HYDRAULIC CONTROL OF SCOUR AT BRIDGES

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INTRODUCTION

Scour around bridge foundations has been a leading cause of bridge failure for the nearly 400,000 bridges over waterways in the United States (Murillo, 1987; Harrison and Morris, 1991; Lagasse et al., 1995; Parola et al., 1995). As a result, a wide variety of studies have been conducted in an attempt to predict and mitigate against scour. Mitigation measures have included armoring techniques as well as flow alteration devices. At small, single span bridges with vertical wall abutments, scour mitigation is often difficult because the use of armor, such as riprap and grout bags, fills a significant portion of the waterway beneath the bridge, which may then result in additional contraction scour. In addition, the channel may have migrated over the years such that the flow in the channel approaches the bridge at an angle which increases scour problems. Replacing these bridges is expensive and the replacement of numerous small bridges is not economically possible. At bridge piers, armoring is often ineffective as large flow events undermine and move the armor away from the pier.

A variety of flow alignment devices, such as vanes, cross vanes, and w-weirs, have been used in recent years primarily as treatment for bed and bank erosion in stream stability and restoration projects. These structures roll the water away from the eroding banks and, in the case of w-weirs, from the center of the channel. They have proven to be very effective treatments over a range of flow conditions. The use of these structures to align flow through a bridge opening could have the effect of moving scour away from the abutments and piers to center channel and mid-span, respectively.

Objective

The objective of this project was to evaluate the effectiveness of vanes, cross vanes, and w-weirs for controlling scour at bridge abutments and piers and to suggest optimum design parameters based on laboratory experiments. In addition, the energy losses caused by these structures were to be assessed.

Background

Scour at Bridges

Local scour at bridges refers to the vertical erosion of the channel bed in the vicinity of the piers or abutments caused by an obstruction to flow. As the flow approaches the obstruction, deceleration of the flow at the boundary causes a pressure gradient and a downward flow. This downflow is thought to be the primary scouring agent. Vortices at the base of the pier or abutment lift and carry sediment downstream. There have been considerable research efforts, particularly over the last decade, to predict and mitigate against scour. Scour depth is a function of many factors including: fluid characteristics (density, viscosity), flow characteristics (velocity, depth), sediment characteristics (size, gradation, critical velocity), and bridge characteristics (pier size and shape, contraction ratio, projected abutment length, angle of attack). Scour at piers and abutments is strongly and directly related to the size of the obstruction. For local abutment scour, the size of the total obstruction, including the embankment leading to the bridge, is important in that it blocks flow across the floodplain. It follows that abutment scour is indirectly a function of the discharge ratio of the main channel flow to the total discharge of the river (Richardson and Richardson, 1993; Sturm and Janjua, 1994).

Contraction scour is the result of acceleration of flow due to a reduction in flow area. From continuity, as the flow area is reduced the velocity must increase. The increase in stream velocity causes an increase in shear stress which in turn causes an increase in sediment transport capacity. The scour process will continue until the bed has eroded a sufficient amount to increase the flow area and reduce the velocity and shear stress below that necessary for sediment transport. Maximum scour under live-bed conditions occurs when the shear stress is reduced to the point such that the sediment that is transported into the reach equals the sediment that is transported out of the reach. Under clear-water conditions, maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material. Most existing contraction scour prediction equations are based primarily on the application of continuity to both the discharge and the sediment load through a long contraction (Straub, 1940; Laursen, 1962, 1963). In this case, contraction scour is also a function of the discharge ratio.

In addition to local and contraction scours, channel incision can adversely effect bridge stability by undermining piers and abutments. Channel bed degradation is a reach wide or system wide erosional problem that stems from a change in either the sediment load or water flow rate delivered to the stream. Migration of bed degradation through a bridge site can be halted by the use of weirs and check dams which effectively provide a control on the bed level.

Mitigation against scour at bridges has been the focus of much research in the past five to 10 years. There are basically two methods of mitigating against scour: (1) armor the piers or abutments to withstand shear stresses during high flow events and (2) alter the approach flow to break up vortices and reduce velocities in the vicinity of the piers or abutments. Other than design constraints, the decision regarding the method of mitigation should reflect maintenance and inspection requirements, enhancement of the physical environment, and the construction methods required. Design specifications for many of these mitigation techniques can be found in HEC-23 (Lagasse et al., 1997).

Armoring techniques for piers and abutments include riprap, precast concrete units, grout filled bags, foundation extensions, concrete aprons, and gabions. Riprap is by far the most commonly used armoring method. Armor tends to work poorly to moderately well at piers and moderately well to well at abutments in most river types and conditions. Problems encountered with armoring at piers include movement of sediment through the armor and an inability to keep the armor in place. At abutments, armor can constrict the channel, causing additional contraction scour.

Flow altering devices at piers and upstream of the bridge have been tried in the field as well as in the laboratory where they have met with limited success. Such techniques include the use of sheet piles and sacrificial piles placed upstream of piers and circular shields or collars constructed around the pier. Recent experimentation on sacrificial piles (Melville and Hadfield, 1999) showed that this method of reducing scour at bridges is only effective for clearwater flow conditions and for approach flow angles less than 20 degrees. In the field these types of devices have been problematic, particularly in that they tend to develop debris and ice accumulations and their effect on scour at high flow angles of attack can be minimal and, in some cases, harmful.

In addition to the flow altering devices described above, it is possible to alter flow alignment upstream of the bridge such that the main flow is concentrated away from the bridge piers and abutments, thereby reducing the velocities and shear stresses on the sediments around the bridge foundations. This type of flow alignment is possible through the use of vanes and other similar structures placed in the channel. Vanes have been used in various forms (e.g., Iowa vanes, bendway weirs) for many years to control bank erosion. Cross vanes and w-weirs, more recent treatments of bank erosion, have the added advantage of limiting bed degradation, particularly when used in a series, and providing environmental benefits. The pools developed downstream of these structures are desirable fish habitat. In the next three sections, we describe these structures. Additional background on bendway weirs, submerged vanes, spurs and groins, and prior work on energy losses at Iowa vanes can be found in Appendix A.

Vanes and similar structures

Vanes are single-arm structures angled to the flow and pitch into the streambed such that the tip of the vane are submerged even during low flow. When properly positioned, vanes induce secondary circulation of the flow thereby promoting the development of scour pools. Vanes are typically constructed from large boulders. They were developed to protect river banks from erosion (Rosgen 1994; 1996). Rosgen found from field experience that optimum results were obtained when the vane was orientated upstream at an angle of 20 to 30 degrees from the bank. At the bank the vanes should correspond to flood plain level and pitch down to intersect the bed of the river no more than one third of the way across the channel (see Figure 1). The vanes create quiescent conditions at the bank face, even during flood flows, while the faster overspilling flow is directed back into mid channel. Several vanes are required around an eroding meander bend to prevent further bank failure. Collectively they generate an outer bank secondary flow cell which opposes that resulting from bend curvature. As a consequence downwelling and scouring occur in mid channel, inside the toe of the vanes, while the former scour hole at the base of the bank is back filled with sediment. Design for these structures are now being incorporated into state guidelines on waterway construction (e.g., see Brown and

Johnson, 1999).

Cross Vanes

Cross vanes are structures that are used to divert erosive flow away from banks (see Figure 2). They consists of two arms, similar to vanes, and a central piece that crosses the stream either at the bed level or at the required bed level if grade control is needed. The arms of the cross vane are set at 20-30° from the bank and extend about 1/3 of the channel width into the channel. The cross vanes are constructed from rock that is large enough to resist movement from shear stresses expected for the design flow. They are best suited for use in moderate to high gradient streams and should be avoided in streams with unstable substrate or bedrock beds. There have been no systematic studies published on the use and design of cross vanes.

W-Weirs

W-weirs are a special case of cross vanes and weirs. Figure 3 shows a typical w-weir which has arms adjacent to the banks designed similarly to vanes and cross vanes and the central apex is about ½ the bankfull height. At the banks the arms extend to the height of the bankfull elevation. W-weirs are constructed from rock that is large enough to resist movement from shear stresses expected for the design flow. W-weirs are best used in rivers with bankfull widths greater than 12 meters. They are best suited for use in moderate to high gradient streams and should be avoided in streams with unstable substrate or bedrock beds. There have been no systematic studies published on the use and design of w-weirs.

BRIDGE SITE SELECTION

Seven bridge sites selected by engineers at the Maryland State Highway Administration were visited. These included Rt. 25 over Jones Falls, Route 25 over Western Run, Route 25 over Georges Run, Route 36 over Georges Creek, Route 219 over Youghigheny River, Route 7 over Bynum Run, and Route 75 over Union Bridge. The purpose of the visit was to identify at least one single-span and one double-span bridge that would be likely candidates for vanes or w-weirs to control flow under the bridge. In addition, these two bridges would become the templates for developing models in the laboratory. The following section provides observations from these site visits.

Route 36 over Georges Creek (Bridge # 1014)

Route 36 over Georges Creek is located in western MD, Garrett County, near Lacoming. This bridge is scheduled to be replaced as a single span bridge within a year. There is considerable degradation at this bridge, on the order of 1.2-1.5 m (4-5 ft). MDSHA is interested in controlling degradation in the vicinity of the bridge. Banks in the vicinity of the bridge are overheightened. Downstream banks are hardened by a railroad on the left bank and a concrete wall on the right bank. Upstream, banks are beginning to widen. The channel appears to have been straightened, at least in the vicinity of the bridge, and is currently about 40 feet wide. Bed material is primarily gravel, cobbles, boulders. The banks are composed of sand, silt, and clay. Vegetation is sparse, with a single row along the banks. The banks are currently too high for vegetation to provide protection from erosion. The bridge consists of two piers (triple span), vertical wall abutments, and piers and abutments skewed at about 10-20°. The footings are exposed, primarily due to degradation. Grout bags have been placed at the piers, but are not effective. The right abutment is severely undercut by erosion. This bridge was judged to be a poor candidate for this project since the bridge is being replaced and because channel degradation is so severe.

Route 219 over Youghigheny River (Bridge # 11024)

This bridge is located in western MD, Garrett County, near Redhouse. The channel is somewhat degraded. The width to depth ratio at the top of the banks is approximately 2.4, which is very entrenched. Meander migration is apparent, causing the channel to be grossly misaligned with the bridge. Outflanking occurs at the left abutment. Banks are composed of primarily sand, silt, and clay while the bed material is sand and gravel. Vegetation consists mostly of shrubs, grass, and a few trees. The bridge is a single span, concrete arch. Abutment setback from the channel is zero. The skew of the channel to the bridge is about 70°. The return flow at high water is probably less than 10%. Very deep scour exists at the right abutment. Grout bags have been placed at both abutments. This bridge was judged to be a poor candidate for this project. The stream needs to be realigned first.

Route 25 over Georges Run (Bridge # 3019)

This bridge is located north of Baltimore in Baltimore County. Channel meander migration has occurred on the right bank upstream of bridge, albeit at a slow rate. The floodplain is wide and the channel is slightly incised. The top of banks width to depth is 10. The channel consists of a pool-riffle sequence with point bars. The left bank has experienced significant erosion and failure on the outside meander with evidence of piping. Right bank exhibits mild erosion on the outside of meander. Bed material is sand and gravel with banks composed of silt and clay. Vegetation consists of grass on both sides and the upper banks. Trees exist in a narrow belt on right bank. The bridge is single span with vertical wall abutments and no set-back. Alignment with the channel exceeds 20°. Footings are exposed. Riprap has been dumped at the right abutment. Return flow at high water is approximately 10-25%. Scour exists at the right abutment, deposition at left. This bridge was judged to be a good candidate for vanes. However, it appears that the high flow angle of attack is about 30° toward the right abutment while the low flow (main channel) angle of attack is about 10° toward the right abutment. Vanes would have to be effective for the high flow which is clearly scouring out the right abutment.

Route 25 over Western Run (Bridge # 3023)

This bridge is located north of Baltimore in Baltimore County. The floodplain is wide. The channel has experienced some widening, mainly at the bridge. The width to depth ratio at top of banks is 11.7. The channel is not incised at the bridge; however incision has occurred upstream. Bank angles are about 60° on the left, 70-90° on the right. The left bank shows evidence of mild erosion while the right bank has experienced significant erosion. Bed material is composed of sand and gravel with silt and clay banks. Vegetation is primarily grass and trees. The bridge is single span with vertical wall abutments, no setback on the left abutment, and 4.6 m (15 ft) setback on the right abutment. Scour exists at the left abutment. Alignment with the channel exceeds 20°. Footings are exposed on the left abutment. Approximate return flow at high water is 25-50%. No scour mitigation is present. This bridge was a possible candidate for vanes.

Route 25 over Jones Falls (Bridge # 3027)

This bridge is located north of Baltimore in Baltimore County. Minor incision of the channel was observed along with some meander progression. The flow is fairly uniform (no pools/riffles present in vicinity of bridge) and the channel fairly stable. The width to depth ratio is 16. Bed material is sand and gravel and the bank material silt, sand, and clay. The banks are covered by woody vegetation. The bridge has a double span with a center pier, vertical wall abutments, and no setback from channel. Its alignment to the channel is less than 5°. Angle of high water approach is less than 5°. Approximate return flow at high water is 10-25%. Riprap is present at the left abutment and pier. Significant debris accumulation is present at the pier and under the right span. Flow divides equally beneath the two spans. This bridge was judged to be a good candidate for the two-span bridge model.

Route 7 over Bynum Run (Bridge # 12010)

The Route 7 bridge over Bynum Run is located in Harford County north of Route 24 near Abington. In this stream channel, the banks are blown out both upstream and downstream of bridge. Channel incision is minor; however, bank widening is prevalent. Pools and riffles exist in the channel. The width/depth at the tops of banks is 7.5. The right bank angle is about 60° and the left bank is 80°. Mass wasting occurs on both the left and right banks. Bed material is primarily sand and gravel, point bar material is gravel, and bank material is silt, sand, and clay. Vegetation on the left bank consists of grass and a narrow, sparse belt of trees. The right bank vegetation is grass. The two span bridge has vertical wall abutments with no setback from the channel. High flow has an angle of attack of about 5-10°. Grout bags are in place around the pier and both abutments. The left span is dry at low water and can only convey limited flow even at high water; it appears to be partially built into the floodplain. The bridge was judged to be a possible candidate for the two-span bridge model. However, the capacity of the left span is very limited. Also, the bank condition upstream will have to be dealt with before installing structures.

Route 75 over Union Bridge (Bridge # 6013)

This bridge is located in Carrol County at Union Bridge. The channel is somewhat incised. The top of bank width/depth ratio is 6.7. Bank angles on either side are about 80-90°. Significant bank erosion exists on the left and right banks (slides, slips, and fluvial erosion). In addition, there is piping on both banks. Bed material is sand and gravel and the bank material is sand, silt and clay. Vegetation is primarily grass on both banks. The bridge has a single span and vertical wall abutments with no setback from the channel. Footings are exposed due to scour. High flow angle of attack is about 5°. Grout bags are placed at both abutments. This bridge was judged to be a possible candidate for vanes if bank erosion can be controlled.

At most sites, there was minor to significant channel incision and/or bank widening observed. This would have to be addressed in order for the structures to be effective. Rosgen (1996) recommends w-weirs for B channels. Based on all of these observations, the two best candidates were the Route 25 bridge over Western Run (single span) and the Route 25 bridge over Jones Falls (double span).

Hydraulic and Bridge Characteristics at the Selected Bridges

Hydraulic characteristics at each of the two selected bridges were determined so that the scale laboratory model could be based on generalized features for these types of bridges. Table 1 provides general bridge and hydraulic characteristics for these channels.

EXPERIMENTAL PROGRAM

The experimental program is briefly described here. Additional detail is given in Appendix B. A 15 meter (50 ft) long, 1.5 meter (5 ft) wide, 0.9 meter (3 ft) deep recirculating flume at Penn State was used to simulate flow patterns and the resulting scour at bridge piers and abutments. It was desired that scaling be chosen to be representative of the range of bankfull widths, depths, slopes and discharges for the rivers under investigation. Tests on scour at bridge abutments were carried out by modeling the full channel (76 cm; 2½ ft) and one floodplain (76 cm; 2½ ft) in order to maintain sensible scaling. Return flows to the river at the bridge abutment, due to the embankment contracting the flow, would be representative of field conditions. The model abutment was a vertical wall type with set-back from the channel representative of the field sites and perpendicular to the channel. The floodplain and channel bank were rigid with a mobile channel bed. The tests were carried out over a range of flow conditions, beginning with the approximate bankfull discharge.

Based on physical constraints of space, a horizontal and vertical scale of 1:18 was selected for the vane and cross vane studies. Both the vanes and cross vanes were studied using the full channel plus one floodplain setup. For the w-weir, the scale was 1:9. This scale was possible because the full width of the flume could be used as the channel.

Six initial runs were conducted with the abutment in place, but with no vanes or cross vanes upstream, over a range of flow conditions to determine the location and extent of scouring that would occur without vanes or cross vanes. For the w-weirs, this process was repeated; five runs were conducted over a range of flows with no w-weir in place so that scour at the unprotected pier could be measured to use as a comparison. The bank angle, structure location, number of structures, and structure height were varied and tested with a range of flows primarily to assess the effect of overbank flow returning to the channel and to examine the ability of the structure to modify flow at the channel bed under flood conditions. The resulting scour depths and channel bed topography were measured and recorded at the end of each run.

The results of the experimental program and additional detail on the experiments themselves are given in Appendix B.

SUGGESTED DESIGN GUIDELINES

In this section, general guidelines are suggested for the various design parameters for vanes, cross vanes, and w-weirs based on the results of the experimental work. It should be kept in mind that these guidelines are based on small scale experiments that were greatly simplified from typical field conditions. Thus, these guidelines should be viewed as tentative until additional field observations can be made. General comments regarding the design and construction of all these structures are appropriate at this point:

- All vanes, cross vanes, and w-weirs will require appropriate footings to withstand expected scour in the channel. The MDE guidelines suggest that footings be placed at least two rock diameters beneath the lowest vane or cross vane rock. Additional guidance can be gleaned from observing channel scour along the thalweg upstream and downstream of the bridge over no less than a reach 20 times the channel width. The vane footers should be at least as deep as the deepest scour observed along the thalweg.

- Each of these structures (vanes, cross vanes, and w-weirs) provide scour reduction at bridge foundations by aligning the flow in such a manner as to reduce velocities and shear stresses in the vicinity of the foundation. They do not armor the bridge foundation against scour. Although these structures have been shown to be very effective in reducing scour at bridge piers and abutments in a laboratory setting, other parameters, such as changing angle of attack with increasing flood level, debris and ice accumulations, and interactions with existing armoring, have not been tested here and could influence the effectiveness of the structures.

- The experiments on the w-weirs have pointed out the possibility that if these structures are improperly designed, they can actually increase scour at the pier for certain weir designs under certain flow conditions. However, when properly designed according to specifications here, w-weirs have the ability to significantly reduce scour at bridge piers.

- The guidelines here are for the general case. Each design will be site specific and, thus, a wide variety of modifications or combination of structures may be needed to meet the design objectives.

- The guidelines can be applied to either new or existing bridges. At new bridges, the bridge should be designed for scour as usual. The vanes, cross vanes, or w-weirs will be an added feature used to control flow through the bridge opening and reduce future maintenance costs.

- Vanes, cross vanes, and w-weirs are not meant to replace riprap; they are meant to be used where riprap is inappropriate or ineffective. It may be desirable to use vanes at abutments under the following conditions: (1) where the abutment footing is very shallow due to years of scour, (2) where the channel has migrated up against one abutment; and (3) where the waterway opening is too narrow for riprap.

Bankfull Elevation

An important aspect of the design of these structures is determining the bankfull elevation. In most cases, the top of the structure will be placed at the bankfull elevation. Bankfull elevation is defined as the elevation of the active floodplain. This elevation does not necessarily correspond to the tops of the banks. The bankfull discharge, corresponding to the bankfull stage, is often considered to be the channel forming discharge (Chang, 1988; Leopold, 1994) and has been shown to occur with an average frequency of about 1.5 years (Leopold, 1994). For more urbanized areas the bankfull discharge can be more frequent, around 1.0 year or less. Others have found that bankfull discharge is sometimes less frequent, on the order of every 2 to 5 years.

The determination of a bankfull stage in an unstable (actively degrading or widening) channel is difficult and sometimes not possible. In an unstable channel, bankfull indicators are continually eroded away as the channel bed elevation drops and the banks collapse. Where the bankfull discharge is difficult to define and where sediment and daily discharge data are available, a dominant or effective discharge may be computed. For a stable channel, the bankfull discharge is equal to the dominant discharge. However, for a channel that has under gone recent channel adjustments, this may not be the case. In these cases, a stable reference channel adjacent to or nearby the channel reach in question can be used to approximate an appropriate bankfull elevation or discharge. For additional guidance in determining the bankfull elevation and discharge, the reader should refer to Leopold (1994), Rosgen (1996), and Hey (1998).

Design of Vanes for Scour Control at Abutments

Angle Adjacent to the Bank

The angle of the vane adjacent to the bank should be set at 25° to 30° (see Figure 4) to provide maximum flow control along the bed adjacent to the bank during overbank flow events. If the vanes are to be placed on a channel bend, then the vanes should be oriented at 25° to 30° to the tangent line of the bend at the attachment point on the bank. If the bridge abutments are skewed with respect to the channel, the vane angle α should be the same as for the unskewed case (with $25° < \alpha < 30°$, measured as in Figure 4) (see Figure 5 for skew angle measurement).

Upstream Location

The distance from the upstream end of the abutment to the upstream tip of the vane adjacent to the bank is a function of the length of the vane. Assuming that the length is such that it extends about 1/3 of the channel width into the channel and is set at 30°, then the distance is also a function of channel width. This optimum distance is given by $d = 2W$, where W = channel width and d = the projected distance along the bank from the upstream corner of the abutment to the upstream tip of the vane (with $d \geq 0$) (see Figure 4).

Number of Structures

In general, two structures provide greater flow control than a single structure, with about an additional 15% reduction in scour at the abutment for the two-vane case. Thus, it may be desirable to place two structures on the same side of the affected abutment upstream of the bridge. However, given that the cost of two vanes will be about twice that of a single vane, it may not be cost effective to add a second vane since the second vane will only provide an additional 15% scour reduction. A case where two vanes may be required is where the first vane is attached to the wing wall of the bridge. In this case, the vane at the abutment may significantly increase the shear stresses at the abutment and may actually increase scour at the abutment, as seen in the experimental results (see Appendix B). The addition of a second vane placed upstream of the first one will alleviate the high shear stresses by aligning the flow well upstream of the bridge, thus reducing scour at the bridge. If two vanes are used, the spacing between the structures should be based on the same calculation as the spacing between the bridge and the vane except that the distance will be from the upstream tip of the first vane (furthest downstream vane) to the upstream tip of the second vane upstream (see Figure 4).

Height of the Structures

The height of vanes at the bank should be at the bankfull elevation, pitching down to the channel invert at its tip. Raising the height of the structure slightly above the bankfull elevation, about ½ foot, will provide somewhat more hydraulic control during overbank events. However, a structure raised above bankfull may also cause additional scour or bank erosion. Thus, careful judgment should be exercised when raising the structure higher than the bankfull elevation.

Horizontal Pitch

The pitch from the horizontal should not exceed about 7.5% on low to moderate slopes, less than about 0.02 (scour depths increase with larger pitch angles because the overtopping flow is drawn over the structure quite abruptly). This geometry dictates that in order to achieve a 7.5% pitch with a 30° orientation to the bank, the channel must have a width to depth ratio of at least 20. If this cannot be met, then for a width to depth ratio of 17 or more, a 25° orientation to the bank can be used. The orientation should not be reduced below 25° as lower angles were shown to be less effective at moving scour away from the bridge abutment and banks. On higher slopes, 0.02 up to about 0.045, a higher pitch can be used, on the order of 12-14%.

Design of Cross Vanes for Scour Control at Abutments

In situations where both abutments require scour protection, then vanes would be required on both banks. By connecting the toe of the vanes with a rock sill, a cross vane is created. The bed invert can be set at the existing level or, if a degree of gradation control is required, at a higher elevation. The design is similar to vanes except for the cross portion.

Angle Adjacent to the Bank

The angle of the vane adjacent to the bank (the arms at the cross vane) should be set at 25° to 30° (see Figure 6) to provide maximum flow control along the bed adjacent to the bank during overbank flow events. If the vanes are to be placed on a channel bend, then the vanes should be oriented at 25° to 30° to the tangent line of the bend at the attachment point on the bank. If the bridge abutments are skewed with respect to the channel, the vane angle α should be the same as for the unskewed case (with $25° < \alpha < 30°$, measured as in Figure 4) (see Figure 5 for skew angle measurement).

Upstream Location

The distance from the upstream end of the abutment to the upstream tip of the vane adjacent to the bank is a function of the length of the vane. Assuming that the length is such that it extends about 1/3 of the channel width into the channel and is set at 30°, then the distance is also a function of channel width. This optimum distance is given by $d = 2W$, where W = channel width and d = the projected distance along the bank from the upstream corner of the abutment to the upstream tip of the vane (with $d \geq 0$) (see Figure 6).

Number of Structures

In general, two structures provide greater flow control than a single structure, with about an additional 15% reduction in scour at the abutment for the two-vane case. Thus, it may be desirable to place two structures on the same side of the affected abutment upstream of the bridge. However, given that the cost of two vanes will be about twice that of a single vane, it may not be cost effective to add a second vane since the second vane will only provide an additional 15% scour reduction. A case where two vanes may be required is where the first vane is attached to the wing wall of the bridge. In this case, the shear stresses at the abutment will be large and may actually increase scour at the abutment, as seen in the experimental results. The addition of a second vane placed upstream of the first one will alleviate the high shear stresses by aligning the flow well upstream of the bridge, thus reducing scour at the bridge. If two vanes are used, the spacing between the structures should be based on the same calculation as the spacing between the bridge and the vane except that the distance will be measured from the downstream junction of the vane and the bank to the upstream tip of the vane (see Figure 6).

Height of the Structures

The height of cross vanes at the bank should be at least as high as the top of the banks. Raising the height of the structure slightly above the bankfull elevation, about ½ foot, will provide considerably more hydraulic control during overbank events. However, a structure raised above bankfull may also cause additional scour or bank erosion. Thus, careful judgment should be exercised when raising the structure higher than the bankfull elevation. Each arm of the cross vane covers about 1/3 of the channel width. The cross portion of the vane should connect the two vanes adjacent to the banks and cover the remaining midsection of the channel at bed level (see Figures 2 and 6).

Horizontal Pitch

The pitch from the horizontal should not exceed about 7.5% on low to moderate slopes, less than about 0.02 (scour depths increase with larger pitch angles because the overtopping flow is drawn over the structure quite abruptly). This geometry dictates that in order to achieve a 7.5% pitch with a 30° orientation to the bank, the channel must have a width to depth ratio of at least 20. If this cannot be met, then for a width to depth ratio of 17 or more, a 25° orientation to the bank can be used. The orientation should not be reduced below 25° as lower angles were shown to be less effective at moving scour away from the bridge abutment and banks. On higher slopes, 0.02 up to about 0.045, a higher pitch can be used, on the order of 12-14%.

Design of W-Weirs for Scour Reduction at Piers

W-weirs may be used to reduce scour at bridge piers. In addition, the side arms of the w-weir will provide bank protection upstream and downstream of the weir.

Angle Adjacent to the Bank and the Interior Angle

In order to provide the appropriate pitch of the arms of the w-weir, the angle at the bank should be 25° (see Figure 7). The interior angle of the weir should be 40°. In any case, the pitch of the arm (labeled L in Figure 7) should be approximately 4° from the horizontal. A pitch that is too steep causes too much water to cascade over the weir contributing to the downward component of flow at the pier. A pitch that is too shallow provides inadequate divergence of the flow, particularly at deeper flood levels, and, thus, scour is not reduced at the pier. If the w-weir is designed such that each of the four arms covers about 1/4 of the cross section, this design should provide an adequate pitch such that the depositional mound downstream of the central apex is maximized.

Upstream Location

The central apex of the w-weir should be located about 0.3 times the channel width upstream of the bridge pier (see Figure 7). This will assure that the pier nose will be within the depositional zone of the w-weir for all flow depths.

Height of the W-weir

The height of the arms of the w-weir at the banks should be at bankfull to provide the appropriate pitch of the arms. The height of the central apex of the w-weir may be set between ½ bankfull height and bankfull height. Although it was shown in the experiments that the higher the central apex, the less scour at the pier, setting the central apex at the bankfull height may result in too much water being diverted toward the banks and, thus, additional erosion at the banks (this was not tested in the experiments).

Energy Losses

The energy losses across each of these structures was measured during the laboratory experiments (see Appendix B). Although the structure increases the flow resistance, and thereby energy loss, locally, when this resistance is averaged over the entire reach that is represented by the cross section, the contribution to resistance and energy loss is negligible. Thus, it is recommended that resistance due to one or two vanes, cross vanes, or w-weirs in a reach can be ignored for out-of-bank flows.

Limitations of the Design Guidelines

The guidelines developed here are tentative in that they have been developed from small-scale experimental evidence and are untested in the field. Vanes, cross vanes, and w-weirs deflect flow away from banks, abutments, and piers. Although the suggested construction material is rock, these structures *do not* armor the banks, abutments, or piers. Therefore, the engineer may desire to use both armor and flow deflectors. The armor will help to protect the bridge foundation in case of failure of the vanes, cross vanes, or w-weirs or in the case that the guidelines prove to be less effective in the field than in the lab.

For a single span bridge with a single abutment which is experiencing scour, it is suggested that one or two vanes be placed according to these guidelines upstream of the bridge. If both abutments are experiencing scour, then a cross vane may be used to deflect flow from either abutment. A two-span bridge which has scour at the pier may require the use of a w-weir. The w-weir not only deflects flow around and away from the pier, but it also deflects flows away from the banks, and thus, the abutments.

In any case, the engineer may wish to install a second vane or set of vane or cross vane just downstream of the bridge to further control flow as it exits the bridge opening or to provide grade control or bank stability. This scenario was not tested in the laboratory.

Monitoring

Monitoring requirements for vanes, cross vanes, and w-weirs at bridges will necessarily be different from those at restoration projects. Bridge inspections are required every two years; thus, if the vanes, cross vanes. or w-weirs are part of the bridge system, they will likely need to be inspected every two years as well. Monitoring only after overbank flood events may not be sufficient, as scour can occur during a flood event and refill during the receding flood, leaving minimal evidence of undermining that may have occurred. In addition, because the bridge must withstand the 100-year flood, the structures must be monitored to assure that the footing and rock size are adequate to withstand such high shear stresses. The Federal Highway Administration (FHWA) provides guidelines for the selection of rock size for a wide range of flow conditions (Lagasse et al., 1997). It is recommended that the footing consist of one to two layers of footer rocks, depending on the bed substrate and predicted maximum scour depth. In addition, the footing depth should be at least as deep as the deepest scour observed along the thalweg upstream and downstream of the bridge over a reach length no less than 20 times the channel width. Finally, monitoring programs following the construction of stream restoration projects typically have durations of three to five years. Where a bridge is located in the reach and is part of the restoration design, the monitoring duration for the structures at the bridge may need to be considerably longer to capture high flow events.

APPLICABILITY

It is anticipated that vanes, cross vanes, and w-weirs will not be acceptable solutions at all bridges. Certain stream and bridge types will not be amenable to the use of these structures. Based on the experimental work as well as previous use of these structures in the field for stream restoration purposes (e.g., see Anacostia Restoration Team, 1992; Haltiner, 1995; Rosgen, 1996; and Brown and Johnson, 1999), it is likely that they will be best suited to stream channels with the following characteristics:

- Gravel-bed streams. Stream bed material with a median sediment size in the gravel to cobble range is most appropriate. Installation in silt or fine sand bed material can be difficult and excessive erosion may occur; thus it is necessary for the installation to be accompanied by the use of geotextiles to promote stability of the structure.

- Bankfull channel width greater than 12 m (40 feet). A channel width of at least 12 meters is desirable for w-weirs so that the appropriate angle at the banks and pitch of the structures can be obtained. Cross vanes and vanes can be constructed in narrower channels; however, this requires a lower bank angle (down to 20°) which was shown in the laboratory to be less effective in moving scour away from the abutments than the 25° to 30° angles.
- Moderate bankfull width to depth ratio. A low width to depth ratio will not provide adequate space for the correct angle and pitch of the structures. The width to depth ratio should be at least 10. A reduced angle can be used to accommodate vanes and cross vanes in lower width/depth channels, as described above.
- Channel pattern. A straight channel or mild to moderate sinuosity is acceptable. Installation of these structures in a braided channel should be avoided.
- Moderate to high flow velocity. Slow flow, pooled reaches, and backwater areas are not appropriate because the structures will not have the intended influence upon the flow field.

In addition to the stream channel characteristics listed above, vanes, cross vanes, and w-weirs can be used in streams with a relatively high bedload transport. Urban streams which experience "flashy" flows can also be acceptable environments for these structures, provided that the other characteristics listed above are present.

The limitations and recommendations for appropriate stream characteristics lead to the following bridge characteristics that are recommended for these structures:

- Single span bridge. For single span bridges, vanes or cross vanes will provide flow transitions through the span and reduce shear stresses and scour along the abutments.
- Double span bridge. For two-span bridges, w-weirs will provide flow transition through the bridge opening and will reduce shear stresses and scour at both the abutments and center pier.
- Blocked floodplain. For bridges where the floodplain is blocked or partially blocked by the bridge approach or embankments, vanes, cross vanes, and w-weirs can be particularly

effective in providing a smoother transition from the upstream floodplain through the bridge opening.

- Span width. As stated above, the bankfull channel width must be at least 12 meters to accommodate these structures. Therefore, the span width should also be approximately 12 meters or greater. For vanes, and cross vanes, a smaller span width may be acceptable.

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Table 1. Summary of hydraulic and bridge characteristics for selected bridge sites.

Variable	Western Run	Jones Falls
Bankfull flow depth (m)	1.60	1.40
Bankfull width (m)	22.4	14.0
Width - depth ratio	14	10
Particle size (mm)	32	37
Particle gradation	2.10	2.20
Channel slope (m/m)	0.0020	0.0004
Bankfull discharge (m^3/s)	5.66	13.00
50-year discharge (m^3/s)	279	368
Bridge length (m)	24.4	14
Bridge height from thalweg (m)	6.1	3

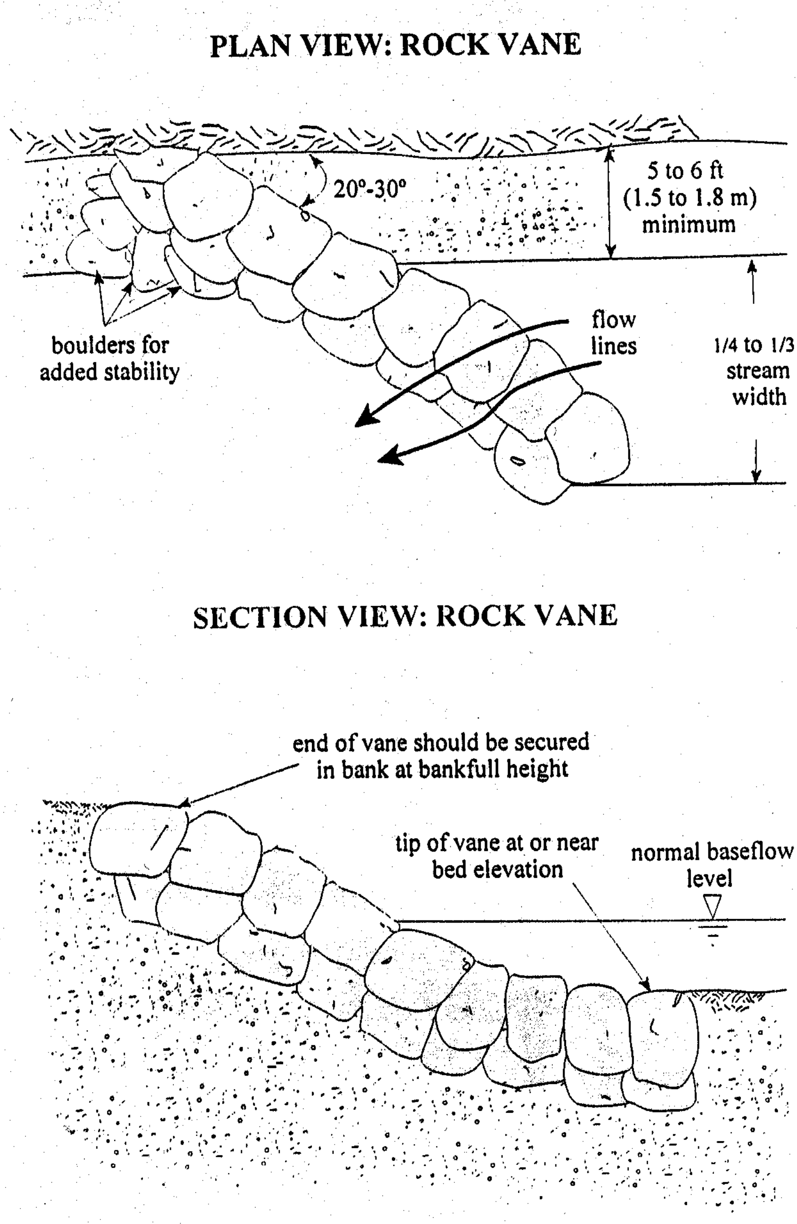
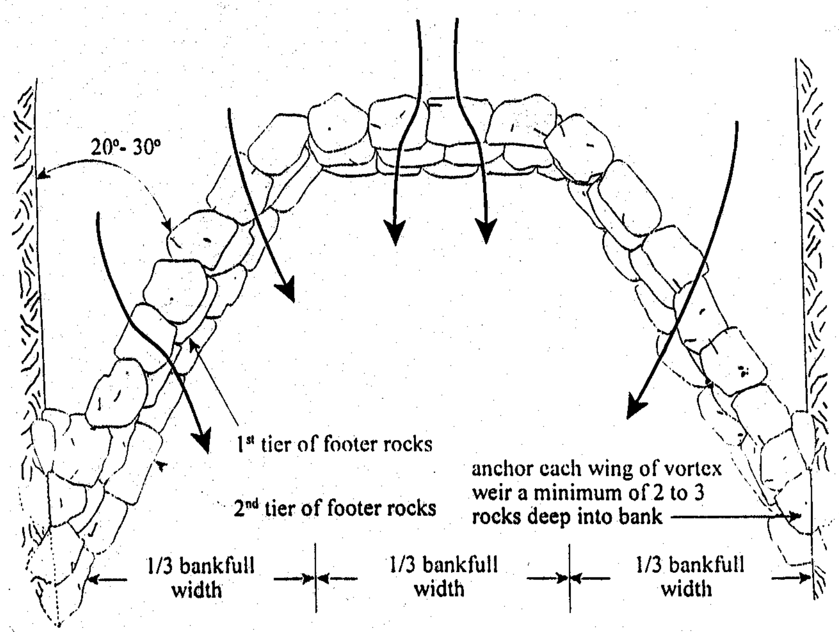


Figure 1. Guidelines for vane design (after Rosgen, 1995; Brown and Johnson, 1999).

PLAN VIEW: CROSS VANE



SECTION VIEW: CROSS VANE

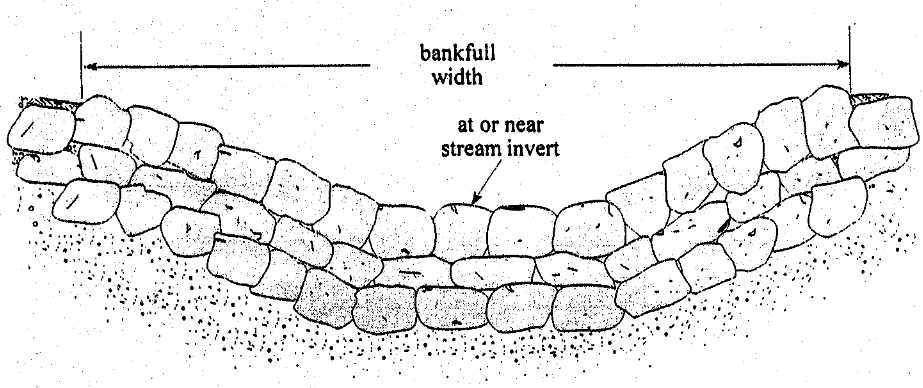
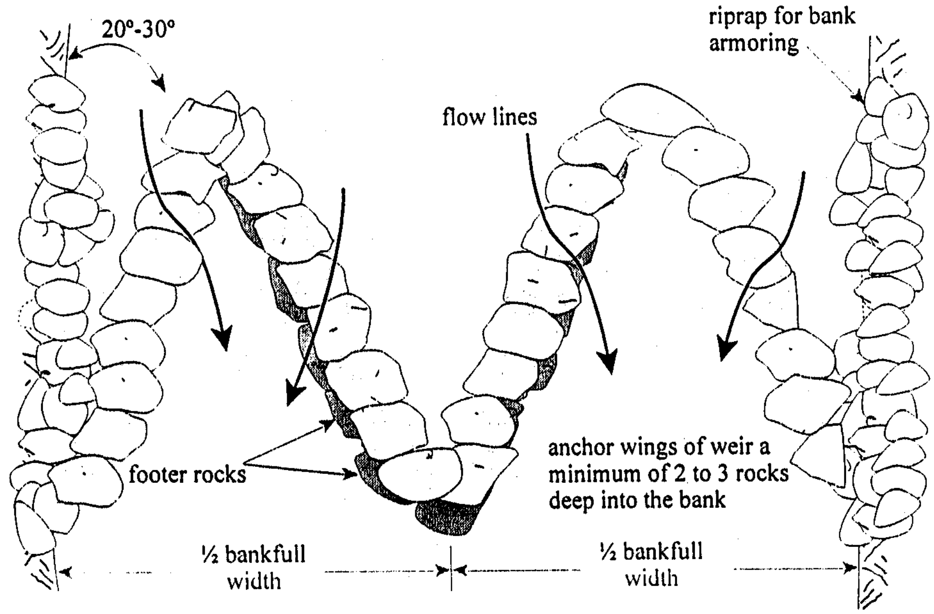


Figure 2. Guidelines for cross vane design (after Rosgen, 1996; Brown and Johnson, 1999).

PLAN VIEW: W-ROCK WEIR



SECTION VIEW: W-ROCK WEIR

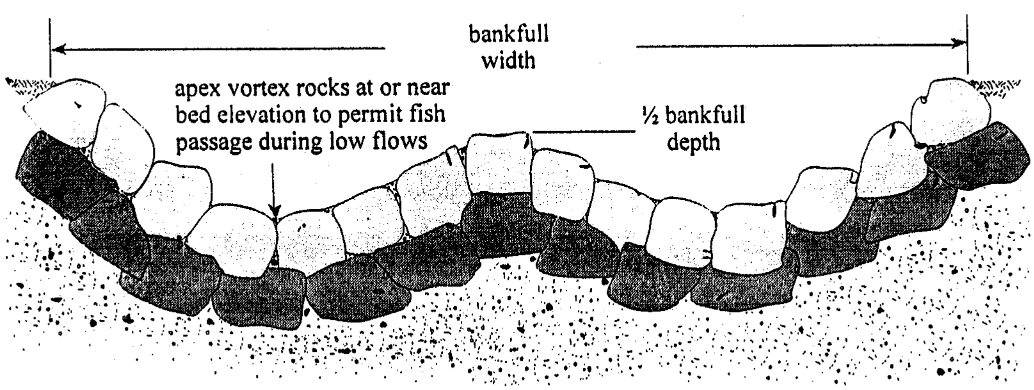


Figure 3. Guidelines for w-weir design (after Rosgen, 1996; Brown and Johnson, 1999).

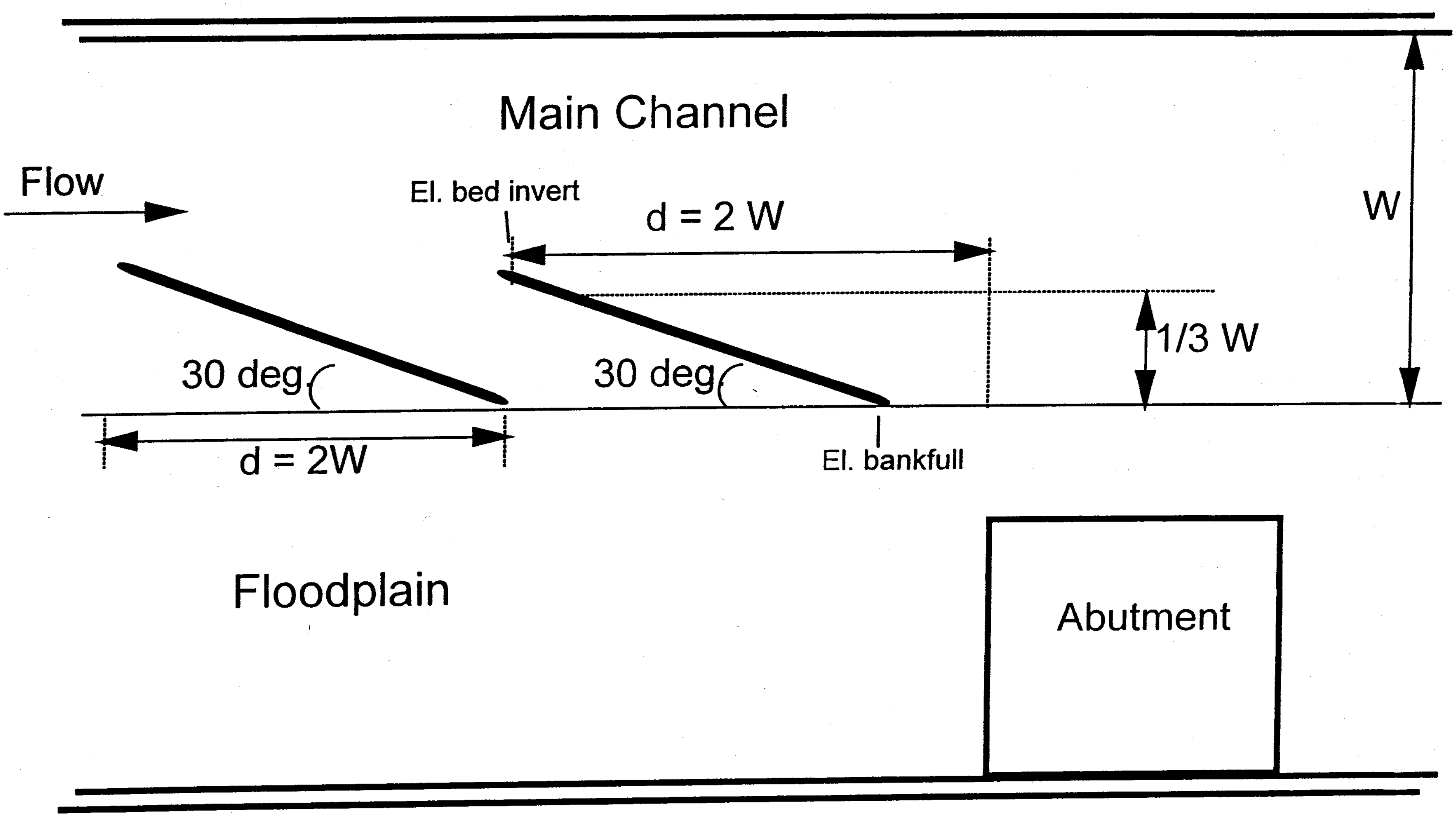


Figure 4. Design configuration for vanes.

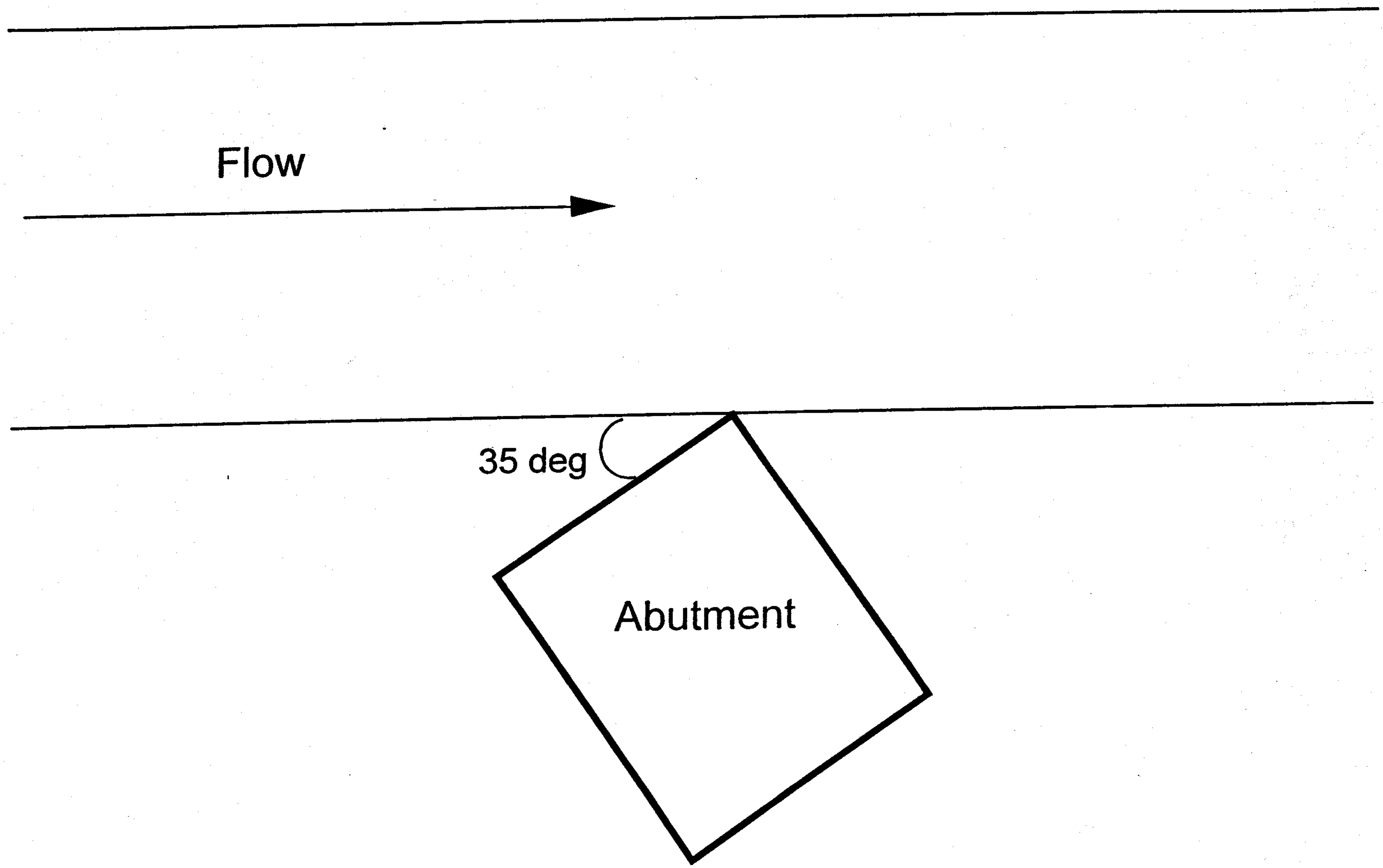


Figure 5. Skewed abutment angle.

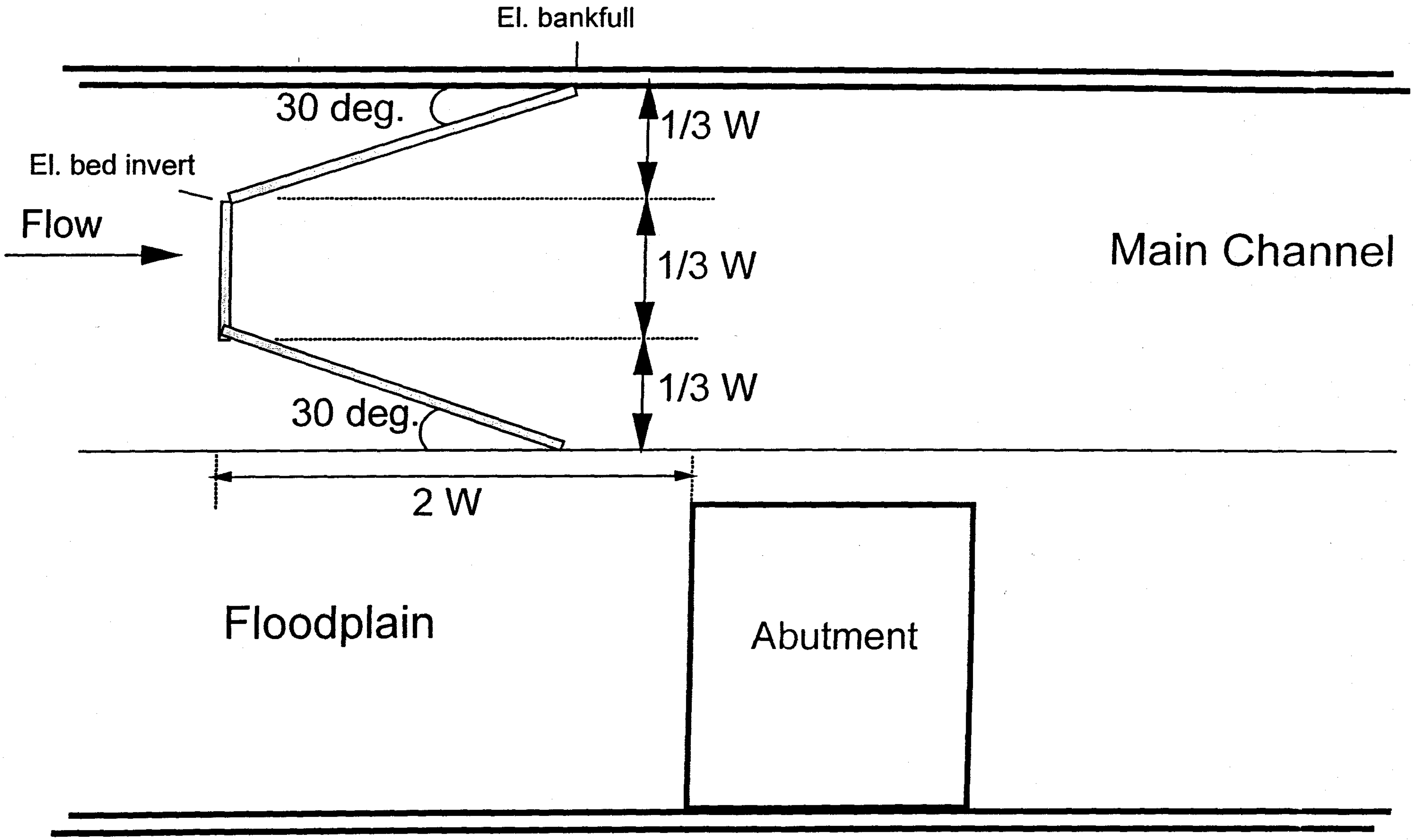


Figure 6. Design configuration for cross vanes.

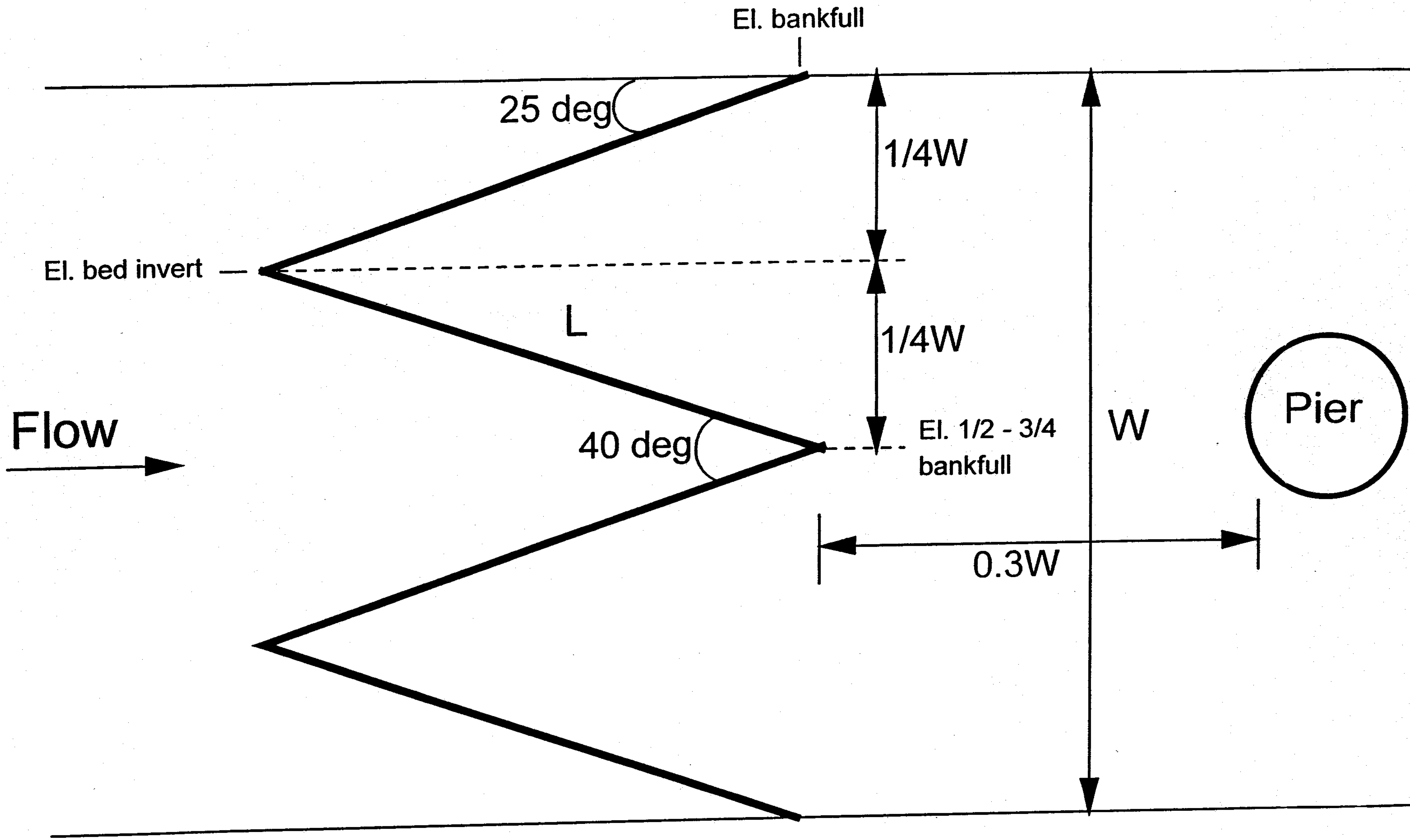


Figure 7. Design configuration for w-weirs.

APPENDIX A

SIMILAR STRUCTURES TO CONTROL OR DIVERT FLOW

SIMILAR STRUCTURES USED TO CONTROL FLOW

Small, isolated, submerged vanes, known as Iowa vanes, have been used for many years to deflect flows and sediment to control spiral flow in bends and erosion at banks. A variety of experimental studies (Odgaard and Kennedy, 1983; Odgaard and Lee, 1984; Odgaard and Mosconi, 1987) have yielded guidance in the design of these types of vanes. Submerged vanes were found to be effective over a wide range of flow depths from two to eight times the vane height. The discharge was determined not to be a primary design parameter; discharge is used only to determine velocity. The ratio of the vane height to flow depth should be between 0.2 and 0.5 at the erosion causing flow rates. The length should be about three to four times the vane height with an optimum angle of about 20° from the primary direction of flow. Lateral spacing of the paired vanes should be less than about twice the flow depth. Vanes are typically constructed from large rocks with footers of adequate depth to resist erosional forces.

The ability of submerged (Iowa) vanes to reduce scour at bridge piers was recently tested at the University of Auckland (Lauchlan, 1999). Arrays of six submerged vanes were tested under both clear water and live bed conditions in laboratory flumes. Two different types of vanes were tested: (1) vanes with length to height ratio (L/H) less than one and (2) vanes with length to height ratio greater than one. In all cases, the vanes were rectangular in shape and set at a shallow angle to the flow in an array upstream of a circular pier with diameter D = 200 mm. Figure A-1 shows the layout of the vanes with respect to the pier. For both clear water and live bed conditions, the vanes with $L/H < 1$ were ineffective. However, the vanes with $L/H > 1$ (similar to Iowa vanes) were shown to markedly reduce scour at piers for certain arrangements of the vanes. For the live bed condition, the best scour reduction appeared to be associated with a vane angle (α) of 30°, a six-vane array, streamwise spacing e = 2D, and lateral spacing z = 2D, resulting in scour reduction greater than 50%. For clearwater conditions, scour was generally reduced less than about 20%. Best results occurred for e = 0.5D and z = 0.5D.

Bendway weirs are low elevation stone sills very similar to vane structures used to improve lateral stream stability and flow alignment problems (Lagasse et al., 1997). Bendway weirs are typically not visible at bankfull flow. They redirect flow by causing the flow to pass perpendicularly over the weir. They are made from stone, tree trunks, and grout filled bags. Based on HEC-23 (Lagasse et al., 1997), a brief summary of design guidelines are given. The weir height should be 30 to 50 percent of the flow depth at the mean annual high water level. The angle from the upstream bank tangent line to the centerline of the weir should be about 50 to 85 degrees (this high angle is due to the placement of the weirs at channel bends). The length should not exceed 1/3 the mean channel width, with typical values between 1/10 and 1/4 of the channel width. Spacing of the weirs is dependent on the channel radius of curvature, weir length, and channel width. The top width of the weir should be two to three times the D_{100} of the rocks used to construct the weir. At least three weirs are used to direct flow around a bend.

Spurs and groins are frequently used structures to protect river banks. Lagasse et al. (1995) recommend that spurs be angled upstream at 70° from the channel bank. Spurs crests can be horizontal or sloping. Maza Alvarez (1989) suggests that a slope-crested groin should be sloped at 0.1 to 0.25 (5.7° to 14°) towards the center of the river channel. Sloping the crest of the groin has the advantage of causing much less scour around the tip of the groin and sediment deposition between adjacent spurs occurs more quickly. Maza Alvarez (1989) developed a method for determining the spacing between groins. He found that for a straight channel, groins should be spaced according to four to six times the length of the groins. The length of the groin should be greater than the mean flow depth of the reach but less than a quarter of the bankfull width. For curved reaches, the suggested spacing is 2.5 to four times the groin length. In terms of channel geometry, the spacing is $W < S < 1.5W$, where W = channel width and S = spacing of the groins. The development of the spacing criteria was based on flow expansion around the groin.

Energy losses caused by vanes in a stream channel can be determined by estimating the change in the friction slope, $\bar{S}_f$ (Odgaard and Mosconi, 1987). The flowing fluid exerts a shear stress which acts over the bed in the reach which contains the vane. As the fluid flows past the vane, the structure exerts a drag force, F_D. The difference between these two forces must be equal to the negative change in momentum:

$$\Delta\tau A_b - F_D = -\Delta M \tag{1}$$

where τ = average boundary shear stress, A_b = area affected by the shear stress along the bed, and M = momentum. The average boundary shear stress is given by:

$$\tau = \gamma y S_f \tag{2}$$

where γ = specific weight of water and y = flow depth. Drag force is given by:

$$F_D = \frac{1}{2}\rho C_D V^2 A_s \tag{3}$$

where ρ = density of water, C_D = drag coefficient, V = average cross sectional velocity, and A_s = surface area of the vane. For A_b = xb, where x = longitudinal distance between the upstream and downstream ends of the vane and b = width of the vane structure within the channel, approximating $A_s \approx$ ½ NLH, where N = number of vanes, L = vane length, and H = vane height, and substituting Eqs. 2 and 3 into Eq. 1 yields:

$$\gamma y \Delta S_f(xb) - \frac{1}{2}\rho C_D V^2\left(\frac{1}{2}NLH\right) = -\Delta M \tag{4}$$

At the upper limit of S_f, $\Delta M = 0$. Thus, Eq. 4 can be solved for ΔS_f:

$$\Delta S_f = \frac{C_D NLH}{2yxb}\frac{V^2}{2g} \tag{5}$$

Assuming that C_D can be approximated for a vane, the change in energy slope due to flow over the vane structure can be estimated from Eq. 5. As an example, assuming $C_D = 0.6$, N = 1, L = 10 m, H = 1 m, V = 0.9 m/s, y = 1.1 m (vane just submerged), x = 8.7 m, and b = 5 m yields ΔS_f = 0.0026. Over the 8.7 m distance, this represents a head loss of 2.3 cm and a Manning roughness coefficient of n = 0.060 for one vane.

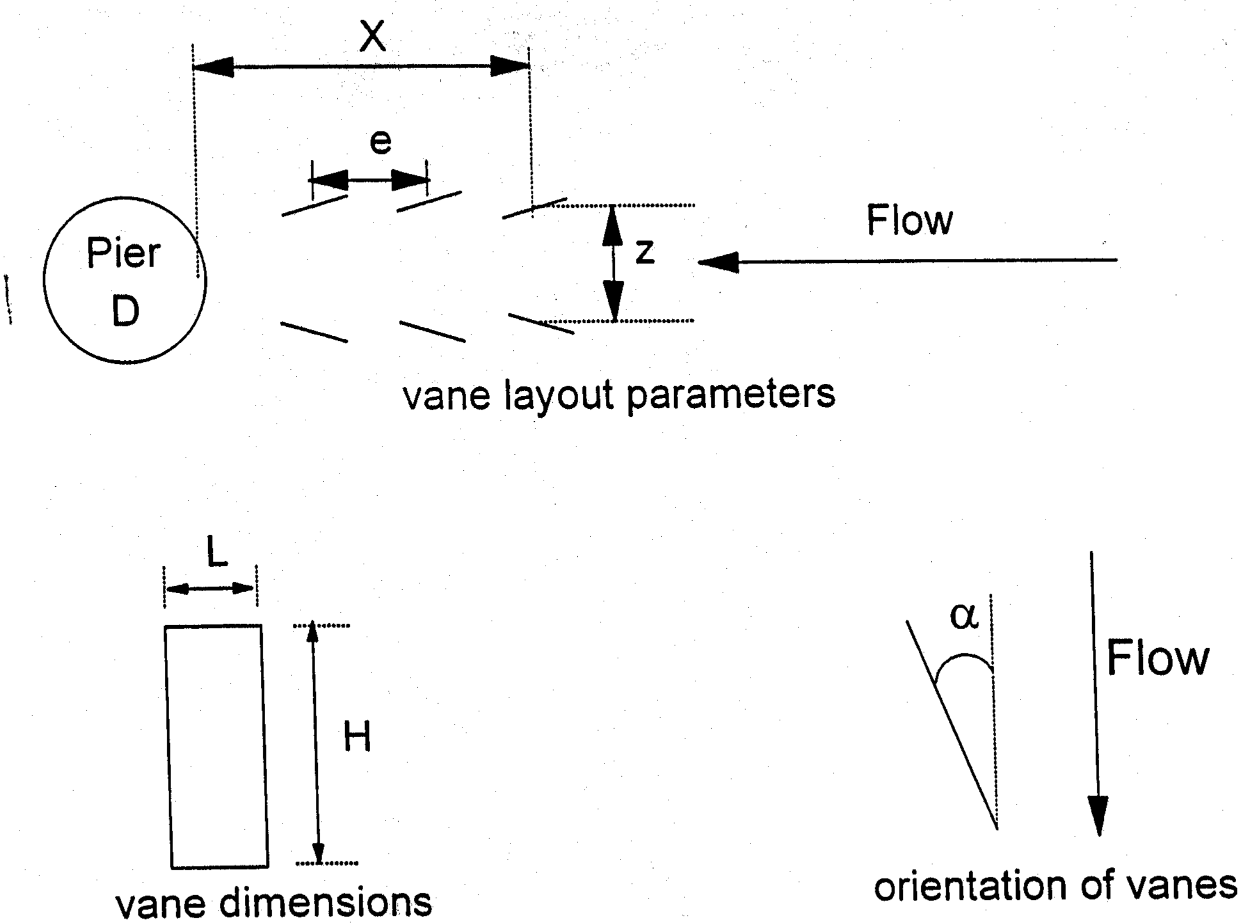


Figure A-1. Submerged vanes for pier scour reduction (after Lauchlan, 1999).

APPENDIX B

DATA AND ANALYSES

EXPERIMENTAL PROGRAM

A 15 meter (50 ft) long, 1.5 meter (5 ft) wide, 0.9 meter (3 ft) deep recirculating flume was used to simulate flow patterns and the resulting scour at bridge piers and abutments. A venturi meter and manometer were used to provide discharge measurements. A point gage was used to measure flow depths in the flume. Flow velocity was measured with an acoustic doppler velocity meter attached to the overhead carriage.

It was desired that scaling be chosen to be representative of the range of bankfull widths, depths, slopes and discharges for the rivers under investigation. Tests on scour at bridge abutments were to be carried out by modeling the channel (76 cm; 2½ ft) and one floodplain (76 cm; 2½ ft) in order to maintain sensible scaling. Although the floodplain would have limited width, as it was to be relatively smooth it could transmit discharges comparable to a wider, rougher, floodplain. Consequently return flows to the river at the bridge abutment, due to the embankment contracting the flow, would be representative of field conditions. The model abutment was a vertical wall type with set-back from the channel representative of the field sites and perpendicular to the channel. The floodplain and channel bank were rigid with a mobile channel bed. Sediment size was to be chosen such that bedforms would be minimized and the sediment would be only approximately scaled to the field size. All flows were to be run at incipient sediment motion to provide maximum scour conditions and a consistent velocity ratio from one experiment to the next. The tests were run at the approximate bankfull condition and for a range of flood flows to determine peak flow effects.

Scaling of bridge, flow, and sediment characteristics was accomplished based on the Froude number, velocity ratio, overbank return flow, and geometry. Froude number similarity is important in any open channel flow problem. Although it is usually impossible to maintain similarity between multiple similarity parameters, the parameters can be maintained within an acceptable range that supplies typical values in the field and which provides the appropriate flow conditions. In the case of mobile bed studies, the Froude number is maintained within an acceptable range so that the velocity ratio (discussed below) can be maintained at the desired value.

The velocity ratio, V/V_c, where V = velocity and V_c = critical velocity, is used to distinguish between live bed and clear water scour. V_c can be approximated using the Neill (1968) equation:

$$V_c = 6.19 y^{1/6} D^{1/3} \qquad \textbf{(1)}$$

where y = flow depth and D = sediment size. The value of V_c was then verified in the flume prior to any experimental runs. V/V_c was held at approximately one in the laboratory experiments for the following reasons: (1) the maximum scour depth occurs at approximately $V/V_c = 1$ (Chiew 1984), and (2) no bedforms are expected at incipient motion.

For each flow depth, the corresponding discharge ratio ($M = q_o/q_m$, where q_o = unit discharge in the overbank areas and q_m = discharge in the main channel upstream of the bridge) computed for the field was maintained in the experiments. Thus, M ranged from 0 at bankfull to about 0.2 at the maximum discharge, similar to the range of M in the field. This was accomplished by changing the roughness on the floodplain using metal brackets. Only one side of the floodplain and bridge abutment are needed provided that the discharge ratio is maintained.

Based on physical constraints of space, a horizontal and vertical scale of 1:18 was selected for the vane and cross vane studies. Both the vanes and cross vanes were studied using the full channel plus one floodplain setup. For the w-weir, the scale was 1:9. This scale was possible because the full width of the flume could be used as the channel. Floodplain-channel dynamics were ignored for the w-weir setup since the concern was at the pier rather than the abutment. The constraints and similarity issues resulted in the model dimensions given in Table B-1.

A total of 75 experimental runs were conducted. Each experiment was run for four hours. Although this length of time at incipient motion does not yield the maximum scour depth, it does provide approximately 75 percent of the total scour based on the Laursen (1963) equation as recommended in Umbrell et al. (1998). Since the objective of the experiments is to determine the change in the scour pattern with the use of vanes and weirs rather than the determination of the maximum scour depth, it was decided that a four-hour time interval provided a consistent and adequate time period.

With the assistance of an Aerospace Engineering graduate student, a computational exercise was undertaken to determine the appropriate initial placement for a single partial-width vane upstream of the bridge. This situation was modeled using an existing two-dimensional finite element flow model of the Navier-Stokes equations to model the flow separation and reattachment caused by the vane. A two-dimensional model was deemed to be adequate for this purpose. The distance from the tip of the vane to where the flow reattached to the channel wall was used to determine the initial distance upstream from the abutment to place the vane. For the model to be constructed and the various flow conditions, the range of distances from the tip of the vane to the flow reattachment point was found to be 0.94 to 1.25 meters. Initially, a distance of 0.94 m was used as the placement for the vane upstream of the abutment, as shown in Figure B-1. In order to keep the entire abutment within the influence of the flow separation zone, the 0.94 m distance is measured from the upstream end of the abutment to the upstream tip of the vane.

Six initial runs were conducted with the abutment in place, but with no vanes or cross vanes upstream, over a range of flow conditions to determine the location and extent of scouring that would occur without vanes or cross vanes. For the w-weirs, this process was repeated; five runs were conducted over a range of flows with no w-weir in place so that scour at the unprotected pier could be measured to use as a comparison. The bank angle, structure location, number of structures, and structure height were varied and tested with a range of flows primarily to assess the effect of overbank flow returning to the channel and to examine the ability of the structure to modify flow at the channel bed under flood conditions. The resulting scour depths and channel bed topography were measured and recorded at the end of each four-hour period.

The general flume layouts for the vanes, cross vanes, and w-weirs are given in Figures B-1 to B-3. The vanes and w-weirs were constructed from marine plywood. All structures were attached to the bank so that flow was not permitted between the structure and the bank. The vanes were constructed to span 1/3 of the channel width. The upstream tip of the vane was set at the bed level, giving a crest slope of about 9.5°. The cross vane spanned the entire channel, as it is not possible to test ½ cross vanes due to scour around the exposed tips. Figure B-4 shows the layout of the measurement scheme for the vanes. At each cross section and each transverse point, flow depths were measured. Velocities were measured at 0.6 times the flow depth. Measurements for the cross vanes were similar. For the w-weirs, measurements were taken as shown in Figure B-5.

RESULTS

The summary of the data collected from the experiments on the vanes, cross vanes, and w-weirs are given in Table B-2. A total of 75 experiments were conducted.

Vanes

A set of runs were conducted with no vane in place to assess the scour depth caused by the abutment itself (Runs 1-6 in Table B-2). These runs provide the base data from which to compare the effects of various vane configurations on scour depth at the abutment. For these cases, a scour hole formed toward the upstream end of the abutment directly against the abutment. Placing the vanes upstream of the abutment caused a reduction in velocity along the channel bank and effectively moved the abutment scour further out into the channel away from the abutment. It should be noted that the maximum scour depth did not change and in many cases increased with the vanes in place; however, the maximum scour was moved laterally away from the abutment into the center area of the bridge opening (see Table B-2). For all runs in which a vane was placed upstream of the abutment (Runs 7-37 in Table B-2), a scour hole was observed at the upstream tip of the vane and immediately downstream of the vane. At lower flows, the scour holes created by the abutment and the vane remained separate. At higher flows, the two scour holes formed by the vane and the abutment joined to form one elongated scour hole toward the center of the channel (see Figure B-6). At the highest flows, (e.g., Runs 10 and 11), a scour hole also developed just upstream of the vane due to secondary currents (as described by Hey, 1995). In addition, a minor secondary scour trough developed at the abutment.

Figure B-7 shows a plot of the effect of the various angles on scour depth compared to the case in which no vane was used (runs 7-21). The scour depth in Figure B-7 is measured at the position of maximum scour depth at the abutment for the no vane case so that the reduction in scour through the use of a vane could be clearly shown. As an example, for a flow depth of 22 cm, the maximum scour depth of 3.5 cm occurred at 12.7 cm laterally away from the abutment. Thus, for all vane angles tested at a flow depth of 22 cm, the scour depth was measured at 12.7 cm out from the abutment. Table B-2 shows the percent scour reduction for each flow depth using the vane angles of 20, 25, and 30° (Runs 7-21). Scour at the abutment is reduced by 62, 58, and 90 for the highest flow at angles of 20, 25, and 30°, respectively. From Figure B-7, it is shown that the optimum angle of the vanes from the bank is approximately 30°, particularly at high flows. This confirmed field observations and studies conducted by Richard Hey at Wallingford which indicate that an identical vane angle (about 30°) provides maximum bank protection in meander bends.

In Figure B-8, the effect of varying the placement of the vane upstream of the abutment on scour depth is shown for a constant angle of 25° (Runs 14-16 and 22-27). The vane placement was measured from the upstream end of the abutment to the upstream tip of the vane as shown in Figure B-1. The 0.94 m and 1.25 m placements were chosen based on the 2-D computational experiments described previously. The 0.41 m placement was such that the vane was just upstream of the abutment. For the highest flow scour is reduced by 22, 58, and 80% for vane placements of 0.41, 0.94, and 1.25 m, respectively. The data and figures show that the 1.25 m position consistently provides the greatest reduction in scour depth adjacent to the abutment for all flows. This is the approximate distance from where the flow detaches from the bank at the vane to where the flow lines reattach to the bank downstream. The vane placed just upstream of the bridge abutment did a significantly poorer job in terms of reducing scour than when the vane was placed further upstream. There are two reasons for this. First, at during an overbank flow, the shear stresses are highest at the upstream corner of the abutment due to the obstruction to the flow and the overbank flow returning to the channel to pass under the bridge. This shearing stress is the primary cause of erosion at an abutment. A vane located in the zone where the shear stresses and flow accelerations are already high may actually cause additional acceleration of flow over the vane.

Figure B-9 (Runs 28-32) shows the effect of the number of vanes for a constant angle of 25°. The use of two vanes rather than one provides somewhat greater hydraulic control in that the scour position is moved further away from the abutment. It appeared that the addition of a second vane upstream started deflecting the flow further upstream so that the downstream vane became more effective in realigning the flow. The first vane was placed such that the distance from the downstream end of the abutment to the upstream tip of the vane was 0.94 m. The second vane upstream was placed with its tip 0.94 m upstream from the downstream end of the first vane (see Figure B-10). The spacing experiments were repeated using the 1.25 m spacing. The 1.25 m placement using two vanes provided optimum detachment area from the banks (see Figure B-9) with a 95% reduction in scour for the highest flow as compared to 80% reduction for a single vane placed 1.25 m upstream.

The height of the vane at the bank was varied from bankfull to ±1 cm (corresponding to ±18 cm or 0.6 ft in the field) for one high flow (Runs 33-34). This variation in height was chosen based on recommendations by Hey (1995) that additional control can be gained by raising the height of the vane slightly (about ½ foot) above bankfull elevation. The results showed that raising the vane slightly higher than bankfull provides additional flow control which yields a lower scour depth and larger distance between the maximum scour depth and the abutment. This is shown in Figures B-11. Scour at the abutment is reduced by 67%. For the same case (i.e., 25° angle, 1.25 m placement, and a single vane) but with the vane at bankfull height (run 27), scour had been reduced by 58% for the same flow depth. Thus, raising the vane slightly above bankfull provided about 9% additional scour reduction.

Three additional experiments were run to briefly examine the changes in parameters for a skewed abutment with respect to the flow lines in the river (Runs 35-37). With the abutment skewed at 35° to the stream channel, measured according to HEC-18 (see Figure B-12), a single vane was angled at 25, 35, and 45° to the channel bank and the resulting scour depth and position measured for a single flow depth and rate. Although the angle of the vane made very little difference with respect to the resulting scour depth, the distance of the maximum scour hole depth from the abutment changed markedly, about 10 cm further away from the abutment over the 10-degree range of angles. This is most likely because the skewed abutment creates a larger obstruction to flow than one that is perpendicular to it. Figure B-13 shows the change in scour depth at the abutment as a function of the vane angle. The conclusion is that the angle of the vane to the bank must be at least as great as the angle of the skew. For example, for a straight alignment, the vane angle should be 30°, while for a 15° alignment the vane should be angled at 30° + 15° = 45°.

Cross Vanes

The results obtained with the vanes, with regard to the angle and upstream location, were assumed to apply to the cross vane configuration. The bank angle was held at 25° and the placement of the structure upstream of the abutment was held at 1.25 m. Flow depth was varied over a range of four depths (Runs 38-41). The results are shown in Figure B-14. Scour depths recorded in Table B-2 were greater for the cross vane (except at the highest flow) than the vane due to greater concentration of flow. However, this effect may be partially due to the experimental setup. In the experiments with the vanes, they were placed only on the floodplain side of the channel (none were placed on the outside glass). For the cross vanes, the entire structure was placed in the channel, spanning from the floodplain to the glass wall.

Although the scour position is centered in the channel for lower flows, at high flows, the results were similar to the results for the vanes. Thus, it can be concluded that the effect of cross vanes are quite similar to those provided by the partial width vanes and their use should be a function of channel dynamics. For example, if the channel bed is degrading, a series of cross vanes may be used to provide bed erosion control as well as control of flow along the banks. In addition, if there is a need to maintain flow alignment into the bridge opening and prevent erosion on both banks, cross vanes may be the preferred solution.

W-Weirs

Data from the w-weir experiments are given in Table B-2 (Runs 42-76). The w-weir experiments were conducted in two sets. The first set consists of Runs 42-57 while the second set consists of Runs 58-75. In the first set, initial runs with just a pier (no w-weir) in the channel were conducted to provide baseline data (Runs 42-46). The angle of the weir at the bank (25°) and the distance upstream of the pier (2.38 m) were held constant. The interior angle was varied which also had the effect of varying the distance from the pier to the inside vertex of the W. For a 20° interior angle, the inside apex of the w-weir abutted the pier (see Figure B-15). Figure B-16 shows the variation in maximum scour depth at the pier face as a function of the interior angle of the w-weir for a specified flow depth. The w-weir with the 20° interior angle clearly did a superior job of reducing scour at the pier; however, this was likely influenced by the weir structure effectively armoring the upstream face of the pier. Figure B-16 shows the variation of maximum scour at the upstream face of the pier with increasing flow depth and discharge for three cases of no w-weir and a w-weir with a 20° and 50° interior angle.

The w-weirs appear to be very effective at decreasing scour at the pier for high flows. However, for low (bankfull) flows, the 50° w-weir (set back from the pier) significantly increased scour at the pier. This same phenomenon has been observed during other experiments conducted by Richard Hey at Wallingford at bankfull flow. It is possible that the pitch of the w-weir is oversteepened (set at approximately 9.5°) in the simple experimental program which may then be responsible for the sizeable scour hole at the pier. However, the angle from the horizontal (or pitch) of the w-weir arms was not a variable in these experiments, so this hypothesis was not substantiated at this time. From the experimental data available at this time, it would appear that the placement of the w-weir is crucial in order to reduce, rather than increase, scour at bridge piers. When the apex of the w-weir is placed directly against the upstream face of the pier, scour is significantly reduced at the pier for all flows (see Figure B-16); however, this w-weir configuration would be rather difficult to construct in the field. Scour reduction for the w-weir with the interior apex abutting the pier was similar to scour reduction provided by the submerged Iowa vanes discussed previously. However, when the apex is placed upstream of a pier, the combination of the hydraulic conditions set up by the w-weir at lower flows and the obstruction caused by the pier resulted in a tremendously increased scour hole.

The results of the w-weir experiments raised concerns regarding the design of the weir itself and the potential for unacceptable scour resulting from the placement of the weir. Thus, an additional set of 19 experiments (Runs 58-76) were conducted using a w-weir design provided by D. Rosgen (personal communication). First, five experiments were conducted with only a bridge pier to provide a new set of control data (see Runs 58-62). Next, five additional experiments were conducted with no pier and only a w-weir (Runs 63-67). The purpose of these experiments was to determine the pattern of scour and deposition downstream of the weir so that the pier could be placed in the deposition zone following these experiments. Rosgen's design estimated that the pier should be approximately 0.5 to 0.6 times the channel width downstream from the central apex of the w-weir. The remainder of the runs (Runs 68-76) were conducted to test various configurations of w-weirs at a bridge pier.

As a result of Runs 63-67, the pier was placed approximately 0.5 meters (1.5 feet) or 0.3 channel widths downstream from the weir. This is about ½ the distance recommended by Rosgen; however, the depositional area in the laboratory was consistently less than 0.5 channel widths. Figure B-17 shows the change in scour depth at the pier with and without the w-weir upstream of the pier. Scour depths in Table B-2 are reported with respect to the original bed level (as opposed to the height of the depositional mound). For the 40° w-weir, the problem of increased pier scour depth at bankfull discharge was eliminated. A greater scour depth occurred for the 22-cm flow depth; however, it was well below that which occurred without the w-weir in place. It appears from Figure B-17 that the w-weirs become increasingly effective with increasing flow depth. The scour hole upstream of the pier was quite deep with respect to the top of the deposition mound for the higher depths; however, relative to the original bed, scouring was minimal. Figure B-18 shows the general deposition pattern around the weir and pier. Figure B-17 also compares the 30° and 40° w-weirs and their effect on scour for a limited number of flow depths. The scour depths obtained for the 30° w-weir were clearly well in excess of those using a 40° w-weir. In fact, the scour depth increased for both the 22 cm and 29 cm runs tested. Figure B-19 shows the effect of varying the height of the central apex from bankfull to 3/4 bankfull to bankfull elevation for a constant flow depth of 22 cm. For a height of ½ the bankfull height (7.5 cm), a single longitudinal ridge of deposition was observed downstream from the central apex of the w-weir. For heights of 11.25 cm and 15 cm, a double ridge was observed which was significantly wider than the single ridge. This larger area of deposition resulted in a lower scour depth, as shown in Figure B-19. Scour depths for the 22 cm flow depth were decreased by 7.7, 75, and 91%, respectively for the ½ bankfull, 3/4 bankfull, and bankfull apex elevations, respectively.

An additional observation was made regarding the flow exiting the w-weirs. In every case, regardless of whether the flow approached the w-weir in a uniform manner, the w-weir was effective in creating uniform flow across the cross section downstream. In a number of the experiments, it was difficult to obtain uniform flow in the transverse direction in the flow exiting the flume headbox; in other words, the velocity was higher on one side of the channel than another. After crossing the w-weir, the flow velocity was uniform across the cross section.

Energy Losses

Energy losses across the vanes were measured by subtracting the energy downstream of the vane from the energy upstream of the vane, where energy is the sum of the flow depth and velocity head. Losses were measured only at the structure itself such that the losses due to flow across the remainder of the bed were ignored. When the vane set at 30 degrees was just submerged, head loss was 0.1 cm over the 73.66 cm length of the vane. This corresponds to a friction slope of $S_f = 0.0014$, which compares favorably to the theoretical results. An effective Manning's coefficient can be calculated for the roughness effect of the vane itself based on the flow conditions for which energy loss was measured, such that n = 0.048. Scaling this value of n by the length ratio, L_r, according to $L_r^{1/6}$ yields a field value of n = 0.078. (This value is a reasonable approximation; however, since the kinematic similarity was skewed in order to maintain a constant V/V_c, the value is only approximate.) This compares very favorably with the theoretical results provided in the previous section. The overall cross sectional roughness is a function of the vane and other frictional losses in the cross section. A weighted roughness coefficient can be computed with n = 0.078 for the portion of the cross section containing the vane or cross vane. At much higher flows, the effect of the vane on the overall flow resistance will be reduced. When the losses are averaged in over the reach that is represented by the cross section, at overbank flows, the effect on the overall resistance, and thus energy loss, will be negligible.

Table B-1. Model dimensions.

Variable	Abutment Model (1:18)	Pier Model (1:9)
Channel width (cm)	76	152
Bankfull depth (cm)	10	15
Sediment size (mm)	1	1
Floodplain width (cm)	76	-----
Critical velocity (cm/s)	42	42
Abutment setback (cm)	25	-----
Pier diameter (cm)	-----	8

Table B-2. Experimental data for vanes, cross vanes, and w-weirs.

Run Number (1)	Structure (2)	Height (3)	Angle (4)	Number of Structures (5)	Distance* (m) (6)	Discharge (m^3/s) (7)	q_o/q_m* (8)	Mean Velocity (cm/s)* (9)	Flow Depth (cm) (10)	Froude Number (11)	Max Scour Depth (cm) (12)	Position of Max Scour* (cm) (13)	Scour Depth at abutment or pier (cm) (14)	% reduction in scour at abutment (15)
1	none	------	------	------	------	0.017	0.000	22.9	9	0.24	0	0	0	
2	none	------	------	------	------	0.030	0.023	25.0	15	0.21	1.4	0	1.4	
3	none	------	------	------	------	0.037	0.067	31.2	19	0.23	1.4	30.5	1.4	
4	none	------	------	------	------	0.053	0.120	31.4	22	0.21	3.5	12.7	3.5	
5	none	------	------	------	------	0.057	0.147	30.2	25	0.19	6.9	10.2	6.9	
6	none	------	------	------	------	0.064	0.194	33.7	28	0.20	8.6	10.2	8.6	
7	vane	bankfull	20	1	0.94	0.021	0.033	23.9	15	0.20	0.9	45.7	2.0	-42.9
8	vane	bankfull	20	1	0.94	0.044	0.125	27.5	19	0.20	2.3	15.2	0.9	35.7
9	vane	bankfull	20	1	0.94	0.057	0.140	31.9	22	0.22	5.3	15.2	1.1	68.6
10	vane	bankfull	20	1	0.94	0.068	0.174	27.9	25	0.18	5.8	17.8	3	56.5
11	vane	bankfull	20	1	0.94	0.084	0.189	33.9	28	0.20	6.8	25.4	3.3	61.6
12	vane	bankfull	25	1	0.94	0.024	0.026	23.8	15	0.20	0	0	0.5	64.3
13	vane	bankfull	25	1	0.94	0.035	0.110	23.9	19	0.18	1.8	45.7	0.4	71.4
14	vane	bankfull	25	1	0.94	0.057	0.137	29.8	22	0.20	4.5	24.8	1.9	45.7
15	vane	bankfull	25	1	0.94	0.067	0.147	32.9	25	0.21	5.6	25.4	2.7	60.9
16	vane	bankfull	25	1	0.94	0.070	0.223	32.2	28	0.19	6.5	25.4	3.6	58.1
17	vane	bankfull	30	1	0.94	0.026	0.011	22.3	15	0.18	1.1	45.7	0.5	64.3
18	vane	bankfull	30	1	0.94	0.035	0.082	22.8	19	0.17	1	45.7	0.5	64.3
19	vane	bankfull	30	1	0.94	0.057	0.128	32.1	22	0.22	4.9	27.9	0.8	77.1
20	vane	bankfull	30	1	0.94	0.059	0.151	30.8	25	0.20	5.2	25.4	1.3	81.2
21	vane	bankfull	30	1	0.94	0.067	0.181	33.6	28	0.20	5.3	24.1	0.9	89.5
22	vane	bankfull	25	1	0.41	0.047	0.114	28.8	22	0.20	3.6	15.2	1.1	68.6
23	vane	bankfull	25	1	0.41	0.062	0.164	32.6	25	0.21	6.5	20.3	5.2	24.6
24	vane	bankfull	25	1	0.41	0.081	0.209	33.3	28	0.20	8.4	20.8	6.7	22.1
25	vane	bankfull	25	1	1.25	0.047	0.102	27.8	22	0.19	0	0	0.2	94.3
26	vane	bankfull	25	1	1.25	0.061	0.141	31.7	25	0.20	2.9	22.9	0.5	92.8
27	vane	bankfull	25	1	1.25	0.054	0.177	21.3	28	0.13	6.5	15.2	1.7	80.2
28	vane	bankfull	25	2	0.94/0.94	0.049	0.135	30.4	22	0.21	1.2	35.6	0.8	77.1
29	vane	bankfull	25	2	0.94/0.94	0.069	0.167	34.4	25	0.22	5.1	34.4	1.7	75.4

Run Number (1)	Structure (2)	Height (3)	Angle (4)	Number of Structures (5)	Distance* (m) (6)	Discharge (m^3/s) (7)	q_o/q_m* (8)	Mean Velocity (cm/s)* (9)	Flow Depth (cm) (10)	Froude Number (11)	Max Scour Depth (cm) (12)	Position of Max Scour* (cm) (13)	Scour Depth at abutment or pier (cm) (14)	% reduction in scour at abutment (15)
30	vane	bankfull	25	2	0.94/0.94	0.079	0.180	33.0	28	0.20	7.4	38.1	2.5	70.9
31	vane	bankfull	25	2	1.25/1.25	0.070	0.159	32.1	28	0.19	4.6	27.9	0.4	95.3
32	vane	bankfull	25	3	1.25*3	0.072	0.23	27.6	28	0.17	5.3	25.4	0.8	90.7
33	vane	- 1 cm	25	1	1.25	0.07	0.251	29.9	28	0.18	7.5	15.2	2.8	67.4
34	vane	+ 1 cm	25	1	1.25	0.07	0.255	28.3	28	0.17	4.6	25.4	0.3	96.5
35	vane	bankfull	25	1	1.25	0.075	0.234	28.7	28	0.17	4.8	25.4	4.8	44.2
36	vane	bankfull	35	1	1.25	0.073	0.251	28.9	28	0.17	4.9	30.5	4.9	43.0
37	vane	bankfull	45	1	1.25	0.072	0.316	28.7	28	0.17	4.5	35.6	4.5	47.7
38	crossvane	bankfull	25	1	1.25	0.031	0.058	22.2	19	0.16	0.0	0	0	100.0
39	crossvane	bankfull	25	1	1.25	0.041	0.089	27.2	22	0.19	2.0	35.6	2.0	42.9
40	crossvane	bankfull	25	1	1.25	0.061	0.159	30.2	25	0.19	5.8	15.2	5.8	15.9
41	crossvane	bankfull	25	1	1.25	0.072	0.171	29.5	28	0.18	5.9	15.2	5.9	31.4
42	none	------	------	------	------	0.920	N/A	24.7	15	0.20	9.7	at pier	N/A	
43	none	------	------	------	------	0.130	N/A	31.1	22	0.21	10.2	at pier	N/A	
44	none	------	------	------	------	0.180	N/A	32.5	29	0.19	10.8	at pier	N/A	
45	none	------	------	------	------	0.208	N/A	31.0	36	0.16	11.5	at pier	N/A	
46	none	------	------	------	------	0.250	N/A	33.1	43	0.16	11.8	at pier	N/A	
47	w-weir	bankfull	50	1	2.38	0.091	N/A	33.0	15	0.27	12.1	at pier	N/A	-24.7
48	w-weir	bankfull	50	1	2.38	0.176	N/A	36.2	22	0.25	6.8	at pier	N/A	33.3
49	w-weir	bankfull	50	1	2.38	0.229	N/A	43.2	29	0.26	6.4	at pier	N/A	40.7
50	w-weir	bankfull	50	1	2.38	0.232	N/A	30.7	36	0.16	6.8	at pier	N/A	40.9
51	w-weir	bankfull	50	1	2.38	0.231	N/A	29.5	43	0.14	9.6	at pier	N/A	18.6
52	w-weir	+2 cm	50	1	2.38	0.269	N/A	34.1	43	0.17	8.7	at pier	N/A	26.3
53	w-weir	-2 cm	50	1	2.38	0.260	N/A	32.4	43	0.16	9.4	at pier	N/A	20.3
54	w-weir	bankfull	35	1	2.38	0.229	N/A	28.6	43	0.14	9.3	at pier	N/A	21.2
55	w-weir	bankfull	20	1	2.38	0.237	N/A	36.4	43	0.15	6.3	at pier	N/A	46.6
56	w-weir	bankfull	20	1	2.38	0.104	N/A	38.3	15	0.32	4.7	at pier	N/A	51.5
57	w-weir	bankfull	20	1	2.38	0.207	N/A	37.2	29	0.22	6.3	at pier	N/A	1.6
58	none	------	40	1	------	0.067	N/A	29.5	15	0.24	10.1	at pier	N/A	
59	none	------	40	1	------	0.098	N/A	29.3	22	0.20	10.4	at pier	N/A	

Run Number (1)	Structure (2)	Height (3)	Angle (4)	Number of Structures (5)	Distance* (m) (6)	Discharge (m^3/s) (7)	q_o/q_m* (8)	Mean Velocity (cm/s)* (9)	Flow Depth (cm) (10)	Froude Number (11)	Max Scour Depth (cm) (12)	Position of Max Scour* (cm) (13)	Scour Depth at abutment or pier (cm) (14)	% reduction in scour at abutment (15)
60	none	------	40	1	------	0.135	N/A	30.5	29	0.18	10.5	at pier	N/A	
61	none	------	40	1	------	0.178	N/A	31.5	36	0.17	11.1	at pier	N/A	
62	none	------	40	1	------	0.204	N/A	31.2	43	0.15	11.2	at pier	N/A	
63	w-weir	7.5	40	1	------	0.075	N/A	33.0	15	0.27	-----	at pier	N/A	
64	w-weir	7.5	40	1	------	0.104	N/A	30.9	22	0.21	-----	at pier	N/A	
65	w-weir	7.5	40	1	------	0.143	N/A	32.4	29	0.19	-----	at pier	N/A	
66	w-weir	7.5	40	1	------	0.176	N/A	32.0	36	0.17	-----	at pier	N/A	
67	w-weir	7.5	40	1	------	0.203	N/A	31.0	43	0.15	-----	at pier	N/A	
68	w-weir	7.5	40	1	0.46	0.079	N/A	34.7	15	0.29	8.2	at pier	N/A	18.8
69	w-weir	7.5	40	1	0.46	0.105	N/A	31.3	22	0.21	9.6	at pier	N/A	7.7
70	w-weir	7.5	40	1	0.46	0.143	N/A	32.3	29	0.19	9.0	at pier	N/A	14.3
71	w-weir	7.5	40	1	0.46	0.174	N/A	31.8	36	0.17	7.8	at pier	N/A	29.7
72	w-weir	7.5	40	1	0.46	0.199	N/A	30.3	43	0.15	7.6	at pier	N/A	32.1
73	w-weir	11.25	40	1	0.46	0.106	N/A	31.7	22	0.22	2.6	at pier	N/A	75.0
74	w-weir	15	40	1	0.46	0.102	N/A	30.3	22	0.21	0.9	at pier	N/A	91.3
75	w-weir	7.5	30	1	0.46	0.105	N/A	31.4	22	0.21	11.1	at pier	N/A	-6.7
76	w-weir	7.5	30	1	0.46	0.139	N/A	31.4	29	0.19	10.9	at pier	N/A	-3.8

***Notes on Table B-2:**

Column 2. Runs 63-67 were run with w-weir only and no pier

Column 3. For vanes and cross vanes, heights are measured at the channel bank. For w-weirs the height is measured at the central upstream apex.

Column 4. Runs 35-37 for abutment at 35 degree skew; Runs 47-57 angle is inside angle of W; Runs 47-57 bank angle = 25 degrees

Column 6. Distance for vanes measured from vane tip to upstream end of abutment. Distance for w-weirs measured from tip of vane end (upstream vertex) to center of pier

Column 8. q_o = unit discharge in overbanks; q_m = unit discharge in main channel

Column 9. Mean velocity is measured as the approach velocity.

Column 13. Position of maximum scour measured laterally from the abutment

Column 14. Scour depth at constant position is measured at the location of max scour depth at abutment or pier for the corresponding flow depth and no structure

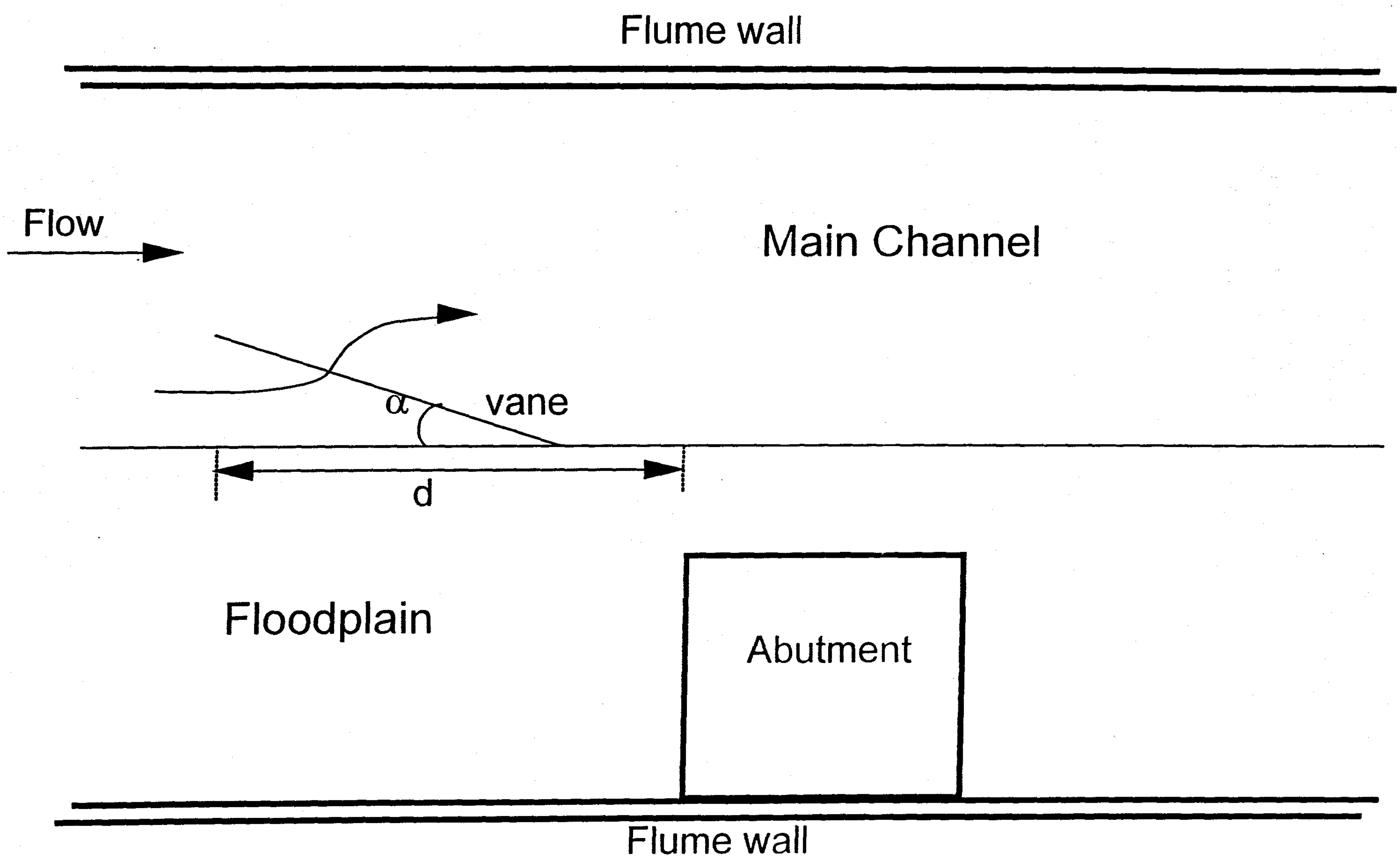


Figure B-1. Placement of vane for laboratory experiments.

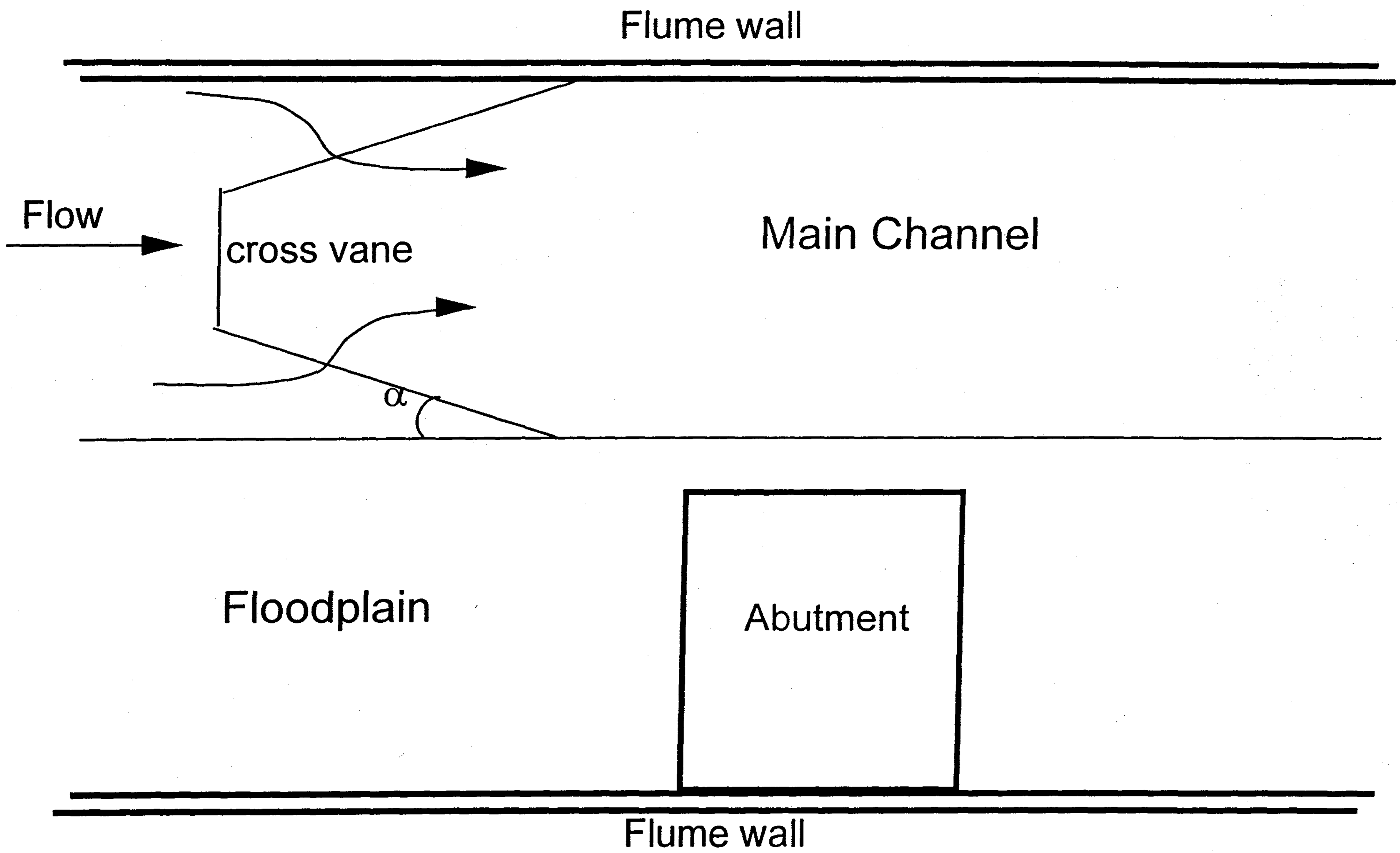


Figure B-2. Layout for cross vane experiments.

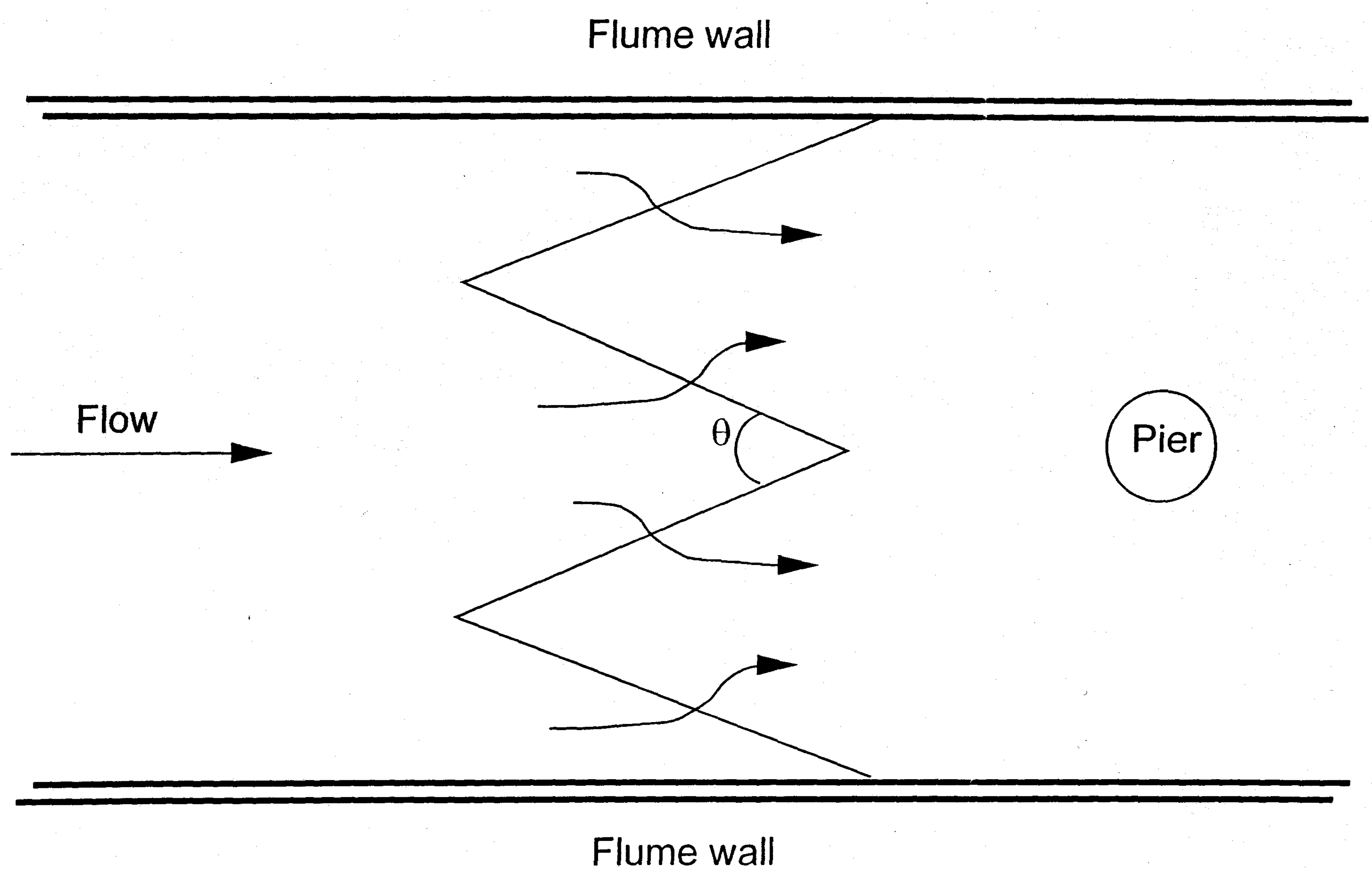


Figure B-3. Layout for w-weir experiments.

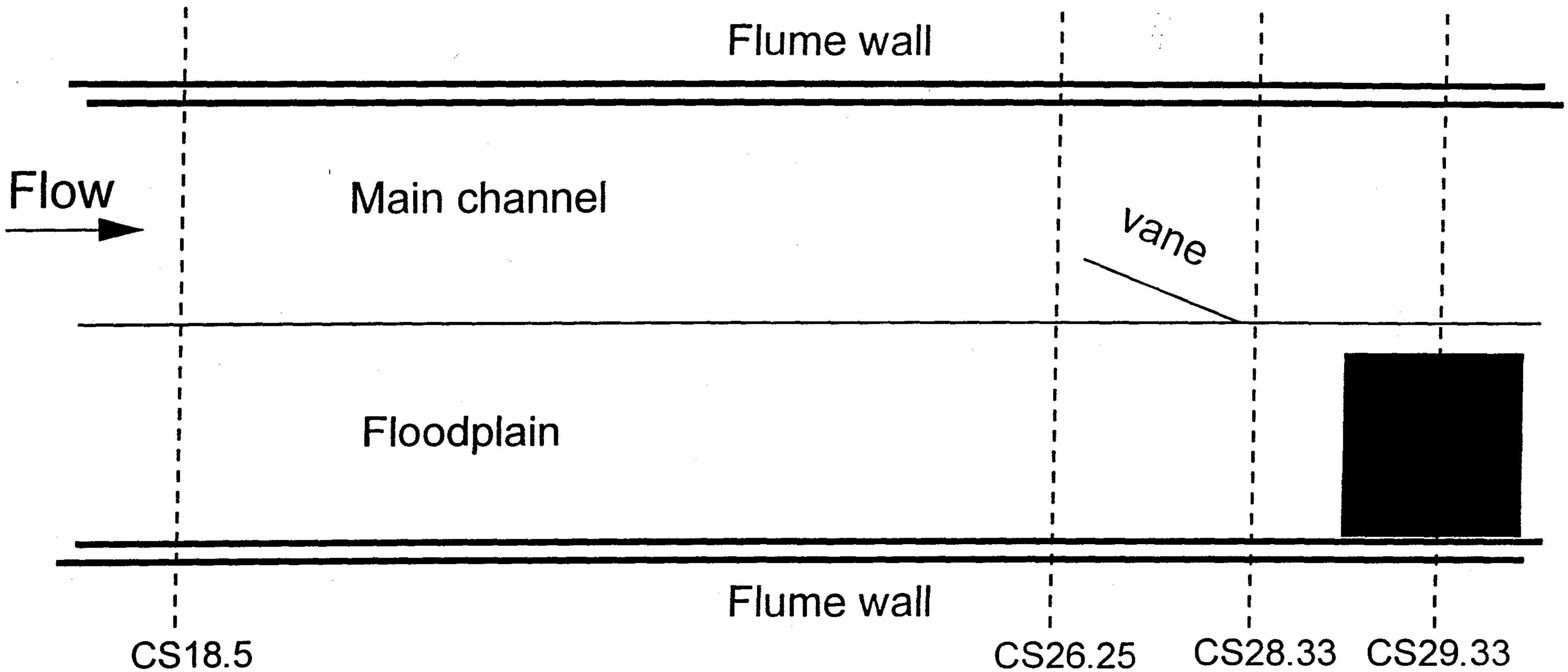


Figure B-4. Measurement layout for vanes. CS# = cross section distance in feet from the inlet.

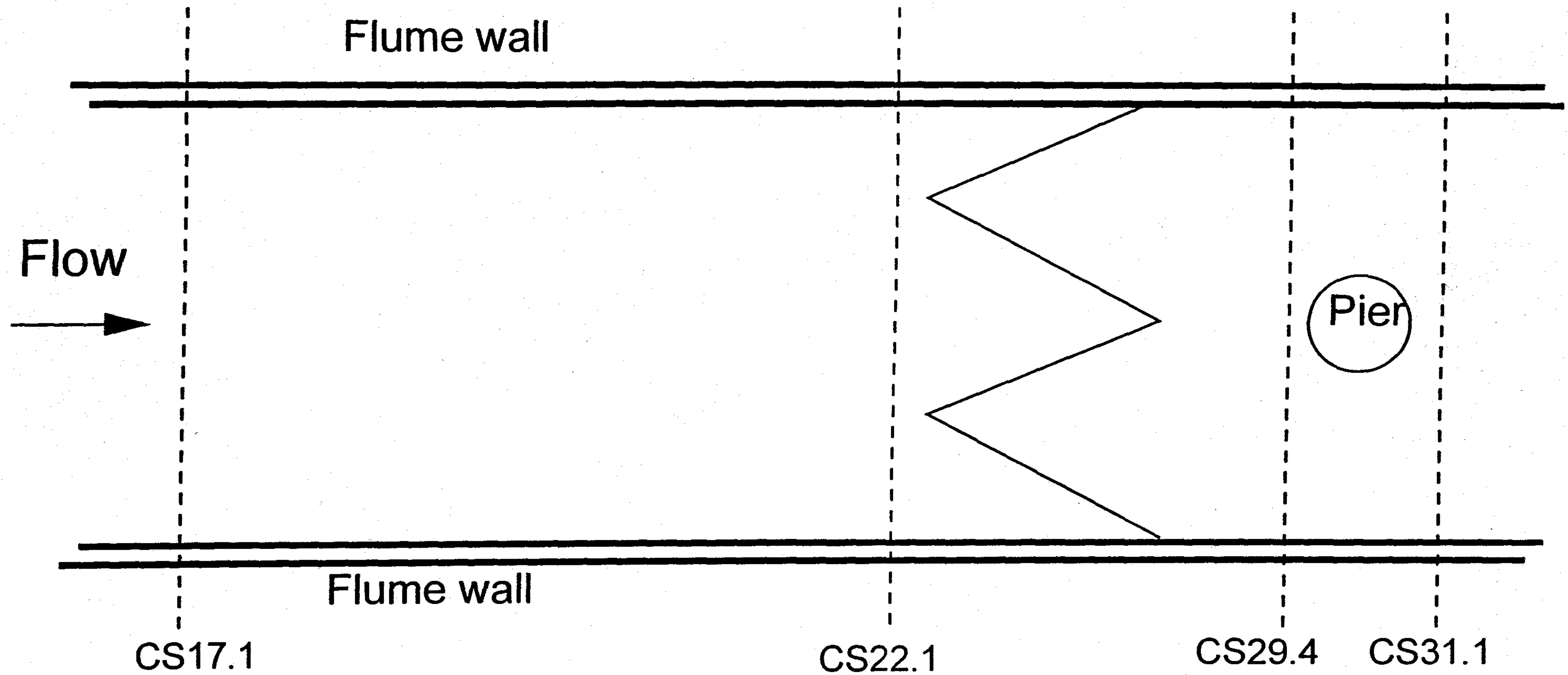


Figure B-5. Measurement layout for w-weirs. CS# = cross section distance in feet from the inlet.

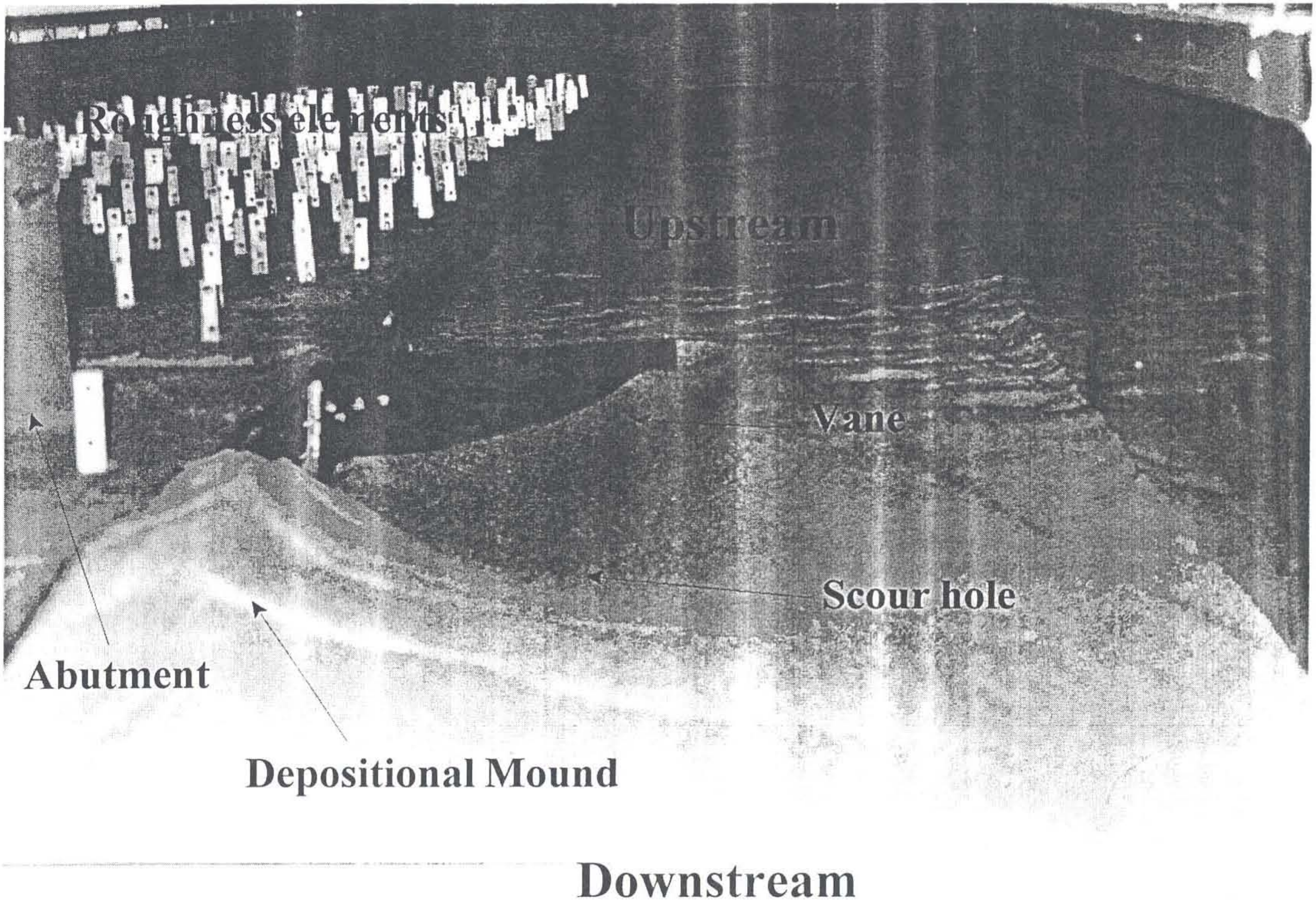


Figure B-6. Scour hole formed by vane and abutment

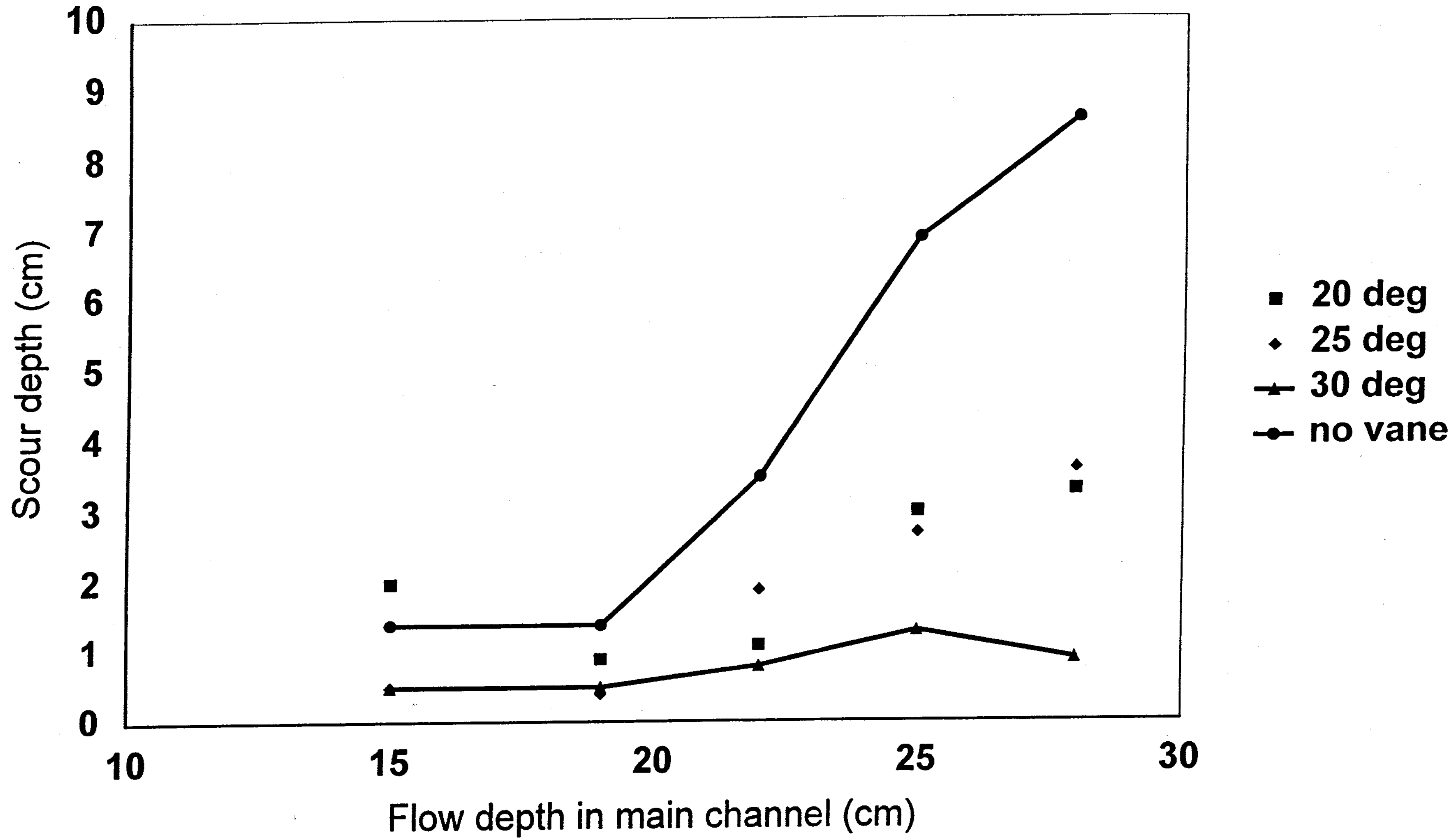


Figure B-7. Effect of vane angle on scour depth (see Table B-2, Runs 7-21).

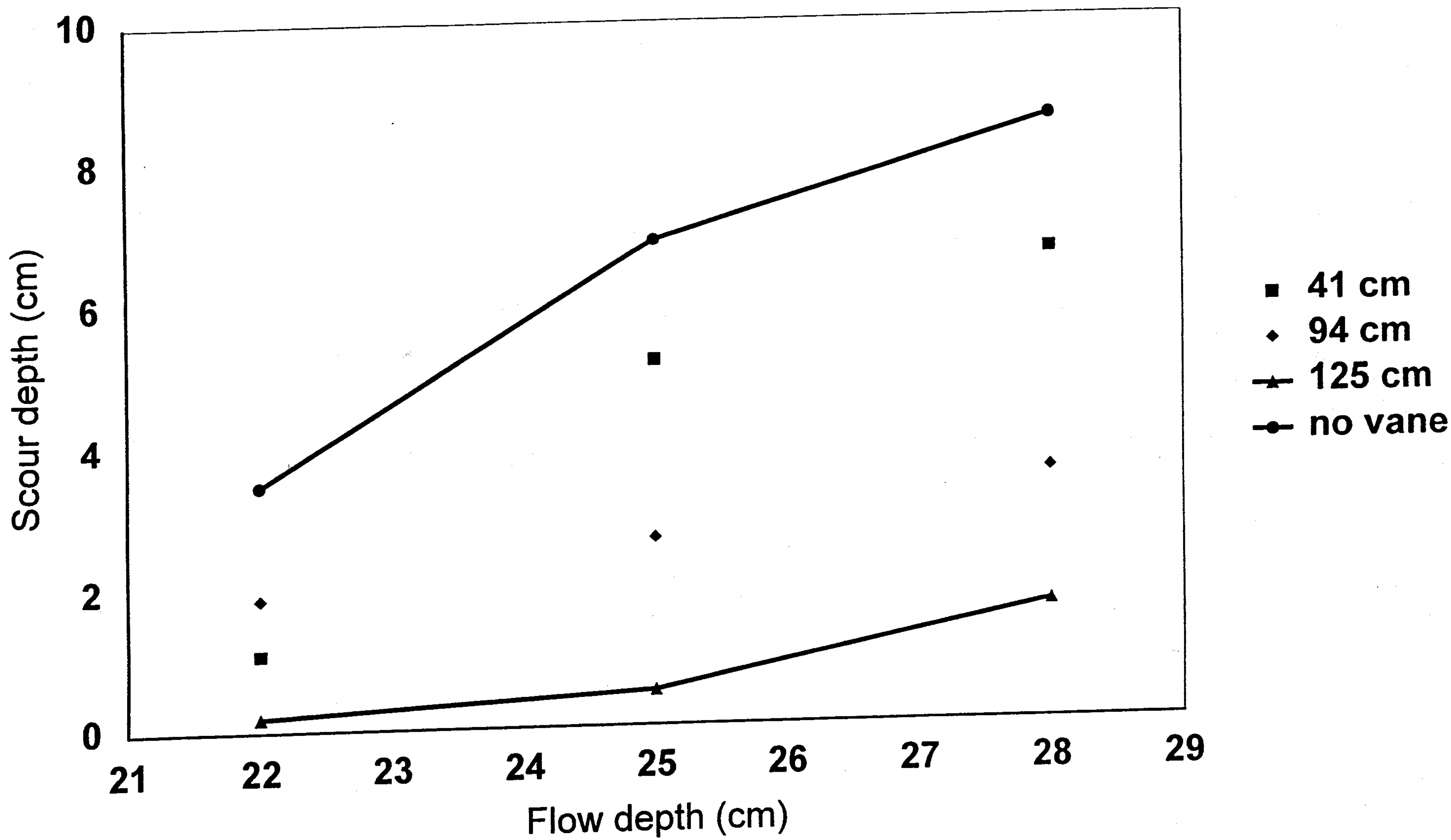


Figure B-8. Effect of placement of vane upstream of abutment on scour depth.

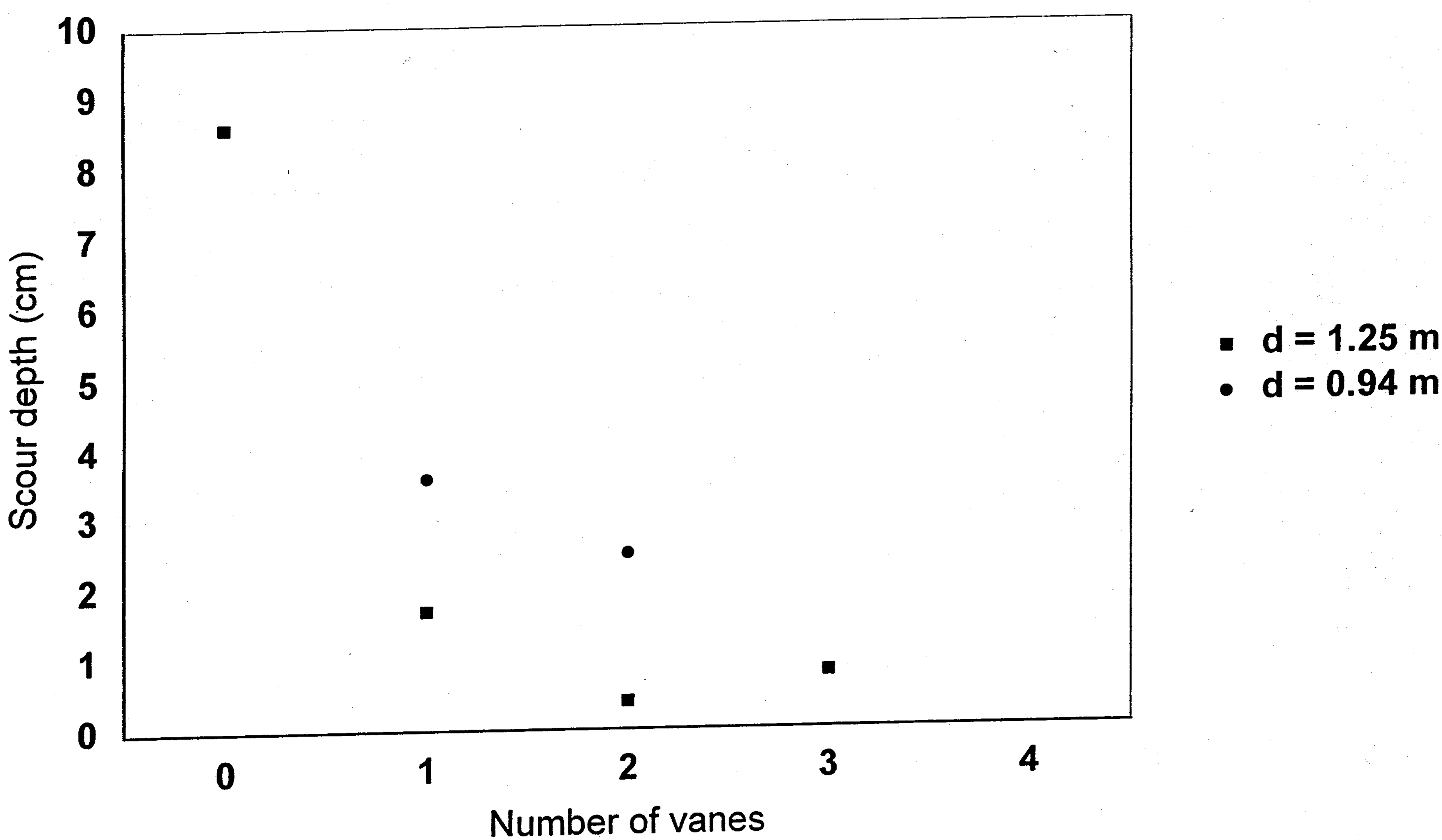


Figure B-9. Effect of number of vanes on scour depth.

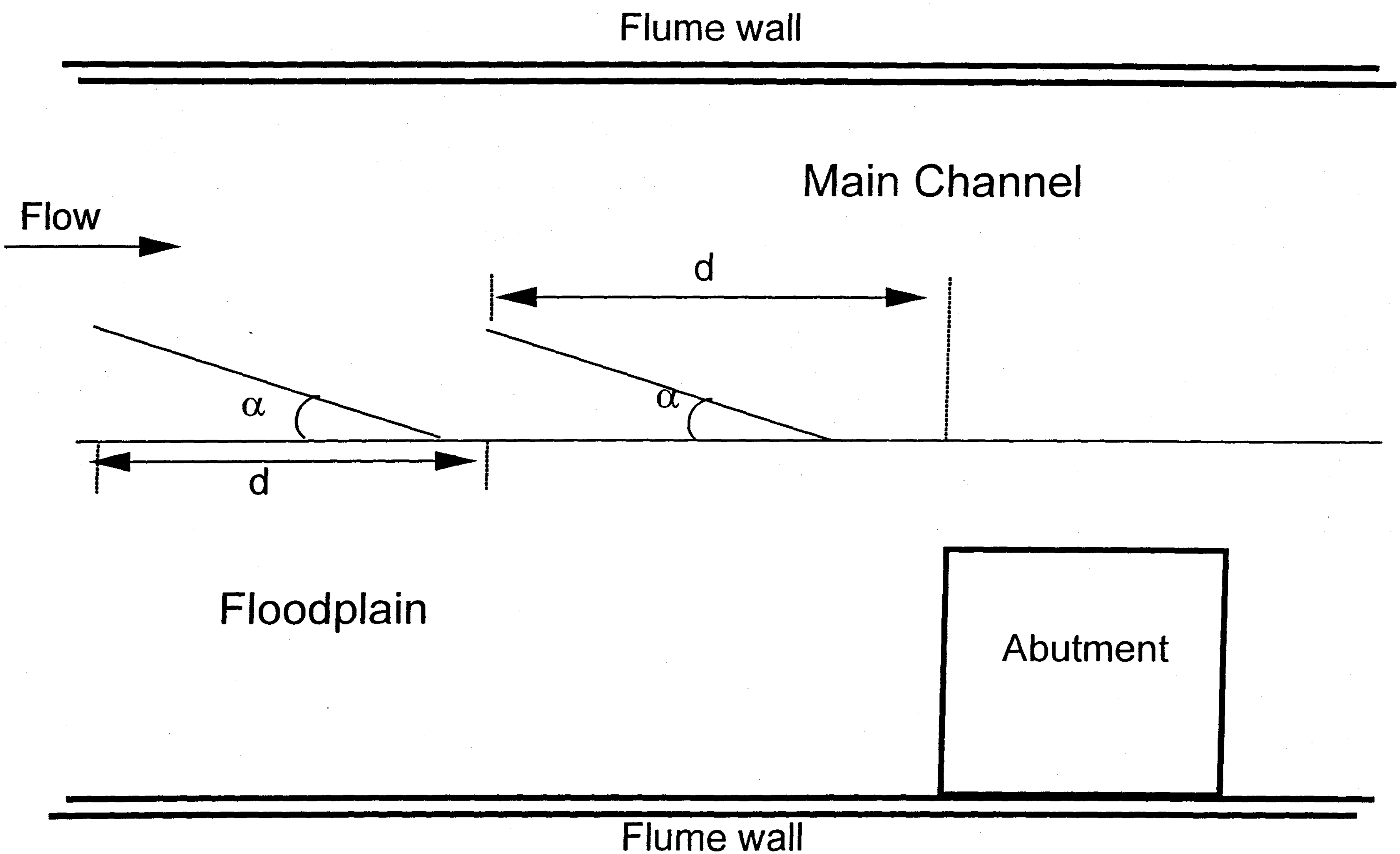


Figure B-10. Placement of multiple vanes.

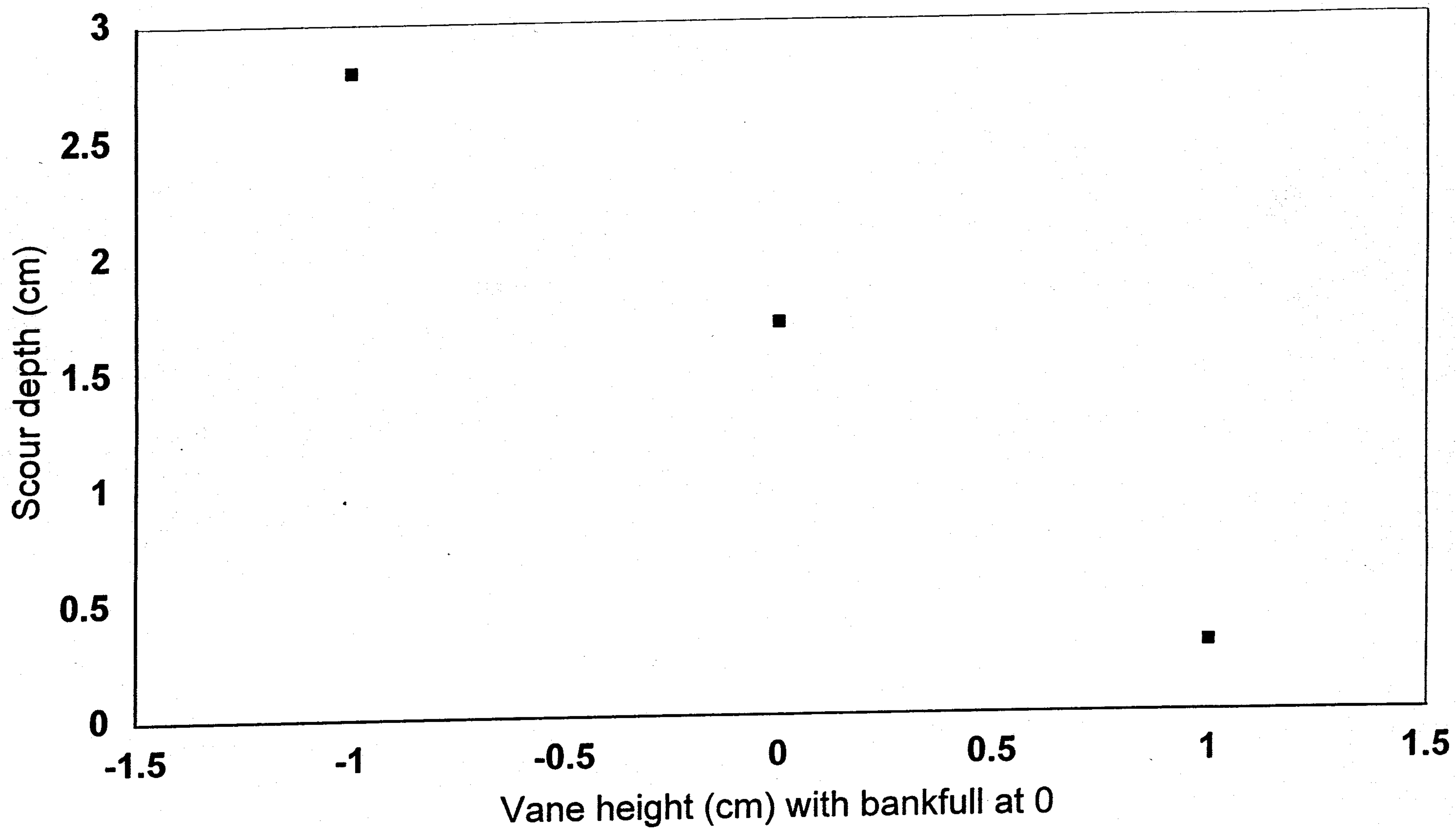
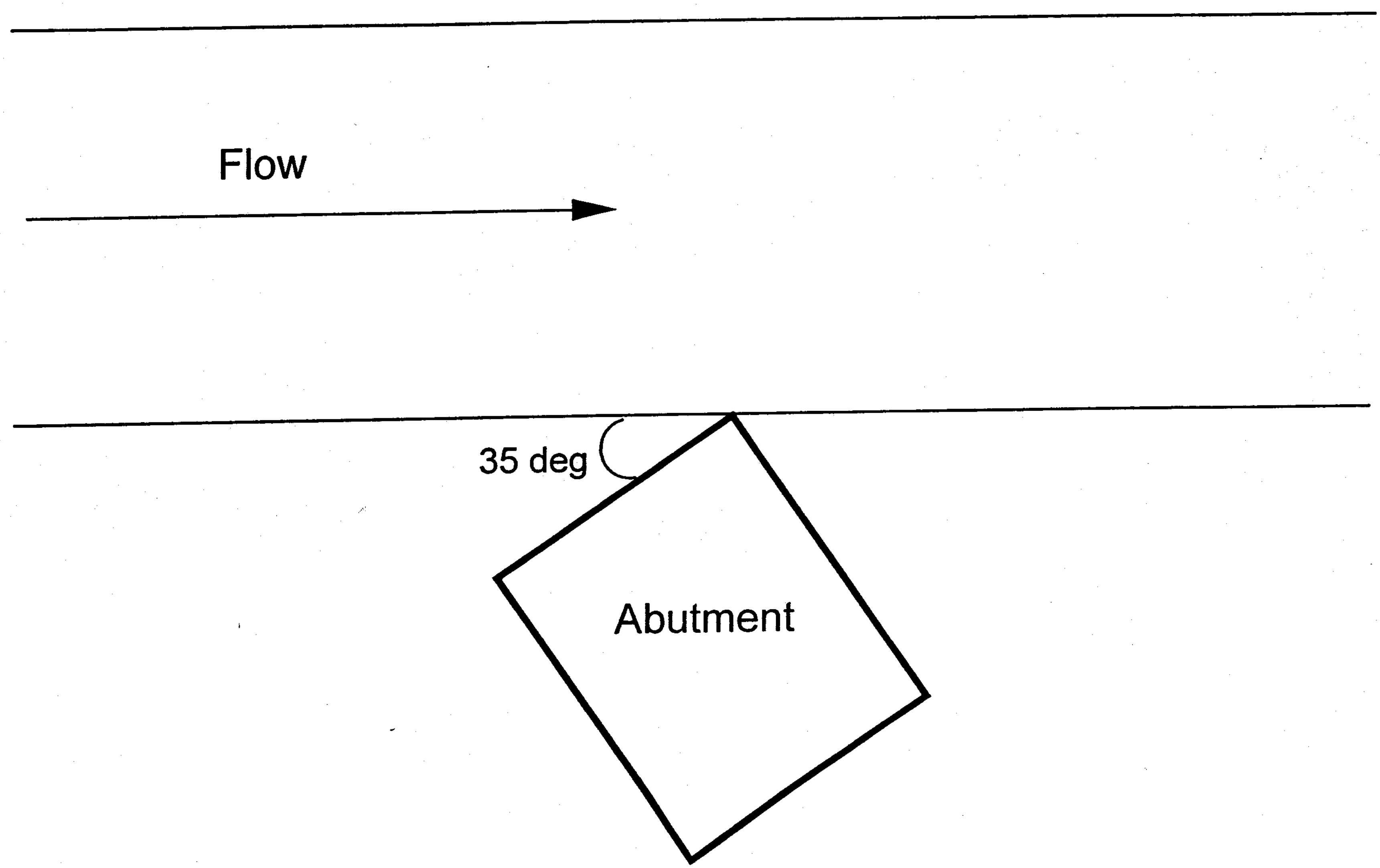


Figure B-11. Effect of height of vanes on scour depth.



B-12. Skewed abutment angle.

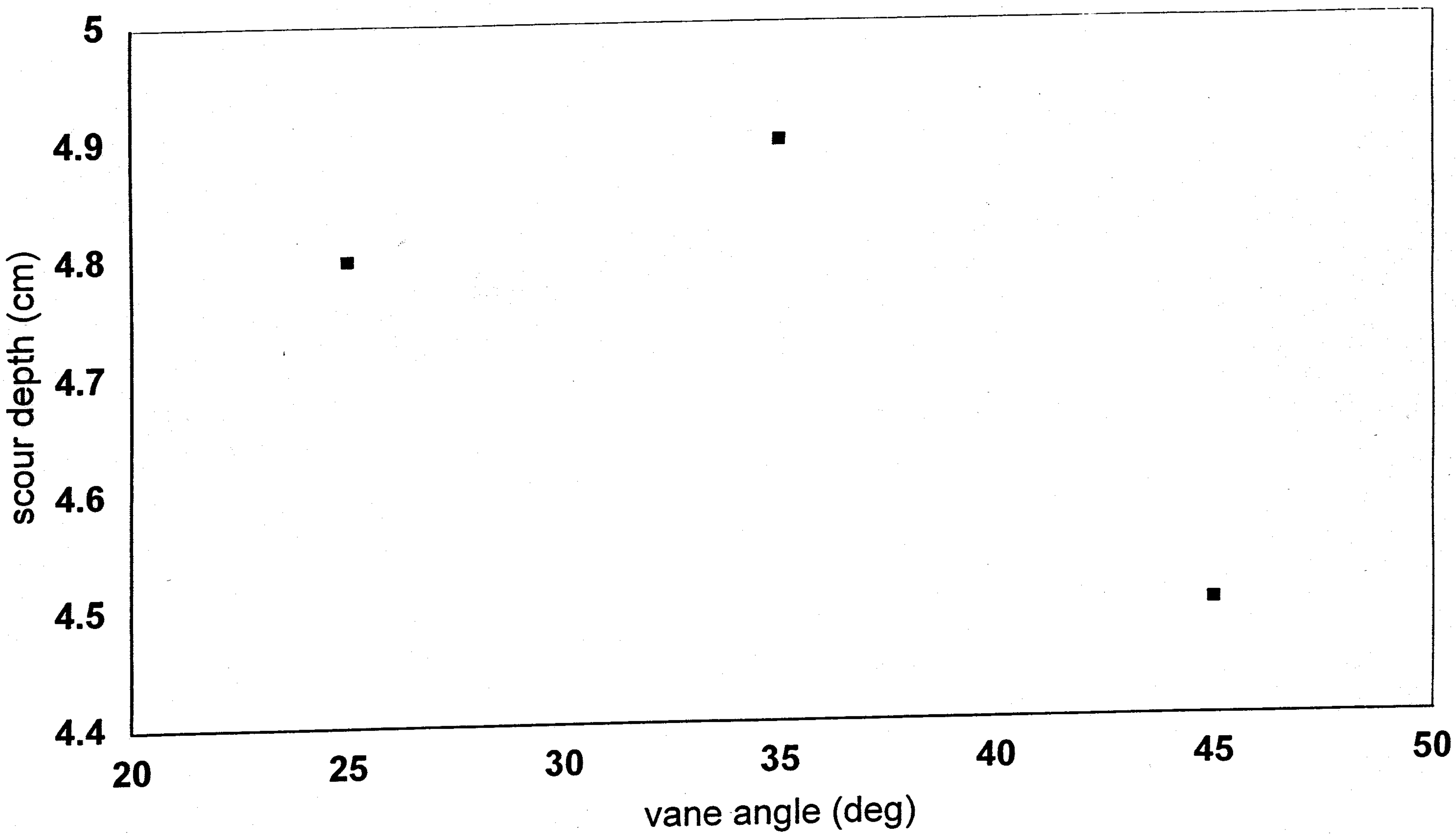


Figure B-13. Effect of vane angle on scour depth at skewed abutment.

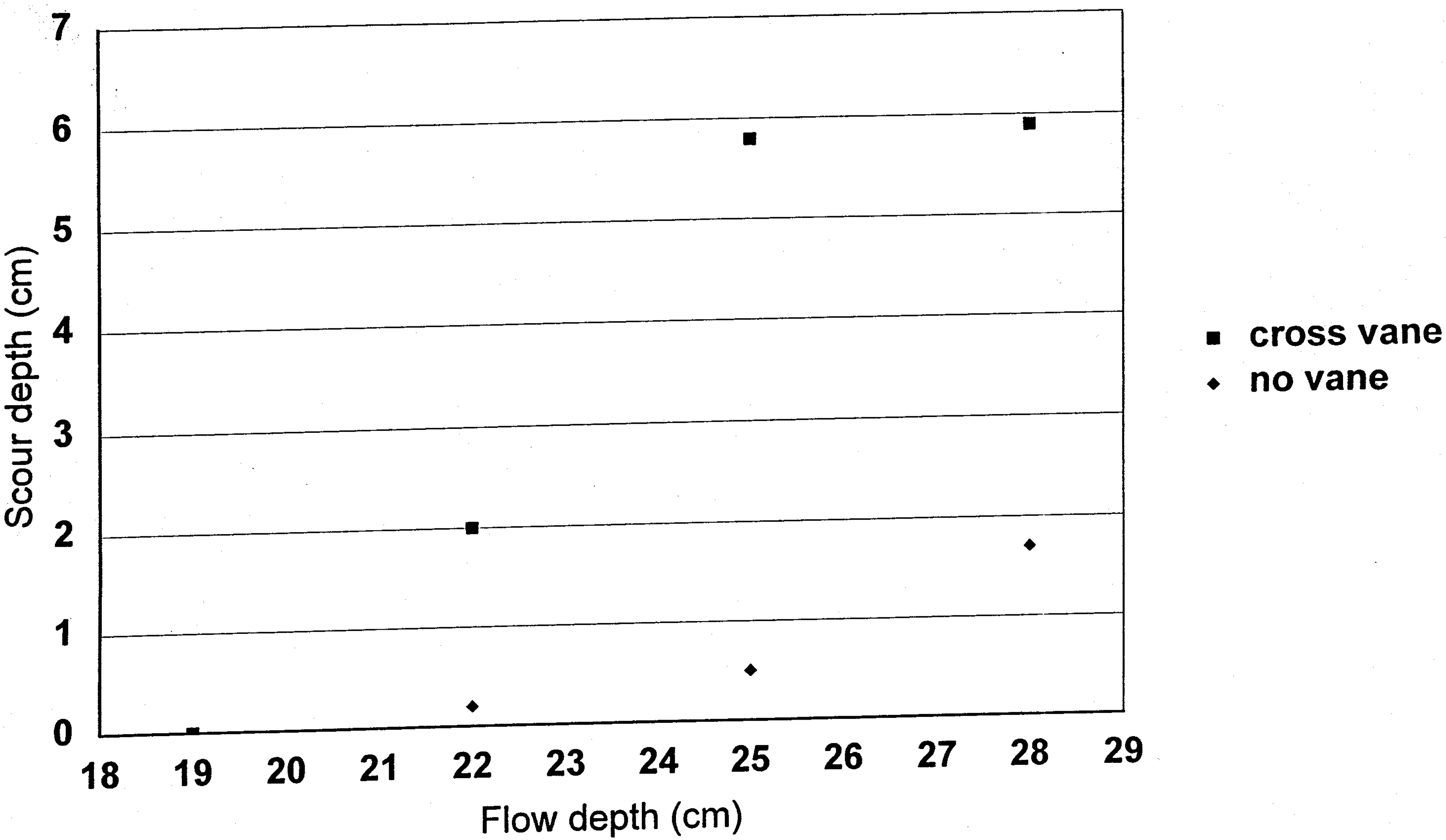


Figure B-14. Effect of cross vanes on scour depth.

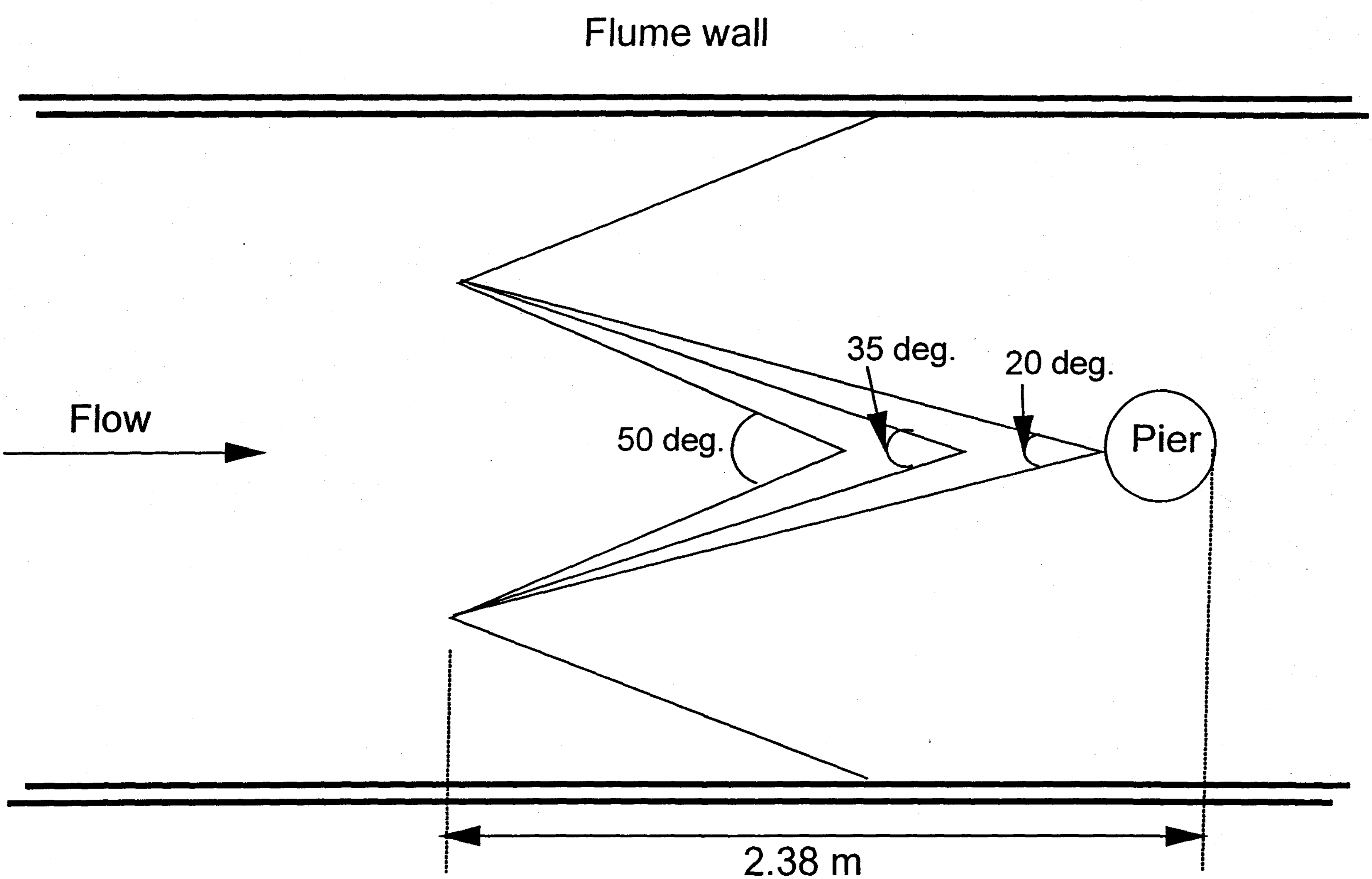


Figure B-15. W-weir configuration for various interior angles.

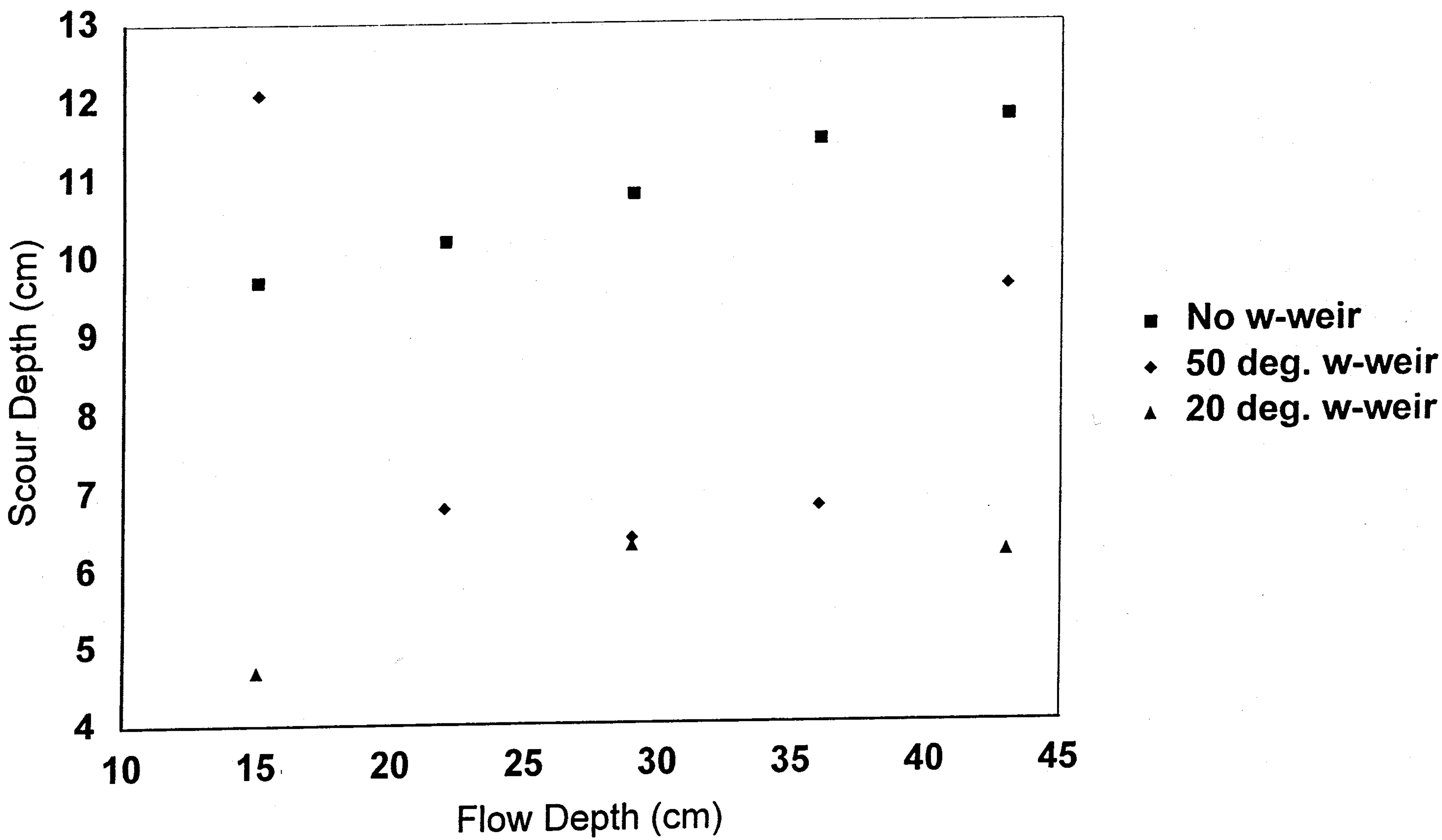


Figure B-16. Scour depth as a function of w-weir central apex angle for 20 and 50 degree interior angle.

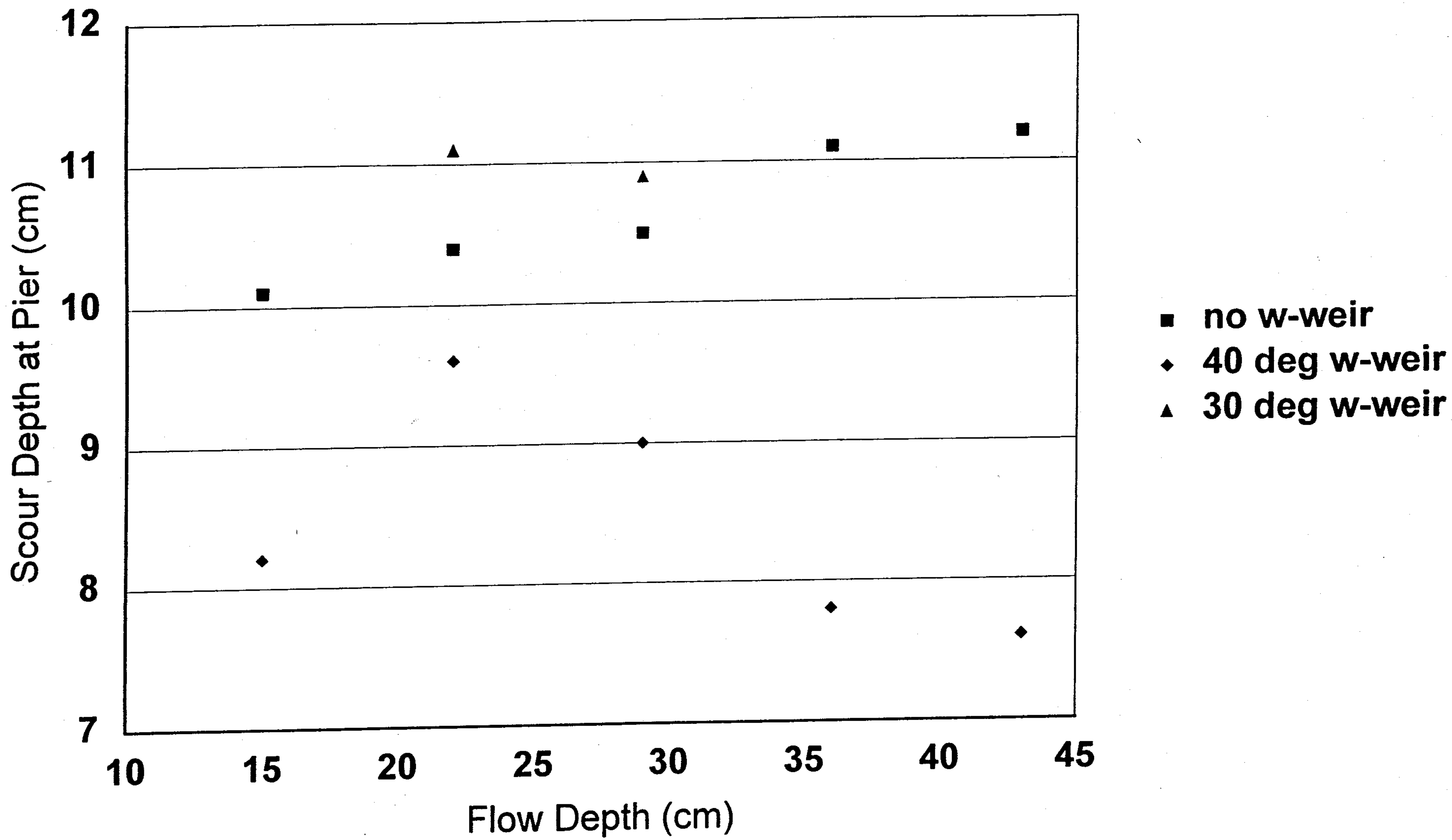


Figure B-17. Scour depth as a function of w-weir central apex angle for 30 and 40-degrees for revised design.

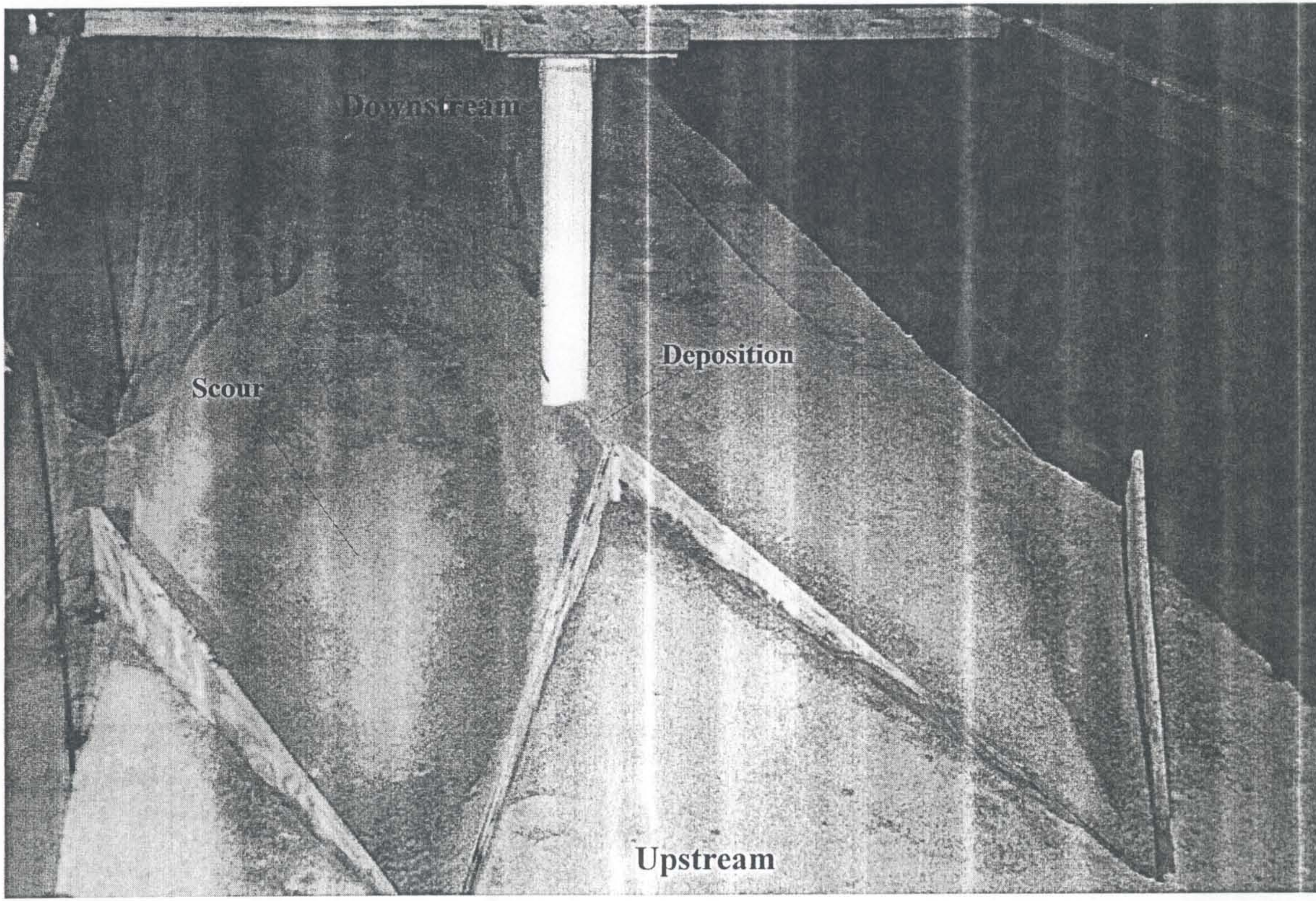


Figure B-18. Deposition and scour pattern produced by pier and w-weir.

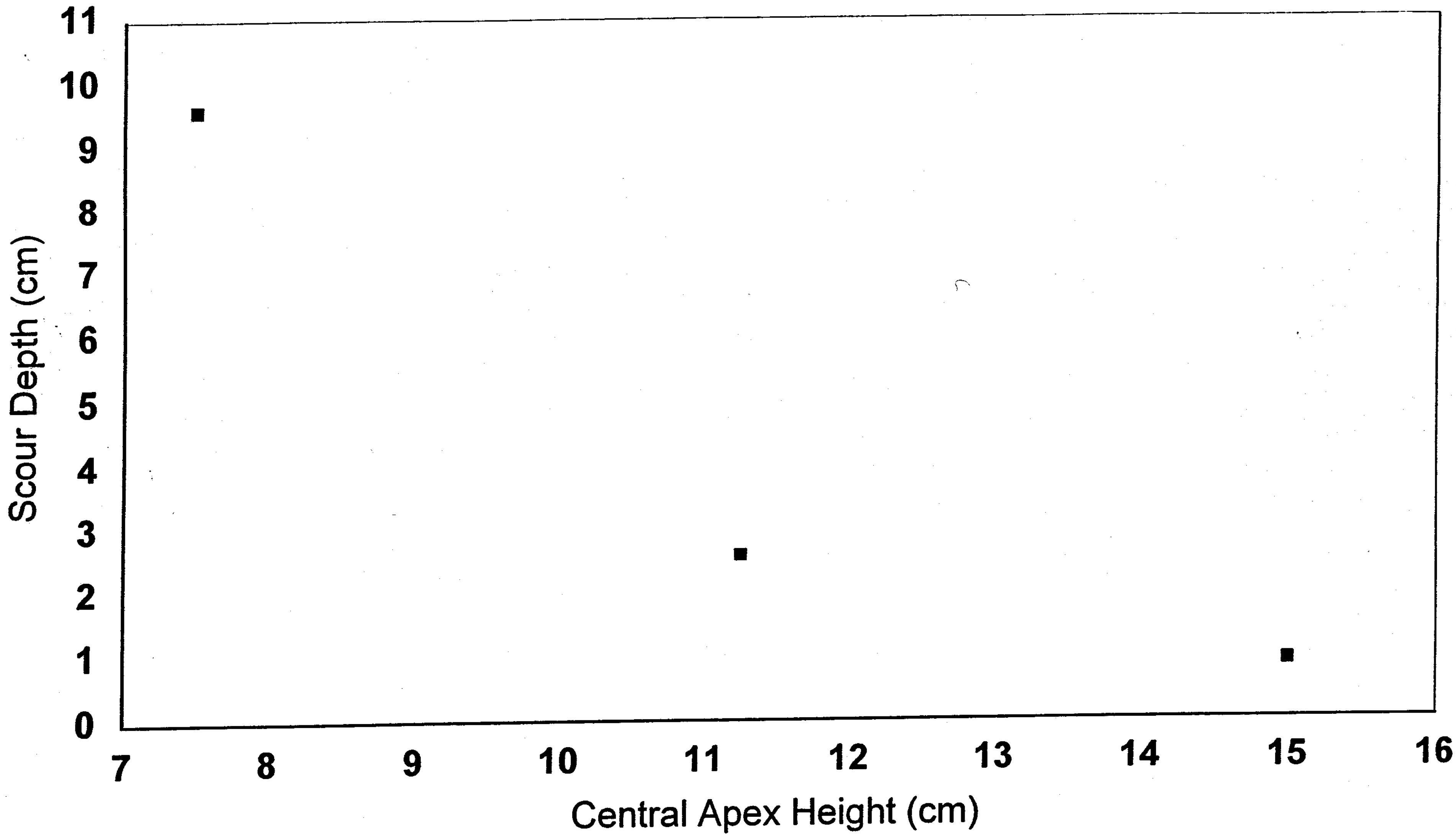


Figure B-19. Scour depth as a function of w-weir apex height.