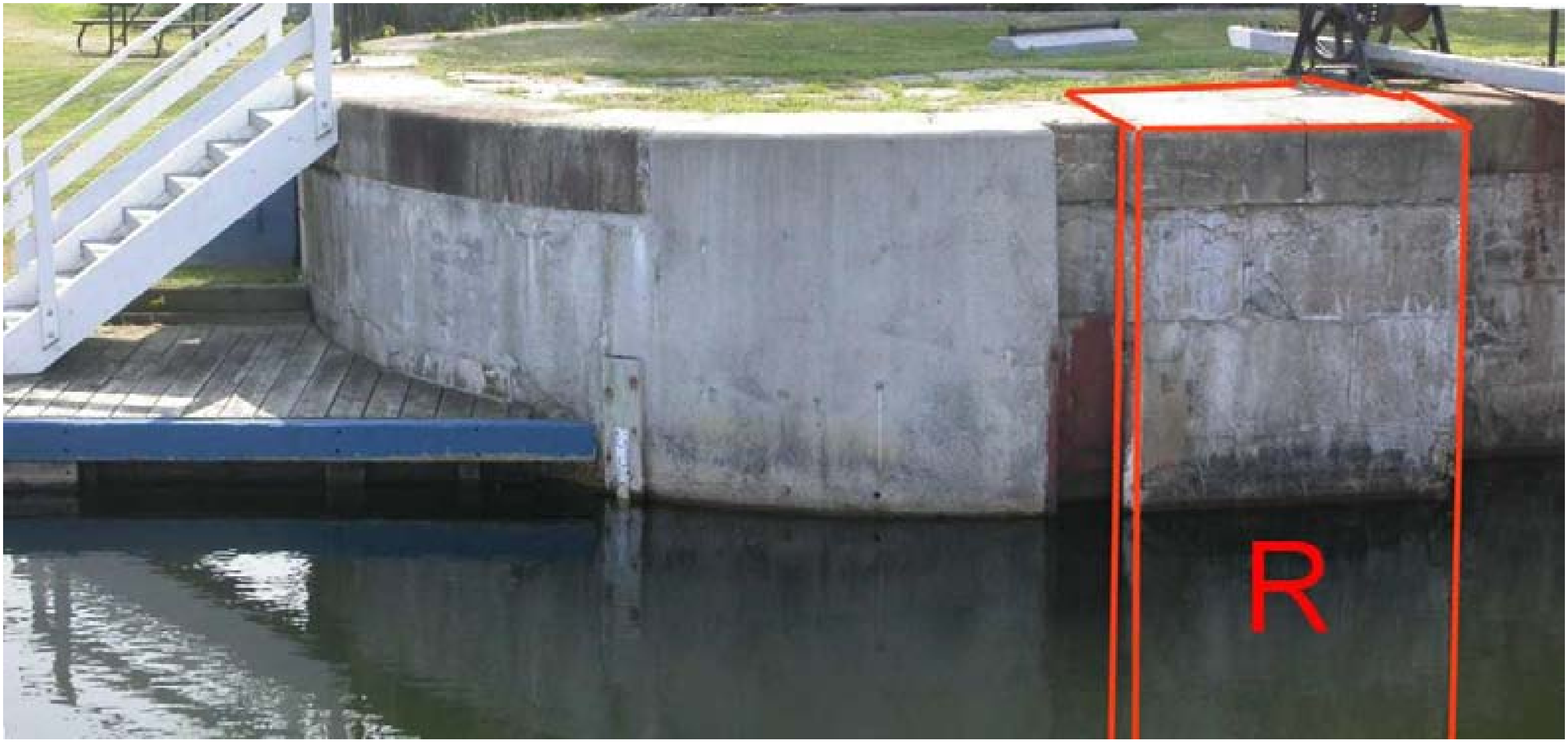

L
C


B
K


K


I
J


A
H


R