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'Barbs' - A Laboratory Study**

**Troy Matsuura**

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## Abstract

This thesis reports on phases 1 and 2 (Matsuura and Townsend, 2004a, 2004b, 2004c) of an ongoing laboratory study of a novel form of streambank protection structure. ‘Barbs’ are groin-like structures designed to protect the (usually unstable) outside-bank regions of channel bends by actively redirecting flow away from the outside-bank. These low-profile, rock-fill structures have trapezoidal-shaped cross-sections and point upstream into the direction of flow. Their overall length typically extends one-quarter way across the channel, and are comprised of ‘key’ and ‘weir’ sections. The former, which is set back into the channel bank material, slopes downwards at 1V:1.5H to meet the low-profile weir section that extends into the stream. The crest of the weir section, the ‘key’ end of which should not exceed the channel-forming flow level (typically taken as the 1.5-year flow event), should have a mild positive slope (<1V:5H) towards the ‘stream’ end for proper operation, (USDA 1999). By disrupting near-bank velocity gradients, barbs promote sediment deposition along the eroding outside-bank region. Their presence also modifies the bend’s ‘helical’- type flow pattern such that secondary currents, that would otherwise attack the outside-bank, are redirected towards the center of the channel.

This novel form of bank protection structure is currently undergoing field tests on selected bends on a number of shallow ‘wide’ streams in Illinois, USA (Kinney, 2000, 2002). While initial results are encouraging, additional studies are necessary to develop design criteria for their wider application. In this study, the effectiveness of different arrangements of model barb groups, in both 90-degree and 135-degree moveable-bed bend sections of a hydraulically ‘narrow’ rectangular channel, are investigated. Physical model experiments were performed to determine the impact of several key parameters on model barb performance, and include: channel forming flow depth ( $y_o$ ), maximum height of the barb weir section ( $h$ ), barb alignment angle ( $\theta$ ), distance along the outside-bank from the bend entrance to the first barb ( $L$ ), and distance along the outside-bank from the bend exit to the last barb ( $L'$ ). For each hydraulic condition considered, the channel-bed scour profiles generated by the different barb groups are compared and also to

corresponding 'reference' profiles generated in the absence of barbs. Judging the effectiveness of the different barb groups in promoting long-term stability of the outside-bank region is based on two criteria: (i) percent reduction in scouring (P) achieved in the vicinity of the outside-bank, and (ii) the degree to which the channel thalweg (deepest portion) is moved from the outside-bank region to a new position closer to the center of the channel.

Experimental results indicate that scouring of the channel bed in the vicinity of barbs is less severe if  $h$  is kept small relative to the channel forming flow depth (i.e.  $0.33 \leq h/y_o \leq 0.5$ ) and that barb system performance largely depends on  $\theta$ ,  $L$ , and  $L'$ . Generally,  $\theta=30$ -degrees is recommended as the preferred alignment for barbs in the bend flow field, and larger  $L$  and  $L'$  result in increased  $P$  as well as the greatest shift in the thalweg away from the outside-bank.

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## List of Symbols

A, B, C, D	barb group types
$D_{15}$	grain size for which 15 percent by weight is finer
$D_{15.9}$	grain size for which 15.9 percent by weight is finer
$D_{50}$	grain size for which 50 percent by weight is finer
$D_{84.1}$	grain size for which 84.1 percent by weight is finer
$D_{85}$	grain size for which 85 percent by weight is finer
$D_{100}$	grain size for which 100 percent by weight is finer
$g$	gravitational acceleration
$h$	maximum barb weir height
$L$	distance along outside-bank from bend entrance to first barb
$L'$	distance along outside-bank from bend exit to last barb
$M$	stream meander pattern
$P$	percent reduction in local scouring
$R$	hydraulic radius
$r$	bend radius from center of bend curve
$Re_*$	sediment Reynold's number
$S$	channel gradient
$u$	local longitudinal velocity
$u_*$	shear velocity
$V$	average longitudinal velocity
$W$	channel width
$x$	point of maximum scour within each cross section (without barbs)
$x'$	point of maximum scour within each cross-section (with barbs)
$y_0$	average channel forming flow depth
$y_s$	bed scour depth (without barbs)
$y_s'$	bed scour depth (with barbs)

## **Greek Symbols**

$\theta$	barb alignment angle
$\sigma_g$	geometric standard deviation
$\tau_{cr}$	critical shear stress
$\tau_o$	boundary shear stress
$\nu$	kinematic viscosity

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# Chapter One

## Introduction

### 1.1 Problem Statement

Bank erosion through bendways within streams and rivers is a major problem in North America that can lead to serious economic, structural, and environmental damage. In 1981, the US Army Corp of Engineers (USACE) reported that nearly 17 percent of the total length of waterways in America requires protection from the effects associated with bank erosion (USACE, 1981). This problem has become so great that a wide range of streambank stabilization and control applications have been studied and implemented throughout North America in an attempt to alleviate the situation.

The USACE recognizes that sediment transport is a direct factor leading to bank erosion. It is a natural occurrence caused by the flow of water and its impact on materials comprising the bed and bank of a waterway. Sediment transport and subsequent bank erosion can lead to changes to a channel, more specifically to its general shape and geometry, which can be observed at distances further downstream, or, at the local area in question. The presence of bendways within a waterway is of great concern due to the complex hydraulic processes associated with these sections. Bed material becomes transported from the outside (concave) bank of the bend to the inside-bank of the bend. If not addressed, this phenomenon can undermine the present boundaries of the outside-bank, resulting in eventual failure. Severe sediment transport away from the outside-bank of the bend is due to the formation and impact of secondary (cross) currents, produced by fluctuations in pressure caused by changes in velocities across the channel when flow enters a bendway. As flow travels through a bend, a helicoidal current forms at the outside-bank and travels towards the center of the channel, normal to the primary direction of flow. To counteract this, another current travels in the opposite direction, creating a sort of natural balance. This flow phenomenon, also known as transverse circulation, can lead to severe damage to the outside-bank. As sediment moves from one

side of the channel to the other, longitudinal flow becomes concentrated in the newly formed deeper sections, increasing its impact and intensity on the outside-bank. Secondary currents can affect point-bar formation, lateral and downstream migration, channel width variation, changes in local depth, navigation constriction, extreme changes in flow velocity, and high-flow current patterns.

Aside from the physical damages that can be caused by sediment transport within a channel, environmental and structural damages in areas surrounding the waterway are key considerations as well. In any situation where sedimentation is encountered, whether as a result of water flowing through a bend or from other human or environmental causes, sediment loads within the flow drastically increases. This can lead to the potential for deposition further downstream such as on private property, at dams, reservoirs, harbors or other navigable waterways etc., resulting in increased costs for removal as well as an increased risk of damage. There are also environmental burdens caused by sediment transport such as water quality reduction, degradation or complete loss of aquatic habitats, and a decrease of the overall aesthetic image of a particular area.

In attempts to address the abovementioned consequences of secondary current action in bendway sections, bank erosion protection and stabilization have become a major focus for river engineers. Years of research have culminated in numerous bank protection devices ranging in size and shape, practicality and effectiveness. More common techniques (termed “traditional” herein) include: various types of revetment (e.g. rock riprap, gabions, etc.), groins (dykes) which are either permeable or impermeable, and trenches. These methods, which have proven to be adequate over time, are frequently costly and difficult to implement and maintain. Newer bank protection methods such as bioengineered techniques (crib walls, vegetation covers, willow posts), and bendway weirs, have focused on lowering costs and labor as well as incorporating more environmentally friendly and aesthetically pleasing aspects.

A novel streambank protection device, called ‘barbs’, is currently undergoing field testing on various streams in the state of Illinois by the United States Department of Agriculture

(USDA) Natural Resources Conservation Service (NRCS). Barbs are low rock structures, similar in form to the more conventional bendway weir, that are placed at an angle from the outside-bank of a stream bend such that they are directed into the flow of the channel and are completely submerged under bankfull flow conditions. Their main purpose is to protect the outside-bank of a stream bend from local erosion processes that result from the aforementioned secondary currents associated with bendway flows by: (i) disrupting velocity gradients near the outside-bank, (ii) deflecting secondary currents away from the bank towards the center of the channel, and (iii) shifting the thalweg towards the center of the channel. This is achieved by the incorporation of a submerged weir segment. Flow becomes forced over the weir portion of the barb, is redirected normal to the structure away from the outside-bank, causing a disruption in the velocity gradient, resulting in a decrease in shear stress along the streambed and thus creating changes in sediment transport patterns. Further beneficial flow effects created by eddies caused by the presence of stream barbs include the promotion of deposition at and near the structure as well as a balance with naturally occurring secondary currents, whose effect is greatly minimized near the bank wall, drastically decreasing local erosion and bed scouring. What makes these structures so unique is that they are active bank protection devices that attack the erosion mechanism, unlike more traditional devices that are passive, counteractive measures for protecting streambanks.

Factors such as stream meander pattern ( $M$ ), channel width to flow depth ratio ( $W/y_0$ ), channel gradient, and geological and geomorphic characteristics are all taken into account when first considering the implementation of a potential barb project, as well as other recommended design guidelines including barb material size and specifications, location and number of structures, barb height ( $h$ ), spacing, upstream angle ( $\theta$ ), profile, barb width, bank and bed key characteristics, and general construction considerations (USDA, 1999). This bank protection technique is so new that published studies are extremely rare. However, through the limited 'internal reports' as well as through personal correspondence with the NRCS (Kinney, 2000, 2002), design notes and recommendations were obtained and have provided a starting point for this study. Research indicates that past field studies (Kinney, 2000, 2002) have focused on the use of barbs in wide, shallow

waterways. Design recommendations (USDA, 1999, 2001) have focused solely on barb applications in the field for this particular condition, and although research and ongoing field studies show great promise, there is clearly a need for controlled laboratory investigations to study the performance of various arrangements of model barb structures for their wider applications. **It is the goal of this study to investigate the performance of model stream barb structures in strongly curved narrow channel bends under steady state 'clear-water' conditions in the hopes of creating a basis for controlled model tests to determine conditions and the effects of barbs prior to their implementation in the field. Primary research objectives include the investigation of any criteria that leads to enhanced barb performance in narrow channel conditions, as well as the verification that a barb axis orientation of 30-degrees is the optimum design value for  $\theta$ , an angle that proved to be superior in previous barb field studies under wide channel conditions.**

## **1.2 Outline of Thesis**

The second chapter of this thesis has three main sections. The first includes a detailed statement of the problem that is bank erosion and scouring and the many factors that influence them. The second section goes over some of the more common bank protection devices, such as rock riprap, revetments, groins, gabions, bioengineering techniques, and bendway weirs. The third section deals with an overview of stream barbs; design and construction details, environmental impact etc.

Chapter three describes the apparatus and experimental procedures that were applied in the study, including such considerations as the bend (test) section(s), experimental details and procedures, model barb characteristics, and other related matters. This chapter also discusses the conditions under which the experiments were designed and performed, including various calibration exercises relating to test facility operation, bed material characteristics, flow visualization, channel slope, flow velocity, and test duration.

In chapter four, the experimental data and analytical methods are discussed. The influence of barb angle ( $\theta$ ), maximum barb height (h), channel forming flow depth ( $y_0$ ), and barb placements are all determined. An overall summary of the study, conclusions, and recommendations as well future research needs are included in chapter five.

## **Chapter 2**

### **Literature Review**

#### **2.1 Introduction**

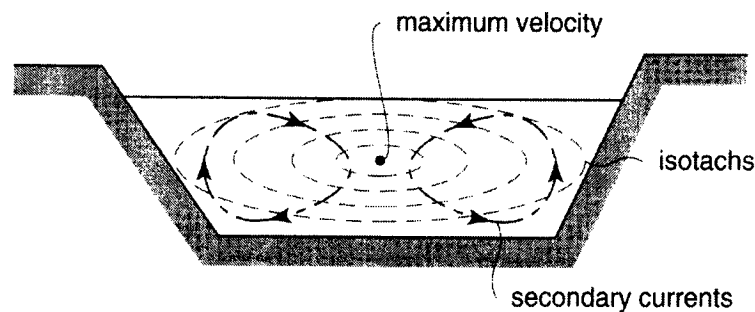
Sediment transport and bank erosion are problems that are commonly associated with the interaction between the flow of water in a river and the various alluvial materials that make up a river's bed and banks. Changes to the shape and geometry of an alluvial channel can have significant impacts to the river itself as well as to surrounding areas. Streambank erosion and subsequent changes to the river bed are major concerns to river engineers. These impacts can result from either human interference such as mining, agricultural activity, the effects of urbanization and transportation, drainage, flood plain development, and boating; or to natural occurrences such as extreme flows, characteristics of the bed and bank material, bank characteristics, subsurface flows, naturally induced waves, climate, vegetation, and animal activities. The impacts of erosion, scouring, and streambank failure, can be experienced not only locally but in many instances for considerable distances upstream and downstream of the actual site. Bank undermining and scouring action at riverbends is of particular importance due to the severe damage that can be caused to the integrity of the channel if not properly dealt with.

##### **2.1.1 Scouring at bends**

Scouring at riverbends can potentially damage a river bank, can influence a river's flow pattern, and can change the general integrity of the river itself. The movement of bed material from the concave (outside) bank of the bend to the inner portion of the bend can lead to bank erosion and bank failure due to the potential for undermining as well as have a great impact on point-bar formation, lateral migration, downstream migration, and width variation. Thus, it is important, especially in bend sections, to reduce the amount of scouring and erosion in order to maintain the viability of the river and the bend.

### 2.1.2 Secondary currents

Secondary currents result when “changes in velocity across the channel cause small changes in pressure” (Kay, 1998). These changes in pressure promote the formation of cross-currents, which occur normal to the direction of flow and travel from the bank towards the center of the channel (fig. 2.1). To balance these surface cross-currents, flow from the center travels underneath the surface currents towards the sides. The interaction and circulation of these opposing cross-flows is the basis of the phenomenon known as secondary currents.



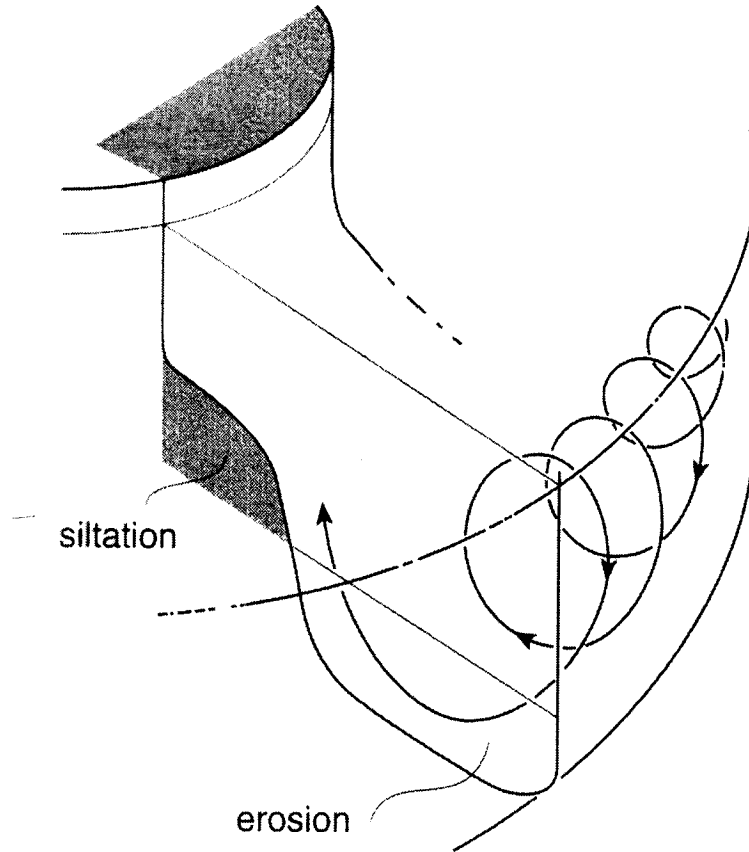
**Figure 2.1:** Cross-currents (Kay, 1998).

The formation and control of secondary currents in channel bends is of particular importance. The phenomenon occurs due to two factors:

- (i) Differences in flow velocity between the water surface and flow near the channel base.
- (ii) The impact of centrifugal forces created when water flows through the bend.

Due to the natural vertical velocity gradient, the maximum longitudinal velocity in a river bend occurs at the water surface, while the minimum velocity occurs at the channel base. Centrifugal acceleration can be defined as  $u^2/r$ , where  $u$  is the local longitudinal velocity and  $r$  is the bend radius (Chang, 1988). As a result of the two abovementioned factors, the vertically differing centrifugal acceleration creates a ‘spiral motion’ flow. It is this spiral motion flow, which is referred to as the ‘secondary current’ or ‘transverse circulation’,

that travels from the outside-bank of a bend to the inside-bank. A major consequence of this natural occurrence is the erosion of outside-bank material and its deposition in the vicinity of the inside-bank (fig. 2.2).



**Figure 2.2:** Secondary flow at a bend (Kay, 1998)

Secondary currents not only have a pronounced impact within the bend itself, but can also be felt downstream of the bend as well. Theoretically, as flow exits a bend, the effects of secondary currents begin to diminish. However, depending on numerous factors such as bend angle and total flow, among others, residual secondary current effects can be felt in reaches far downstream from the bend exit. As a result, it is important for river engineers to not only combat the effects of bend generated secondary currents within a riverbend, but in downstream sections as well.

## **2.2 Streambank erosion**

Streambank erosion and the subsequent disruption of a channel's bed and bank geometry over a period of time can create problems, particularly during a period of high flow such as a flood. In riverbends for example, the movement of sediment towards the inside-bank of the bend can create a restriction of flow due to a decrease in flow area within the bend. Besides channel geometry, streambank erosion and overall bank stability is also dependent on the interaction between flow and sediment material. The 'stream boundary' of a channel can be greatly altered due in part to changes to streamwise slope, and velocity and depth of flow. Other factors affecting the stream boundary include the bed-load and suspended-load discharge and concentration, as well as bed material characteristics such as size, gradation, shape, and specific weight of the sediment.

Three major factors that can lead to streambank erosion, scouring, and sediment transportation and deposition are (Peterson, 1986):

- (i) Removal of soil particles from the bank surface either continuously or intermittently over a period of time.
- (ii) Sequential failure of small segments of bank material.
- (iii) Failure of a single large segment of bank material.

According to the USACE, there exist various mechanisms that can help evolve the process of surface bank deterioration. They include abrasion, animal activity, the effects of vegetation, chemicals and chemical reactions, debris, gravity, human activity on the bank or within the channel, ice, precipitation, waves, wind, freeze-thaw action, and factors of flow such as flow rate and velocity, transport capacity, turbulence, secondary currents, seepage flows, and overbank flows.

The USACE also classify conditions under which bank stability can be affected (USACE, 1981):

- (i) At the surface
  - a) Severe surface deterioration that may result in an unstable bank configuration, such as erosion by streamflow at the toe of the bank; erosion at the water surface due to waves; and erosion along the bank surface due to overbank flows.
  - b) Deep tension cracks due to excessive drying of a cohesive soil, or a different but similar type of structural change, can cause the bank to weaken and become unstable. Crumbling may occur if excessive drying is followed by submergence.
  - c) Overburden along the top of the bank that may cause a stable bank to become relatively unstable.
  
- (ii) Moisture content within the bank
  - a) The slope of a bank composed of cohesionless material may become temporarily steeper than the angle of repose of the bank material due to capillarity or other effects. When these effects are no longer present, the bank may become unstable.
  - b) Piping, the phenomena in which subsurface flow carries sediment from permeable layers resulting in the failure of any overlying layer(s), can make the bank more prone to erosion. An example includes when cohesionless material is eroded from the bank by seepage flow, creating a cavity that can extend deep into the bank along the seepage path.
  - c) Fine-grained and loose material within the bank can become liquefied and can be affected by increases in pore pressure. This can result in the sliding or slumping of large chunks of bank material as a fluid mixture into the flow of water.
  - d) High water tables and/or low stream levels can lead to an increased hydraulic load on the bank, resulting in potential failure.
  - e) Clay type soils can be affected by wetting and drying, which can lead to swelling and shrinking of the soil, creating an unstable condition that can lead to failure.

- f) Free flowing water into deep tension cracks and openings between different bank materials can lead to an increase in hydraulic pressure and subsequent failure.
  - g) The shear strength of clay is dependent on pore pressure and saturation.
- (iii) Miscellaneous conditions
- a) Artesian and gravity flow in a cohesionless or porous layer can remove soil material via piping, which can result in shear failure within above layers.
  - b) Saturated clay layers can weaken and compress, causing shear failure in the upper bank.
  - c) Lubrication by water and high hydrostatic pressure along interfaces between bank materials can cause low resistance to sliding, which can lead to severe bank failure.
  - d) Combination of site-specific failure mechanisms.

A common cause leading to bank failure is due to the effects associated with the impact of flow into the region at and surrounding the bank toe (ASCE, 1965). Erosion at this location of the bank and the subsequent movement of eroded material leads to a more steepened bank slope, eventually leading to failure. Two events recognized by the ASCE Task Committee: (i) 'flow slides', a rapidly occurring phenomenon that has the potential to affect large areas and occurs when low density soil deposits of silt and sand exposed to flow become saturated and subsequently liquefies; and (ii) 'bank sloughing', when saturated cohesive banks that are unable to drain fail due to a lowering of the water surface, were determined as major factors leading to bank failure.

### **2.2.1 Bed and bank material**

The erosion process within a channel can vary with changes in bed and bank material. There exist three types of generally classified bank material: cohesive, noncohesive, and composite (stratified) bank material. Cohesive bank material is considered the most resistant against erosion due to flow within a river. It has a low permeability and can resist

the negative effects associated with piping, seepage, frost heaving, and subsurface flows. Banks made up of cohesive material however are prone to failure due to sliding when undercut or saturated, and can become extremely unstable in situations where water levels are quickly falling. Banks consisting of noncohesive material tend to fail 'grain by grain'. Surface erosion and particle removal rates are affected by velocity, flow direction, turbulence fluctuations, shear stress magnitude, shear stress fluctuations, seepage force, wave forces, and piping. Composite banks are created by the transportation and deposition of sediment from other parts of the channel and are found mostly on alluvial rivers. The banks of these types of channels are made up of different layers of material ranging in size, permeability, and cohesion. Layers primarily consisting of noncohesive material are prone to surface erosion, but are often times protected by neighboring cohesive layers. Composite banks are also prone to sliding due to subsurface flows as well as piping.

### **2.2.2 Subsurface flows**

Rivers that experience subsurface flows can have an effect on the riverbank in two ways: seepage into the bank, or seepage into the river. Rivers that experience the former usually exhibit larger flow depths and smaller channel widths for a given discharge. Conversely, rivers that experience the latter tend to have larger widths and smaller flow depths. It is this latter case that is of the utmost concern to river engineers as an inflow of water from the riverbank creates a seepage force which often times becomes a destabilizing influence on the bank. One can always tell whether or not to expect flow into or out of the banks. When water flows from the bank into the river it means that the water table is higher than the free surface of the river, and can be caused by (Simons, 1995):

- (i) A wet period during which water draining from adjacent watersheds saturates the flood plain to a higher level.
- (ii) Poor drainage conditions resulting from deterioration or failure of surface drainage systems.

- (iii) Increased infiltration resulting from changes in land use causing an increase in water level.
- (iv) Irrigated flood plains.
- (v) Development of the adjacent flood plain for homes and businesses that utilize septic tanks and leach fields to dispose of wastewater and sewage.

It should be noted that fluctuations in the river stage lead to both the inflow and outflow of water to and from the bank, and at all times, subsurface flows and seepage forces have a major impact on overall bank stability.

### **2.2.3 Piping**

Piping, as mentioned earlier, is the process in which subsurface flow carries sediment from permeable layers, resulting in the formation of cavities within said layers, leading to the subsequent failure of the bank. When material is slowly removed by subsurface flow from these permeable layers, the entire bank can become undermined. Layers lying atop these newly formed cavities have no underlying support, and thus fall into the empty spaces below. This leads to the formation of tension cracks, which also enhances the erosion process further by allowing water at the surface to enter the ground. Eventually, as more and more material is carried away by subsurface flow, and as more and more water enters the newly formed voids, the section will ultimately slide into the river, creating a massive bank failure.

### **2.2.4 Mass wasting**

The processes described in section 2.2.3 show the impact of piping, subsurface flows, and mass wasting. As previously mentioned, according to the USACE, local mass wasting is a common occurrence when the bank of a river becomes saturated, as entire blocks of bank material can be undercut by the flow of the river under these conditions. This results in the slumping or sliding of these sections into the river. Other factors that can influence the amount of local mass wasting in a riverbank include human activity on or around an

adjacent flood plain such as construction, the leaching of waste from septic tanks, and increased infiltration due to the use of the surrounding land, gravitational forces influenced by vegetation growth, and poor drainage conditions due to either human influence or natural circumstances.

### **2.3 Streambank stabilization**

Generally, the two main reasons for riverbank stabilization are: (Peterson, 1986)

- (i) To prevent bank erosion and the subsequent loss of land that threatens life and property.
- (ii) To improve stream alignment, cross-sectional shape, and depth to reduce flood stage and to improve navigation conditions.

Stabilization and rectification projects on rivers have been accomplished by utilizing a range of structures and devices whose purpose could be to either control or direct the flow of water away from the riverbank and subsequently guide it naturally without forcing or creating unnatural-like conditions, or, to protect the bank from erosion by creating resistance against negative forces associated with the contact made between channel flow and the riverbank.

Riverbank stabilization also limits evaporation, transpiration, and losses due to seepage by concentrating flow through a deep, narrow channel. In practice, stabilization and rectification works can help save money by preventing damage and protecting valuable property and infrastructure, as well as create alternative means of income for industry and commercial use by improving long-term channel stability.

As previously mentioned, avoiding unnatural conditions is key in a successful stabilization project. Flow within a river must be allowed to follow its natural path throughout the course of the channel without creating major disruptions to the flow and

sediment regime(s). This is important with regards to an economic aspect because costs are drastically minimized when a channel follows its natural alignment.

There exist a wide variety of stabilizing methods and techniques that range from the traditional, such as dikes, revetments, and other bank protection devices, to the modern, more innovative and rapidly growing techniques for bank stabilization, such as bioengineered protection, bendway weirs, and stream barbs.

## **2.4 Riverbank stabilization techniques**

Traditional bank protection techniques include revetments, rock riprap blankets, groins, gabions, and soil-cement blocks just to name a few. The one common characteristic shared by many bank protection devices is that they are constructed and designed for sections well above the estimated high water stage and are implemented within the channel in an area well below expected scour levels. These traditional bank protection devices have the advantage of being designed for extreme and often times unforeseen situations. They are also relatively easy to design and install, and are considered to be very effective. However, most of these techniques are expensive, involve a lot of time and labor in the installation process, and when completed, do not blend in well with its surrounding environment.

### **2.4.1 Rock riprap**

Rock riprap is a common method used for protecting riverbanks against erosion and scour and can best be described as a layer or blanket of large stones acting as a protective armor against erosion and scour. If sufficient sized rocks are available, riprap is considered an extremely economical and efficient bank protection technique compared to other traditional protection measures due to the low cost and simplicity involved in constructing a riprap blanket. Aside from its relatively low cost, rock riprap is considered to be very flexible compared to other protection techniques, is extremely durable, can be installed and placed with ease, can be quickly repaired when damaged, can be self-repairing, is

recoverable, does not encounter foundation problems, usually is not affected by small settlements, slight adjustments, or small movements of the bank or the bank toe, requires relatively low maintenance, does not require complicated or highly technical construction procedures, combats wave run-up and large currents effectively, and is natural, salvageable, and environmentally friendly.

The overall stability of rock riprap is dependent upon several factors, which include individual rock durability, shape, density, and angularity, the range of velocities within the channel, gradation, the direction of flow, bed and bank slope to be protected, riprap angle of repose, weight of the blanket, blanket thickness, and the distance below the bed the riprap must exceed to avoid scour.

Rock riprap is usually composed of hard, durable, angularly shaped rock ranging in size from 10 to 50 cm, depending on the expected hydraulic load, and whose mass has been known to range from 10 to 500 kg. In terms of individual rock angularity, the best rocks are usually those that are nearly cubical. Thin rocks that resemble a flat sheet are the least desirable due to their susceptibility to hydrodynamic forces. The rock riprap should also be well graded so that any voids created by larger stones can be filled by smaller ones.

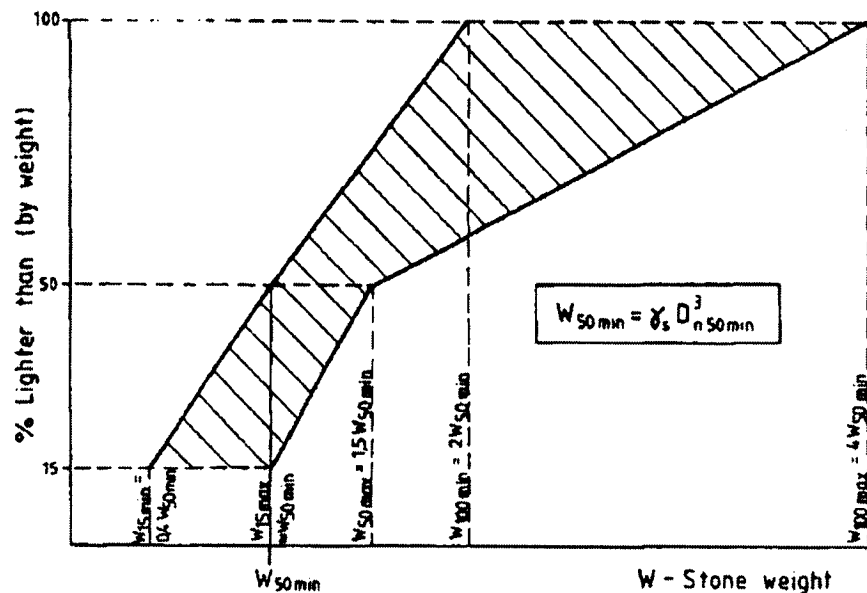
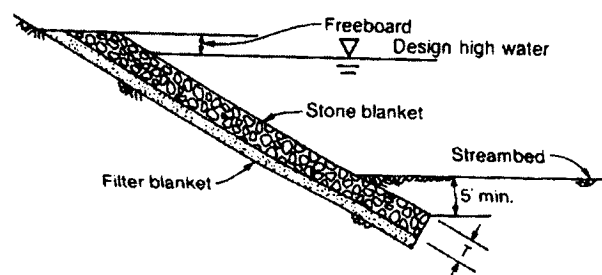


Figure 2.3: Riprap gradation- US Army Corps of Engineers (Przedwojski et al., 1995)

The USACE recommend that the riprap gradation resemble that shown in fig. 2.3. The thickness of a riprap layer should not exceed 30.5 cm, the  $D_{100}$  stone size, or 1.5 times the upper limit of the  $D_{50}$  stone size, whichever is largest (USACE, 1970). However, it is recommended that if placed underwater, the layer thickness should be increased by about 50 percent. The minimum stone density used in the layer is usually around  $2 \text{ Mg/m}^3$  with a compressive strength no less than 40 MPa.

Installed either by hand or dumped by a truck or a machine, a riprap layer can be placed either directly on the bank slope that is to be protected or upon a pre-made filter layer. A filter layer prevents the movement of bank and bed material through the voids of the riprap and is usually made up of fine gravel or a synthetic cloth-like material. In order to be effective, the filter must consist of very fine material to prevent leaching, but must also be more permeable than the bank material itself. In cases where the bed material is very fine, a gravel filter consisting of many layers, each about a tenth of a meter thick, is utilized. Finer layers are placed closer to the bed. While moving towards the riprap, the gravel consisting of these layers become larger and larger. A 'filter ratio' is defined as  $D_{15}/D_{85}$ : the ratio between the coarser filter layer (a single layer consisting of coarser material) and the finer filter layer (a single layer consisting of finer material). In order to promote stable conditions,  $D_{15}/D_{85} < 5$  is often time desirable. Other important requirements involve the  $D_{15}$  of the coarser layer and the  $D_{15}$  of the finer layer such that  $5 < D_{15(\text{coarser})}/D_{15(\text{finer})} < 40$ , and the  $D_{50}$  of the coarser layer and the  $D_{50}$  of the finer layer such that  $D_{50(\text{coarser})}/D_{50(\text{finer})} < 40$  (Chang, 1988 & Przedwojski et al., 1995).



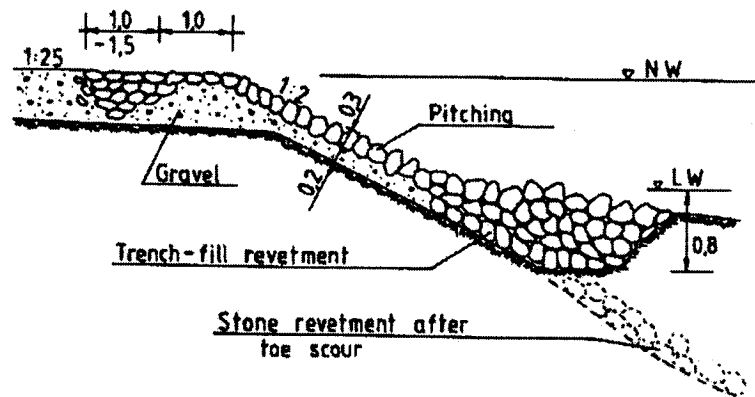
**Figure 2.4:** Riprap layer (Chang, 1988)

Synthetic fabrics or geotextile materials can also be used for a filter layer. These types of filters are quick and easy to install, are cost-effective, and can be used without worrying about the availability of adequate gravel sizes. The major drawback to these types of filters is the difficulty of in-situ installation. Also, these filters may cause sliding or slumping of embankment riprap layers, especially on steeper slopes. If these types of filters are employed, an opening area of about 25 to 30 percent is often used in order to provide filtration and drainage, which is necessary to ensure adequate piping, clogging resistance, and adequate strength (Peterson, 1986). A thin 100-150 mm layer of sand or fine gravel is often times spread between the synthetic layer and the riprap to prevent the ripping of the filter. Figure 2.4 shows a general riprap layer with a filter blanket.

### **2.4.2 Revetments**

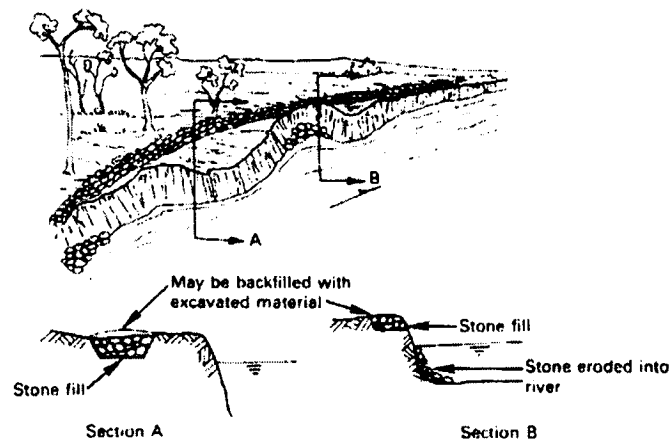
Revetments are structures placed on the slope of a riverbank and are designed to protect its surface from eroding mechanisms such as waves, currents, groundwater flow, and surface flow. The process involved in designing revetments usually takes into consideration several important conditions, which include a bank slope graded between 1V:2H to 1V:4H, depending on the type of bank material, in order to ensure bank and revetment stability; protective blankets that consist of porous material to ensure proper bank drainage so as to avoid buildup of excessive pore water pressures; the use of a filter layer under the protective blanket; protective measures at the toe of the bank extending riverward into the channel to avoid excess scour at the toe; and the type of material suitable for the protective blanket.

It must be noted that the use of rock riprap, as outlined in section 2.4.1, is a common type of material used in the design and construction of revetments, and as such, many of the guidelines followed in the construction and installation of rock riprap protective armor blankets is followed in the design process for revetments.



**Figure 2.5:** Trench-fill revetment (Przedwojski et al., 1995)

Standard trench-fill revetments involve the placing of stones on the riverbank in the form of a riprap-like blanket, usually no more than 30-60 cm thick. Stones are also placed into trenches that are dug and located at the riverward edge of the stone blanket, as seen in fig. 2.5. When flow erodes away at the bank, the pile of stones at the revetment toe slide down the bank slope to provide more than adequate protection at the toe. A different type of stone-trench revetment, known as a windrow revetment, involves a trench dug above the slope of the bank, parallel to the stream, and filled with rocks (fig. 2.6)



**Figure 2.6:** Windrow revetment (Peterson, 1986)

Flow is meant to erode the sides of the bank to the point where it reaches the rock filled revetment. The stones then roll down the bank into the river at the foot of the bank slope, providing protection from further erosion. In both types of trench-filled revetments, the amount of rock placed in the trench is entirely dependent upon the desired extent of protection along the bank. Trenches can be dug by hand or by machine, and rocks can be easily placed into the trenches, making these types of revetments very economical and practical. However there are other types of revetments that are designed without the use of rock riprap.

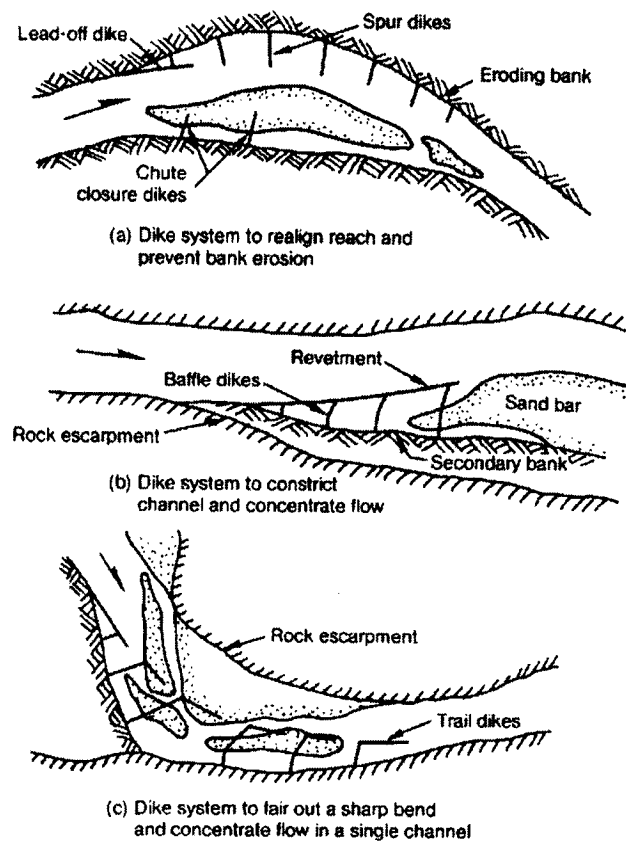
Standard revetments with mattresses are considered the one of the first devices used to protect streambanks in the US, and were first used on various sections of the Mississippi River. The mattress itself can consist of different types of material such as wood, willow, asphalt, or concrete, and extends from the edge of the water at low water on the underwater bank to the streambed beyond the bank toe. The two most common mattress types are the woven wood mattress and the articulated concrete mattress.

Woven wood mattresses consist of wooden boards (usually 2.5 cm thick by 10 cm wide) that are perpendicularly woven together and mounted on a frame made of wood that is designed to provide stability and strength. These types of mattresses are commonly used in the middle portion of the Mississippi River. Installation involves the mattress being sunk from a barge by the weight of rocks placed in cribs that are designed within the frame. Woven wood mattresses have the advantage of being very flexible and being able to conform to any shape of channel bottom. The major disadvantage however is that material can enter the gaps between the boards, which require mattresses to be installed with bank paving and/or a filter layer on the upper bank.

Articulated concrete mattresses consist of rectangular blocks of concrete cast onto and held via metal reinforcing fabric. Used predominantly in the lower Mississippi River, articulated concrete mattresses involve high construction and installation costs and are only used for situations where large amounts of underwater protection is required.

### 2.4.3 Dikes (Groins)

Dikes, also known as a groynes, groins, or spurs, are transverse river training structures designed to deflect flow away from the riverbank. Normally made of stone, gravel, dirt, or piles, dikes extend outward from the bank either perpendicular to the flow or, often, at an angle in the downstream direction. Aside from preventing bank erosion, dikes also create improved navigable conditions, cut off side channels and chutes, concentrate a braided river into a single channel, constrict a wider channel to increase flow depth, realign a river reach, train a river along a particular course, create regions of low velocity in order to increase siltation, increase the potential for flood control, and protect structures along the bank.



**Figure 2.7:** Typical groin system (Chang, 1988)

Figure 2.7 shows a system of groins in the Arkansas River. The design of a groin is dependent upon a number of factors, which include flow and flood depths, the ratio of

suspended load to bed load, channel slope, flow velocity, bed material characteristics, channel size, the presence of debris and ice, and materials available for groin construction.

Taking into consideration all of these factors, one must decide what type of groin to use. There are two types of groins, either permeable or impermeable, which differ from one another in the type of material used for its construction. Permeable groins, which can be made from timbers, bamboo, or piles, are as their name implies, and therefore have the ability to allow flow through them. Their main purpose is to slow flow velocity down, which thereby decreases the amount of bank erosion. They are considered to be most effective in alluvial streams and in rivers where bed load and sediment concentration is relatively high, which allows for the deposition of sediment around the groin itself. In many cases, permeable groins have been shown to effectively dampen the erosive forces of the current and eliminate any potential local erosion within clearer streams. It has also been determined that permeable groins are more effective than their impermeable counterparts in streams whose bed and bank materials contain high levels of sand and silt. They are also relatively inexpensive compared to impermeable groins, and since they do not create high amounts of disturbance within the flow, eddies, turbulence, and scour holes are avoided within the stream. Figures 2.8 & 2.9 show a diagram of a permeable bamboo groin and a typical groin system set-up respectively.

Impermeable groins deflect flow away from riverbanks as well as encourage a more desired stream alignment. The major advantage of solid groins is that they can be used in streams that encounter high velocity levels, whereas permeable groins lack the strength to withstand such flows. Impermeable groins are structures that mainly consist of rocks that are well graded so as to eliminate the possibility of the formation of voids. As seen in fig. 2.10, the head and toe are heavily reinforced with stone due to the fact that impermeable groins induce a significant amount of scouring. As a preventative measure, the groin is placed deep into the erodible bed due to the potential for local scouring at its toe, and is often times further reinforced with dumped stones around the toe for added protection.

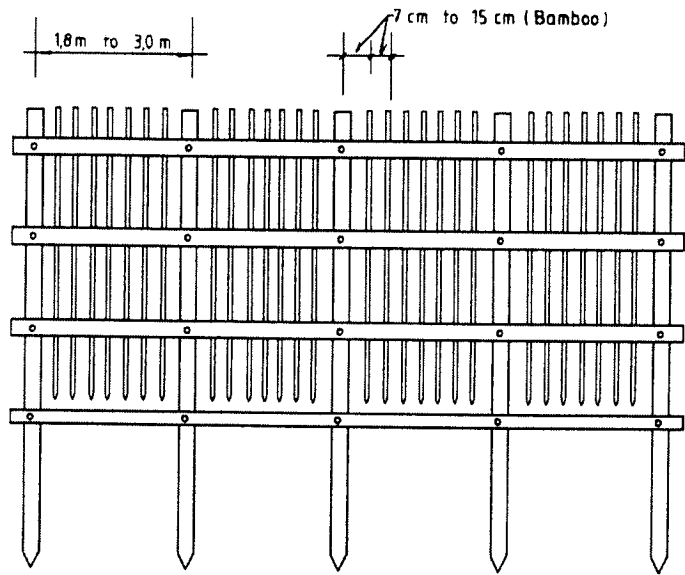


Figure 2.8: Bamboo groin (Przedwojski et al., 1995)

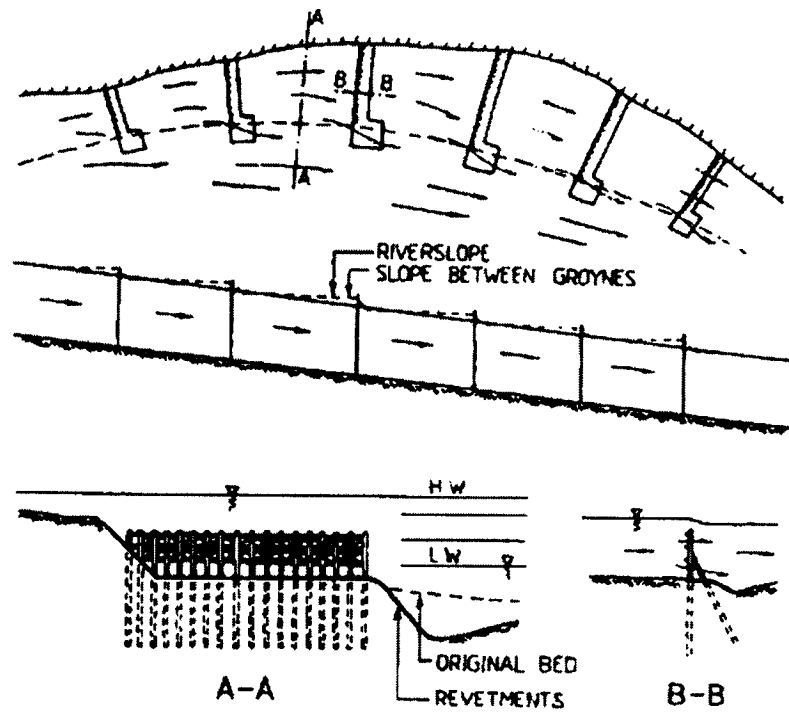
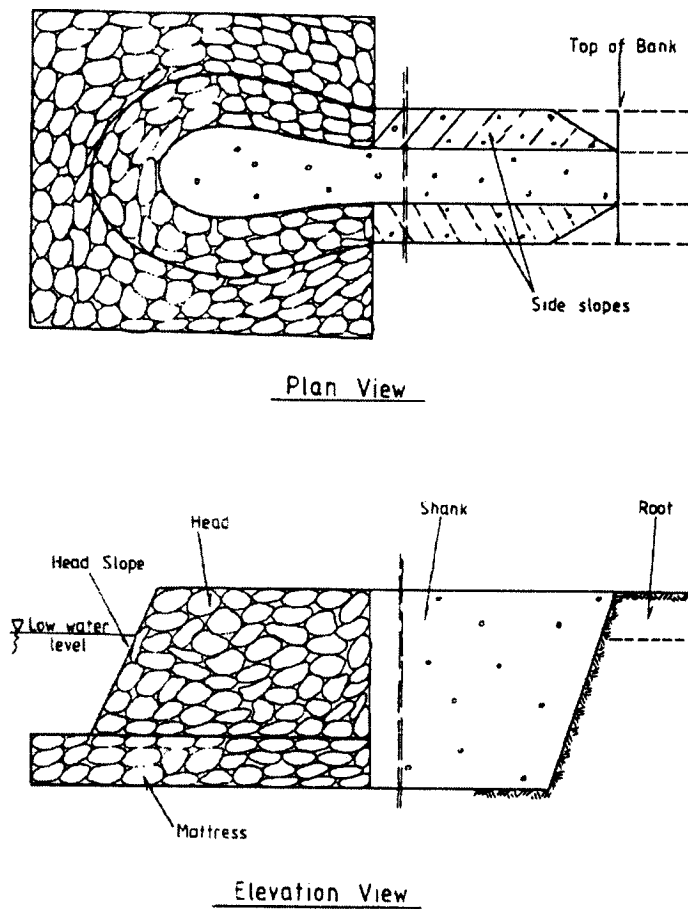


Figure 2.9: Groin system (Przedwojski et al., 1995)



**Figure 2.10:** Basic impermeable groin (Przedwojski et al., 1995)

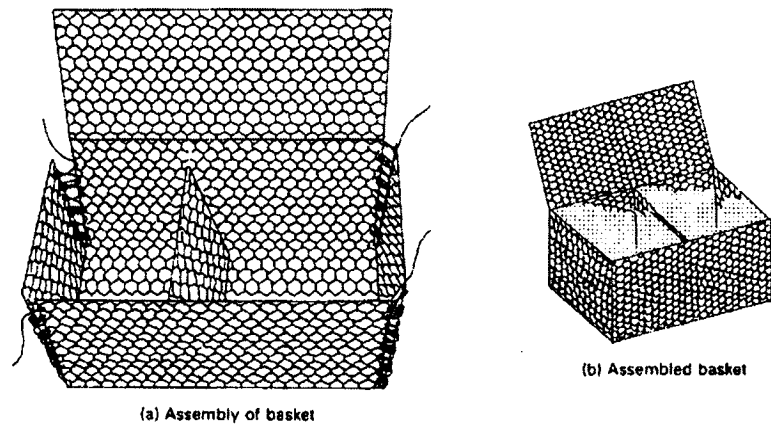
Another decision that must be incorporated into the design of a dike system is whether or not they are submerged or non-submerged. Impermeable groins are often times non-submerged. Its solid state promotes conditions of erosion along the shank of the groin if totally submerged, which can result in the flow of water over its top. Permeable groins however are very capable under submerged conditions due to the lack of flow disturbance associated with them.

Groins can also be classified as being attracting, deflecting, or repelling. Attracting groins have the ability to attract flow to themselves and are angled downstream. They are designed not to direct flow towards the opposite bank, and as such should not be used on a concave bank. Deflecting groins are usually shorter than the other types of groins and

change flow direction without repelling flow. They are usually used in cases where small, local protection is required. Repelling groins are angled upstream and repel flow away from themselves.

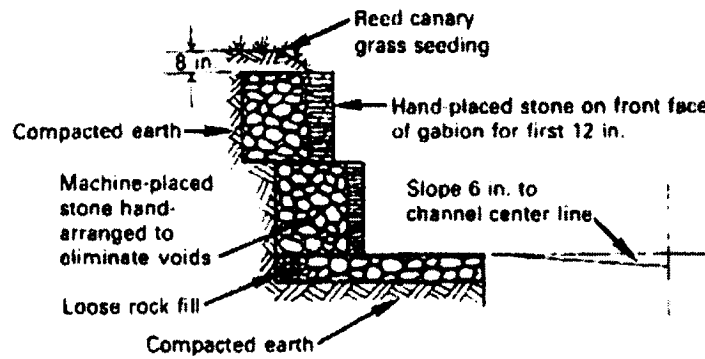
#### 2.4.4 Gabions

Gabions (fig. 2.11) consist of rectangular shaped baskets made of heavy galvanized metal wire that are packed with rocks and are used as a revetment-type structure in situations in which riprap blankets do not provide enough stability for a given flow velocity. It is recommended that gabions be used in conjunction with maximum flow velocities ranging between 2 and 5 m/s. Rock sizes packed within the structure have been known to range between 10.2 and 20.3 cm (TerraAqua Gabions, Inc.)



**Figure 2.11:** Gabion wire mesh basket (Peterson, 1986)

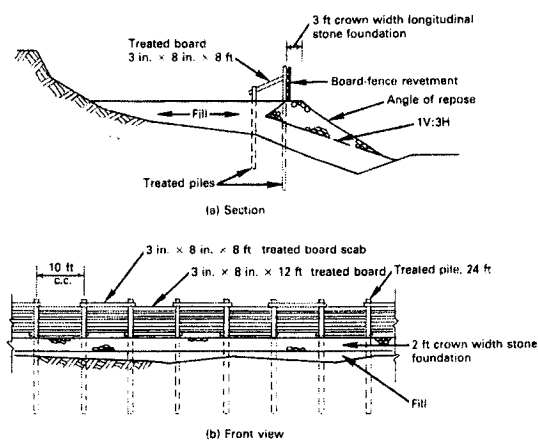
The major advantage of gabions is the fact that the structure itself is very heavy and very porous, and thus can provide adequate bank drainage as well as act as a suitable stabilization device. Often times a filter layer, usually made of granular stone, is applied underneath the gabion in order to prevent leaching and undermining. The size of the basket mesh openings depend upon the conditions at hand as well as the stone size used. A typical gabion revetment structure as designed by the USACE is shown in fig. 2.12 and was designed for a stabilization project against a 100 year flood on Deadman's Run and Antelope Creek in Lincoln, Nebraska.



**Figure 2.12:** Gabion revetment structure (After USACE, 1981)

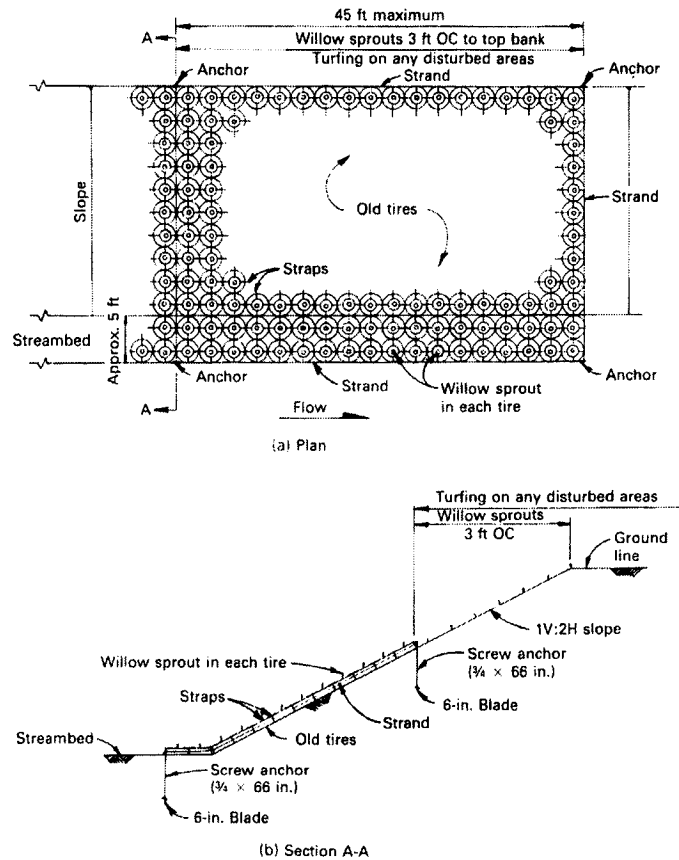
### 2.4.5 Other bank protection devices

Protecting banks from erosion is a process that often times involves using different methods in order to achieve the desired result. Permeable fencing is an economical technique that has been used to protect riverbanks in cases where the channel in question has a low gradient. A common design by the USACE is shown in fig. 2.13. It can be installed longitudinally with respect to the flow and parallel to the bank as well as set at an angle against the flow. Fences can consist of wood, rails, beams, pipes, or concrete, but is often times made of steel wire. Advantages of fencing include the sufficient reduction of local velocities, the trapping of debris in the fence itself, and the encouragement of deposition as well as the growing of native vegetation at the fence.



**Figure 2.13:** Fence structure (Peterson, 1986)

Sack revetments, which consist of burlap sacks filled with sand, soil, soil-cement, or sand-cement mixtures, can be placed and lined up as a layer along the bank of a river. Cellular block revetments, which as its name implies incorporates the use of commercially manufactured cellular blocks, is the method of tying together such blocks and anchoring them to the ground. Soil-cement block revetments is a common protection device utilized in Asia which involves the creation of blocks made from a compacted soil-cement-water mixture and used in place of stones or rocks as revetment material. Used car tires can be utilized as an efficient bank protection measure by using tires tied together with wire, rope, or chain, and anchored flat on the bank to form a blanket-like layer. The voids within the tire(s) are then filled with rock, dirt, or concrete. A tire bank protection system is shown in fig. 2.14.



**Figure 2.14:** Tire protection system (After USACE, 1981)

A major problem with many of these as well as other previously mentioned traditional bank protection techniques is their inability to blend in aesthetically with their surrounding natural environment. These structures also tend not to provide much environmental benefit to the area, which has become almost as important a requirement as their ability to effectively stabilize and protect the streambank.

## **2.5 Bioengineering techniques**

A great deal of attention has been directed towards the interaction between traditional bank stabilization techniques and the environment, and as such, the use of natural bioengineered bank protection measures has increased at a rapid rate throughout the last decade. A wide range of bioengineering streambank stabilization and protection techniques that have utilized different species of plants, trees, bushes, grasses, and other types of vegetation have been applied to various sites including, among others: Little Miami River, Buffalo Bayou, Norton Branch, Little Sugar Creek, and Raccoon Creek (Fotherby et al., 1998; Sotir, 1998). However, the use of bioengineered techniques involves tiresome research and planning in order to determine the right type of plant as well as the right type of technique that is suitable for a given site and situation. Some of the more popular bioengineered techniques involve the implementation of transverse structures such as crib walls, longitudinal structures such as live timber frames, simple vegetative covers, and the vertical planting of such devices as willow posts along the streambank.

### **2.5.1 Willow posts**

The use of willow posts as a means of soil bioengineering and the stabilization of streambanks has garnered a great deal of attention recently. Using willow trees as a means of streambank stabilization has been a reliable and successful technique for years. The process of taking willow posts, which are large cuttings from the trunks or branches of willow trees, and planting them on the streambank, has been shown to be a successful endeavor in the short and long term for controlling erosion and maintaining stability along

bank slopes. Often times, willow posts are used in conjunction with other types of bank stabilization or protection measures to fully ensure the sustainability of the project.

Willow trees are sturdy, can consume large quantities of water, can grow at a rapid rate in a wide variety of conditions, and can easily grow from trunk or branch cuttings. Willow posts are harvested during the willow's dormant season and are usually about 3 to 4 m long and have a diameter ranging between 8 to 15 cm at the time of installation (Watson et al., 1997). It is important that they be planted during the dormant season, which is usually between early December and early March, depending on local meteorological conditions. They are placed vertically into holes along the course of the streambank, and in the case of bends along the entire length of the outside-bank. The holes are drilled 2 to 2.5 m deep via a steel ram or auger and spaced at regular intervals, usually between 1 to 1.5 m apart from the center of the hole. Other general recommendations include that posts be no more than 1.2 m above the ground surface, prior to installation posts should be soaked in water and installed no more than 48 hours later, posts be in good condition with undamaged bark, and that the seeding of non-woody species be planted in the surrounding area of the willow post installation to minimize erosion, promote a diverse range of plant life, and enhance aggradation which can provide an adequate rooting medium within the ground in areas where soil conditions are poor for growth.

The advantages of willow posts include (Watson et al., 1997; Derrick, 1998a, 1998b)

- (i) The strengthening and binding of the streambank by soil arching and pinning (stabilizing) of possible failure surfaces.
- (ii) Depletion of soil water and bank flow within the eroding bank by evapotranspiration.
- (iii) The mechanical reinforcement of the soil by root growth and the mechanical stabilization of the bank by the trunk.
- (iv) The ability of the foliage to retard stream velocity impinging against the bank by providing increased hydraulic roughness.
- (v) Its ability to enhance sedimentation along the bank due to decreased flow velocity.

- (vi) Its ability to provide a microclimate for colonization by other species.
- (vii) The trapping of debris and soil blocks from upstream failures.
- (viii) By providing a sound anchor for natural plant growth and other bioengineering techniques that may be placed along the toe of the bank.
- (ix) Bank protection is long term.
- (x) The willows do not grow into the stream or above the bank.
- (xi) The natural habitat (aquatic & terrestrial) is improved, restored, and/or created.
- (xii) Can be used on mild to steep bank slopes.

One disadvantage of sorts to the use of willow posts is the unknown factor associated with installation costs due to the fact that pricing varies from site to site, project to project. However, the overall cost of willow post stabilization projects is considered to be relatively low compared to conventional techniques and other bioengineered protection measures. The most costly aspect of willow post projects are the initial costs, which cover those incurred through the installation process and include time spent and moneys used for machines for drilling, transportation, and installation of the cuttings. Once installed however, maintenance and upkeep costs are considered to be very minimal.

The major concern with willow post stabilization projects is the survivability of the planted posts to ensure the long-term success and sustainability of the project. Premature willow post deaths immediately after installation have been caused by many factors, including: insects or other animals, damage during installation, the lack of aerobic rooting conditions due to impermeable soils or the incompatibility with certain types of soils, damage from high flow velocities, damage due to large amounts of sediments in the flow causing a retardation in growth, lack of adequate moisture especially near the trunk, anaerobic rooting conditions caused by standing water, and damage due to flooding. Another major concern is the filling of the hole in which a willow post has been planted. Improper back filling can create air pockets and subsequent soil failure around the post, as well as the possibility of the willow's roots growing into air pockets rather than the soil, which can potentially allow the post to displace itself via any sort of applied force. Clearly, it is extremely important to make sure conditions are suitable for growth prior to

installation, and detailed site investigations which include the determination of soil moisture levels, the type of soil at the site, and determining the right species of plant to use in the area are extremely important before implementing any type of bioengineered stabilization project.

Upon installation, careful monitoring and frequent site inspections during the first and second growing season and subsequent periods thereafter can also avoid high mortality rates. It has also been determined that survivability increases if erosion can be prevented at the toe of the bank slope at willow post installation sites. Watson et al. (1997) recommend that a longitudinal toe dike consisting of rock riprap be used in these areas as a continuous toe reinforcement mechanism. And while survivability of the planted willows is one of the marks for a successful project, longer-term success often times heavily depends on the growth and survival of native vegetation along the stabilized banks.

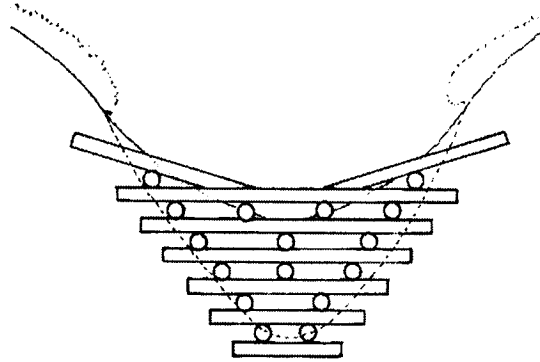
Numerous studies and field experiments have resulted in the growing admiration and use of willow post stabilization projects. Potential cost savings, increased effectiveness, high material availability, and low environmental disturbance are among the strong benefits of this type of stabilization technique. Often times however willow posts are used in conjunction with other stabilization methods, is usually not used without first altering the channel flow, and has not been successful where flow velocity is high.

### **2.5.2 Crib walls**

Crib walls are live barriers that consist of hollow-single or double-walled box structures made up of wood, logs, or prefabricated concrete, and are filled with free draining soil (Schiechtel and Stern, 1997). Branches of trees or shrubs with the ability to root itself and propagate are also placed in the structure for increased stability, strength, and efficiency.

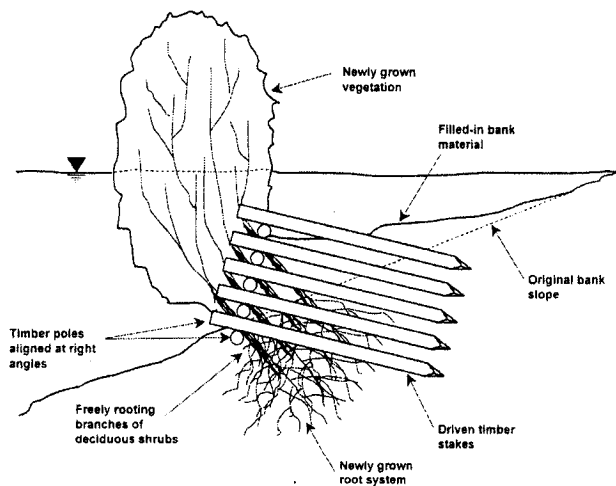
The most common crib wall structures are made of wood poles or timber sections that are tied together to form a box-like shape. Poles/timbers are aligned at right angles to the

flow. More poles are placed at intervals perpendicular to the first row of poles, as seen in figure 2.15, which shows a cross-sectional view of the structure.



**Figure 2.15:** Crib wall cross-sectional view (Schiechl & Stern, 1997)

Throughout the construction of the crib wall, whose dimensions may vary as the situation requires, the soil fill and branch cuttings are placed within the structure. The branches are placed at a downward angle into the soil. As they are installed, the timbers are continuously tied together, and the final structure is placed inclined against the flow and anchored onto the streambank. Another view of a live timber crib wall is seen in fig. 2.16.



**Figure 2.16:** Live timber crib wall (Voisin, 2001)

They should be constructed and implemented during the dormant period of the cuttings. This makes for much easier soil and branching installation. Also, it is recommended that treated timber be used in the construction process as it can help extend the life span of a crib wall to about 60 years (Schiechtel and Stern, 1997).

Crib walls eliminate any shear action along the bank and bed of the channel as well as withstand any heavy bed load forces that it may encounter. They literally act as a live drop structure that can reduce steep gradients via aggradation. The level bed sections can be adequately maintained once rooting and vegetation become established, and by the time the crib box structure has rotted and deteriorated, the crib wall will have become a soil drop structure reinforced by vegetation.

The major advantage to crib walls is that they are inexpensive due to the fact that local materials can be used in the construction process. Installation is considered to be extremely fast and relatively easy.

### **2.5.3 Vegetation covers**

A vegetation cover is considered to be a complimentary protection measure and is often used in conjunction with other, more conventional stabilization techniques. They provide two basic levels of protection (Thompson and Green, 1994). Their root systems act as a reinforcement agent with the ability to strengthen the stability of the bank. Also, exposed portions of vegetation covers act as a protective barrier by absorbing flow energy.

Generally, vegetation covers are effective at protecting riverbanks at the water surface/riverbank interface and are most common especially in smaller channels that experience relatively low flow velocities. Grasses, shrubs, and trees literally create a shield against waves, surface velocities, and small variations in surface water elevations. Vegetation covers have been known to have the ability to redirect flow from the bank, lower flow velocities near the bank to a level below that which may cause erosive damage, can act as a buffer against debris and ice, can increase bank stability by

promoting deposition, can act as a drain and a means of turning horizontal seepage flow into vertical flow and thus increasing the stability of the bank slope, can effectively deal with flood water, and can provide protection against damage associated with precipitation, runoff, and animals. The roots of trees and shrubs are especially effective at providing strength and reinforcement within the soil in sections where the bank is low. However, in cases where banks are high, the roots cannot reach the toe of the bank, and thus can actually have a negative effect on bank stability especially in cases where erosion and undercutting are common along the lower section of the bank underneath the reach of the roots.

General vegetative cover guidelines indicate that the bank slope must be stable and relatively flat with a horizontal to vertical ratio of more than 2 to 1. Past studies have shown that the effectiveness of grass depends on grass height and density, and that younger grasses are preferred due to their increased sturdiness and resiliency. Shrubs and trees provide much more stability than do grasses, however take much longer to mature and grow. Generally, Bermuda grass has shown to be very effective as have other species of grass that have the ability to create underground rootstocks. Near the upper bank, where effects from precipitation and runoff are high, water-tolerant grasses such as canarygrass, reedgrass, cordgrass, and fescue have been deemed some of the most effective at preventing erosion. Grasses such as cattails, bulrushes, reeds, knotweed, and manna grass have been shown to greatly reduce velocities in shallow flow as well as induce deposition, and thus, have been deemed more effective at the lower bank region where high erosive forces are prevalent. Often times, the selection of plant species to be used as part of a vegetative cover at and around streambanks are tested on a site to site basis and are greatly dependent upon the conditions at hand. It is also important that growth not get out of hand as it can restrict the flow area and lead to more damaging effects downstream.

Providing relatively inexpensive bank protection and controlling flood water, runoff, precipitation, and erosion are the primary functions of vegetation covers. They also have the added benefit of improving the aesthetic value of a site and can help improve overall

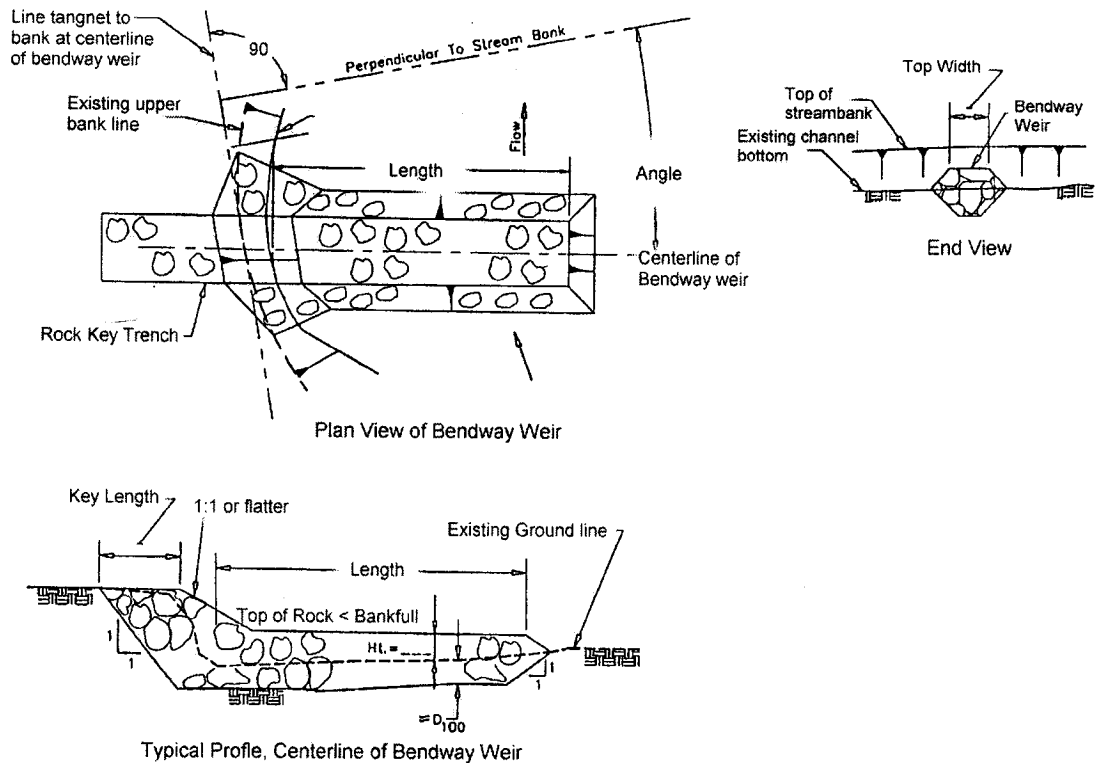
water quality and other environmental aspects, which have become an ever increasing objective in the implementation of bank stabilization methods.

## **2.6 Bendway weirs**

Bendway weir systems are a relatively new concept that was first implemented on river bends on the Mississippi River in 1989 by the USACE (Davinroy, 1990; Derrick et al., 1994; USACE, 1990). They are structures that can best be described as submerged rock sills made of riprap stones that are attached to the bank and placed on the bed of a river. Their purpose is to direct flow away from the outside-bank, to destroy any naturally occurring secondary currents that may develop in a river bend by producing a counteracting current, induce bed material deposition on the outside-bank of the bend, and encourage scouring at the inside-bank. Bendway weirs extend from the concave section of the bank of the river bend into the channel flow and underneath the water surface at an optimum upstream angle of about 30-degrees with respect to the midbank flow line and the tangent to the outside-bank. They have lengths that have been known to range from approximately 120 to 500 m, depending on the conditions and constraints of the channel in question, and are spaced with respect to one another depending on the flow's projected intersection with the bank as it leaves the weir, ranging anywhere between 120 to 490 m (CHL, 1998). The weir height is usually about one  $D_{100}$  stone size above the downstream riffle elevation, and its crest width should be at least  $2xD_{100}$  (USDA, 2000). Cross-sectional side slopes are usually assumed to be the natural angle of repose. The amount of weirs used at a particular section may vary significantly and have been known to range between 4 and 14. Figure 2.17 shows a typical bendway weir.

Because they are submerged, bendway weirs do not compromise or impede navigation through the waterway. They must however have sufficient length and height in order to intercept flow and subsequently redistribute the velocity within the stream, which optimizes, widens, aligns, and safeguards the area available for navigation within the channel. The length of the weir must always extend to at least the thalweg but should not exceed half the channel base width. Also, weirs must be keyed into the stream bank and

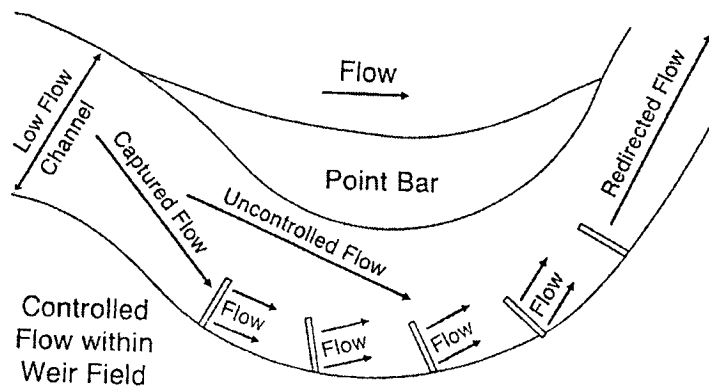
bed. In the case of the riverbank, the structure(s) must extend up to at least one  $D_{100}$  stone size into the base of the channel, must run up the bank a distance of at least 75 percent the low bank height, must be extended into the bank a minimum length of 25 percent the channel base width; in the case of the bed, the structure must be keyed up to a maximum of one  $D_{100}$  stone size or until bedrock (USDA, 2000). Generally, the weir should be level along the length of the structure with a slight rise near the bank.



**Figure 2.17:** Bendway weir (USDA, 2000)

Bendway weirs can also be applied to smaller streams with unrevetted banks. General design guidelines indicate that in these instances, the weirs should be angled anywhere from 5 to 25 degrees facing upstream, spaced approximately 15 to 30 m apart, maintain a height of about 60 cm at the stream end and 120 cm at the bank, and have lengths ranging from about 25 to 50 percent of the total stream floor width (CHL, 1998).

As previously mentioned, the purpose of a bendway weir system is to reduce scouring and erosion in a river bend by redirecting flow away from the bank, retarding flow velocity (especially in the outside-bank region of flow), and by limiting the formation of secondary currents at the outer bank while at the same time aligning the current throughout the entire length of the bend. Unaltered currents at the water surface of a river bend have a tendency to move from the inside portion of the bend to the outer region of the bend. This concentrates the flow as well as increases flow velocity near the outside-bank, which can potentially lead to severe erosion. The presence of a set of bendway weirs allows flow to run over the weir such that it becomes redirected in the direction downstream perpendicular to the length of the weir away from the bank (fig. 2.18). This induced redirection of flow subsequently negates naturally formed secondary currents.



**Figure 2.18:** Bendway weir theory (Derrick, 1997)

Other benefits include better channel alignment, flow direction predictability at and downstream of the bend, the ability to blend in with the environment, the ability to work well with other stabilization methods (particularly bioengineered techniques such as willow posts), effectiveness under high flow and high energy conditions, low cost relative to other traditional stabilization techniques, increased outside-bank stability due to deposition at the toe-end of the structure, uniform surface water velocities, and its ability to shift the thalweg towards the center of the channel. The added environmental benefit to bendway weirs over the years has been its ability to improve aquatic and terrestrial

habitat. Increases in native wildlife populations and native vegetation growth have been observed in many bendway weir project locations.

It is important to note that there is no one set of written rules and guidelines for the design and implementation of bendway weirs. Every instance in which bendway weir systems are applied to waterways must be evaluated individually and not be reliant on past bendway weir systems on other channels and bends. It is also important to use bendway weirs only on channels with: (i) a stable bed and stable bend entrance and exit conditions, (ii) channels whose width at base flow is no less than 6 m and a bend radius/stream width ratio no less than 6, (iii) streams that are accessible by machinery and equipment for weir installation, (iv) areas with adequate construction material, and (v) channels with minimal disturbances to the upper bank.

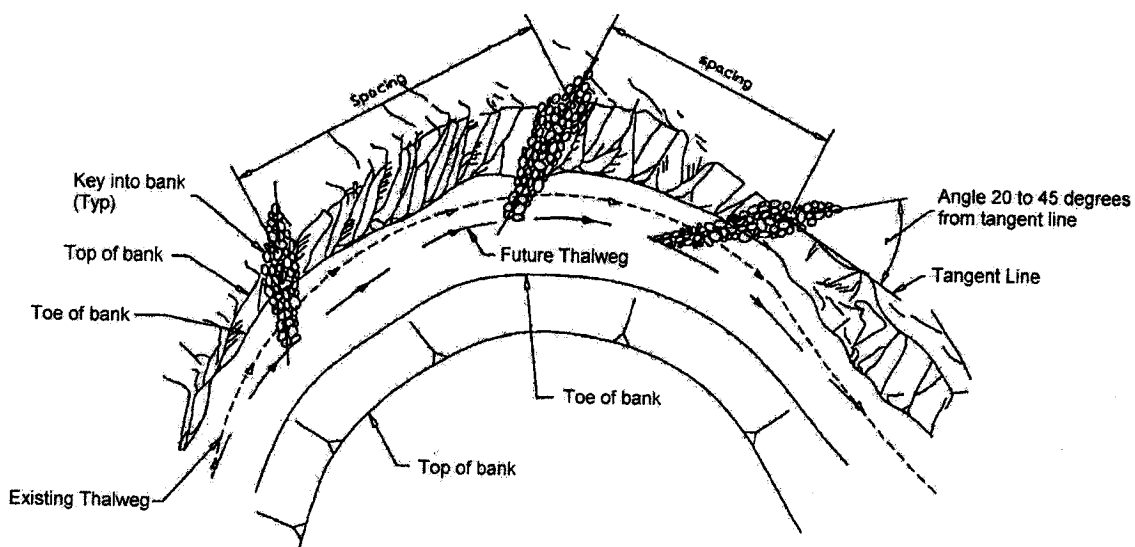
It is very important to inspect bendway weirs following installation and after any major event at the site, such as significant floods or rainfalls. Scouring at the toe of a weir is not uncommon, and the sliding of stones into the scour hole compensates for any potential problems.

## **2.7 Stream barbs**

As mentioned in chapter 1, stream barbs are structures that are very similar to bendway weirs. They are made of rock, extend from the outside-bank of a bend at an upstream angle into the flow of the channel, and have a low-profile such that they are completely submerged under 'channel forming' flow conditions (a 1.5 year flow event according to the USDA NRCS). The design of stream barbs incorporates a submerged weir segment, which will be discussed in detail later, whose purpose is to redirect flow and break up the velocity gradient near the outside-bank region of a channel bend. This leads to decreased sediment transport, altered sediment transport and deposition patterns, an increase in scour of point and lateral bars, and a channel width to depth ratio reduction. Flow is forced over the weir segment into an artificial hydraulic jump, which flattens the water slope upstream of the barb. Like flow over a bendway weir, flow over a stream barb

becomes redirected normal to the structure, away from the bank. Because the weir segment is constantly submerged, it continually disrupts the velocity gradient near the channel base, which in turn, decreases bed shear stress and limits the amount of sediment transport. As a result of the effects of river barbs, an increased amount of deposition occurs at and near the structure. This is due to the formation of eddies caused by the local flattening of the upstream water slope as well as eddies created by the hydraulic jump and the redirection of flow at the barb.

The plan view of a general barb system at a stream bend is shown in fig. 2.19.



**Figure 2.19:** River barb system plan view (Kinney, 2000)

### 2.7.1 Materials

Barbs consist of stones not unlike those used for riprap blankets. The most suitable stones must be durable, dense, and must sustain weathering, freezing, thawing, and severe climate variations. Smaller rock fragments are also used in the construction of river barbs. Suitable fragment shapes are classified as angular to subrounded whose smallest size must be at least one-third that of the largest fragment size. All stones and fragments in question are recommended to have a specific gravity of at least 2.5 (USDA, 1999, 2001). Large rock with a diameter of at least 90 cm are the preferred stone size of choice

due to their low cost and ease in installation. They are however often times unavailable, and the NRCS recommends that suitable stone size be determined via the Far West States-Lane Method for standard riprap stone size under turbulent flow conditions (USDA, 1999, 2001). Using this method, median riprap stone size can be determined, and using the following equations, suitable barb stone size can be calculated.

$$D_{50\text{-barb}} = 2 \times D_{50\text{-riprap}} \quad \text{Eq. 2.1}$$

$$D_{100\text{-barb}} = 2 \times D_{50\text{-barb}} \quad \text{Eq. 2.2}$$

$$D_{\text{minimum}} = 0.75 \times D_{50\text{-riprap}} \quad \text{Eq. 2.3}$$

The NRCS states that when the ratio between bend radius and channel width is less than six, the barb rock size becomes very large, resulting in a conservative design.

The weir section of the barb should contain well graded rock ranging in size from  $D_{50}$  to  $D_{100}$ . Smaller diameter stones should be used as the primary material installed into the bank key while the larger stones should be used in the exposed weir section. The NRCS warns against using traditional riprap stone sizing procedures as they usually result in smaller than desired rock sizes.

It is important to incorporate such local aspects as the stream size, maximum flow depth, planform, entrenchment, and potential ice or debris effects in order to make any necessary adjustments in barb stone sizes.

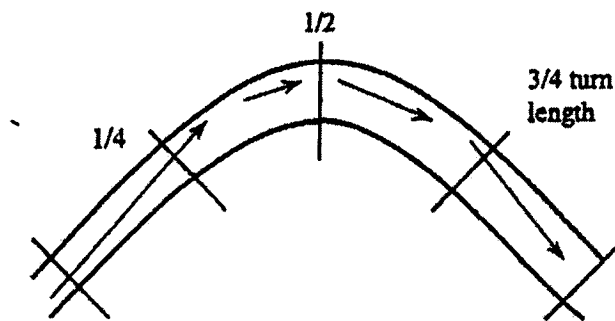
## **2.7.2 General design guidelines**

### *2.7.2.1 Location and quantity*

Barbs are usually installed on the outer bank of a river bend in a section where the thalweg is closer to the streambank than it is to the middle of the channel. If the latter case is encountered, the NRCS recommends that another stabilization technique be used

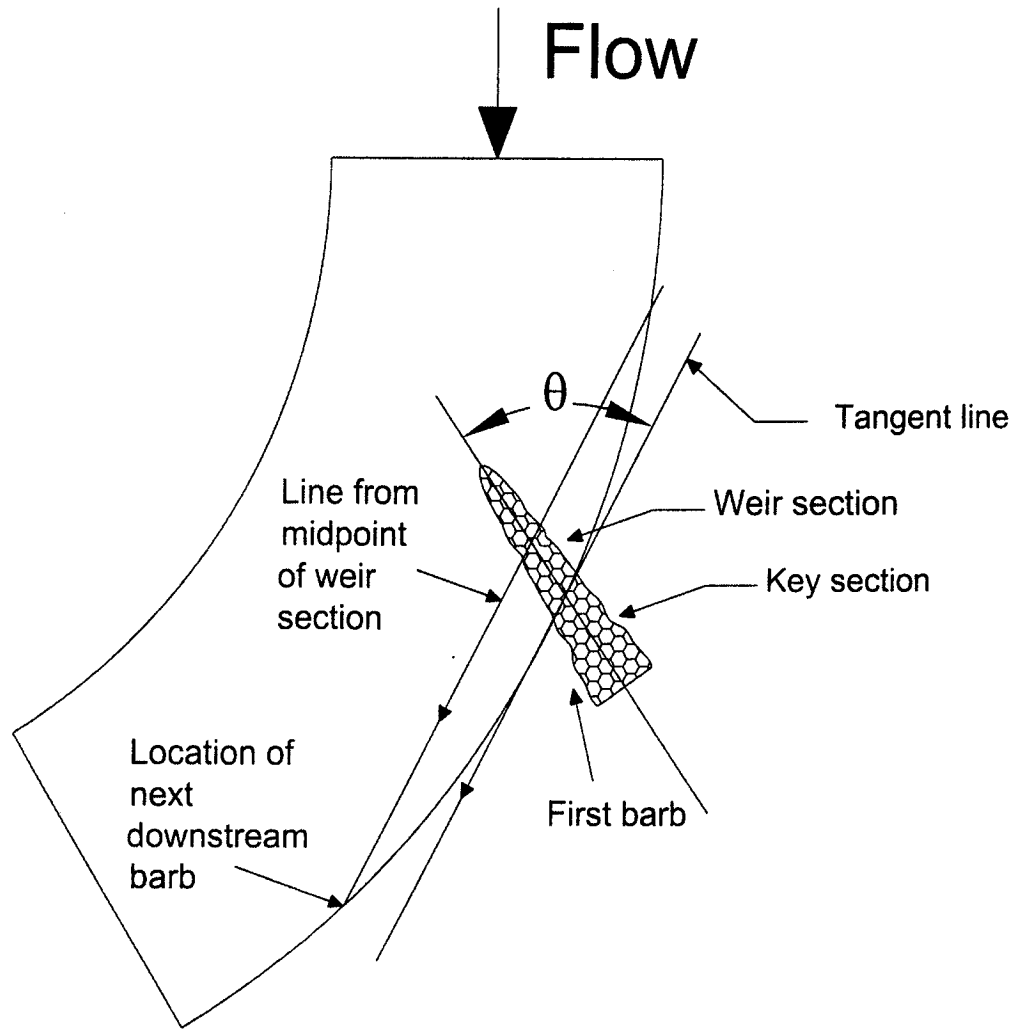
for streambank protection. It should also be noted that barbs cannot protect banks against erosion caused by rapid drawdown or mass slope failure.

The number of barbs in each section is dependent upon a number of factors such as spacing, length of the eroding bend section, channel geometry, bedload, flow characteristics, and the overall desired results. The first barb is usually placed at an area where high flow erosion is first experienced upstream of the actual bend section. The NRCS recommends that the furthest downstream that a barb be placed is three-fourths the length of the entire turn (fig. 2.20).

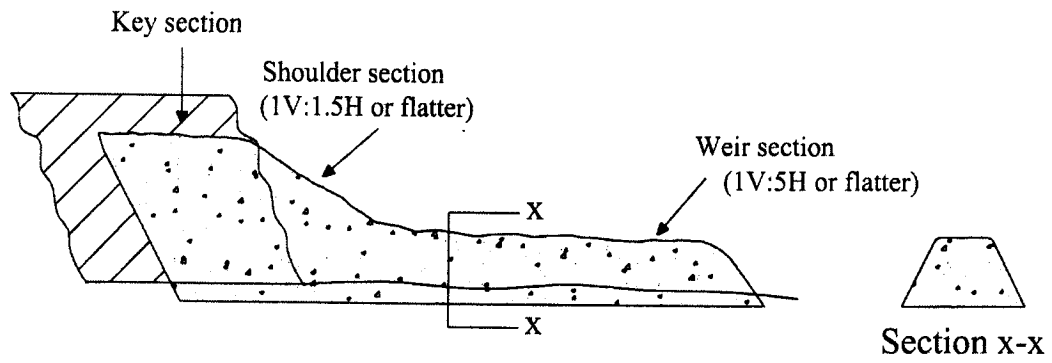


**Figure 2.20:** Stream barb schematic (USDA, 1999)

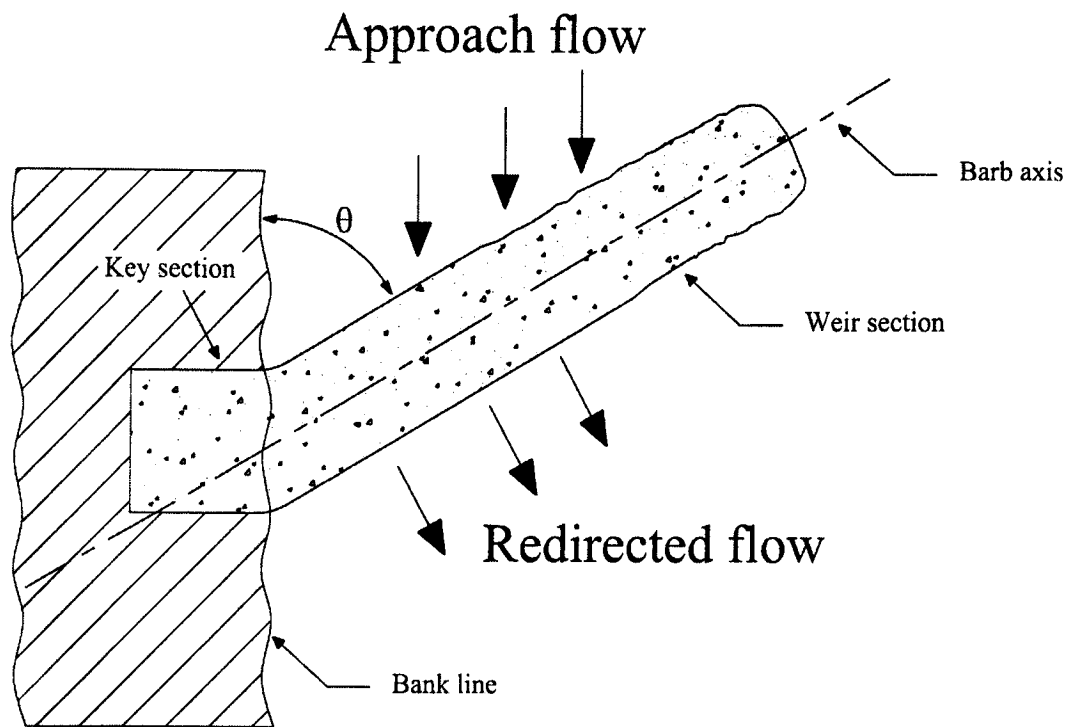
It is critically important that barbs upstream of the bend be placed properly so as to effectively re-direct the angle of flow away from the bank. Proper upstream placement subsequently limits the amount of barbs that are needed in the downstream reach of the channel. Figure 2.21 shows the typical barb placement methodology within a stream bend (which will be discussed in detail in chapter 3), while figure 2.22 shows the basic overview of a general barb structure.



**Figure 2.21:** Barb placement methodology (Matsuura and Townsend, 2004a)



Side View



Plan View

**Figure 2.22:** Individual barb structure (Matsuura and Townsend, 2004c)

### 2.7.2.2 Barb height

The main factor in determining the height of river barbs is the bankfull flow elevation (elevation of channel-forming flow discharge), which is usually equal to or slightly higher than the ordinary high water level. Most channels maintain flow levels below that which is required to visually determine this particular height, and for ungaged streams especially, the bankfull flow can be determined from bed features or the amount (or lack thereof) of vegetation in the area in question.

The slope of the barb is nearly flat and is recommended to be no greater than 1V:5H, with the slope always being positive. If the slope of the weir section becomes negative at any time, the flow then becomes forced towards the bank. This usually occurs when the slope of the barb is too flat, which often leads to a loss of rocks at the top of the structure, resulting in a negative slope. The NRCS recommended height design equation is:

$$h=1/3y_0 \text{ to } 1/2y_0 \qquad \text{Eq. 2.4}$$

(where  $y_0$ =average channel-forming flow depth)

The relative height of barbs when compared to one another, especially barbs in succession, is a very important design factor. For a set of barbs, the height difference between successive structures should represent that of the energy grade line of the stream, or a close proximity to it. It is important to note that the higher the barbs, the more scour is experienced. Conversely, lower barbs experience a reduction in scour depths. This is due to the fact that barbs that approach the bankfull flow elevation experience greater flow convergence, which leads to larger scour depths.

### 2.7.2.3 Barb spacing and angle

If successive barbs are not spaced adequately, flow can cut between the spacing and directly erode the bank. Since flow is directed normal to the barb, the optimum placement of the next barb downstream is at a location that intercepts the flow near the center of the

barb prior to the flow attacking the outside-bank. Vector modeling can often times illustrate the direction of flow lines after being influenced by the presence of a river barb. Designers should also keep in mind that, according to NRCS, barb structures affect flow patterns downstream at a distance of about 5 to 10 times the barb's perpendicular projection into the stream, and thus should be designed for accordingly. However, there are no set guidelines as to the spacing of barbs as it is very much dependent on each individual situation.

Barbs are directed at an angle upstream of about 20 to 45 degrees with respect to the tangent line of bank to the centerline of the barb, depending on what is most effective. Tighter angles closer to 20 degrees are usually reserved for situations when the desired result is to maintain the thalweg near the outside-bank.

#### *2.7.2.4 Length and width*

The maximum effective length of the barb is recommended not to exceed 25 percent of the bankfull flow width. Anything longer would result in a change to the stream's meander pattern as well as to the flow affecting the opposite bank. The minimum effective barb length should be greater than 10 percent of the channel-forming flow width. As previously mentioned, barbs are used when the thalweg is close to the outer bank. In order for barbs to be effective, they must cross the thalweg. Barbs that do not cross the thalweg only affect secondary and near-bank currents and not the primary flow pattern of the stream. Thus, it is essential, that in order to achieve its full potential, river barbs must cross the thalweg. Cases where the thalweg is closer to the center of the stream mean that longer barbs must be used, which in turn could potentially change the meander pattern of the stream. The NRCS recommends that if the thalweg is far from the riverbank, other stabilization techniques should be used for the reason that short barbs would not be fully effective, while longer barbs would cause more harm than good.

Barb width can range from the  $D_{100}$  barb stone size to  $3xD_{100}$ . Often times, the width of the barb does not exceed 2 rock diameters, as wider barbs would not be economically

feasible nor would it add much benefit. The only case in which a wider barb would be used is if deep scour holes are expected downstream. Wider barbs have the ability to create uniform, stronger hydraulic jumps, which would reduce the depth of scour holes past the structure. The width of the outer tip of the barb can be as small as one rock diameter.

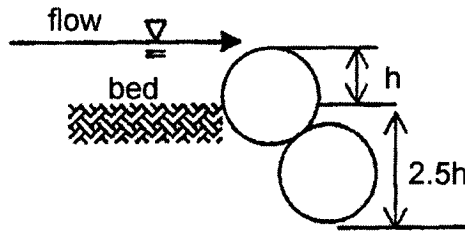
#### *2.7.2.5 Barb profile*

Barb side slopes are usually based primarily on the natural angle of repose for the stones being used, and is often times in the order from anywhere between 1V:1H to 1V:2H from the exposed section to the bank key. The bank key must have a height sufficient enough that water will not flow around the structure, which would lead to erosion at the bank behind it. If the bank section is often overtopped with flow, the key must extend further into the bank than normally designed for. The weir section of the barb must always be lower than the bankfull flow elevation.

#### *2.7.2.6 Bank and bed key*

The bank key is a protective device used to combat flanking caused by erosion in the near bank. The length of the bank key is usually around  $4xD_{100}$  or half the length of the barb. In cases of longer barbs, the key length is about 20 percent the total barb length. Another design requirement is that the bank key must exceed 1.5 times the height of the bank, or 2.4 m, whichever is larger.

The depth of the bed key is calculated using the depth of expected local scour near the tip of the barb. Scouring downstream of the barb may occur if the bed key is too shallow or non-existent, leading to the sliding of barb stones into the scour hole, and thus compromising the structure of the barb. The bed key is usually placed at a depth of about  $D_{100}$ .



**Figure 2.23:** Bed key depth (USDA, 1999)

Figure 2.23 is a diagram that can assist in calculating expected local scouring. For beds made of gravel or cobble, the NRCS estimates expected scouring as  $2.5h$ , where  $h$  is the height of exposed rock relative to the bed elevation. For beds consisting of sand, expected scouring is 3 to 3.5 times  $h$ . Again, decreasing barb height decreases scour hole depths, as lower barbs reduce flow convergence.

#### *2.7.2.7 Barb construction*

It is recommended that barbs be constructed during the low flow season. This minimizes the amount of disturbance to the stream and the bank during installation. The NRCS points out two major construction warnings:

- (i) Rocks should not be end-dumped.
- (ii) Construction of the barbs should always begin upstream and progress sequentially downstream.

Through studies conducted by the United States Environmental Protection Agency (USEPA), it was estimated that on average, the cost of a single barb structure can range between \$2000 and \$3000 (USEPA, 1998)

### 2.7.3 Environmental impact

An added benefit to a stream barb system is its ability to enhance aquatic habitat and water quality. Stabilizing the outside streambank of bend sections can potentially create new or improve upon existing habitats by improving a stream's structural diversity and by creating ideal conditions for the establishment of riparian vegetation. Recent studies that have involved the use of stream barbs in conjunction with the planting of willow posts have led to extremely positive results (USEPA, 1998). Barbs have the ability to direct high flow velocities away from the outside-bank, which in turn leads to movement of the thalweg away from the bank. Subsequent reduction of scouring and erosion at the bends' outside-bank creates what is known as slack water. Locally, scouring occurs at the tip of the barb while sediment trapping and collection occurs behind it, eventually leading to the formation of a bar behind the barb. Willow posts planted on the upper and lower bank as well as on the bar behind the barb have been shown to effectively stabilize the bank as well as the bar. The barb/willow post configuration is often times followed up by the planting of other vegetation species (USEPA, 1998).

This type of barb/bioengineered approach acts as a two-stage stabilization method. Stream barbs prevent sediment transport and provide an immediate means of bank protection while vegetation covers and willow posts act as a long-term stabilizing influence. Numerous studies and programs conducted by the USEPA have shown that in instances where barbs were used in conjunction with willow posts, a deeper, narrower channel was formed under low-flow conditions. The creation of deeper pools and the reduction of water surface area resulted in maintainable water temperatures, which led to a significant increase in the survivability of fish species in these rivers. The barb structures also restored channel diversity and the stream's morphology to a desired, more natural appearance, which are also key factors to an improved aquatic habitat.

Barbs also have the ability to trap significant amounts of organic material. High levels of organics in the stream, especially near the bank, improves channel complexity, benefits plantings, allows for an increase in organic material and natural nutrient input in the area,

and creates more habitat. An increase in habitat not only restores the area but also provides more canopy cover and shading for the regulation of stream temperatures.

## **2.8 Overview**

Streambank stabilization is an important aspect in maintaining the navigability and sustainability of a natural watercourse and its surrounding environment. Controlling erosion and scouring, managing floods, runoff, and precipitation, and dealing with the effects of natural and human activities are just some of the more important factors that a successful stabilization project must accomplish. Furthermore, doing these things in a cost effective and environmentally safe manner have become equally as important as actually achieving the physical goals of the project. The use of traditional stabilization and protection applications is still commonplace. However, due to the success of new bank protection devices, combined with the advantages of bioengineered techniques, it is this form of streambank stabilization methods that are now being encouraged. Field tests and actual projects show however that most of if not all bioengineered methods should not be used alone for the purpose of stabilizing and protecting streambanks. The use of bendway weirs, and more recently stream barbs, has been shown to be very successful in complementing most bioengineered techniques. Structures such as the stream barb immediately and effectively control scouring and erosion. The bioengineered method, on the other hand, provides long-term stabilization, while at the same time creating streambank diversity and an aesthetically pleasing environment.

## Chapter Three

### Experimental Settings, Boundary Conditions, and Equipment

#### 3.1 Introduction

To remain within the overall budget for the study and to reduce construction costs and time, the experimental channel used in the testing program was designed to utilize existing channel 'straight' sections from an earlier similar-type study performed in the hydraulics laboratory. The existing straights were used to construct the 'entrance straight' and 'exit straight' sections of the channel. This meant that the only additional section required for the present study were separate (and interchangeable) 90-degree and 135-degree bend sections.

#### 3.2 Model channel

The experimental channel used in this study is comprised of three separate sections made from acrylic sheet: (i) a 460 mm wide x 254 mm deep x 9.75 m long 'entrance straight' section, (ii) a 460 mm wide x 406 mm deep x 2.44 m long 'exit straight' section, and (iii) separate (and interchangeable) 90-degree and 135-degree bend sections, each 460 mm wide x 406 mm deep, connecting (i) to (ii) (fig. 3.1 (a) & (b)). A 900 mm radius of curvature for the bend sections, dictated by space limitations in the laboratory and by the width of the adjustable slope platform on which the entire channel rested, was utilized in the study. The bend sections had an inner radius of 0.7 m, and outer radius of 1.13 m, and an invert set 152 mm below that of the 'entrance straight' section, which continued for the remainder of the channel. The 152 mm deeper bend and 'exit straight' sections allowed for the placement of 460 mm x 152 mm deep beds of sand therein. At the start of each test, the surfaces of these sand beds were carefully leveled to be continuous with the invert of the channel 'entrance straight'. Thus, when assembled, the channel was

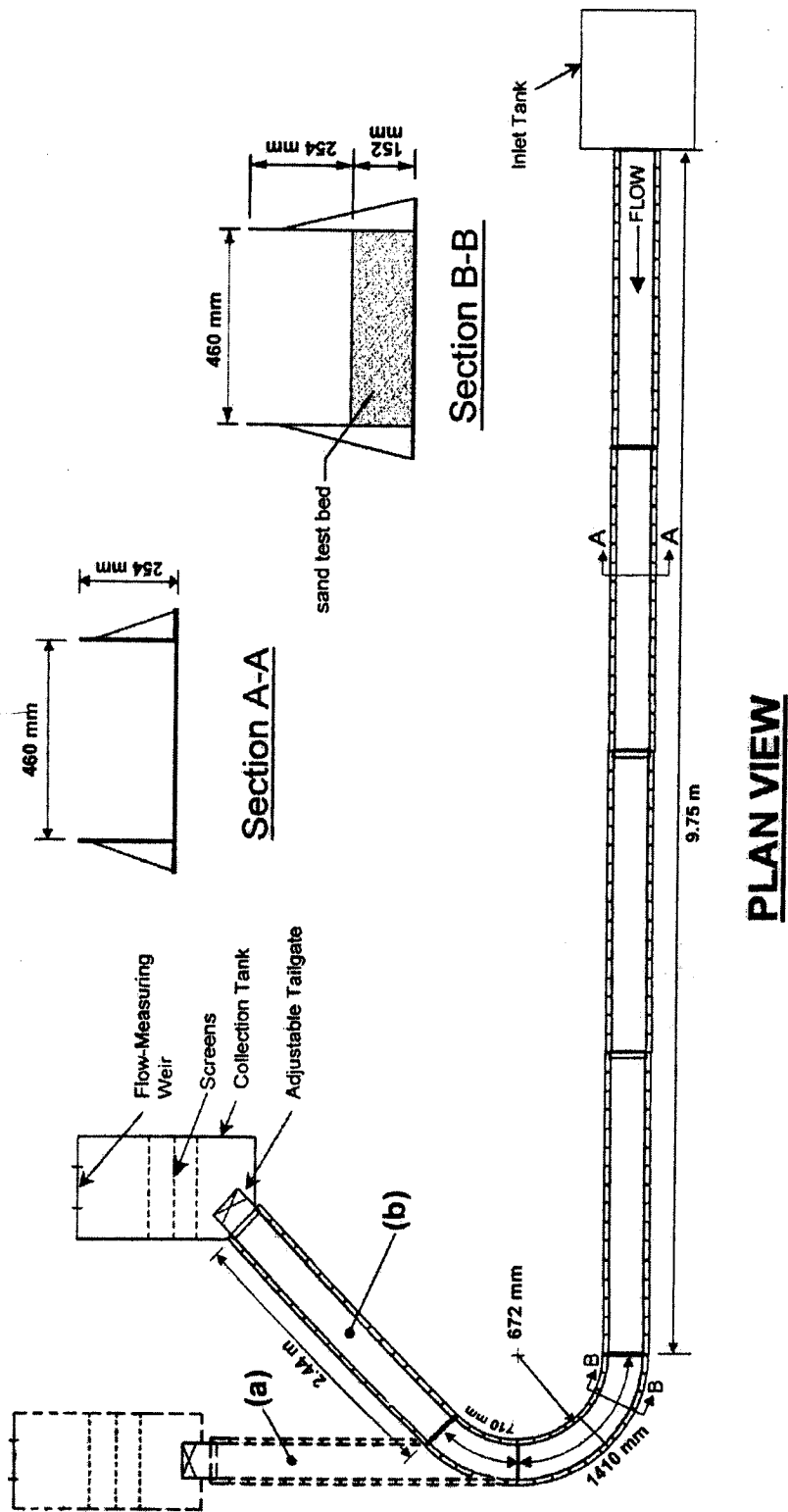


Figure 3.1: (a) 90-degree bend arrangement; (b) 135-degree bend arrangement (Matsuura and Townsend, 2004b)

comprised of a fixed-bed 'entrance straight' followed by mobile-bed bend and exit straights.

The final 2.44 m length of the entrance section is surfaced with a thin layer of the same sand used in the test sections so as to avoid an abrupt change from 'smooth' to 'rough' surfaces as the flow leaves (i) and enters (iii), (fig. 3.2).

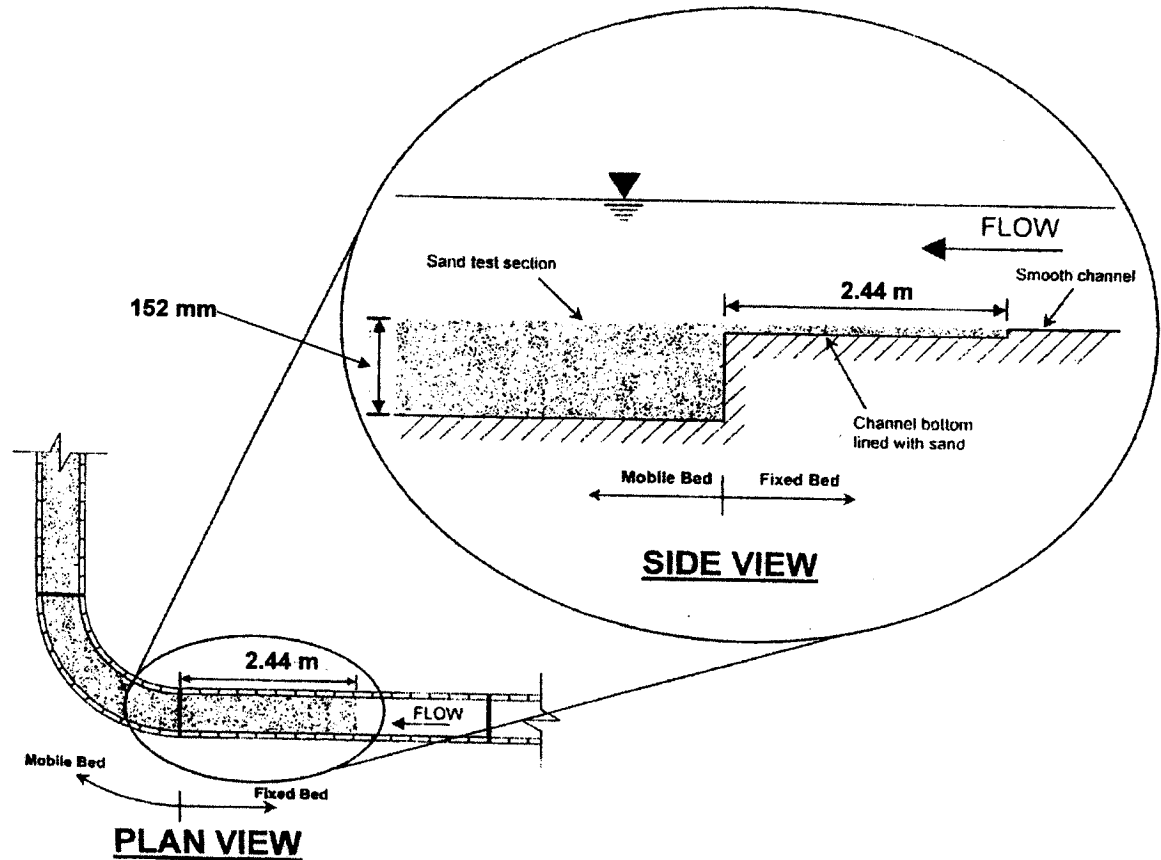
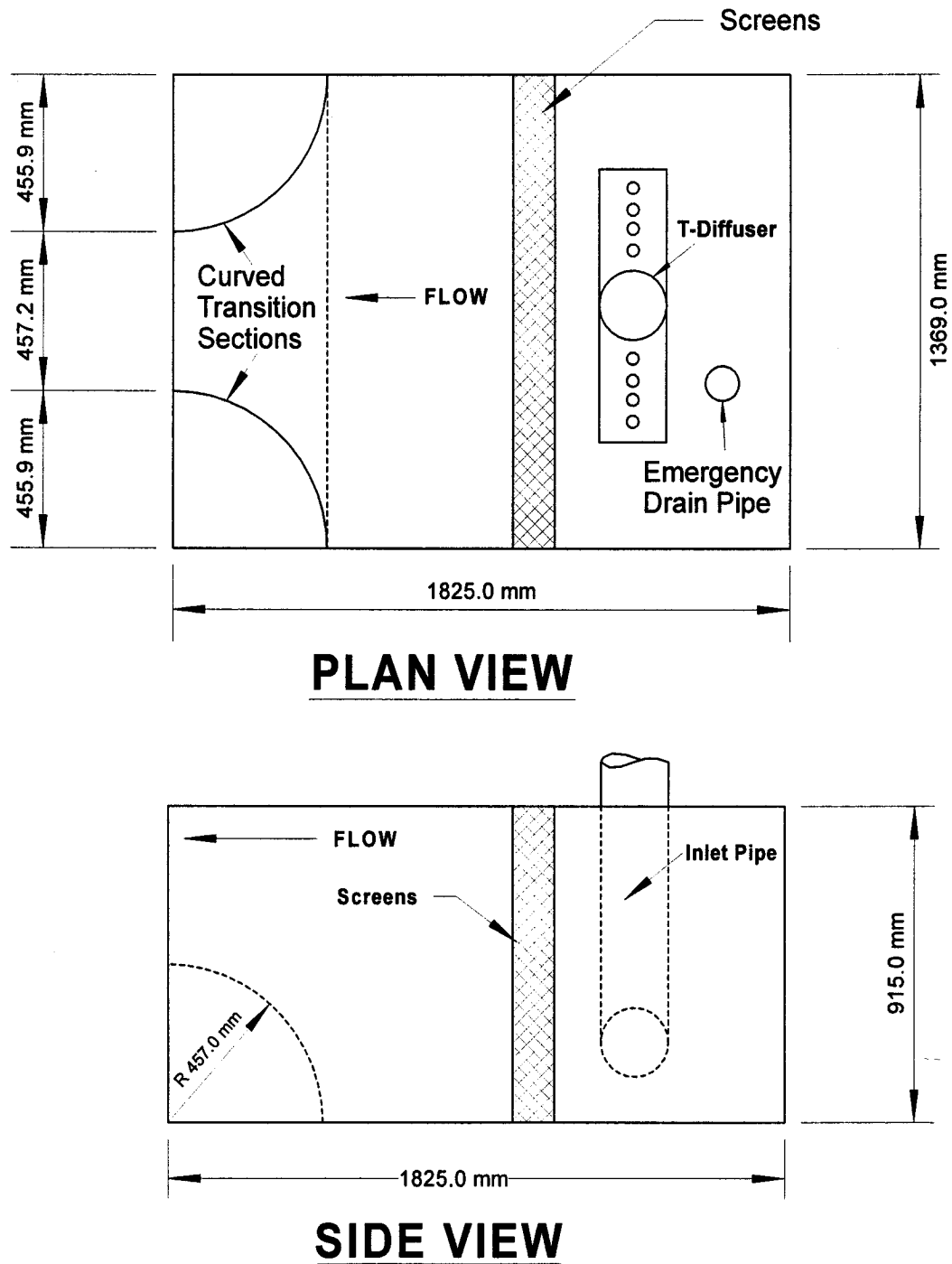


Figure 3.2: Entrance section invert

Water was supplied to the channel via a large head tank (fig. 3.3). A 0.1 m diameter pipe supplies water to the tank from the laboratory's 380 m<sup>3</sup> underground storage reservoir through the pump delivery system. The latter is a constant-rotational speed (870 rpm) centrifugal pump able to maintain a maximum flow of 0.315 m<sup>3</sup>/s at a total head of 18.28 m. A perforated 'T' diffuser is attached to the end of the water distribution pipe so as to create evenly-distributed flow and minimal large-scale turbulence structures within the

tank. Vertical mesh screens are installed near the middle of the entrance head tank to help eliminate large-scale turbulence structures prior to the flow entering the channel.



**Figure 3.3:** Entrance head tank

A smooth-curved transition section between the bottom of the tank and its outlet helps ensure a smooth acceleration of steady state flow into the channel. Initial velocity measurements performed on the entrance flow indicated that the maximum velocity filament was located near the channel invert. To correct this, a 1 cm-diameter plastic tube was placed along the bottom of the channel entrance normal to the flow direction. This resulted in movement of maximum velocity filament to near the free surface. Follow-up tests showed that flows in the downstream half of the entrance straight exhibited near-uniform cross-stream velocity distributions.

Connected to the outlet of the exit straight channel section located downstream of the bend section is an adjustable tailgate, which controls flow depths throughout the entire channel. Upon exiting the channel, the flow discharges to a sediment trap and from there into a large collection tank. The sediment trap consists of a metal rectangular open-topped box lined with a 0.425 mm #40 mesh screen. A 90-degree sharp edged weir is installed at the outlet of the tank to measure the discharge rate. Two baffle-plates placed across the tank help reduce turbulence therein.

The entire channel rests on an adjustable-slope platform to permit adjustment of the former's longitudinal slope.

### **3.3 Bed-elevation measurements**

Channel bed point-elevations were recorded using a *Disto<sup>TM</sup> Pro<sup>4</sup>a* laser meter manufactured by Leica Geosystems. The instrument was securely fastened to a portable carriage which traversed the entire channel in both the longitudinal and transverse directions. The instrument is accurate to  $\pm 0.8$  mm. Bed surface measurements were recorded prior to each test to provide initial points of reference. Elevations recorded after each test run were compared to the initial readings to provide accurate scour and deposition measurements throughout the test sections.

### 3.4 Velocity measurement

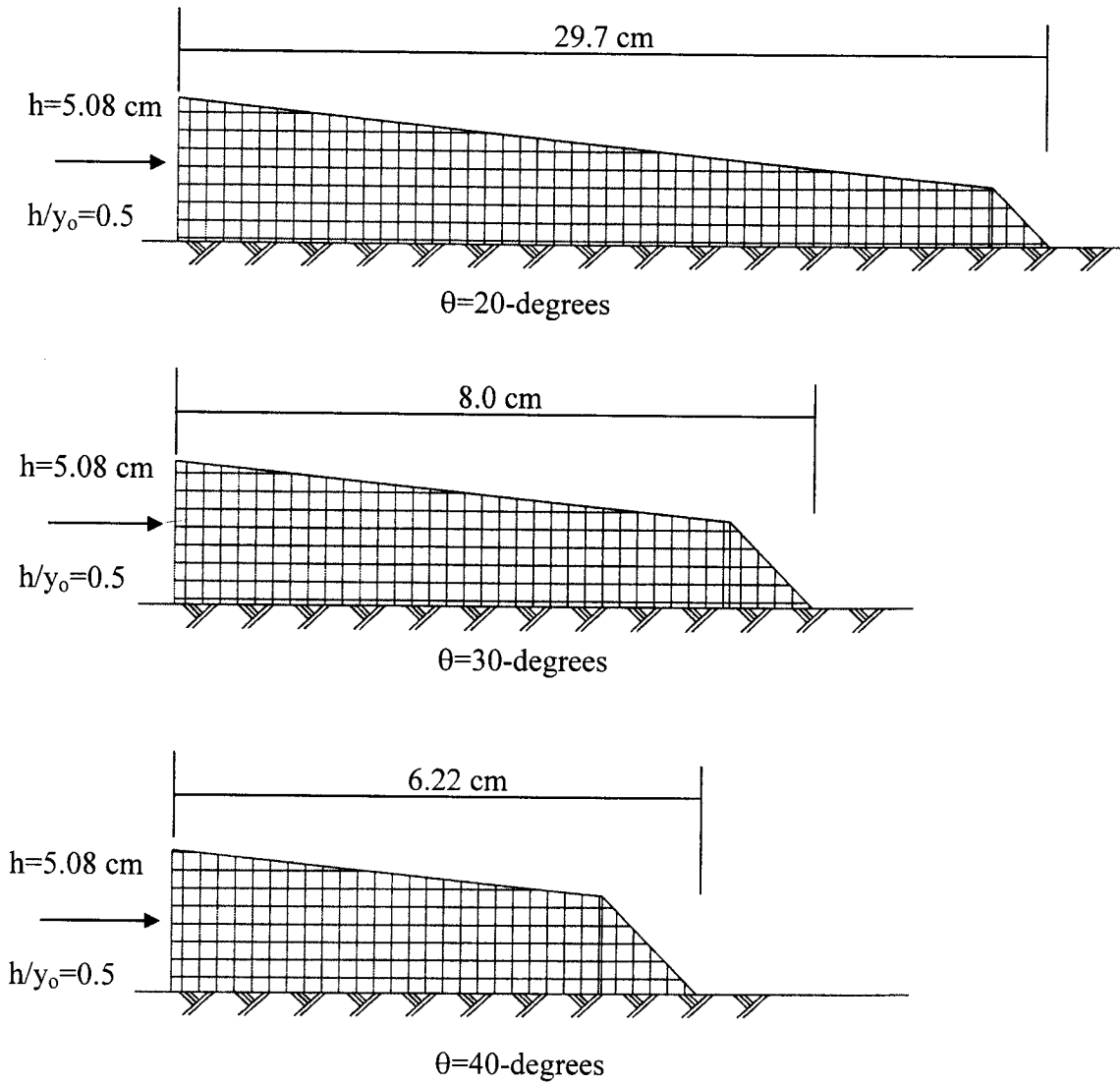
A 'Streamflow-Series' propeller-type velocity meter manufactured by Nixon Instrumentations Ltd. was used to measure flow velocity. This instrument, which is accurate within the range  $5.0 \leq V \leq 70$  cm/s, was used primarily during preliminary flow tests to determine the correct settings and hydraulic conditions for the planned testing program. This particular means of velocity measurement was also used to check the accuracy of flow measurements by the 90-degree sharp edged weir on the outlet tank.

### 3.5 Barbs

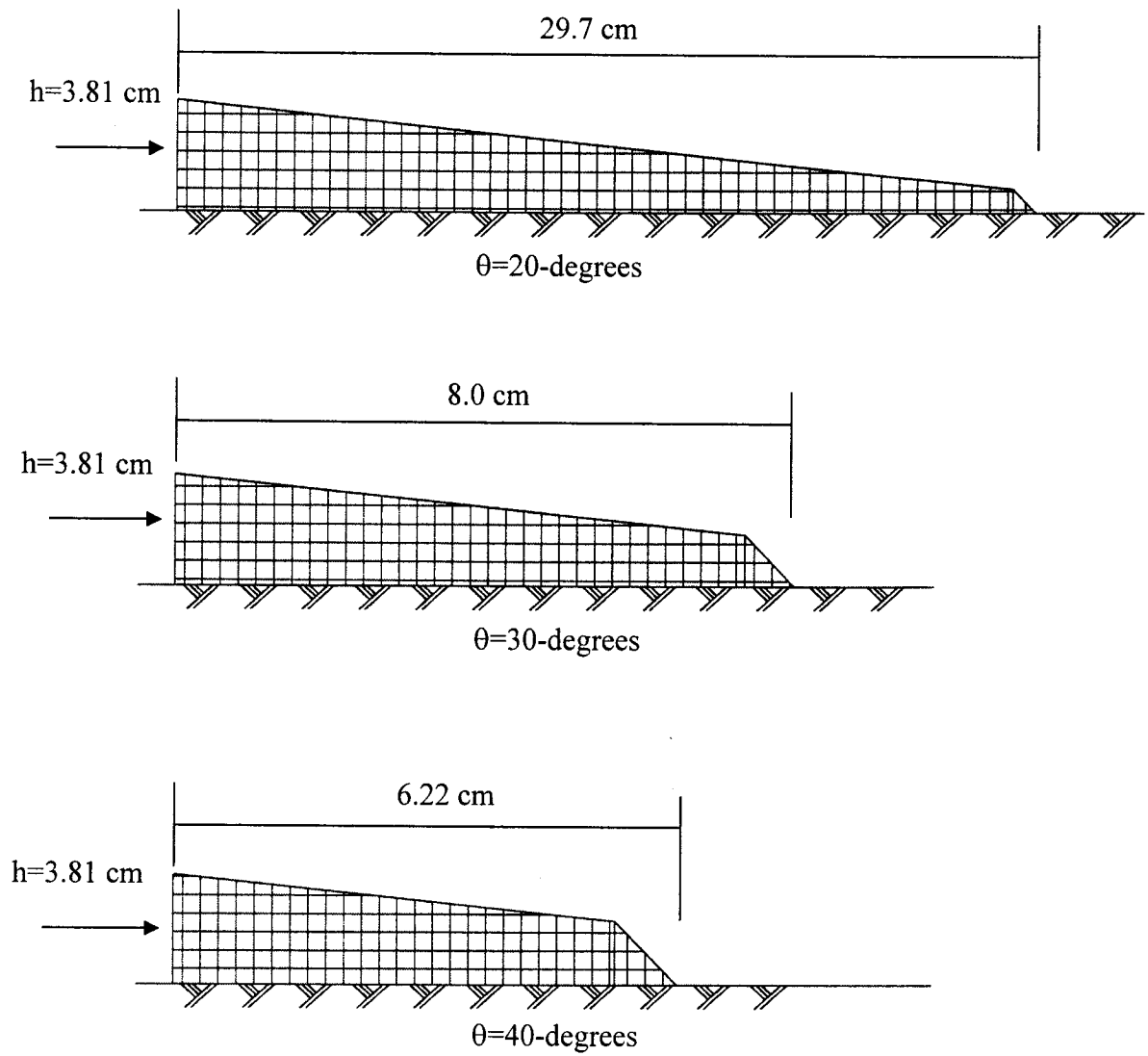
Based on the past experience of USACE personnel, regarding suitable material for the construction of model groin-like structures, wire mesh was used to construct the model barb structures (R.D., 2001<sup>1</sup>). Mesh sheets with various size openings were shaped to conform with the model barb dimensions selected (fig. 3.4 & 3.5). The model structures were initially tested in separate experiments to determine their effectiveness under pre-determined flow conditions. Given the dimensions of the experimental channel, the most realistic mesh size was determined to be 2.743 mm. Preliminary tests with the presence of barb structures within the test sections were conducted to determine the most effective and feasible cross-sectional characteristics of the model barbs. These tests showed that in cases where barb width exceeded 10 mm, severe local scouring around the structure would occur. This resulted in the use of two pieces of mesh, identically shaped and cut, and pieced together using plastic ties. This option provided the structure with suitable flow resistance, while at the same time avoiding any unwanted local scouring. This particular design also adequately simulated openings that would resemble those found in barbs in the field. Various tests using wider structures as well as structures with a more trapezoidal shaped cross section resulted in severe local scouring that rendered model performance obsolete.

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<sup>1</sup> D.Ronald Townsend personal communication with Robert Davinroy, (2001), Potomology Section, US Army Corps of Engineers, St.Louis, M.O.



**Figure 3.4:** Model barbs ( $h=5.08\text{ cm}$ )



**Figure 3.5:** Model barbs ( $h=3.81\text{ cm}$ )

### 3.5.1 Placement procedure and experimental conditions

Tests conducted with the presence of barbs include:

- (i) Two different channel width:flow depth ( $W/y_0$ ) ratios,
- (ii) Two different barb height:flow depth ( $h/y_0$ ) ratios,
- (iii) Three different barb alignment angles ( $\theta$ ), and
- (iv) Four different types of barb groups (A, B, C, and D).

The barb groups tested in the experimental program were largely defined by the location ( $L$ ) of the first barb in the group, relative to the bend entrance section. In the aforementioned NRCS field-testing program, first barbs were located “just upstream of the bank area that is first impacted by flood flow erosion”, (USDA 1999).

Two different uniform flow depths ( $W/y_0=4.5$  and  $6.0$ ) under steady state conditions were used throughout the study to simulate a reasonably narrow channel as well as to establish a variety of hydraulic conditions. To avoid excessive scouring of the channel bed in the immediate vicinity of barbs, the height ( $h$ ) of the barb (at the bank) should not exceed 50% of a channel’s bank-full depth, (USDA 1999). Accordingly, all experiments involving barbs were performed using four different barb groupings (A, B, C, and D), set at alignment angles,  $\theta = 20^\circ, 30^\circ$  and  $40^\circ$ , and for  $h/y_0=0.5$ . These experiments were then repeated for  $h/y_0=0.375$  to investigate the influence of  $h$  on barb system performance.

During the preliminary ‘reference’ tests that preceded the main testing program, it was observed that bank erosion due to secondary currents commenced in the region  $2.67 < L/W < 3.08$ . Accordingly, in the testing program involving different barb groups, the key of the first barb of groups A and B were respectively located at the extremes of the above range to examine the relative effectiveness of placing first barbs at these locations. In the cases of groups C and D, first barbs were installed at  $L/W=1.78$  and  $1.33$  respectively to determine if placing first barbs further upstream of those in A groups would result in enhanced barb system performance.

The positions of all other barbs in a group were determined as follows (Kinney, 2002): a line from the mid-point of the first barb is drawn parallel to the tangent to the outside-bank at the key of the first barb location. The point downstream where this line intercepts the bank is where the key of the next barb is located (fig. 2.21). This location procedure is repeated until the projected line from the mid-point of the last barb in a group is essentially parallel with the channel banks. An effective length of approximately 22.22% of the channel width was taken as a constant for all experiments.

The crest of the weir section of a barb should have a 10-degree (downward) slope. However, in order to satisfy the conditions of the aforementioned effective length criteria, certain barb groups required that the crest slope end suddenly. In order to avoid an abrupt change of the crest slope from 10-degrees to 90-degrees (i.e. a vertical drop), the structures were modified to incorporate a 45-degree end slope near the toe. The bed keys of the model barbs were deep enough to avoid undermining action.

The various barb locations for each barb grouping tested in the 90-degree and 135-degree bends are listed in tables 3.1 and 3.2, respectively. The rationale for testing only 2 types of barb groups in the larger bend is given in chapter 5.

**Table 3.1:** Barb locations in the 90-degree bend

$\theta$	Barb Group	Barb Location (L/W)			
		Barb 1	Barb 2	Barb 3	Barb 4
20	A	2.67	3.56	4.82	N/A
	B	3.08	4.01	4.98	N/A
	C	1.78	3.14	4.40	N/A
	D	1.33	2.31	3.27	4.40
30	A	2.67	3.56	4.82	N/A
	B	3.08	4.08	5.07	N/A
	C	1.78	2.75	3.61	4.57
	D	1.33	2.17	3.08	4.08
40	A	2.67	3.39	4.22	N/A
	B	3.08	3.87	4.68	N/A
	C	1.78	2.58	3.36	4.23
	D	1.33	2.11	2.94	3.78

\*Note: L/W>3.90 indicates that the key of the barb is in the last straight exit section, not the bend.

**Table 3.2:** Barb locations in the 135-degree bend

$\theta$	Barb System	Barb Location (L/W)					
		Barb 1	Barb 2	Barb 3	Barb 4	Barb 5	Barb 6
20	B	3.08	3.98	4.82	5.72	7.20	N/A
	C	1.78	3.14	4.06	4.95	5.87	N/A
30	B	3.08	4.01	4.82	5.57	6.87	N/A
	C	1.78	2.75	3.61	4.41	5.29	6.29
40	B	3.08	3.87	4.62	5.24	6.20	N/A
	C	1.78	2.58	3.36	4.18	5.07	5.98

\*Note:  $L/W > 5.87$  indicates that the key of the barb is in the last straight exit section, not the bend.

### 3.5.2 Barb Sizes

In order to satisfy both the effective barb length and maximum height requirements, individual barbs associated with the different alignment angles required different lengthed structures. The barb sizes corresponding to their upstream angle are shown in figs. 3.4 and 3.5. Length-wise, structures with a 20-degree angle are the longest, whereas those used in a 40-degree alignment are the shortest. Prior to testing, preliminary measurements were made to determine what length a certain structure should be to satisfy the boundary conditions at a designated barb location in the test section.

### 3.6 Flow visualization

Flow visualization techniques, including the use of dye and confetti, were employed in order to examine and visualize the impact of secondary current flow patterns within the bend section(s). It was impossible to examine the physical structure of secondary currents within the bend due to the fact that (i) dye breaks up due to the high amount of turbulent flow within the bend, and (ii) confetti remains at the surface of the flow. However, these techniques showed the clear direction of secondary currents, the proximity along the outside-bank where flow would initially hit the channel wall, and flow deflection caused by the presence of barbs.

### 3.7 Bed material

The experiments were run near the threshold condition for the movement of sediment such that  $\tau_o \approx \tau_{cr}$ . Sediment size characteristics were determined based on the laws that govern ripple formation and armouring. Ripples tend to form when the sediment Reynold's number ( $Re_*$ ) is below 11.6 (Garcia, 1999), where

$$Re_* = \frac{u_* D_{50}}{\nu} \quad \text{Eq. 3.1}$$

Ripple formation also occurs when the mean sediment size  $D_{50} \leq 0.7$  mm (Raudkivi, 1990). Preventing the formation of ripples is of extreme importance due to the small model scale adopted in this study. Any such occurrence would have a relatively large impact on the resulting bed profile, and would distort the experimental data.

Initial assumed values of parameters that are relevant in determining the sediment Reynold's number include:

- Kinematic viscosity,  $\nu = 1 \times 10^{-6} \text{ m}^2/\text{s}$ ,
- Gravity,  $g = 9.81 \text{ m/s}^2$ ,
- Uniform channel flow depth,  $y_o = 10.16 \text{ cm}$  &  $7.62 \text{ cm}$  ( $W/y_o = 4.5$  &  $6.0$  respectively)
- Mean grain size,  $D_{50} = 0.85 \text{ mm}$  (determination explained later)
- Slope,  $S = 0.0010$  (preliminary assumption)
- Hydraulic radius,  $R = \text{flow area} / \text{wetted perimeter}$ :  $R_{(W/y_o=4.5)} = 0.046 \text{ m}$  &  $R_{(W/y_o=6)} = 0.035 \text{ m}$
- Shear velocity,  $u_* = \sqrt{gRS}$  :  $u_{*(W/y_o=4.5)} = 0.0212 \text{ m/s}$  &  $u_{*(W/y_o=6)} = 0.0185 \text{ m/s}$

$$\text{For } W/y_o = 4.5, Re_* = \frac{(0.0212 \text{ m/s})(0.00085 \text{ m})}{(1 \times 10^{-6} \text{ m}^2/\text{s})} = 18.02$$

$$\text{For } W/y_0=6, \text{Re}_* = \frac{(0.0185 \text{ m/s})(0.00085 \text{ m})}{(1 \times 10^{-6} \text{ m}^2/\text{s})} = 15.73$$

In both cases,  $\text{Re}_* > 11.6$ , which implies that ripple formation will be avoided.

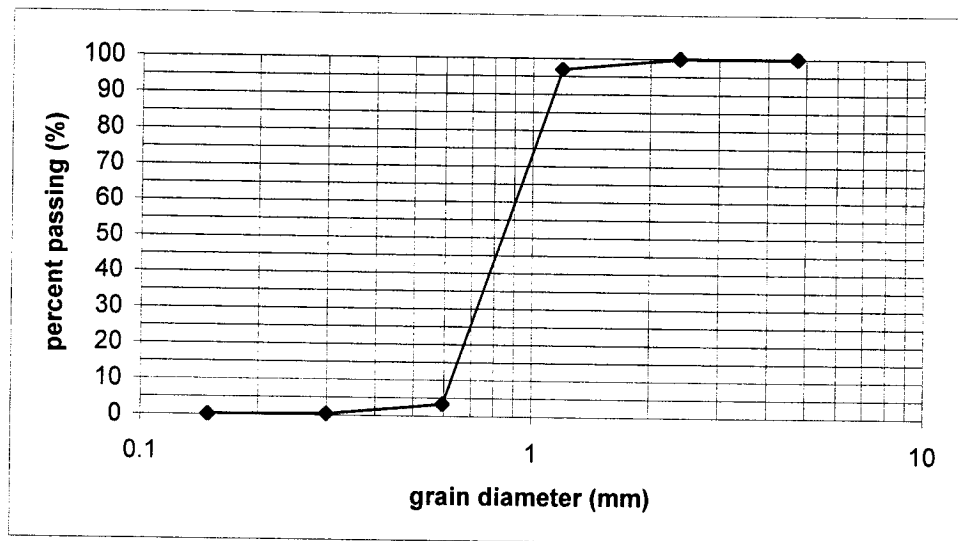
### 3.8 Sediment size and uniformity

Sediment size distribution and uniformity are extremely important parameters when dealing with the movement of bed material. Non-uniform sediment would result in sedimentation and scouring less severe than would be encountered from a more uniformly distributed material. A way to determine whether or not a particular sediment is uniform would be to calculate the geometric standard deviation ( $\sigma_g$ ) of the grain size distribution where:

$$\sigma_g = \sqrt{\frac{D_{84.1}}{D_{15.9}}} \quad \text{Eq 3.2}$$

$D_{84.1}$  and  $D_{15.9}$  are those grain sizes of the sediment where 84.1 percent and 15.9 percent of the material is finer, respectively.

For sediment to be classified as ‘non-uniform’, the  $\sigma_g$  value must be greater than 1.5. As  $\sigma_g$  increases, scour levels and sedimentation rates decrease. This phenomenon is known as armouring. Fine particles are easier to move than larger, heavier particles. If a sediment sample is considered non-uniform, the bigger the difference between the smallest and largest particles. As the finer particles are transported, the larger, coarser grains tend to remain in the bed. Since coarser particles are a lot harder to move, the scouring rate and the magnitude of scour decreases with time. Studies conducted by Kwan (1988) and Melville (1975) verify this process. Therefore, to avoid potential distortion of the data, as well as to simulate ‘worst-case’ scouring scenarios, it is of the utmost importance to ensure that the sediment used in the study is uniform.



**Figure 3.6:** Final sand gradation curve

Previous studies performed in the laboratory on scouring within a stream bend utilized a washed quartz sand from an Ottawa-area quarry. The identical sand from this quarry was used in the present study. Sieve analysis revealed that characteristics of the delivered sand were:  $D_{50} \approx 0.70$  mm,  $\sigma_g = 1.75$ . In order to produce a more uniform sediment with a slightly larger mean grain size, the delivered sand was screened in a *Gilson* mechanical sieve. The final mixture had the following characteristics:  $D_{50} \approx 0.85$  mm,  $\sigma_g = 1.29$ , producing the gradation curve shown in fig. 3.6.

### 3.9 Channel slope

Shields criterion is the primary consideration for selecting channel slope. In order to achieve near-uniform flow at the 'threshold' condition for sediment transport, other factors, such as mean grain size, hydraulic radius, and shear stress must be taken into consideration. Using the Shields' diagram (fig. 3.7) and sediment Reynold's numbers 18.02 and 15.73,

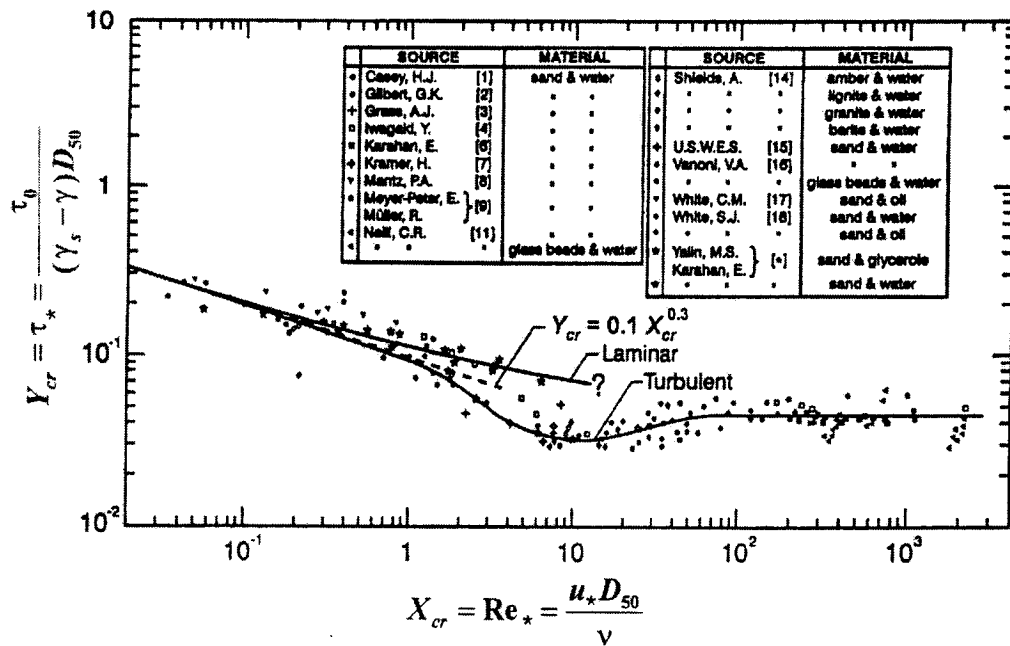


Figure 3.7: Shields' curve (Yalin and Karahan, 1979)

$$\text{For } Re_* = 18.02, \tau_* = \frac{\tau_{cr}}{(\gamma_s - \gamma) D_{50}} = 0.033 \approx \tau_* \text{ for } Re_* = 15.73$$

Thus, the critical shear stress:

$$\tau_{cr} = 0.033(2.65 - 1)(9810 \text{ kN} / \text{m}^3)(0.00085 \text{ m})$$

$$\tau_{cr} = 0.45 \text{ N} / \text{m}^2$$

For the commencement of sediment movement,  $\tau_{cr} = \tau_o$ :

$$\tau_{cr} = \tau_o = \gamma R S$$

Eq. 3.3

$$\tau_{cr} = 0.45 \text{ N} / \text{m}^2 = (9810 \text{ kN} / \text{m}^3)(0.046 \text{ m})(S)$$

$$S = 9.97 \times 10^{-4}$$

Similarly, for a hydraulic radius  $R=0.035 \text{ m}$ ,  $S = 1.31 \times 10^{-3}$ .

Although the two above-noted slopes are not equal, they are extremely close to the assumed initial slope used to determine  $Re^*$ . Due to the fact that the straight entrance section of the channel is very long (9.75 m), the small slope difference noted above can be neglected. Accordingly, the platform slope was set at 0.001 and this remained constant throughout the study.

### **3.10 Velocity**

The velocity at which sediment begins to move under the influence of flow is called critical velocity. It is difficult to determine an exact value at which sediment begins to move instantaneously. Past studies have given a basis for which critical velocity can be determined: (i) visually, and (ii) published values (Voisin, 2001). Preliminary tests determined that critical velocity is the value that would correspond to the mean velocity at which 10 to 20 percent of the surface sand particles in the test section would move. This condition was used as the basis to determine critical velocity in similar studies conducted previously in the laboratory where critical velocity within a bend was studied. However, any criterion that deals with the human eye can be very subjective, thus, a second method was used to determine critical velocity. Based on published values, the critical velocity corresponding to similar sands with  $D_{50} \approx 0.85$  mm is 0.28 m/s (Yang, 1996). Establishing this mean velocity within the channel produced adequate particle motion for both flow depths, resulting in discharges of 0.0101 m<sup>3</sup>/s and 0.0132 m<sup>3</sup>/s for  $W/y_o = 6.0$  and 4.5 respectively.

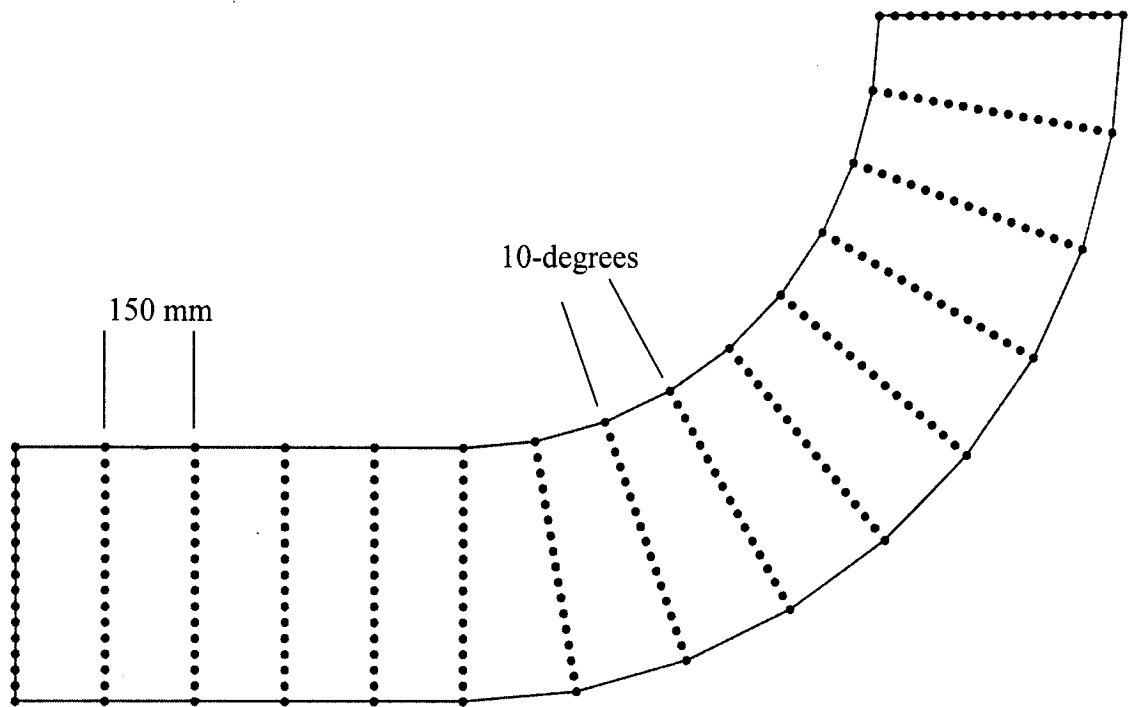
### **3.11 Scour tests**

An initial series of calibration test runs were conducted to establish equilibrium conditions in the test sections without the presence of barbs. These tests, which determined maximum scour depth, scour profile, thalweg location, and bank scour in both bend sections for the two  $W/y_o$  conditions, indicated that equilibrium was established at approximately 28 hours after commencing the tests. Channel bed elevations were recorded in a fixed grid pattern through the bend sections using the

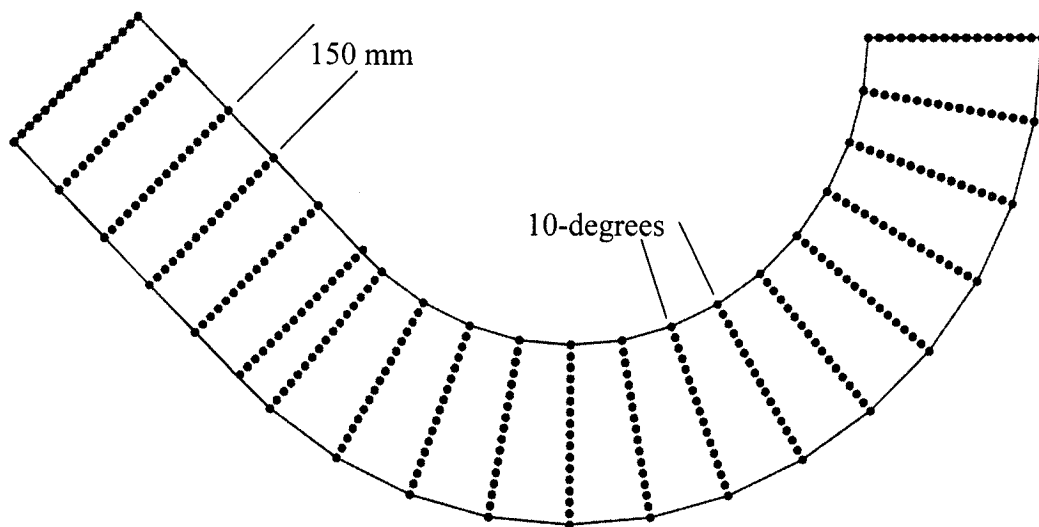
*Disto™ Pro 4a* laser meter. Elevations were recorded at 17 equal-spaced sites across the channel on cross-sections spaced at 10-degree intervals through the bends, starting at 10 degrees from the entrance and ending at its exit. Cross-section bed level profiles downstream of the bend were measured at 5 cross-sections spaced at 150 mm intervals starting 150 mm downstream of the end of the bends. Figures 3.8 and 3.9 show the points of measurements in the 90 and 135-degree bends, respectively.

Maximum scour depths and longitudinal profiles obtained from these initial calibration tests became the references for which results from all other tests would be compared. These ‘reference’ bed contour profiles (figs 3.10-3.13) show that within both the 90 and 135-degree bends, outside-bank erosion due to secondary currents commenced approximately in the range  $2.67 < L/W < 3.08$ , where  $L$ =distance along the outside-bank from the bend entrance. The upstream and downstream limits of the region of maximum scour for both bends were roughly at the 70-degree cross-section and the cross-section 750 mm downstream of the bend, respectively.

The two parameters that became the main focus of the study and the criteria under which successful barb system performance would be judged were: (i) scouring near the vicinity of the outside-bank, and (ii) location of the channel thalweg relative to the outside-bank. Barb structures are designed to protect the usually unstable outside-bank region of a stream bend by reducing the amount of scouring near the bank and moving the channel thalweg towards the center of the channel. The latter is of particular importance in order to maintain the long-term stability of the channel. Barbs are designed to promote increased scouring near the ‘toe’ end of the structure, which accompanies the eventual lateral movement of the thalweg away from the bank. In practice, local erosion near the ‘toe’ end of the structure is welcome, and would not compromise the structural nature of the barb, as the individual stones consisting of the structure are designed to re-adjust in such instances. Although the test facility in this study consisted of channel bends made from acrylic material with perfectly vertical walls, scouring near the outside bank would give a good indication as to what would be encountered in the field. Taking into consideration the region of maximum scour resulting from these ‘reference’ tests the



**Figure 3.8:** Bed measurement points; 90-degree bend



**Figure 3.9:** Bed measurement points; 135-degree bend

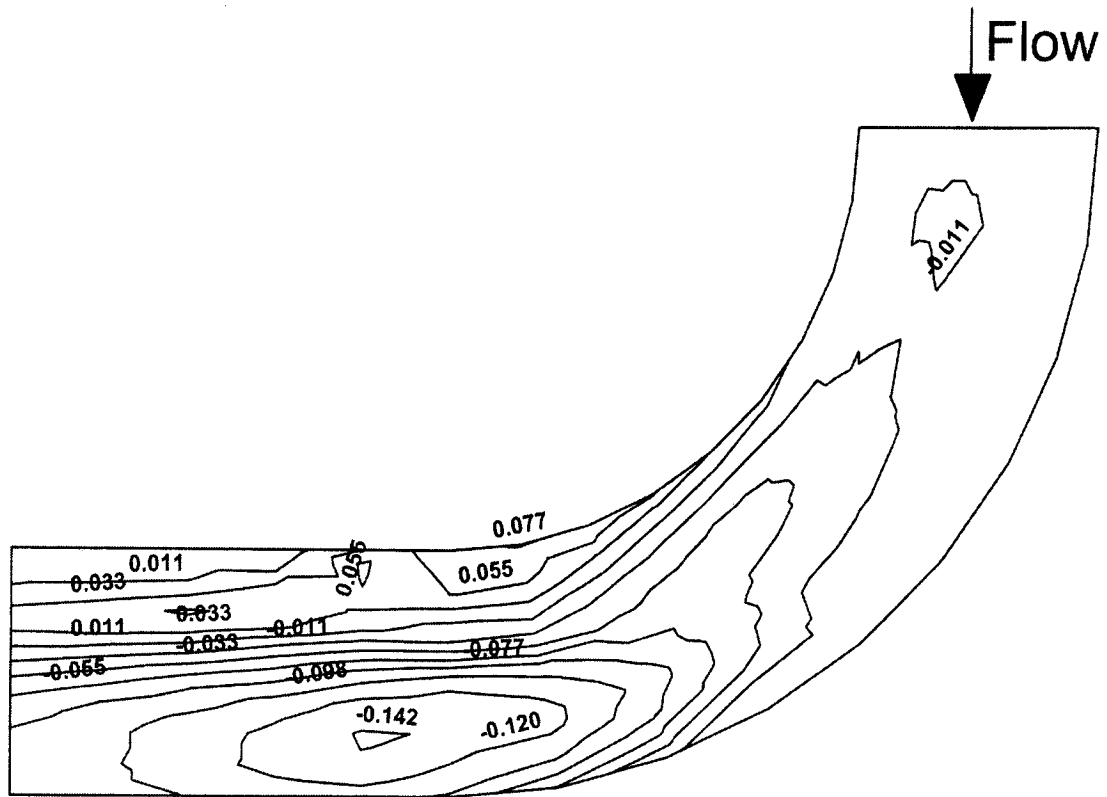
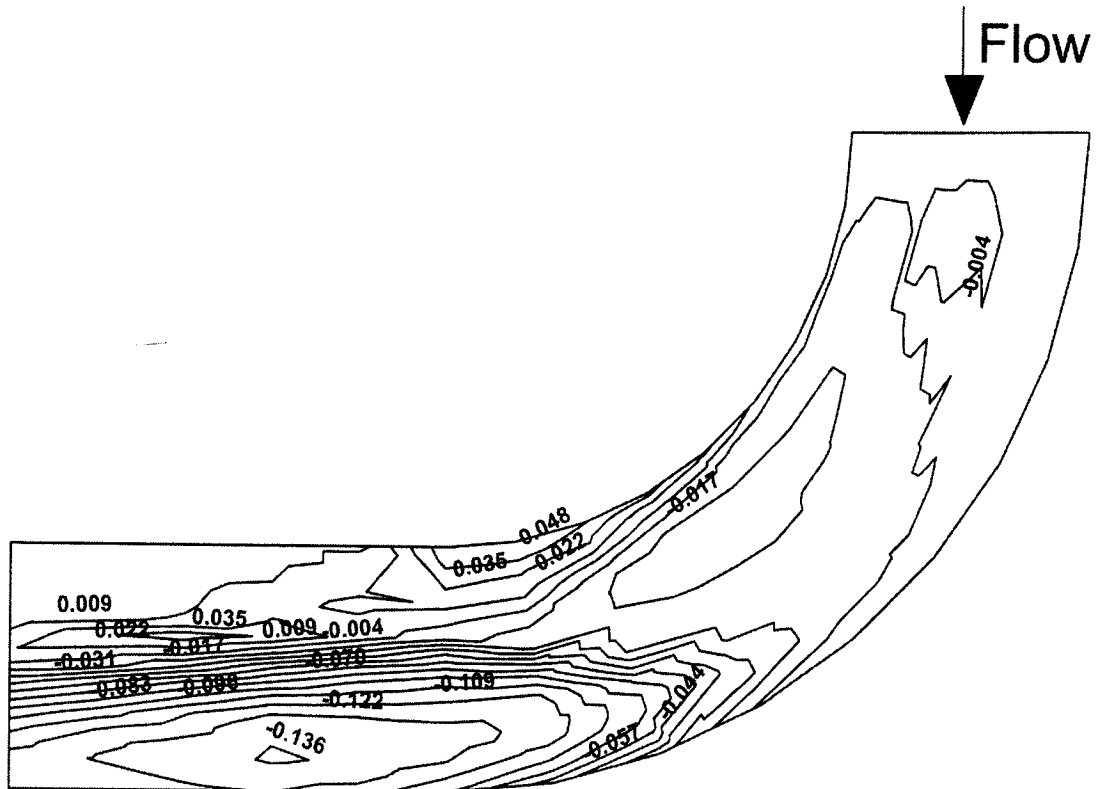
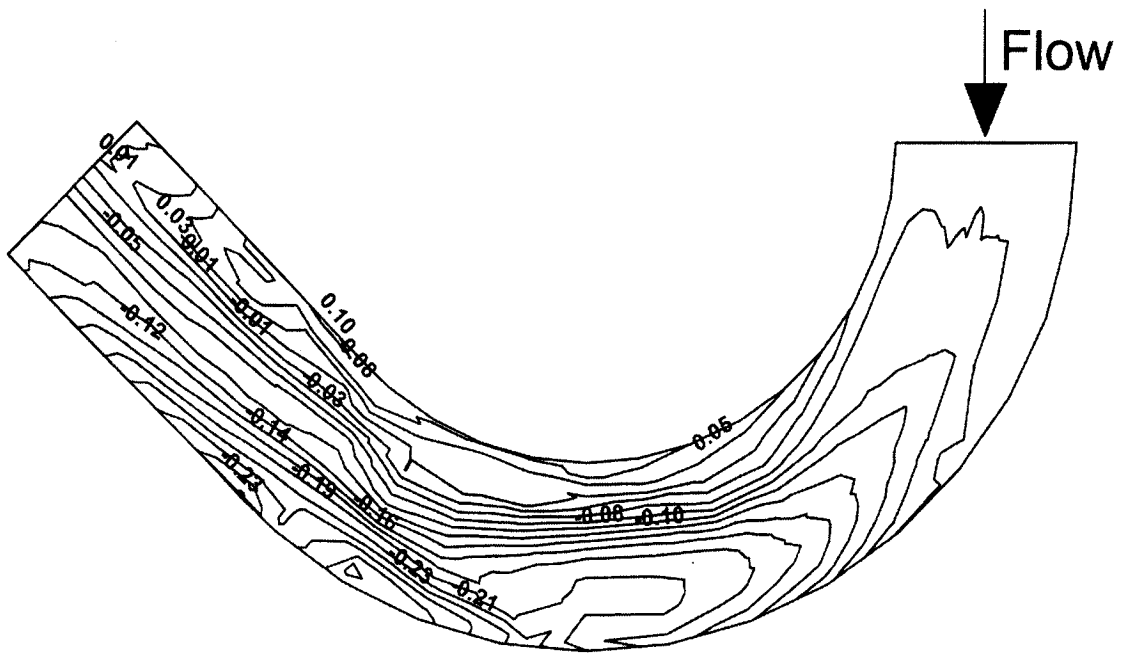


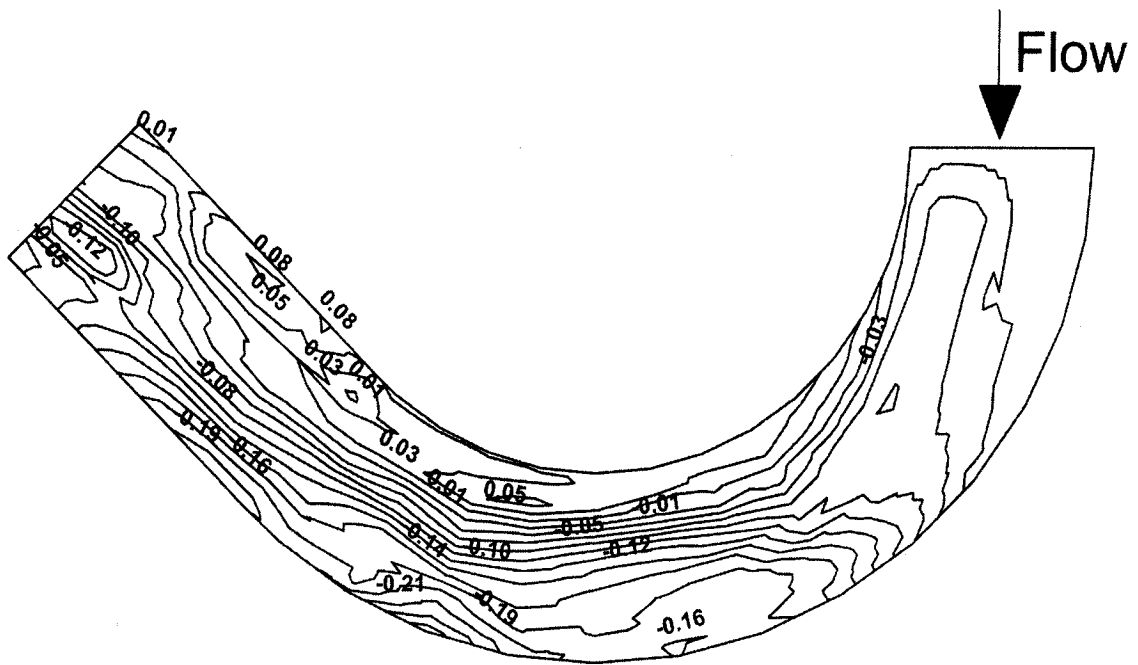
Figure 3.10: 'Reference' bed contour pattern (90-degree bend);  $y_0=10.16$  cm



**Figure 3.11:** 'Reference' bed contour pattern (90-degree bend);  $y_o=7.62$  cm



**Figure 3.12:** 'Reference' bed contour pattern (135-degree bend);  $y_0=10.16$  cm



**Figure 3.13:** 'Reference' bed contour pattern (135-degree bend);  $y_o=7.62$  cm

the region of maximum scour resulting from these ‘reference’ tests, the analysis of outside-bank bed scour and the calculation of scour reduction was determined as follows: for the 90-degree (135-degree) bend, within the aforementioned region of maximum scour, scour depths,  $y_s$  were measured at 8 (13) stations running parallel to the outside wall of the bend and located 25.4mm from the latter. These measurements were done for ‘scour depth without barbs’,  $y_s$ , and for the equivalent ‘scour depth with barbs’,  $y_s'$  case. The resulting percent reduction in local scouring, P was estimated based on the following relation:

$$P = \frac{\sum y_s - \sum y_s'}{\sum y_s} \quad \text{Eq. 3.4}$$

Scouring action at points 25.4 mm distant from the bend wall would be representative of the type of scouring and hydraulic action expected near the stream bank through the course of a real bendway. Scouring action any closer to the outside-bank was considered suspect due to the presence of the vertical boundary. As previously mentioned, the upstream and downstream limits of the region of maximum scour for both bends are the 70-degree cross-section and that cross-section 750 mm downstream of the bend, respectively. For the 90-degree (135-degree) bend, this region was represented by 8 (13) cross-sections, and thus, 8 (13) points where scour depths with (without) barbs,  $y_s'$  ( $y_s$ ) were measured.

Movement of the thalweg from next to the outside-bank of a stream bend is particularly important for the long-term stability of the waterway. The channel thalweg was determined by locating the deepest point of scour for each cross-section within the aforementioned region of maximum scour. In reporting results relating to thalweg movement, the location of the thalweg relative to the outside wall of the bend is defined as the cross-stream (horizontal) distance from the wall to the point of maximum scour of the bed. For those scour profiles generated without (with) barbs in place, this distance is defined as  $x$  ( $x'$ ), respectively. For tests without barbs, the average location of the thalweg generally occurred at  $x/W=0.236$  and  $0.201$  for tests in the 90-degree bend with

$W/y_0=4.5$  and  $6.0$ , respectively, and  $x/W=0.126$  and  $0.133$  for tests in the 135-degree bend with  $W/y_0=4.5$  and  $6.0$ , respectively.

### **3.12 Test duration**

As previously mentioned, the 'reference' tests for determining maximum scour lasted approximately 28 hours. Studies analyzing sediment transport and maximum local scour usually include tests conducted to run until a quasi-equilibrium state is reached. However, since the 'equilibrium' condition is approached asymptotically, it was not feasible to run all experiments for this length of time and complete the study in a reasonable time frame. Preliminary calibration tests demonstrated that highly representative stable scour depths with barbs in place were produced at approximately 5 hours. These tests showed that resulting maximum scour values were extremely close to those of the equilibrium 'reference' tests without barbs (approximately 87-percent). However after 5 hours, local scouring at the toe and around the model barbs began to undermine the structure. These results led to the decision to allow all model barb tests to run for 5 hours, at which time bed level measurements would be conducted and scour depths determined.

### **3.13 Scour volumes**

Measuring changes in scour volumes is an issue relevant to studies concerned with decreasing the total amount of scour. In this study however, it is not an important criterion to determine barb performance. The purpose of stream barbs is to decrease the amount of scour near the outside-bank and to laterally displace scour profiles towards the center of the channel, not to decrease the total amount of scour. The important criteria in this study is the reduction of scour near the outside-bank, and the movement of the deepest point of scour away from the outside-bank.

# Chapter Four

## Results and Discussion

### 4.1 Introduction

In this study, the impacts of the different barb arrangements considered on local scouring in the two bend test sections were determined by comparing two aspects of local scouring for the case “barbs installed” with corresponding data for ‘reference’ tests (i.e. “without barbs”). The two aspects of local scouring selected for these comparison exercises were: (1) percent reduction in local scouring achieved in the vicinity of the bend’s outside-bank, and (2) cross-stream displacement achieved in the channel thalweg. Table 4.1 shows total amount of scour ( $\Sigma y_s$ ) observed during these ‘reference’ tests, and the total scour: channel width ( $\Sigma y_s/W$ ) ratio near the outside-bank within the designated region of analysis for each flow condition and bend arrangement, respectively.

**Table 4.1:** ‘Reference’ test scouring near outside-bank

Bend arrangement	$W/y_o$	$\Sigma y_s$ (mm)	$\Sigma y_s/W$
90-Degrees	4.5	292.9	0.64
	6.0	355.7	0.78
135-Degrees	4.5	1097.8	2.40
	6.0	996.9	2.18

For the 90-degree bend, a lower flow depth (higher  $W/y_o$ ) produces considerably more scouring near the outside-bank. Since all tests are designed to run at critical velocity for incipient motion of the sediment particles, a decrease in depth (while maintaining a constant velocity) would naturally result in a smaller overall cross-sectional flow area. As such, bend generated secondary currents are confined within a smaller area, thus creating a greater impact on the bed of the channel more pronounced than what would be encountered when using a larger flow depth. The difference of total scouring near the outside-bank comparing the different  $W/y_o$  exceeds 21 percent.

When comparing 'reference' scour results between the 90-degree bend and the 135-degree bend, there is no question that the severity of scouring is much greater in the latter bend arrangement. Theoretically, secondary currents strengthen throughout the course of a bend until flow exits the turn, whereupon the 'decay' process starts. Considerably larger  $\Sigma y_s/W$  values are experienced for the 135-degree bend because of the increase in both time and space for the formation and development of secondary currents within the channel turn. As well, depending on the angle of the bend, the flow rate, and the nature of the bank and bed material, residual secondary currents may continue to influence erosion patterns well downstream of a bend. What results is a greater residual effect in the downstream straight section of the channel immediately after the bend within the 135-degree bend, especially compared to what was encountered in the 90-degree bend.

What must also be considered is the greater depths of scour near the outside-bank experienced for the larger discharge/flow depth (i.e. smaller  $W/y_o$ ) ratios within the 135-degree bend, a trend that is opposite to what was observed in the 90-degree bend. The main reason for this is simply the presence of a larger bend. Figures 3.10-3.13 show that scouring after the 90-degree section of the 135-degree bend is particularly severe. This reinforces the abovementioned statement of greater secondary current effects in the larger channel turn. There is little question that, for lower flow depths, the effects of bend-generated secondary currents have a major impact close to the channel bed. However, a larger flow depth in the larger bend negates this trend. In the case of the 90-degree bend, scouring near the outside-bank was more severe for the smaller flow depth because the concentration of secondary currents was closer to the bed and bank of the channel. What is seen within the larger bend is that the strength and impact of secondary currents within a larger flow depth is much more relevant than what is experienced in a smaller bend. Thus in the 90-degree bend, the major factor contributing to more severe outside-bank scouring is the confinement of bend-generated secondary currents within a smaller cross-sectional area of flow, promoting greater sediment transport for a lower discharge. In the 135-degree bend, the additional space allows the larger secondary currents formed within the larger flow depth to have a greater impact within the bend as well as a greater downstream residual effect.

## 4.2 Barb groupings

Chapter three outlines the different barb groups used in the experimental program. Up to four different groupings were used to determine the most effective location  $L$  ( $L'$ ) of the first (last) barb in a group relative to the bend entrance (exit). Other effects influenced by experimental conditions, such as  $W/y_0$ , maximum barb height  $h$ , barb alignment angle  $\theta$ , and bend arrangements, are also noted.

## 4.3 Analysis of results

The relative effectiveness of a particular barb grouping, with regards to protecting the outside-bank region against undermining and local-erosion processes, was judged on the basis of two criteria: (i) percent reduction achieved in local scouring next to the bend's outside-bank, and (ii) lateral displacement of the channel thalweg from its 'reference' position (next to the outside-bank) to a new position closer to the center of the channel. For each barb group type tested, the bed level is measured within the entire test section, as specified in section 3.11, after each experimental run. The data of most interest in this study are obtained in the region between the 70-degree cross-section and the section 750 mm downstream from the end of the bend. As noted in chapter three, this region consists of 8 sections of measurement for the 90-degree bend and 13 sections for the 135-degree bend. Outside-bank scour is determined by taking all depth measurements ( $y_s'$ ) of points nearest to the outside-bank within this region and taking the summation ( $\Sigma y_s'$ ). Based on this data, outside-bank scour reduction can be calculated as a percentage (P):

$$P = \frac{\sum y_s - \sum y_s'}{\sum y_s} \quad \text{Eq: 4.1}$$

A judgment on how effective a particular barb arrangement is relative to others can be determined by a comparison of P values. A 100 percent reduction indicates no scouring

in the section under analysis, and 0 percent reduction indicates the same amount of total scouring as that achieved without the presence of barbs (i.e. the reference test). Percent reduction values at any single point do not exceed 100 percent. Deposition at a particular point is considered to be a 100 percent reduction of scour so as not to create a bias among points of measurement that would skew the overall scour reduction value.

As mentioned in chapter three, in reporting the results relating to thalweg movement, the location of the thalweg relative to the outside walls of the bend section(s) is defined as the horizontal distance from the outside-bank wall to the point of maximum scour in the bed. Final thalweg location is determined by taking the average of these distances throughout the course of the aforementioned section in question and representing it as a ratio with respect to the width of the channel relative to the outside-bank ( $x'/W$ ). Comparing this value against the value corresponding to the thalweg location for tests conducted without the presence of barbs ( $x/W$ ) determines the level of effectiveness of a certain barb grouping.

Based on the criteria discussed in section 4.3, barb performance analysis includes the following:

1. Outside-bank scour reduction (P) vs. upstream barb angle ( $\theta$ )
2. Outside-bank scour reduction (P) vs. location of the last barb relative to the bend exit ( $L'$ )
3. Thalweg location ( $x'/W$ ) vs. upstream barb angle ( $\theta$ )
4. Comparisons between data collected for tests in the 90 and 135-degree bends

Tables 4.2 and 4.3 show a summary of all tests performed in both bend arrangements. Appendix A.5 includes bed contours of all tests performed in both bend arrangements (values above contour lines indicate  $W/y_s'$ ).

**Table 4.2: Summary of tests conducted in the 90-degree bend**

Barb grouping	$\theta$	$W/y_0$	$h/y_0$
A	20	4.5	0.5
	20	4.5	0.375
	30	4.5	0.5
	30	4.5	0.375
	40	4.5	0.5
	40	4.5	0.375
B	20	4.5	0.5
	20	4.5	0.375
	20	6.0	0.5
	30	4.5	0.5
	30	4.5	0.375
	30	6.0	0.5
	40	4.5	0.5
	40	4.5	0.375
C	20	4.5	0.5
	20	4.5	0.375
	20	6.0	0.5
	30	4.5	0.5
	30	4.5	0.375
	30	6.0	0.5
	40	4.5	0.5
	40	4.5	0.375
	40	6.0	0.5
D	20	4.5	0.5
	20	4.5	0.375
	30	4.5	0.5
	30	4.5	0.375
	40	4.5	0.5
	40	4.5	0.375
No Barbs	N/A	4.5	N/A
No Barbs	N/A	6.0	N/A

**Table 4.3:** Summary of tests performed in the 135-degree bend

Barb grouping	$\theta$	$W/y_0$	$h/y_0$
B	20	4.5	0.5
	20	4.5	0.375
	20	6.0	0.5
	30	4.5	0.5
	30	4.5	0.375
	30	6.0	0.5
	40	4.5	0.5
	40	4.5	0.375
	40	6.0	0.5
C	20	4.5	0.5
	20	4.5	0.375
	20	6.0	0.5
	30	4.5	0.5
	30	4.5	0.375
	30	6.0	0.5
	40	4.5	0.5
	40	4.5	0.375
	40	6.0	0.5
No Barbs	N/A	4.5	N/A
No Barbs	N/A	6.0	N/A

### 4.3.1 90-degree bend

#### 4.3.1.1 Outside-bank scour reduction ( $P$ ) vs. upstream barb angle ( $\theta$ )

Table 4.4 shows results for all barb group types tested in the 90-degree bend.

**Table 4.4:** Percent scour reduction (90-degree bend)

Barb grouping	$W/y_0$	$h/y_0$	$\theta$		
			20°	30°	40°
A	4.5	0.5	60.94	71.01	52.03
B			76.72	75.59	77.50
C			36.05	45.27	33.56
D			47.42	36.05	23.28
A		0.375	67.93	66.77	63.31
B			73.10	74.71	79.57
C			54.86	53.53	53.80
D			65.47	41.58	38.06
B	6.0	0.5	68.74	76.81	74.75
C			37.28	49.79	32.16

For a visual analysis of the results shown in table 4.4, accompanying graphs show percent scour reduction ( $P$ ) versus the upstream barb angle ( $\theta$ ) for each barb group type for all  $y_0$  and  $h$  tested in the study. Figure A.1.1 shows the results for all 90-degree bend tests for  $W/y_0=4.5$  and  $h/y_0=0.5$ . This figure shows that group type B consistently provides superior outside-bank protection whereas by comparison systems C and D consistently provide inferior results. Figure A.1.2 shows test results corresponding to the same parameters as fig. A.1.1, but for  $W/y_0=4.5$  and  $h/y_0=0.375$ . Again, group B is the most effective in protecting the outside-bank from scouring while groups C and D provide the least amount of protection following these particular parameters. The main differences between the two sets of results are: (i) overall there is less variation in  $P$  with respect to  $\theta$  for each respective barb group for lower  $h/y_0$  (i.e. fig. A.1.2) compared to those results in which  $h/y_0$  is increased (i.e. fig. A.1.1); (ii) consistently higher  $P$  was obtained for most  $\theta$  in most barb groups for lower  $h/y_0$ ; (iii) as  $L$  increases, the difference in  $P$  between the same barb group type for different  $h/y_0$  for each respective  $\theta$  decreases; and (iv) when B

group types are utilized,  $h/y_0$  appears to be a non-factor (i.e. when  $L'$  increases,  $h/y_0$  becomes irrelevant). Figure A.1.3 shows a graphical comparison between all tests conducted for the conditions ( $h/y_0=0.5$  &  $0.375$  for  $W/y_0=4.5$ ).

Following the same criteria as figs. A.1.1 and A.1.2, fig. A.1.4 shows results for tests in which discharge is decreased, such that  $W/y_0=6.0$  for  $h/y_0=0.5$ . Due to time constraints, only selected tests were performed under this condition. Groupings B and C, two barb systems whose respective results lie at both ends of the performance spectrum for previous conditions, were chosen. Scour reduction values for this criteria appear to be very similar to those observed in fig. A.1.1, and a graphical comparison of results combining all tests with  $h/y_0=0.5$  are shown in fig. A.1.5.

The data presented in figs. A.1.1, A.1.2, and A.1.4 show that for a 90-degree bend, flow depth is not so much a factor as far as outside-bank protection (P) is concerned so much as maximum barb height (h) is. A change in flow depth generates very little variation in P for constant  $h/y_0$ . However, as shown in fig. A.1.2, a change in h results in a significant decrease in the variation of P respective to both changes in  $\theta$  and changes in L (i.e. changes in barb group type) when compared to fig. A.1.1 (as seen in fig. A.1.3). Another major finding is the impact and effectiveness of certain barb groupings. In all three instances where flow depth and/or barb height are variable, group B is clearly the most effective of the four barb groupings, with outside-bank scour reduction values ranging between 70 and 80 percent. Group B is configured such that L and  $L'$  are the largest compared to the other barb groups. Conversely, groups C and D, which are shown to be the least effective in protecting the outside-bank region from the effects of flow through the bend, have lower L and  $L'$  values respectively compared to barb groupings A and B. These tests have shown that for a narrow channel, it would appear that locating first barbs upstream of those points where outside-bank erosion is expected to begin (as in the case of C and D-groups), does not greatly enhance barb system performance. For improved performance, first barbs are required to be located as close as possible to the point where outside-bank scour commences, a location clearly shown by the 'reference' test results of figs. 3.10-3.11. Conversely, having a barb further downstream, within the last straight

reach of the channel after the end of the bend, appears to be a critical factor in the overall performance of a barb arrangement.

The effects of flow through a bend include those attributed to normal and lateral velocity, but more importantly, the creation of transverse velocity; secondary currents caused by the combined effects of centrifugal forces due to flow in a bend and the differences in velocity between that near the free surface and the channel base. In nature, transverse velocity becomes a dominant factor due to the increase in force associated with centrifugal acceleration. The strength of secondary currents continues to grow through the course of a bend until flow exits the turn, at which time its strength begins to weaken. However, residual effects are still felt in the reach immediately downstream of a bend.

Without protective devices such as barb structures, secondary currents are overwhelming relative to naturally occurring bankline frictional resistance. Barbs however promote the formation of eddies which in turn partially balance the effects of secondary currents throughout the bend. This balance minimizes the impact of secondary currents near the bank, and increases frictional cell resistance, resulting in a decrease in scouring near the outside-bank. Placing the final barb in an arrangement downstream of the bend, as was the case for group B, provides insurance that the entire region of maximum scour will be protected. Even though a structure is placed in the final straight reach of the channel, the effects due to the hydraulic action caused by the presence of such a structure are minimal in comparison to the damage that secondary currents and their residual effects can inflict on an unprotected region. It is the position of this final structure that becomes a major factor as to the overall performance of a barb grouping.

Figures A.1.1-A.1.5 show that  $\theta$  has a strong influence on barb group performance. The major trend appears to be that  $\theta=30$ -degrees is the angle that yields superior results. However, as seen in both figs. A.1.1 and A.1.2, in group D, a 20-degree angle produces the best results. One can conclude from these results that, as  $L$  increases for barb groups where  $\theta$  remains constant, the optimum upstream angle is 30-degrees. It also should be noted that for group B, the variation in  $P$  for all  $\theta$  tested is the smallest of the four groups.

This data shows that for large  $L$  and  $L'$ ,  $\theta$  becomes less of a factor than when  $L$  and  $L'$  approach zero.

#### 4.3.1.2 Thalweg location

The movement of the thalweg towards the center of the channel is of particular importance in achieving continued long-term stability of the outside-bank region. Initial 'reference' tests without the presence of barbs showed that  $x/W=0.236$  and  $0.201$  for  $W/y_0=4.5$  and  $6.0$ , respectively. Table 4.5 shows thalweg location ( $x'/W$ ) for all tests performed in the 90-degree bend. Figures A.2.1-A.2.4 show the comparison between  $x'/W$  and  $\theta$  for each barb grouping under the various flow and  $h/y_0$  conditions.

**Table 4.5:** Mean thalweg location,  $x'/W$  (90-degree bend)

Barb grouping	$W/y_0$	$h/y_0$	$\theta$		
			20°	30°	40°
A	4.5	0.5	0.316	0.323	0.306
B			0.292	0.330	0.333
C			0.274	0.340	0.313
D			0.267	0.302	0.281
A		0.375	0.309	0.288	0.313
B			0.299	0.302	0.313
C			0.246	0.285	0.309
D			0.278	0.292	0.292
B	6.0	0.5	0.250	0.299	0.292
C			0.205	0.264	0.230

Based on this method of calculating a value to qualitatively represent thalweg location, a higher value, or value closer to 0.50, implies a thalweg position closer to the middle of the channel. Anything less than 0.50 means a thalweg that is closer to the outside-bank, and the lower the value, the closer the average location of the thalweg is to the outside-bank. The commonality between all four barb groupings tested in the 90-degree bend, irregardless of flow and  $h/y_0$ , is that the presence of barbs has a positive effect and shifts the thalweg closer to the middle of the channel in each experimental run. In every instance where there was a barb grouping present,  $x'/W > x/W$ . The amount of movement

depends on  $\theta$ , the particular barb grouping, and  $h/y_o$ . Comparing figs. A.2.1-A.2.4 for  $W/y_o=4.5$  and variable  $h/y_o$ , group-types B and A are the most effective in moving the thalweg away from the outside-bank. For groupings C and D,  $\theta=30$ -degrees appears to be, for the most-part, the optimum angle that achieves the most significant thalweg movement for these particular group types. This holds true in some instances for systems A and B. However, for these two barb group types, the differences between  $x'/W$  values for variable  $\theta$  are not as great as they are for group types C and D. Once again, this shows that an important factor in achieving consistency and overall performance of a barb system is the establishment of a grouping with larger  $L'$  values (i.e. group types A and B)

Taking into consideration a third flow condition ( $W/y_o=6.0$  and  $h/y_o=0.5$ ) for barb groupings B and C, it is apparent that this lower discharge (lower flow depth) results in reduced thalweg movement. This trend occurs in every test for both barb systems. The differences in  $x'/W$  values for tests using group B for corresponding values associated with the same  $\theta$  for  $W/y_o=4.5$  and variable  $h/y_o$  are smaller however compared to the differences in  $x'/W$  values resulting from group C.

#### *4.3.1.3 Location of last barb*

As discussed in section 4.3.1.1, the location of the last structure in the sequence within a barb system appears to be a dominant factor regarding overall protection that barb groupings provide to the outside-bank. Secondary currents, resulting from the combined effects of bend-generated centrifugal forces and the differences between flow velocities near the free surface and those near the channel bed, greatly influence local scouring processes. Well-designed barb systems generate flow circulation patterns that largely counteract bend-generated secondary currents. Placing the last barb in a group downstream of the bend exit section promotes a more rapid decay of any residual secondary current effects.

Figures A.3.1-A.3.3 show graphical representations between  $P$  and  $L'/W$  for all  $\theta$ . [A negative  $L'$  value denotes a final barb location for a particular grouping within the bend section]. Figure A.3.4 provides a complete overview of all results within the 90-degree bend. The general trend appearing in all four figures is a linear increase in outside-bank scour reduction as the location of the last barb in an arrangement is placed further downstream from the bend exit. For each  $\theta$ , it appears that the slope of the trends for each flow/barb condition is similar. Of the three conditions tested; (i)  $W/y_o=4.5$ ,  $h/y_o=0.5$ , (ii)  $W/y_o=4.5$ ,  $h/y_o=0.375$ , and (iii)  $W/y_o=6.0$ ,  $h/y_o=0.5$ , similar trends appear when  $h/y_o=0.5$  (i.e. conditions (i) and (iii)).

Figures A.3.1-A.3.3 also show that, for the aforementioned three experimental conditions under which barbs were tested, the best results are obtained under the second condition (i.e.  $W/y_o=4.5$ ,  $h/y_o=0.375$ ). As well, figs. A.1.1-A.1.3 show improved results for tests performed under this same condition compared to the exact same barb arrangements tested under the other two conditions. An argument could be made that the ideal  $h/y_o$  ratio is closer to 0.375 than it is to 0.5.

### **4.3.2 135-degree bend**

#### *4.3.2.1 Outside-bank scour improvement ( $P$ ) vs. upstream barb angle ( $\theta$ )*

Due to time constraints, the full range of tests performed in the 90-degree bend was not repeated in the 135-degree bend test section. The 90-degree bend test results showed that the best barb arrangement for all three testing conditions was group type B. Accordingly, this arrangement was selected for testing in the 135-degree bend. System C was also tested, due to the fact that the results for this barb grouping arrangement in the 90-degree bend were close to the lower limit of  $P$  values relative to the other group types, but not too close to that of group B. It was important to determine the consistency of the results obtained for tests performed in the 90-degree bend and to observe any similarities or differences that may occur for these particular tests in the 135-degree bend.

Table 4.6 shows scour reduction values for all barb group types tested in the 135-degree bend. Figure A.1.6 shows P vs.  $\theta$  for tests with  $W/y_o=4.5$  and  $h/y_o=0.5$ . Figure A.1.7 shows the same for  $W/y_o=4.5$  and  $h/y_o=0.375$ , and fig. A.1.8 for  $W/y_o=6.0$  and  $h/y_o=0.5$ . Figures A.1.9 and A.1.10 show all results for tests performed in the 135-degree bend for  $W/y_o=4.5$  and  $h/y_o=0.5$  respectively. As was observed in the 90-degree bend, it is clear that grouping B offers best protection for the outside-bank throughout the entire test section while a  $\theta=30$ -degrees appears to be the optimum alignment angle for all  $W/y_o$  and  $h/y_o$  tested.

**Table 4.6:** Percent scour reduction (135-degree bend)

Barb Grouping	$W/y_o$	$h/y_o$	$\theta$		
			20°	30°	40°
B	4.5	0.5	81.56	84.67	74.12
C			77.89	78.30	66.67
B		0.375	81.85	85.89	75.63
C			60.58	77.43	68.55
B	6	0.5	78.26	79.06	64.96
C			65.28	63.98	55.97

Figure A.1.9 shows that there is very little difference between the results presented in figs. A.1.6 and A.1.7. Similar P trends for all  $\theta$  are observed for variable  $h/y_o$ . Unlike in the 90-degree bend, barb performance in the 135-degree bend appears to depend heavily on discharge and/or flow depth rather than barb height, the latter being the predominant case in the smaller bend arrangement. Changing barb height while maintaining a constant flow depth achieves very little difference in P, however, a change in flow depth (comparing fig. A.1.4 and A.1.6, as seen in fig. A.1.10) results in a major difference in P for both barb group types tested.

As indicated in the above figures, although P versus  $\theta$  trends were similar for different  $W/y_o$ , improved scour reduction was achieved for all  $\theta$  at the larger flow depth (i.e. smaller  $W/y_o$ ). For those tests in which  $W/y_o=4.5$ , very little variation in P with  $\theta$  was observed for different  $h/y_o$ . As previously noted, the effectiveness of B-type barb groups was found to be generally superior to C-type groups. The following conclusions can be

drawn: (i) at large flow depths ( $W/y_o < 4.5$ ), scouring in the vicinity of barbs is only marginally affected by changes in weir height  $h$ , (ii) when  $W/y_o < 6.0$ , barb system performance is only marginally affected by changes in  $\theta$ , (iii) scouring near the outside-bank is generally less severe when the local flow depth is large ( $W/y_o < 6.0$ ), and (iv) locating first barbs upstream of those points where outside-bank erosion is expected to begin (as in the case of C-groups), has a largely negative impact on barb system performance. For improved performance, first barbs require to be located as close as possible to the point where outside-bank scour commences.

#### 4.3.2.2 Thalweg location

'Reference' tests (i.e. without the presence of barbs) showed that the thalweg was located at that  $x/W = 0.126$  and  $0.134$  for  $W/y_o = 4.5$  and  $6.0$  respectively, as determined in the same manner outlined in section 4.3.1.2. Table 4.7 shows the mean thalweg location ( $x'/W$ ) for all tests conducted in the 135-degree bend.

**Table 4.7:** Mean thalweg location,  $x'/W$  (135-degree bend)

Barb grouping	$W/y_o$	$h/y_o$	$\theta$		
			20°	30°	40°
B	4.5	0.5	0.293	0.316	0.282
C			0.291	0.312	0.261
B		0.375	0.284	0.307	0.271
C			0.220	0.301	0.259
B	6.0	0.5	0.308	0.325	0.246
C			0.207	0.280	0.237

Figures A.2.5 and A.2.6 show the comparison between  $x'/W$  vs.  $\theta$  trends for barb group types under all flow and  $h/y_o$  conditions within the 135-degree bend. The general trend is that the location of the thalweg is most improved when  $\theta = 30$ -degrees. It also appears that both barb height and flow depth have a major influence on the movement of the thalweg away from the outside-bank. However, a very interesting scenario that did not occur in the 90-degree bend occurs in the 135-degree bend. For tests conducted in the 90-degree bend, results show that  $\theta = 20$ -degrees is the least effective barb alignment for

moving the thalweg towards the center of the channel. In the 135-degree bend however, particularly for barb grouping B,  $\theta=20$ -degrees produces  $x'/W$  values close to those resulting from  $\theta=30$ -degree. However, when  $\theta=40$ -degrees, particularly for group type B,  $x'/W$  values, on average, are lower than corresponding values for  $\theta=20$ -degrees. This is an exception to the trend found in the 90-degree bend results.

One explanation for this occurrence could be the location of the last barb in the channel. Usually,  $\theta$  as low as 20-degrees would produce the deepest scouring close to the outside-bank than would occur for  $\theta=40$ -degrees. This would be expected due to a larger deflection of flow in both lateral and transverse directions that would occur when larger  $\theta$  (i.e.  $\theta=40$ -degrees) is applied. However, in the case of barb grouping B in the 135-degree bend,  $L'$  for  $\theta=20$ -degrees is considerably larger than  $L'$  for  $\theta=30$  and 40 degrees. In the 90-degree bend, even though  $L'$  for  $\theta=20$  degrees for grouping B is larger compared to the  $L'$  for  $\theta=40$  degrees (table 3.2), the difference in length is not significant. The results corresponding to group type B in a 90-degree bend show that  $\theta=40$ -degrees more than makes up for a lower  $L'$  in terms of thalweg movement (fig. A.2.4). However, in the case of the 135-degree bend, when comparing the results for  $\theta=40$ -degrees and  $\theta=20$ -degrees, not even a barb grouping with a larger  $\theta$  can compensate for a considerably smaller  $L'$ . Elsewhere, it has been emphasized that the placement of the final structure downstream has a major impact on the performance of a barb system. This claim also holds for the case of thalweg location.

In general, thalweg movements for barbs aligned at  $\theta=30$ -degrees were observed to be greater than those when  $\theta = 20$  and 40-degrees. Field studies indicate that as  $\theta$  decreases, the closer the thalweg's position is to the outside-bank, (USDA, 1999, 2001). Tests in the 90-degree bend support this statement. However, as indicated in Table 4.7, when dealing with bend angles greater than 90-degrees, barbs aligned at large  $\theta$  ( $\geq 40$ -degrees) result in thalwegs much closer to the outside-bank than one would expect. When comparing scour data associated with barb groups aligned at 30 and 40-degrees, the

groups aligned at 40-degrees generated considerably more scouring action near the outside-bank (table 4.6).

#### *4.3.2.3 Location of last barb*

Figures A.3.5-A.3.7 show the relationship between outside-bank scour reduction and the location of the last barb within a system with respect to the distance of the location of its bank key from the bend exit for all tests conducted in the 135-degree bend arrangement. For all three  $\theta$  tested, it is apparent that the major factor leading to improved barb performance is the flow condition rather than barb height characteristics. Figure A.3.8 shows  $P$  vs.  $L'/W$  for all groupings tested in the 135-degree bend. As mentioned in the previous section, a final barb placed further downstream (increased  $L'$ ) for a smaller  $\theta$  (e.g. 20-degrees), compared to decreased  $L'$  for a larger  $\theta$  (e.g. 40-degrees) leads to improved results (i.e.  $L'$  is more of a factor than  $\theta$ ). This can be seen in the differences between outside-bank percentage scour reduction ( $P$ ) in figs. A.3.6 and A.3.7.

However, when comparing performances for barb groupings of different  $\theta$ , constant  $y_o$  and  $h$ , and similar  $L'$ ,  $\theta$  does become a factor in overall performance (fig. A.3.5.8). There is a linear trend where an increase in  $L'$  leads to an increase in  $P$ .

#### **4.3.3 Bend comparison**

The most obvious difference between flow characteristics in the 90 and 135-degree bends is the strength and impact of secondary currents within the channel bend. As mentioned in section 4.1, the difference in the amount of scouring in the two bend arrangements is significant (table 4.1). However, the performance of barbs, especially those from tests performed in the 135-degree bend, prove to be very significant as well. Percent scour reductions near the outside-bank are closely similar for tests between the two bends. However, one must consider that, while percentage reduction values are similar, the total amount of protection (i.e. total scour depth reduction) resulting from the presence of barb

structures within the 135-degree bend compared to the 90-degree bend is also very significant.

For the most-part, trends observed in both bends were very similar. Figures A.4.1-A.4.6 reinforce the statement that, when operating under the same flow conditions, barb groupings within the 135-degree bend provide just as much (and often-times more) protection for the outside-bank compared to corresponding groupings within the 90-degree bend. This result is very important considering that scouring in the larger bend without the presence of barbs is quite severe (table 4.1). The fact that these structures can offer a good level of protection to the outside-bank region of a 135-degree bend, where secondary flow effects are significant, is very promising.

From a reduced scouring standpoint,  $\theta=30$ -degrees appears to be the optimum alignment for barbs in the bend flow field in both bend arrangements studied herein. As mentioned previously, results for the 90-degree bend indicate that, in general, larger thalweg movements are achieved when barbs are aligned at both 30 and 40-degrees. However, in the case of the 135-degree bend, thalweg movements for barbs aligned at 30-degrees were much greater than those observed when  $\theta=20$  and 40-degrees.

The relationship between P and  $L'$  is shown in figs. A.3.5 and A.3.8 for all conditions tested in both bend arrangements. Looking at both figures, one commonality corresponding to the results observed in both bend arrangements is a similarity between the slopes of trend-lines projected through the data points. As noted earlier in this chapter, one major difference between the two bend arrangements is that, more often than not, outside-bank scour reduction percentages are higher for any  $L'/W$  for barb groups in the 135-degree bend. This again demonstrates the effectiveness of barb use in a strongly curved bend, particularly one in which the bend angle exceeds 90-degrees. These figures also show that, from a barb system design viewpoint,  $L'$  is an important parameter.

The one major finding that results from a comparison between results for both bend arrangements is the encouraging fact that barbs also offer an effective means of

protection in the larger bend. The substantial local scouring that occurred within the bend barb systems in place was worrying, especially in the case of the 135-degree bend. All results in the 135-degree bend are significant in the fact that P exceeds those observed in the 90-degree bend.

# Chapter Five

## Summary, Conclusions, and Research Needs

### 5.1 Summary

Barbs are a novel form of streambank protection that attack erosion mechanisms (i.e. secondary currents) associated with flow through a bend. Although several Illinois-based field studies of barbs conducted by the USDA NRCS at selected bends on wide shallow streams are underway, general design criteria for a wider application of these streambank protection structures are still not available (Kinney, 2002). This study includes the first two phases of an ongoing study, and focuses on the performance of model barb structures in two strongly curved channel bends (90 and 135-degrees) in which narrow channel flow conditions ( $W/y_o=4.5$  and  $6.0$ ) are simulated. Most of the scour tests were performed using: (i) two different barb height: flow depth ( $h/y_o=0.5$  and  $0.375$ ) ratios, (iii) three different barb alignment angles ( $\theta=20, 30,$  and  $40$ -degrees), and (iv) four different barb group-types (A, B, C, and D).

Of the two bend arrangements used in this study, the majority of tests were performed in the 90-degree bend. In order to keep within the budget and time frame designated for this project, only selected barb groupings were tested in the 135-degree bend. Types B and C were chosen because their results lay at opposite ends of the performance spectrum with regards to overall performance in the 90-degree bend. All data resulting from tests with the barbs in place were compared against 'reference' tests (without the presence of barbs). The latter were run at approximately 28 hours in order to establish 'benchmark' equilibrium conditions for subsequent comparison studies. In these experiments, it was noted that approximately 87 percent of the final (equilibrium) scour depths was reached after 5 hours of testing. Accordingly, in order to reduce the total testing time to a manageable level, all subsequent tests were performed for a standard 5 hour period.

Water was supplied from a tank containing a smooth transition section equipped with fine-mesh screens in order to promote steady state flows. Surface profiles were controlled by an adjustable tailgate located at the downstream end of the testing facility, while flows exiting the channel were measured in a collecting tank located underneath the outlet. The entire channel rested on an adjustable-slope platform. In phase 1, longitudinal slopes were selected that generated near-uniform and ‘clear-water’ channel flows corresponding to the ‘threshold’ condition (i.e. where boundary shear stress acting on the sand bed,  $\tau_o$ , approached the critical value corresponding to incipient motion of the sand particles,  $\tau_{cr}$ ). For ‘clear-water’ conditions, sediment is not re-introduced into the test section. However, in the field during ‘flood’ conditions, any sediment in a bendway that is transported from a particular section is re-filled with sediment from upstream reaches of the stream. It was the goal of this study to simulate the ‘worst-case’ scenario (re: maximum local scour depth relative to the channel bed level) for scouring within the stream bend. The bed material in the test sections had a  $D_{50} = 0.78\text{mm}$ , a sediment Reynolds number  $> 11.6$ , and  $\sigma_g \approx 1.3$ , all of which satisfy the conditions for prevention of ripple formation and armoring.

Published data on the aforementioned USA-based field studies are scarce. While the barb structure’s general shape is reasonably well defined, there is still some debate regarding preferred alignment in the flow field and also its height above the channel bed. Also, although a simple procedure has been developed for barb spacing, locating the first and last barbs at a particular site relies heavily on good judgment and experience. For a wide channel condition it is recommended that the first barb be placed just upstream of the outside-bank section first affected by erosion and that the last barb be located no further downstream than 75 percent of the entire bend turn length, (USDA 1999). Barb spacing and placement methodology in this study were loosely based on those outlined by the NRCS (Kinney, 2002; USDA, 1999, 2001), and were largely defined by the location of the first and last barb ( $L$  and  $L'$  respectively) and the structure’s upstream angle with respect to the outside-bank tangent line ( $\theta$ ).

The material used in constructing the model barbs was a USACE recommended 2.743 mm opening wire mesh.

## 5.2 Conclusions

The objectives of this study outlined in chapter 1 were clearly met. Generally, the model stream barb systems tested in this study offered a consistently high level of streambank protection for the experimental conditions studied. Outside-bank scouring was reduced in every instance, and results varied depending on group type,  $\theta$ ,  $y_o$ , and  $h$ . Not only did barbs promote sedimentation in the vicinity of the outside-bank, but the channel thalweg was also moved from a position close to the outside-bank to a new location at the ‘toe’ ends of the barbs. In other words, the thalweg was moved to a more benign location closer to the center of the channel.

Locating the first barb upstream of the bend’s outside-bank region first affected by erosion reduces barb system performance. This was evident from the results of those experimental involving barb group-types C and D, whose first barbs were located well upstream of aforementioned region first affected by outside-bank bed scouring:  $1.78 < L/W < 3.48$ . Compared to barb group-types B and C, group-types A and B were found to be the most effective at reducing scouring near the outside-bank.

Similarly, locating the last barb just downstream of a bend significantly reduces local scouring caused by residual secondary currents in the channel reach immediately downstream of the bend. Generally, group-types A and B involved the largest  $L'$  of all barb groups. From a design standpoint, it is clear that large  $L'$  is an important parameter in regards to enhanced decay of residual secondary currents in the channel reaches downstream of bends. This is especially important under narrow channel conditions, where stronger secondary currents are prevalent. Conversely, group-types C and D, whose  $L'$  were generally much less than those of groups A and B, experienced greater amounts of scouring near the outside-bank downstream of the bend as well as a thalweg

much closer to the outside-bank than one would want. These  $L'$  and  $L$  values appear to have the most impact on barb system performance.

Barb alignment angle  $\theta$  also plays a major role in barb system performance. A 30-degree barb axis of orientation with respect to the structure and the outside-bank tangent line, proved to be the superior alignment for barbs in the bend flow field.

Generally speaking, the results of this study show that movement of the thalweg towards the center of the channel increases with increase in  $h/y_o$ .  $P$  generally decreases when  $h/y_o$  increases. These opposing trends may present problems in determining optimum  $h/y_o$  for design purposes. For example, the movement of the thalweg away from the outside-bank promotes greater long-term channel stability. However, achieving greater  $P$  is also an important consideration. Group B achieved minimal variations in  $P$  and  $x'/W$ , regardless of  $h$ . Thus, when comparing the relative importance of  $h$  and  $L$ , as long as the former is within the range:  $0.33 \leq h/y_o \leq 0.5$ , first barb location  $L$  is the more important 'design' parameter.

Although scouring without barbs in the 135-degree bend was significantly greater than that experienced in the 90-degree bend under similar flow conditions, barb groups within the 135-degree bend performed as well, if not better, than in the 90-degree bend. This important discovery is key in the related studies that will follow this one.

### **5.3 Research Needs**

This study examined the performance of model stream barbs in strongly curved narrow channel bends. In nature however, most streams are of the wide, shallow variety. Also, 'clear-water' conditions are not typical of flood flows in alluvial channels. This study resulted in extremely positive and useful results in terms of model barb testing and their success under narrow channel conditions. Since most studies dealing with stream barbs have been in-situ 'field' studies, further laboratory type studies are necessary to develop comprehensive design criteria. Comparison studies between barb systems operating in

narrow-and wide-channel bends would be useful. Continuous supply of sediment at the entrance of the test section(s), to simulate field-like 'live-bed' conditions, would also be an important next step in the continuing study of these novel structures.

In terms of geometric model similarities, the data produced by this study covers all bend angles in between 90 and 135-degrees. Any design criteria may be applicable to smaller bend angles due to lesser flow effects that would be experienced in such situations. However, criteria may not be appropriate for bend angles greater than 135-degrees due to the increased severity in secondary currents experienced in such strongly curved bends.

This study utilized one type and size of bed material: washed quartz sand. Future research should not only consider other sand sizes, but also other materials, including gravel, small stones, silt, and clay. Also, this study utilized acrylic to form the entire channel. Although, for the most-part, the laboratory channel performed well, a channel comprised of soils and bank materials found in nature would represent a major improvement in the simulation process. Rather than studying only effects of scouring in the channel bed near the outside-bank, bank erosion and changes to bank geometry could also be analyzed in this type of channel.

Another suggestion for future research on this topic is to use a larger laboratory channel. This would permit the use of larger-scale model barbs which would reduce the potential for the introduction of possible scale effects. A larger channel would also permit the width of these structures to be more correctly simulated. Although this laboratory represents a good beginning for the necessary experimental studies that are required to develop comprehensive design criteria, there is still a need to study the performance of these structure in different 'field' situations. Most importantly, there is a need to test the behavior of these structures in channels formed in clay soils. [To date, all past field tests have been restricted to channels formed in coarse sands and gravels]. This novel form of bank-protection structure appears to be a very attractive alternative option for providing long-term stability to the outside-banks of alluvial channel bends.

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## **Appendix A.1**

**P vs.  $\theta$**

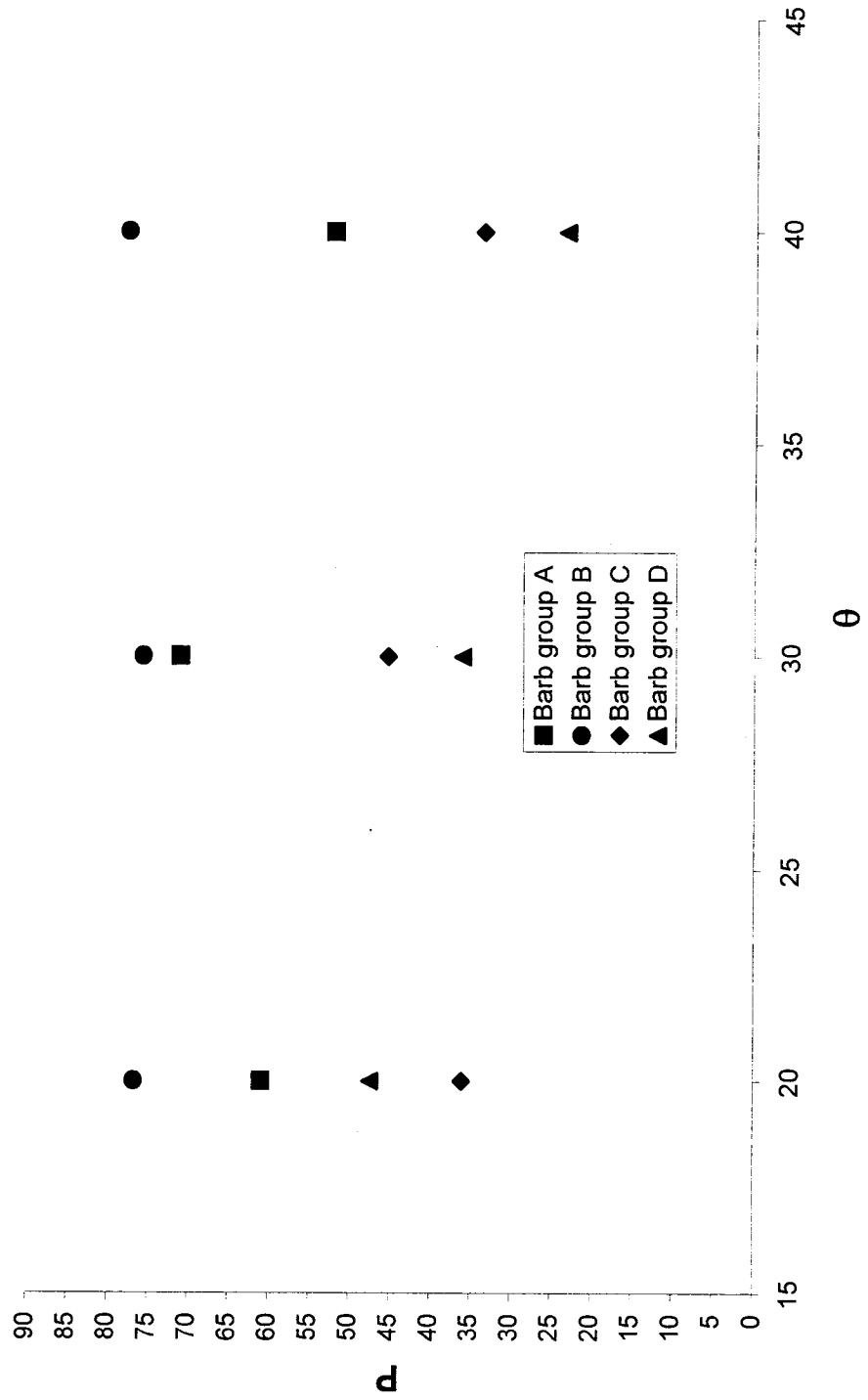


Figure A.1.1:  $P$  vs.  $\theta$  (90-degree bend,  $W/y_0=4.5$ ,  $h/y_0=0.5$ )

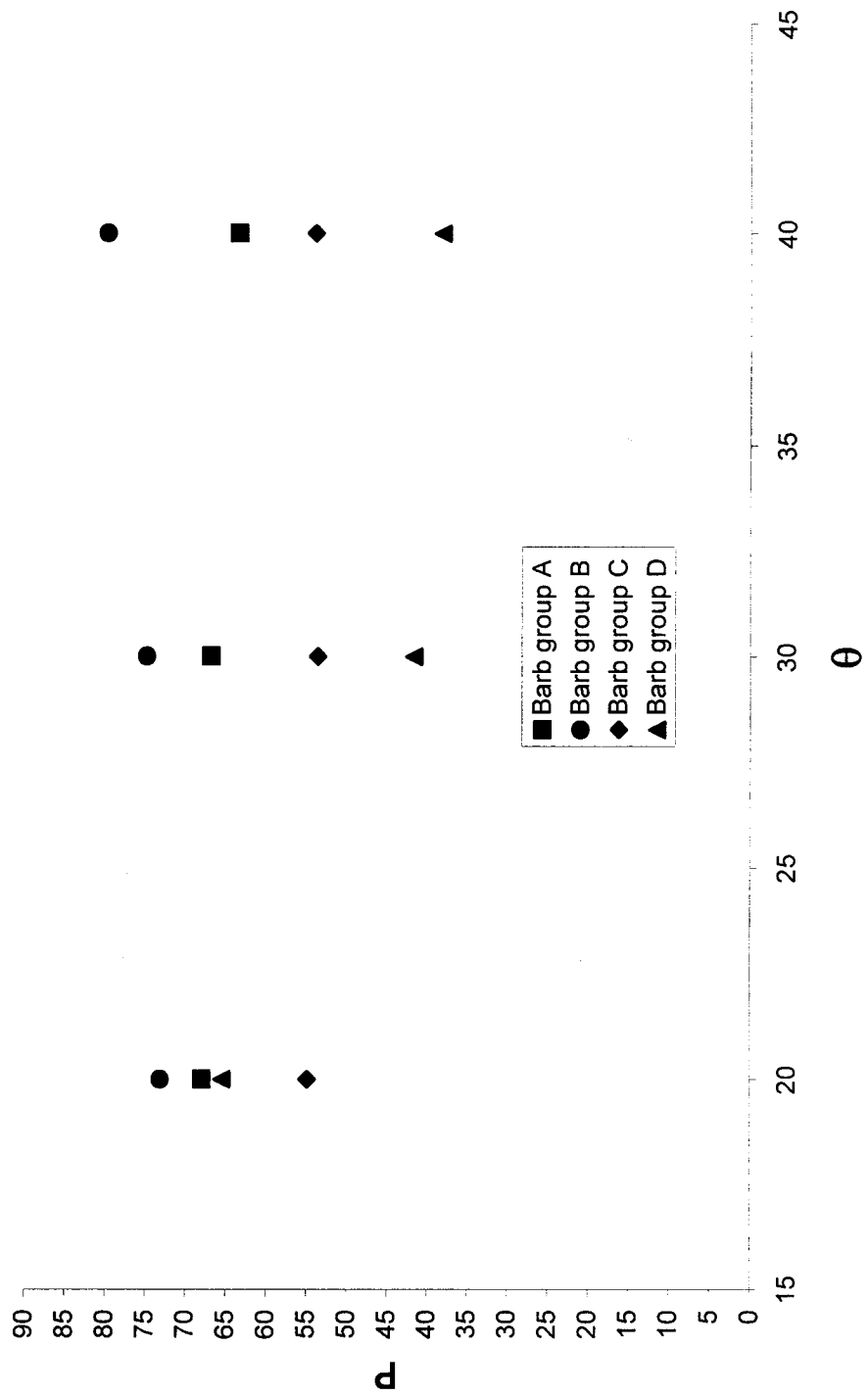
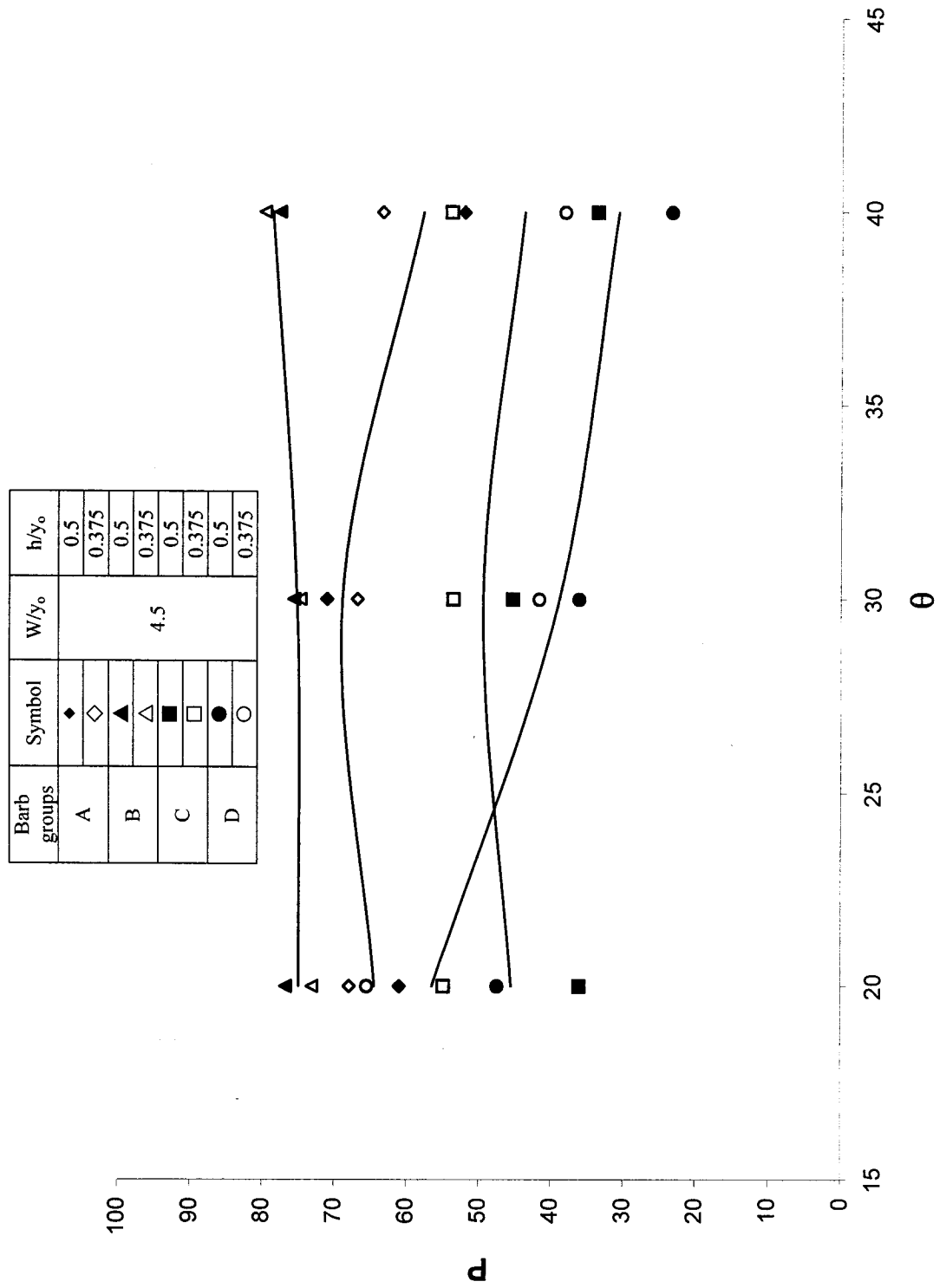


Figure A.1.2:  $P$  vs.  $\theta$  (90-degree bend,  $W/y_0=4.5$ ,  $h/y_0=0.375$ )



**Figure A.1.3:** P vs.  $\theta$  (90-degree bend,  $W/y_0=4.5$ ,  $h/y_0=0.5$  &  $0.375$ )

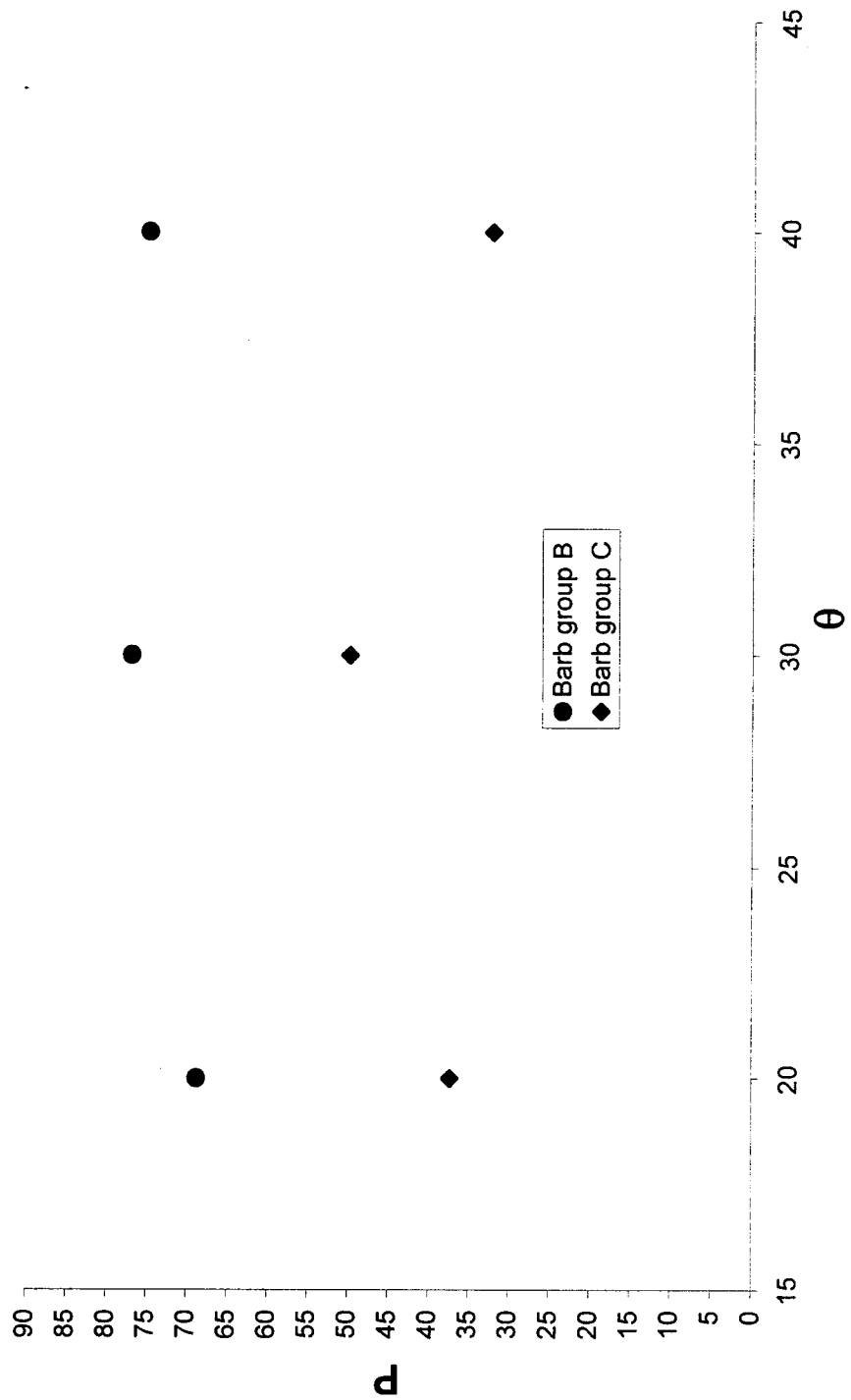
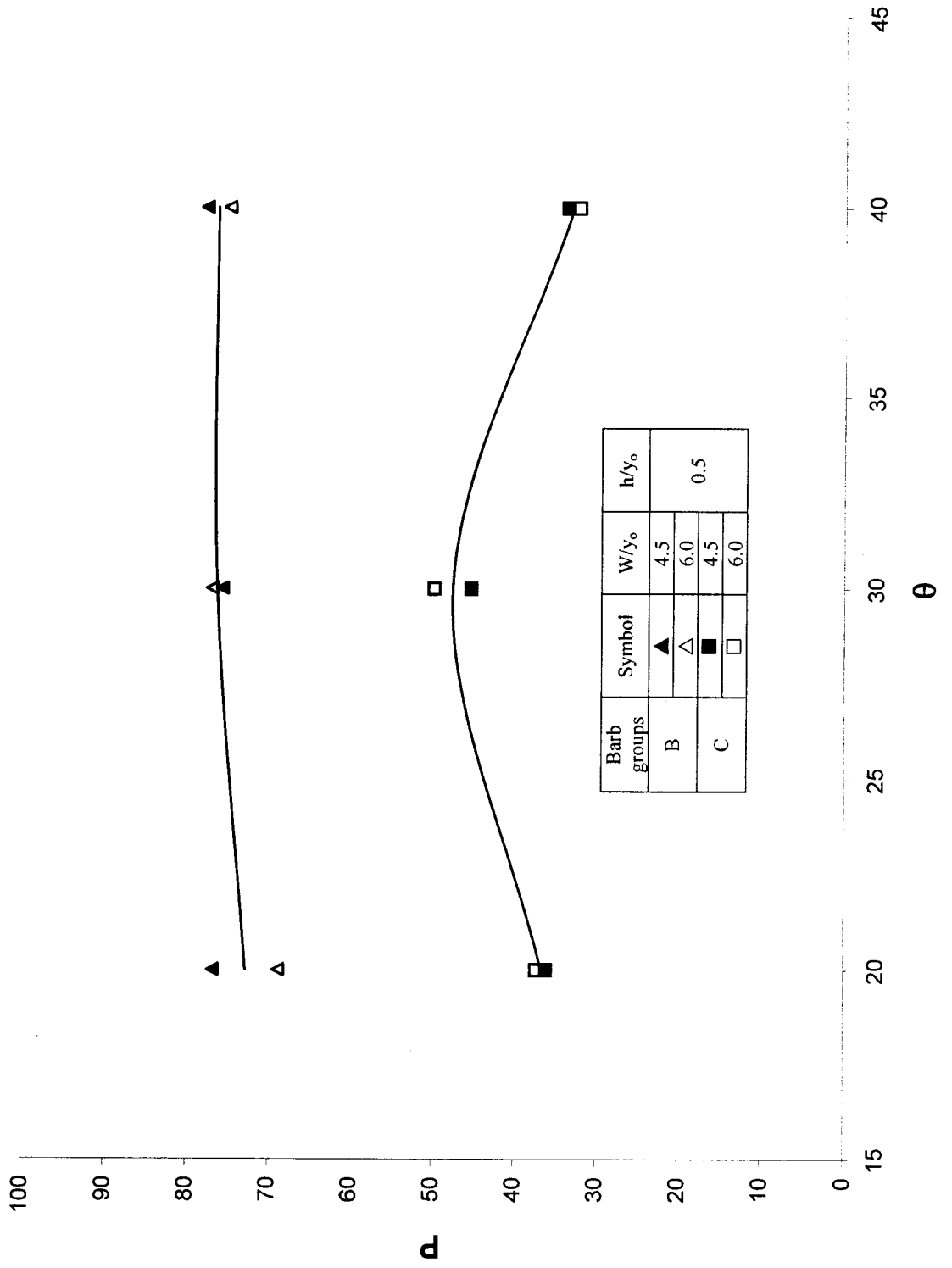
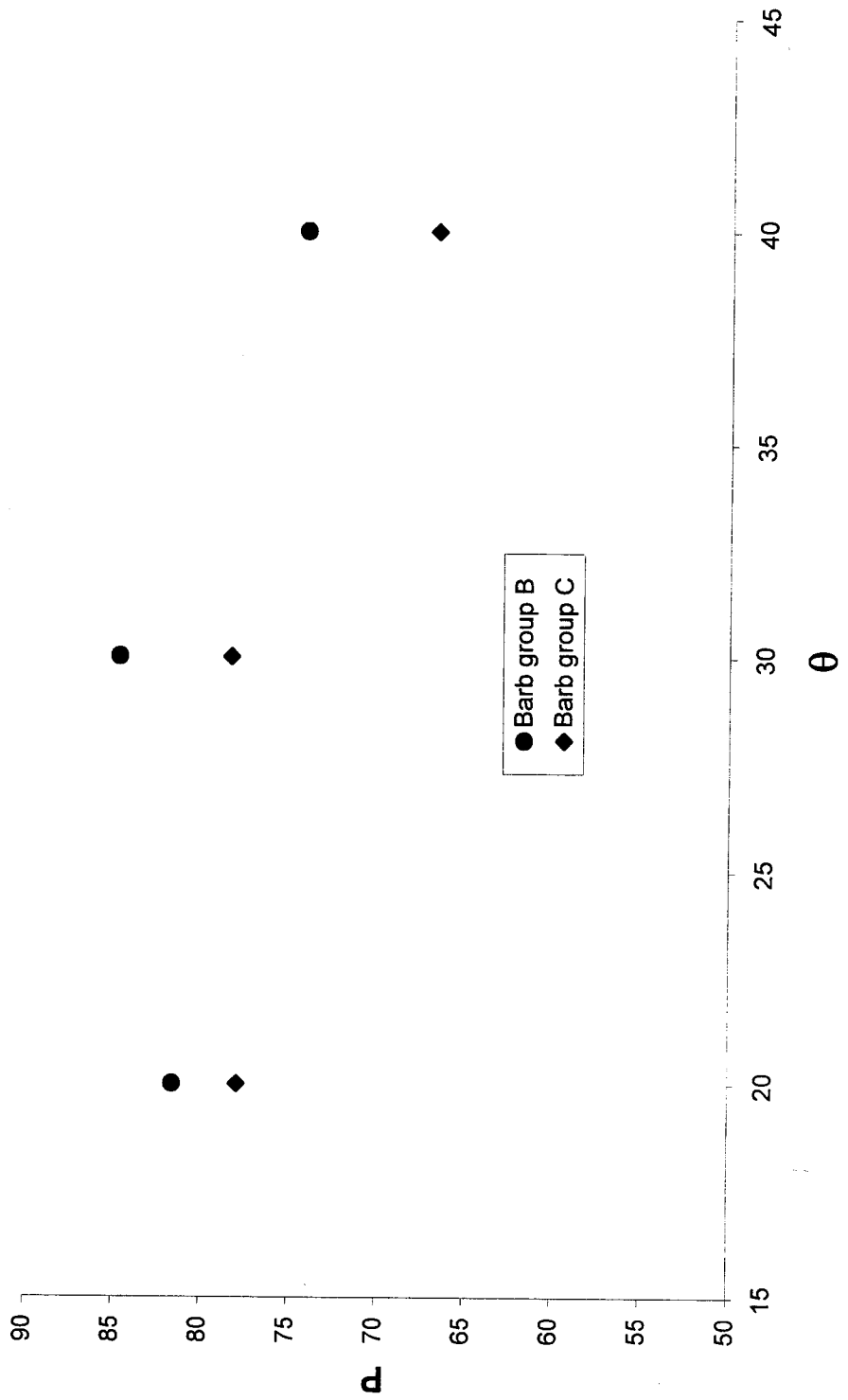


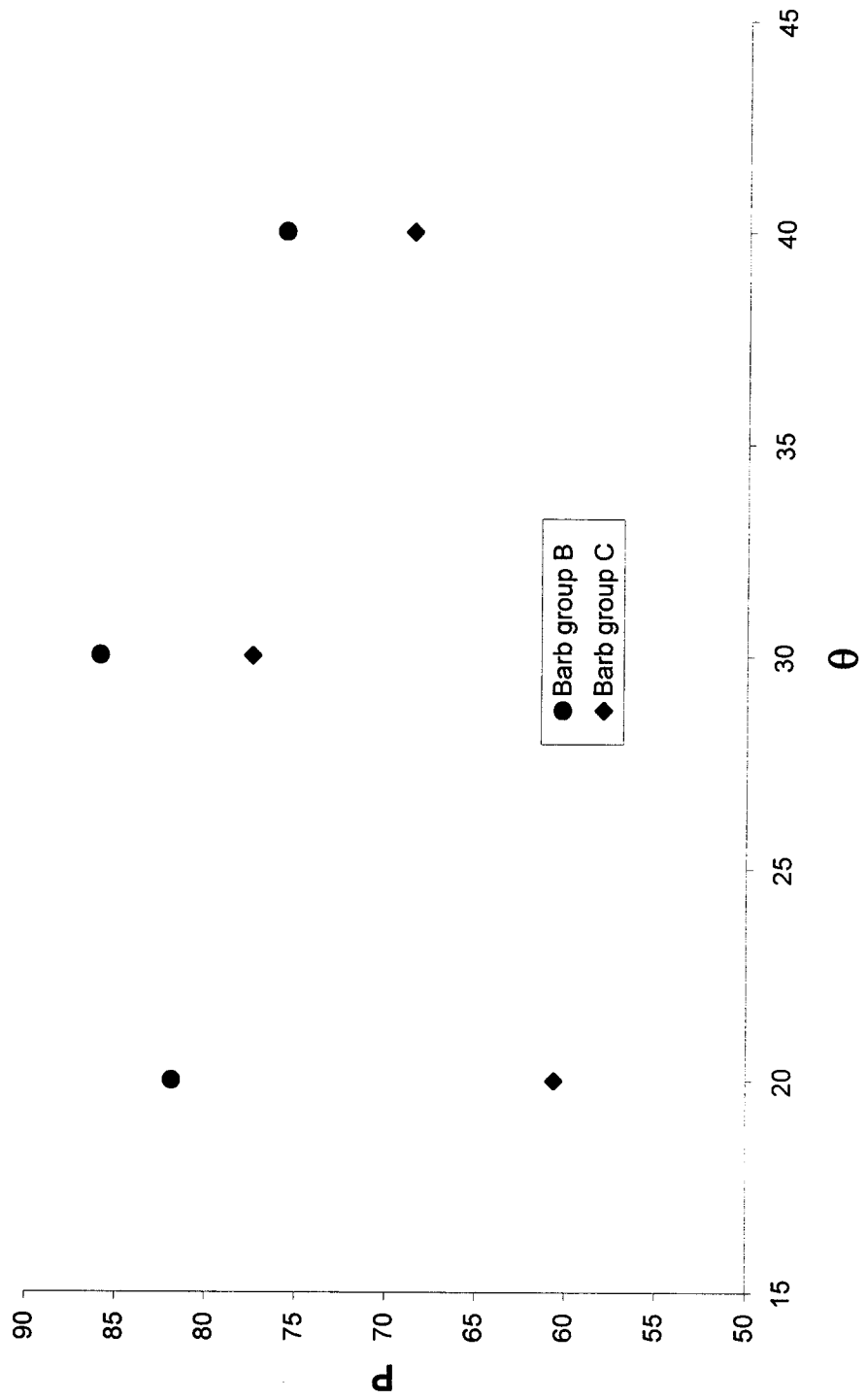
Figure A.1.4:  $P$  vs.  $\theta$  (90-degree bend,  $W/y_0=6.0$ ,  $h/y_0=0.5$ )



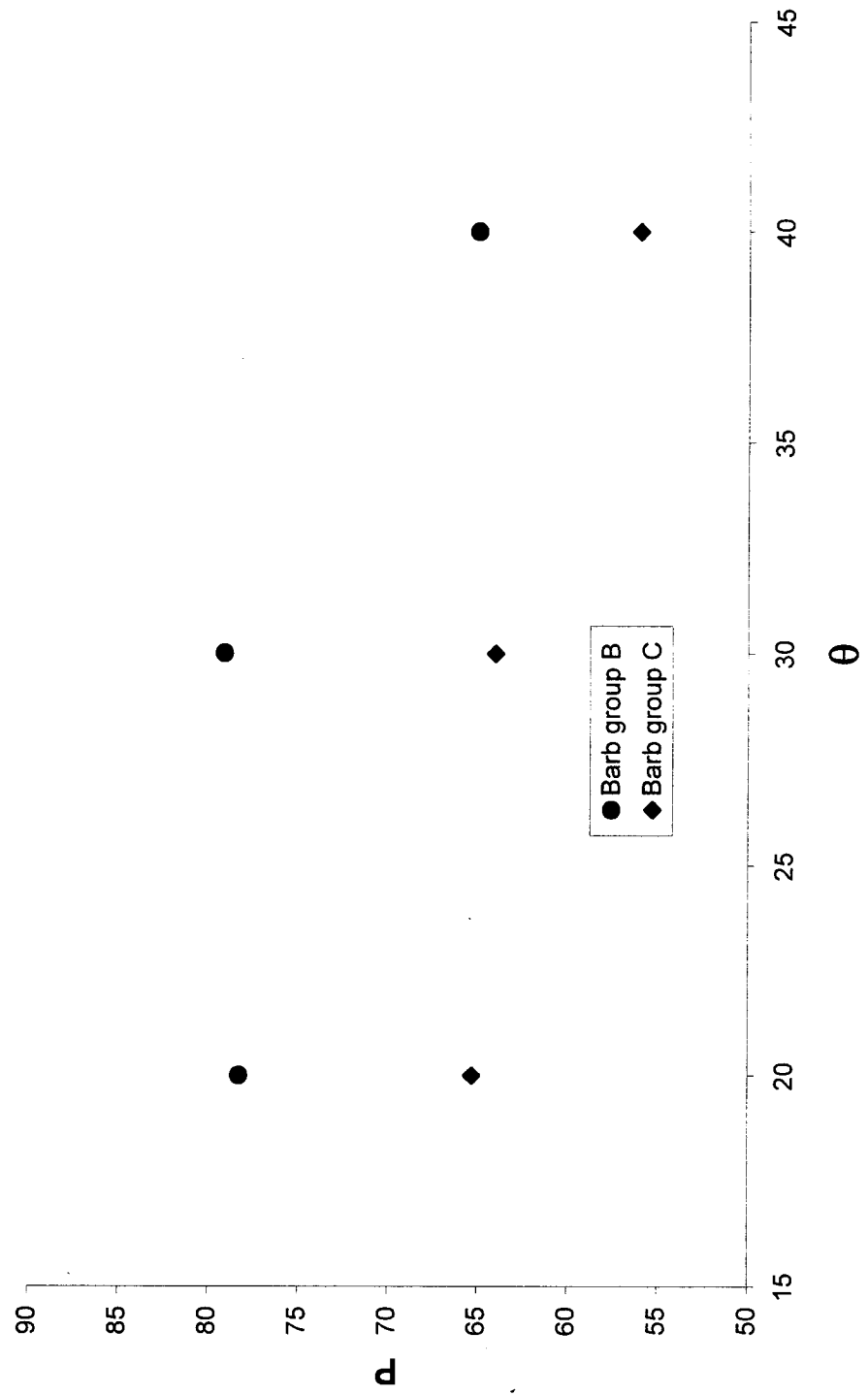
**Figure A.1.5:** P vs.  $\theta$  (90-degree bend,  $W/y_0=4.5$  &  $6.0$ ,  $h/y_0=0.5$ )



**Figure A.1.6:**  $P$  vs.  $\theta$  (135-degree bend,  $W/y_0=4.5$ ,  $h/y_0=0.5$ )



**Figure A.1.7:**  $P$  vs.  $\theta$  (135-degree bend,  $W/y_0=4.5$ ,  $h/y_0=0.375$ )



**Figure A.1.8:**  $P$  vs.  $\theta$  (135-degree bend,  $W/y_0=6.0$ ,  $h/y_0=0.5$ )

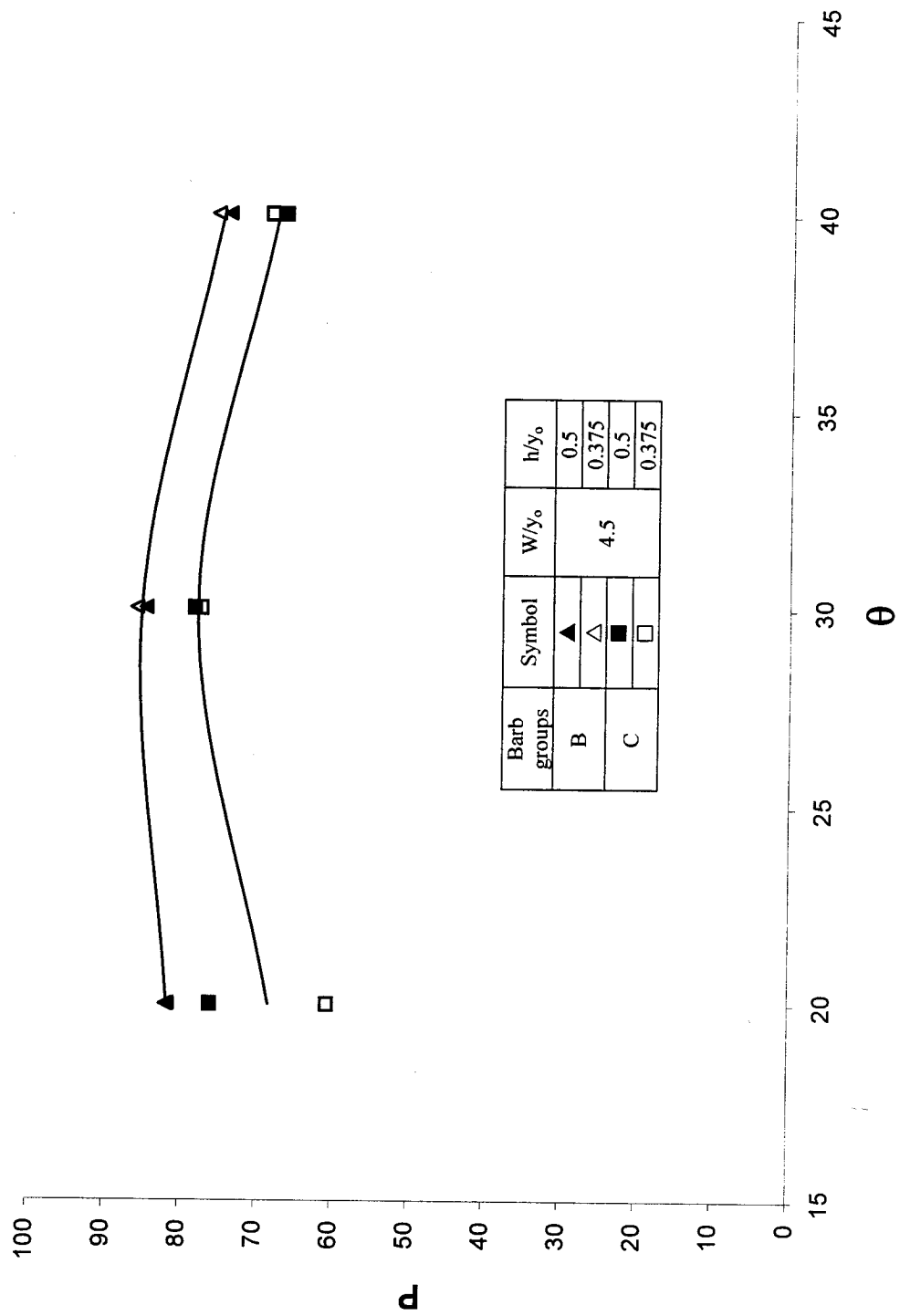


Figure A.1.9: P vs.  $\theta$  (135-degree bend,  $W/y_0=4.5$ ,  $h/y_0=0.5$  &  $0.375$ )

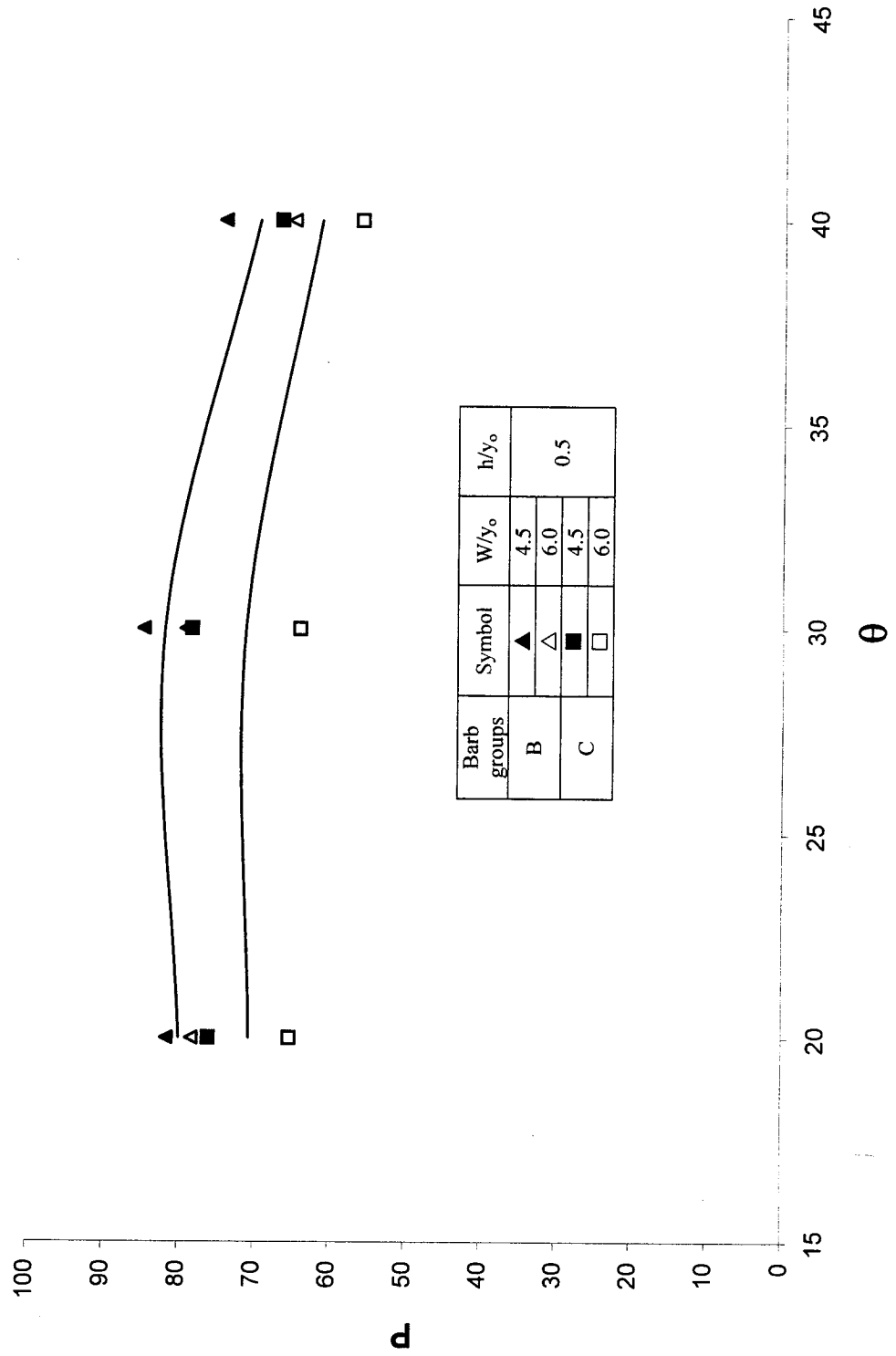


Figure A.1.10: P vs.  $\theta$  (135-degree bend,  $W/y_0=4.5$  &  $6.0$ ,  $h/y_0=0.5$ )

## **Appendix A.2**

**$x'/W$  vs.  $\theta$**

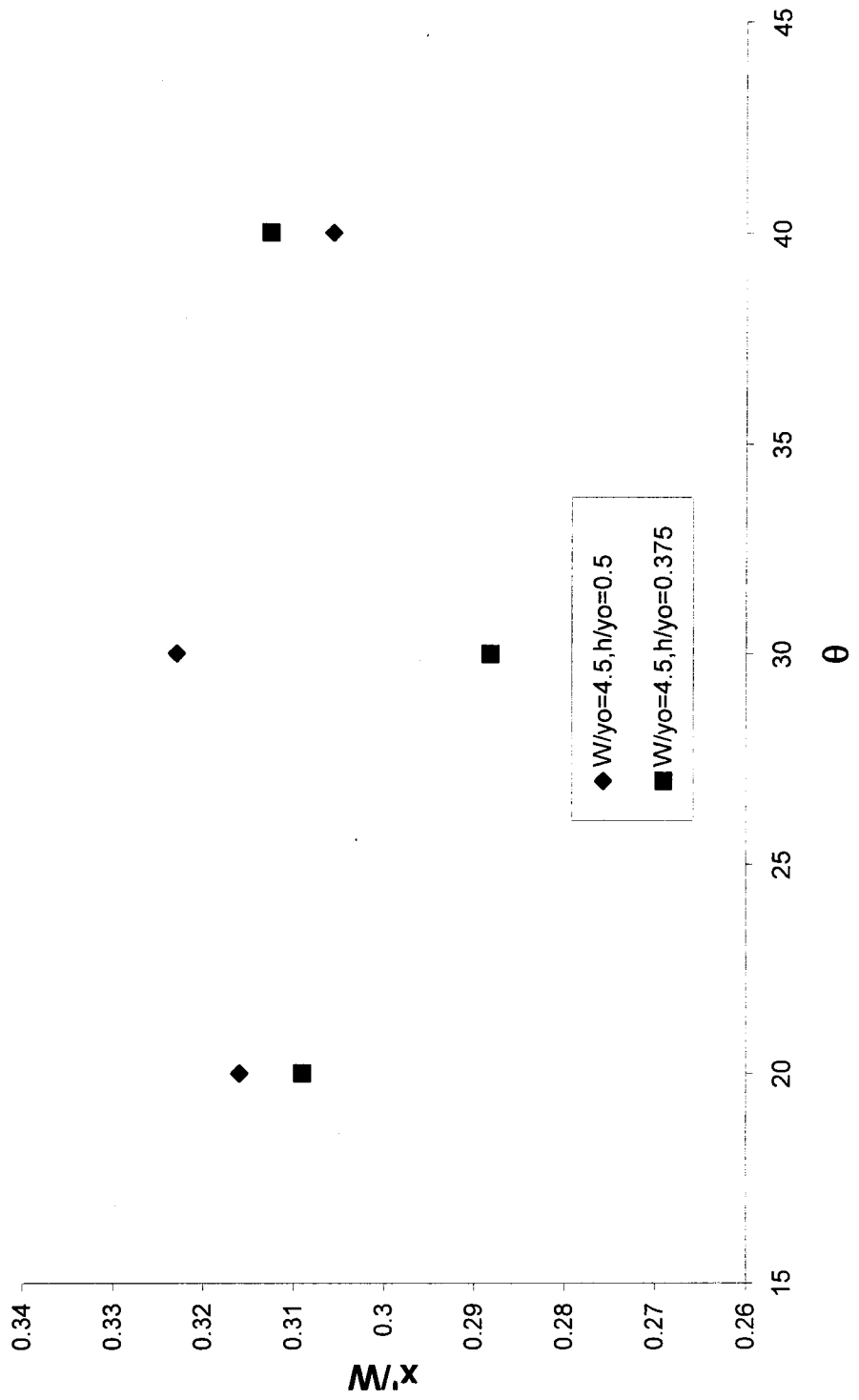


Figure A.2.1:  $x'/W$  vs.  $\theta$  (Barb group A, 90-degree bend)

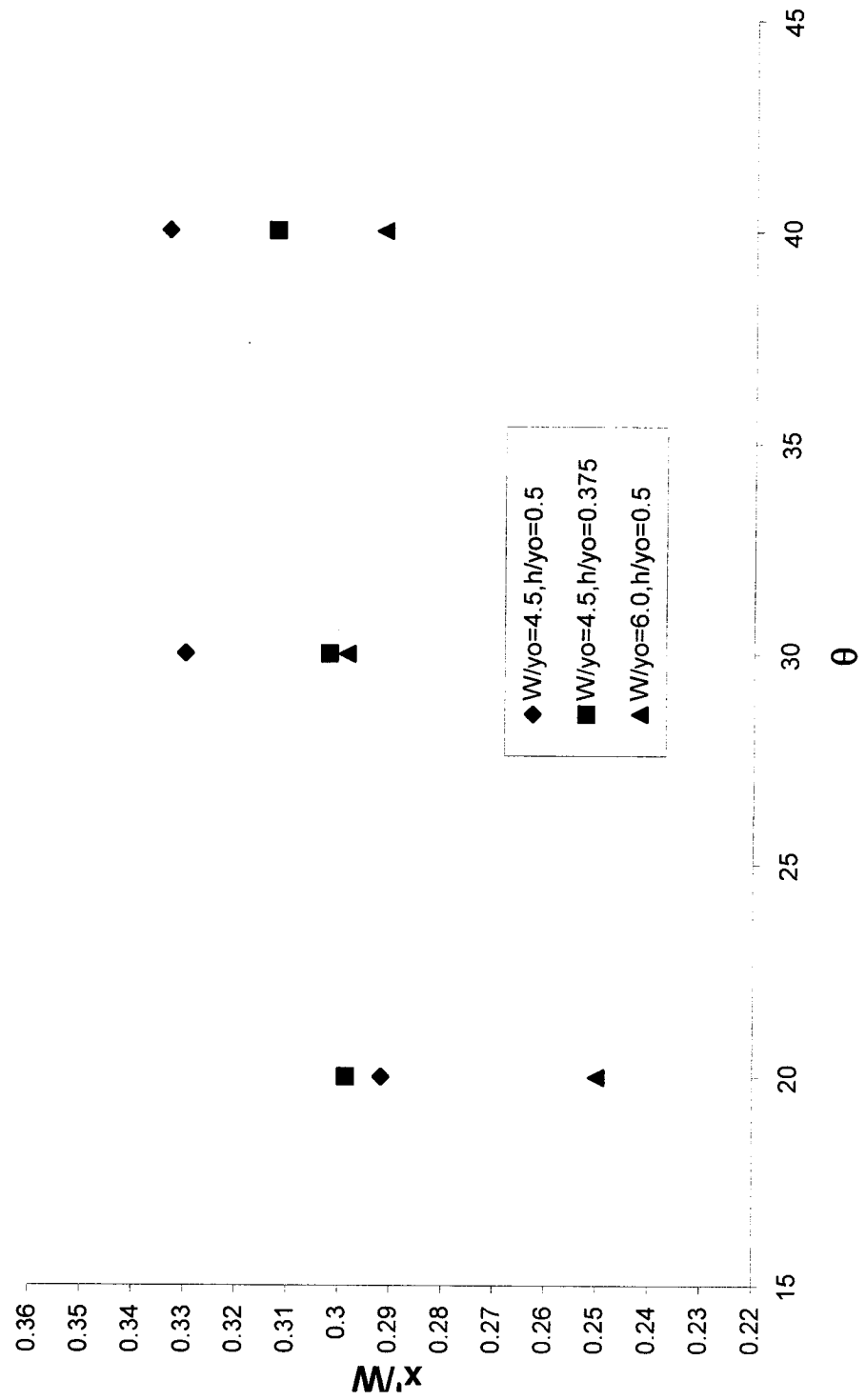


Figure A.2.2:  $x'/W$  vs.  $\theta$  (Barb group B, 90-degree bend)

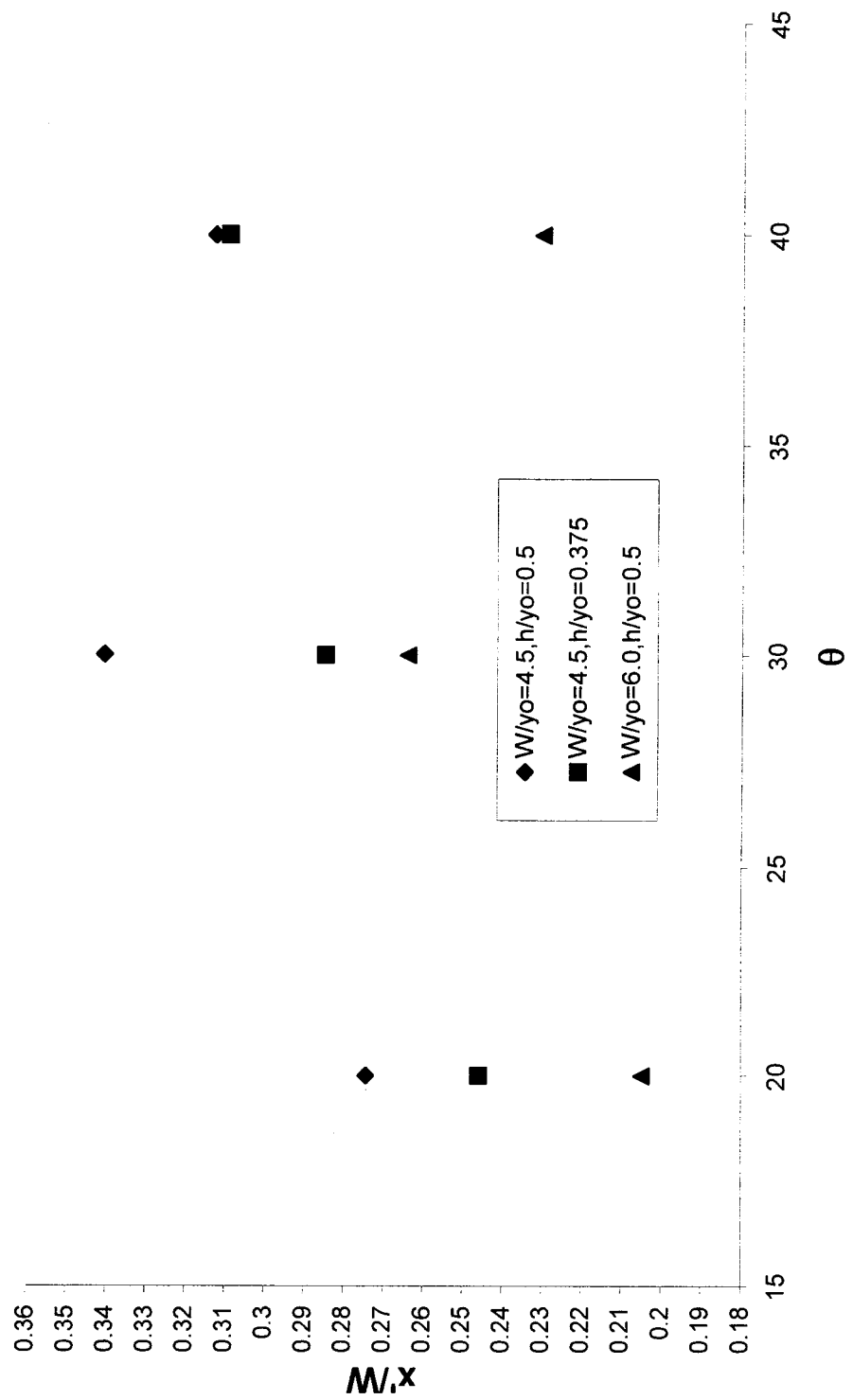
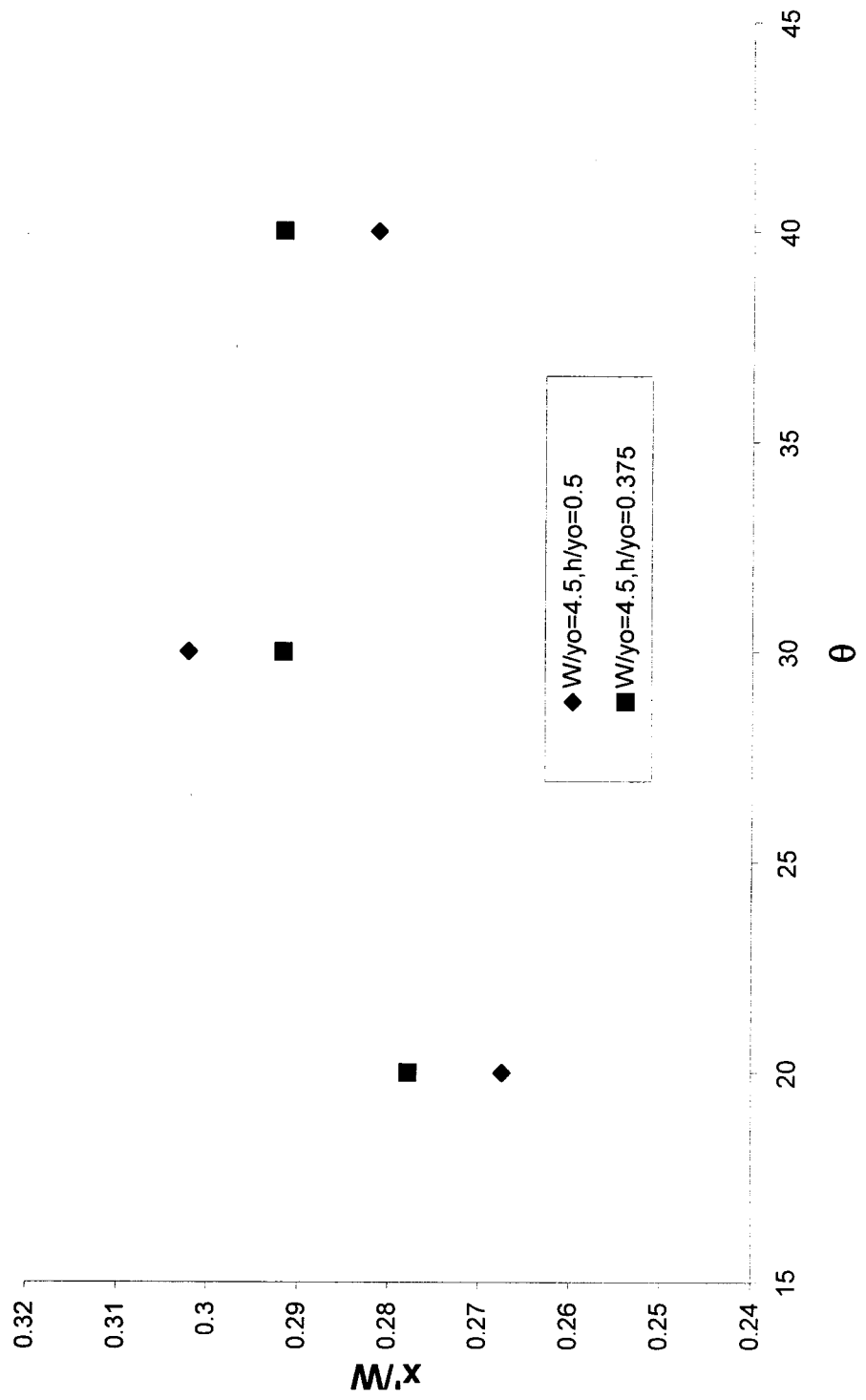


Figure A.2.3:  $x'/W$  vs.  $\theta$  (Barb group C, 90-degree bend)



**Figure A.2.4:**  $x'/W$  vs.  $\theta$  (Barb group D, 90-degree bend)

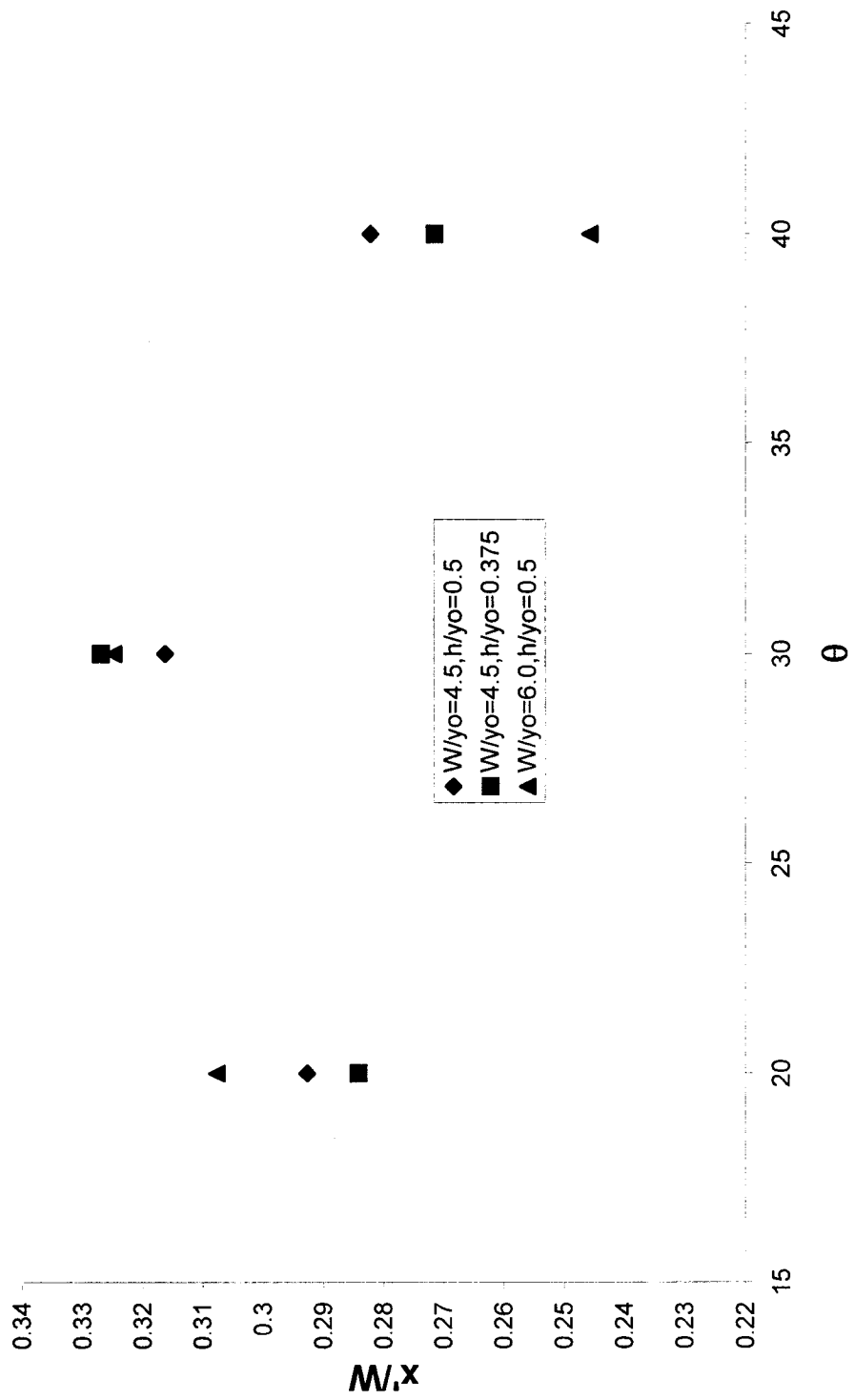


Figure A.2.5:  $x'/W$  vs.  $\theta$  (Barb group B, 135-degree bend)

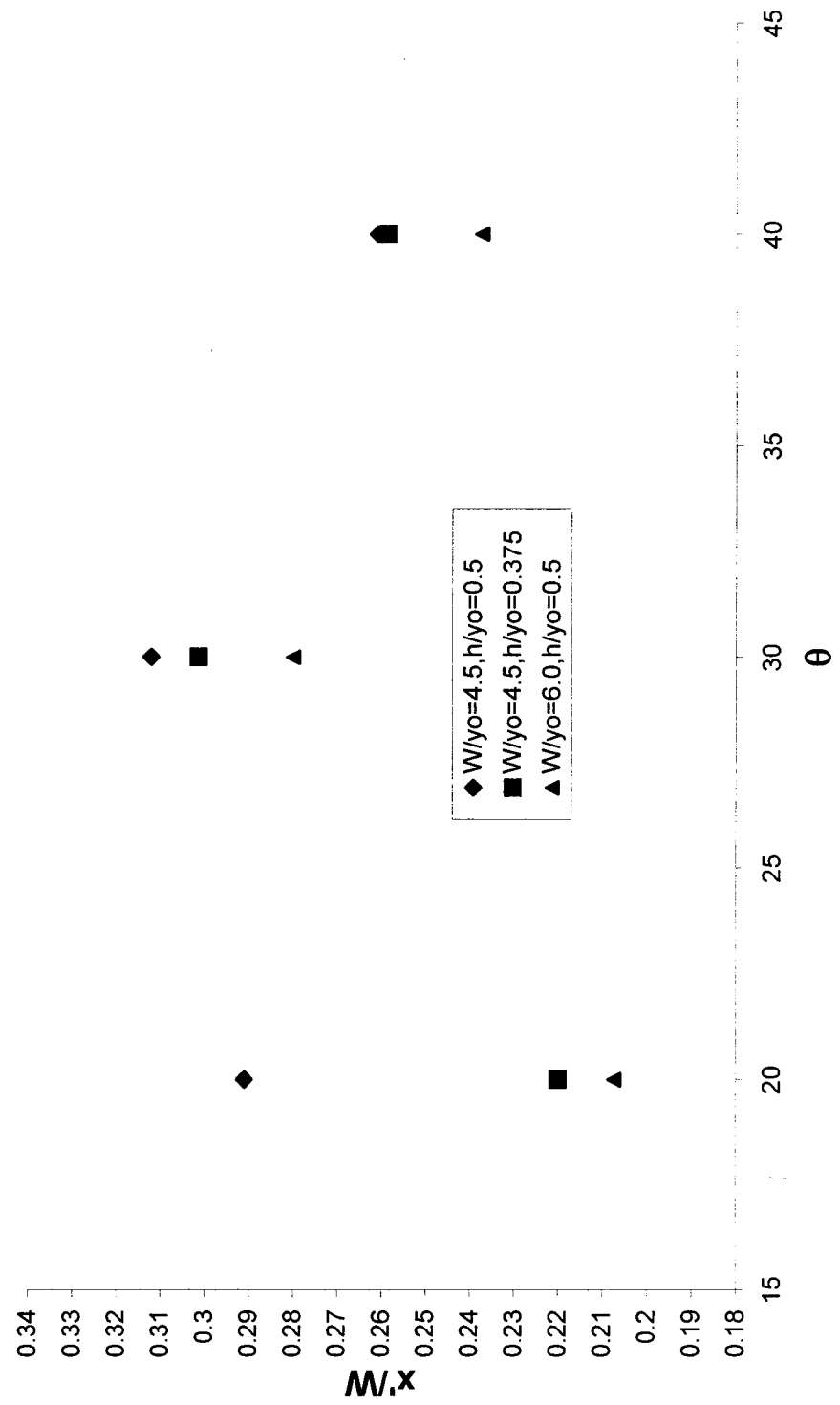


Figure A.2.6:  $x'/W$  vs.  $\theta$  (Barb group C, 135-degree bend)

## **Appendix A.3**

**P vs. L'/W**

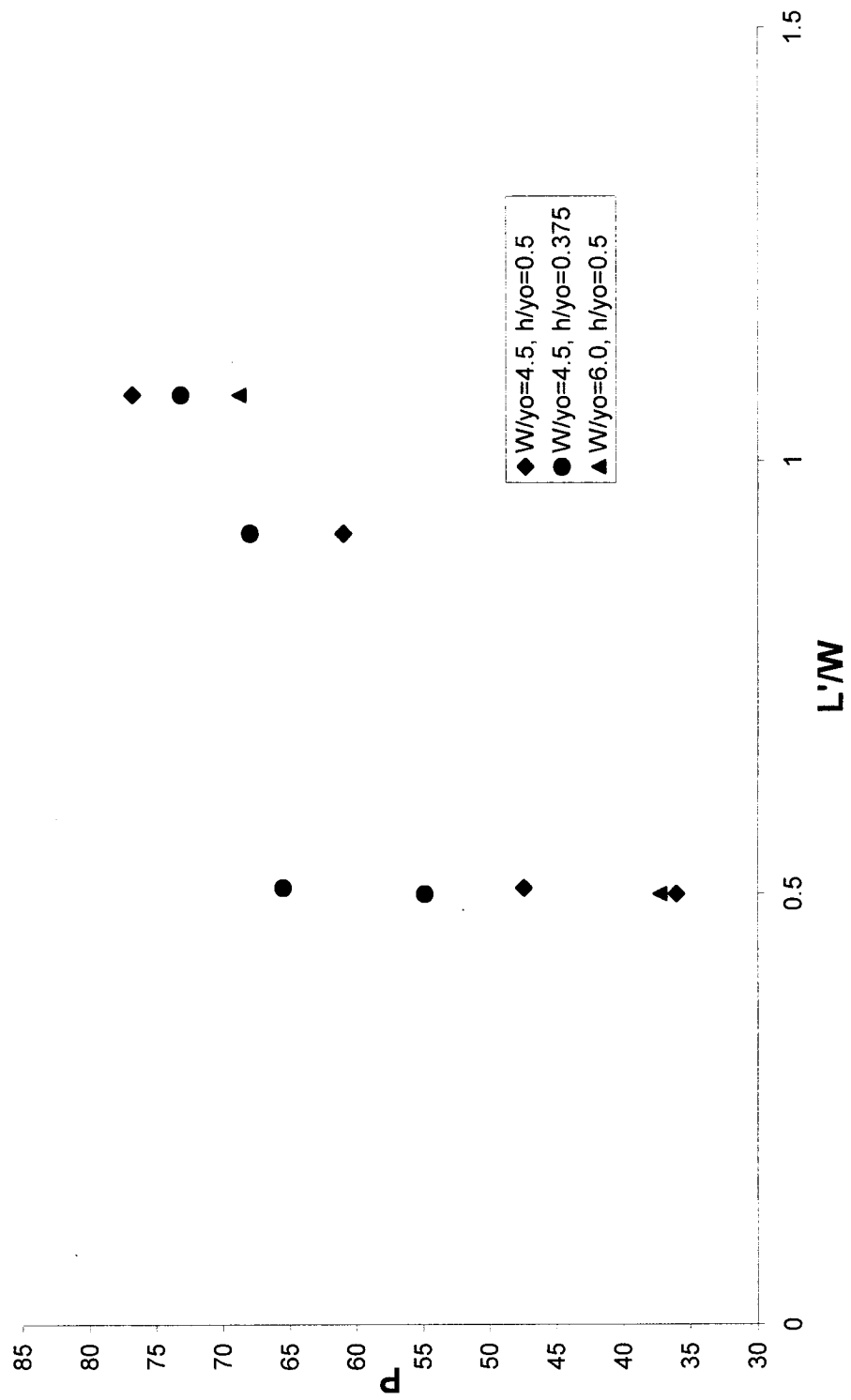


Figure A.3.1:  $P$  vs.  $L'/W$  (90-degree bend,  $\theta=20$ -degrees)

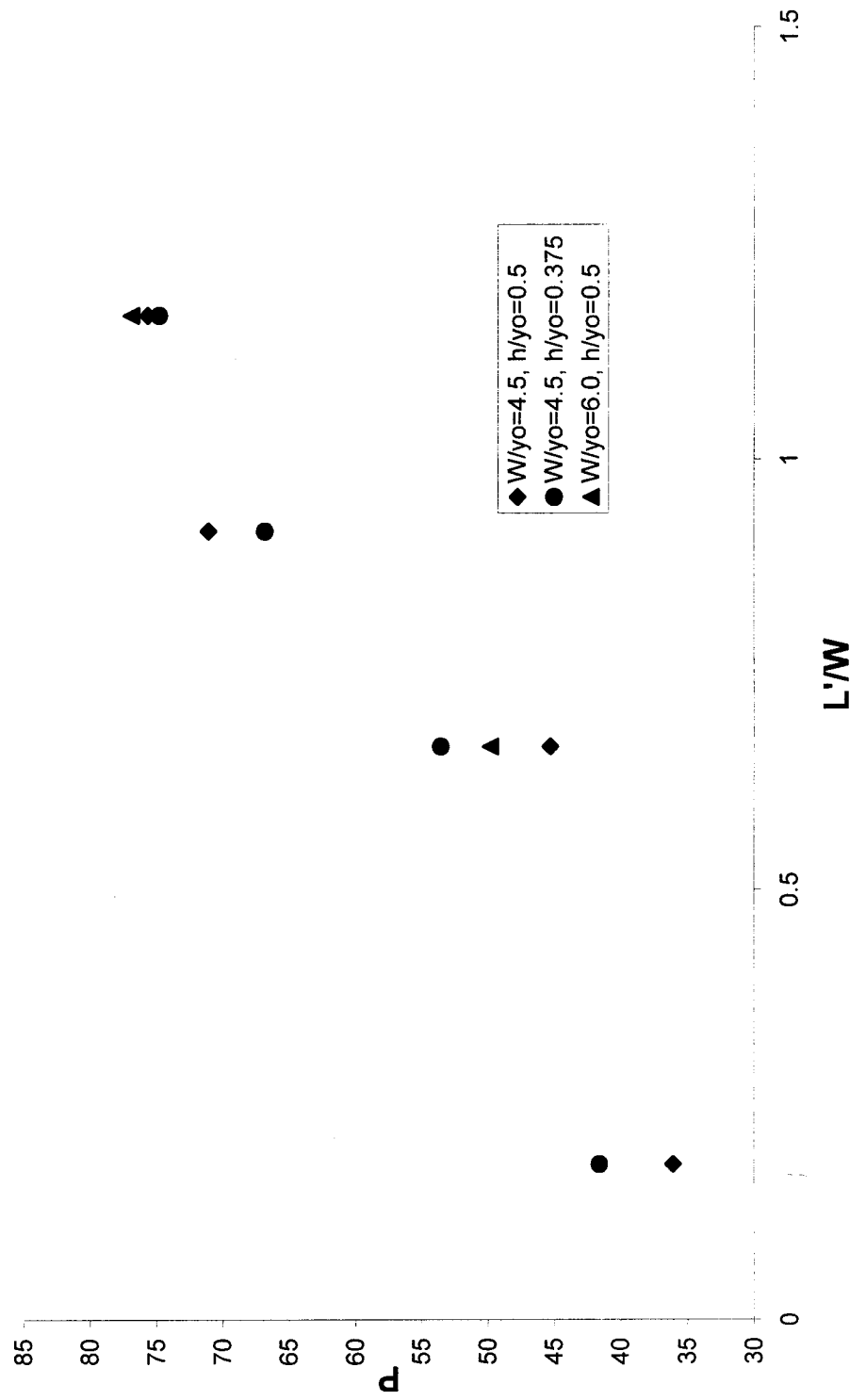


Figure A.3.2: P vs. L'/W (90-degree bend,  $\theta=30$ -degrees)

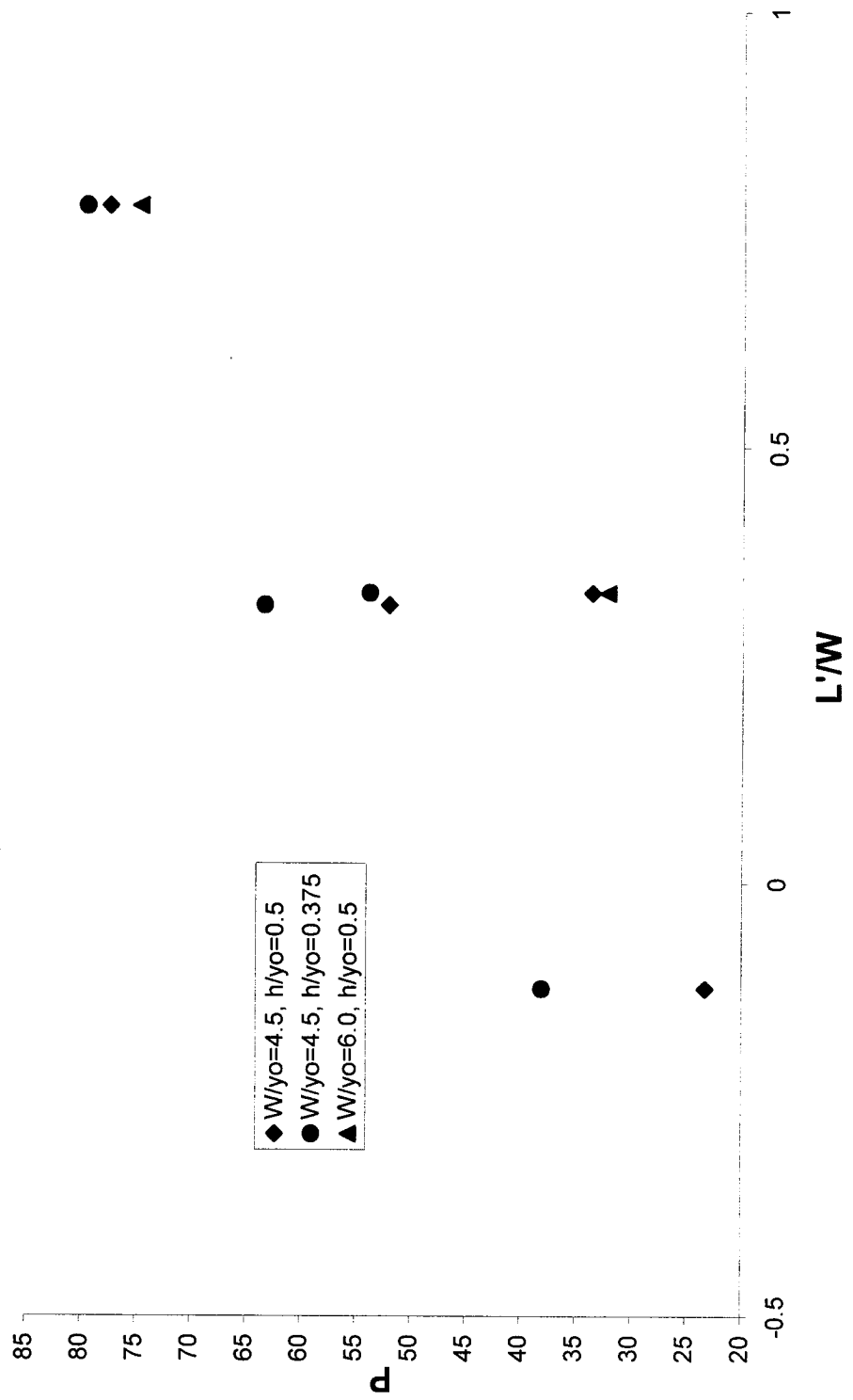


Figure A.3.3: P vs.  $L'/W$  (90-degree bend,  $\theta=40$ -degrees)

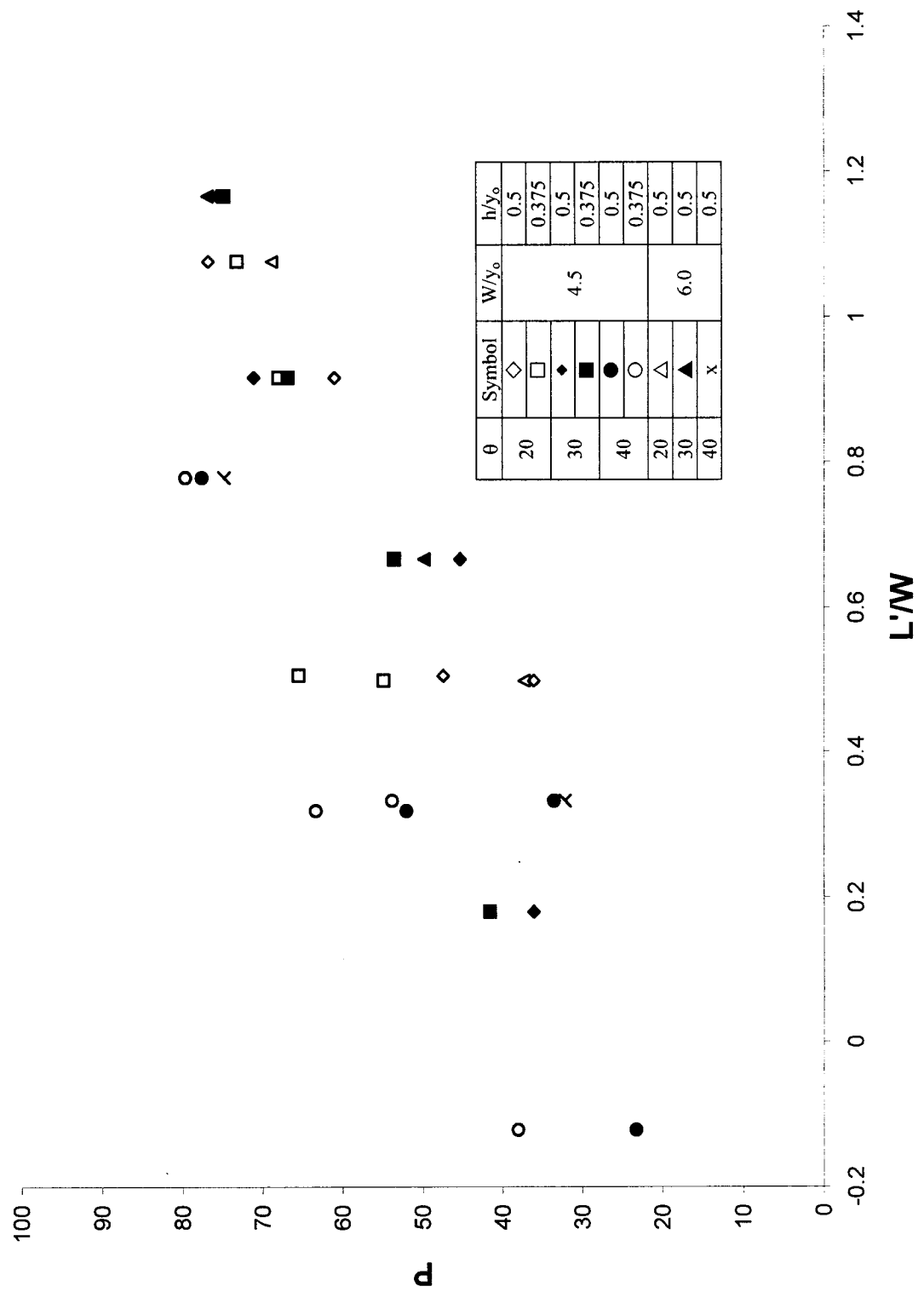


Figure A.3.4: P vs. L'/W (90-degree bend, All tests)

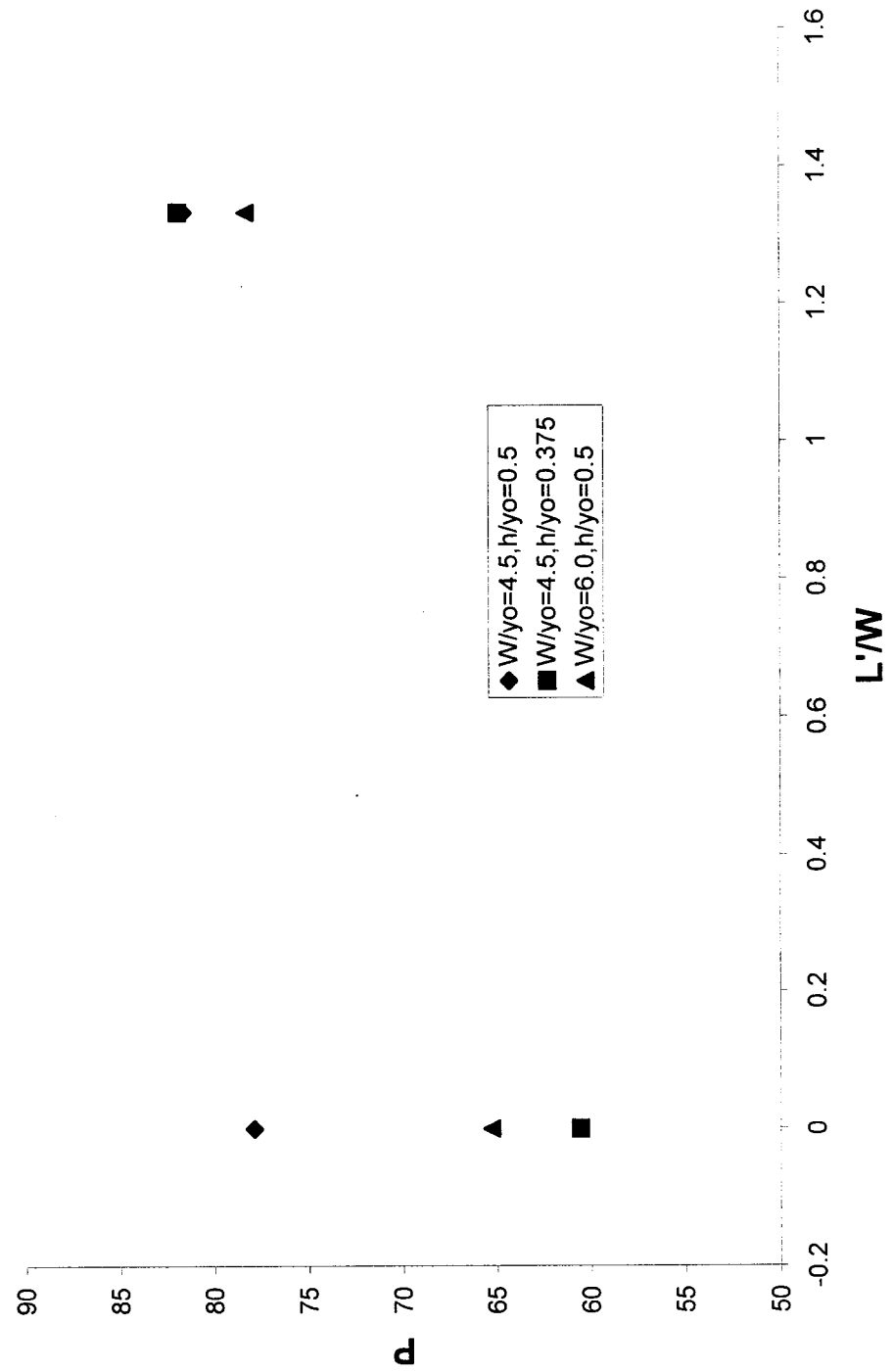


Figure A.3.5: P vs. L'/W (135-degree bend,  $\theta=20$ -degrees)

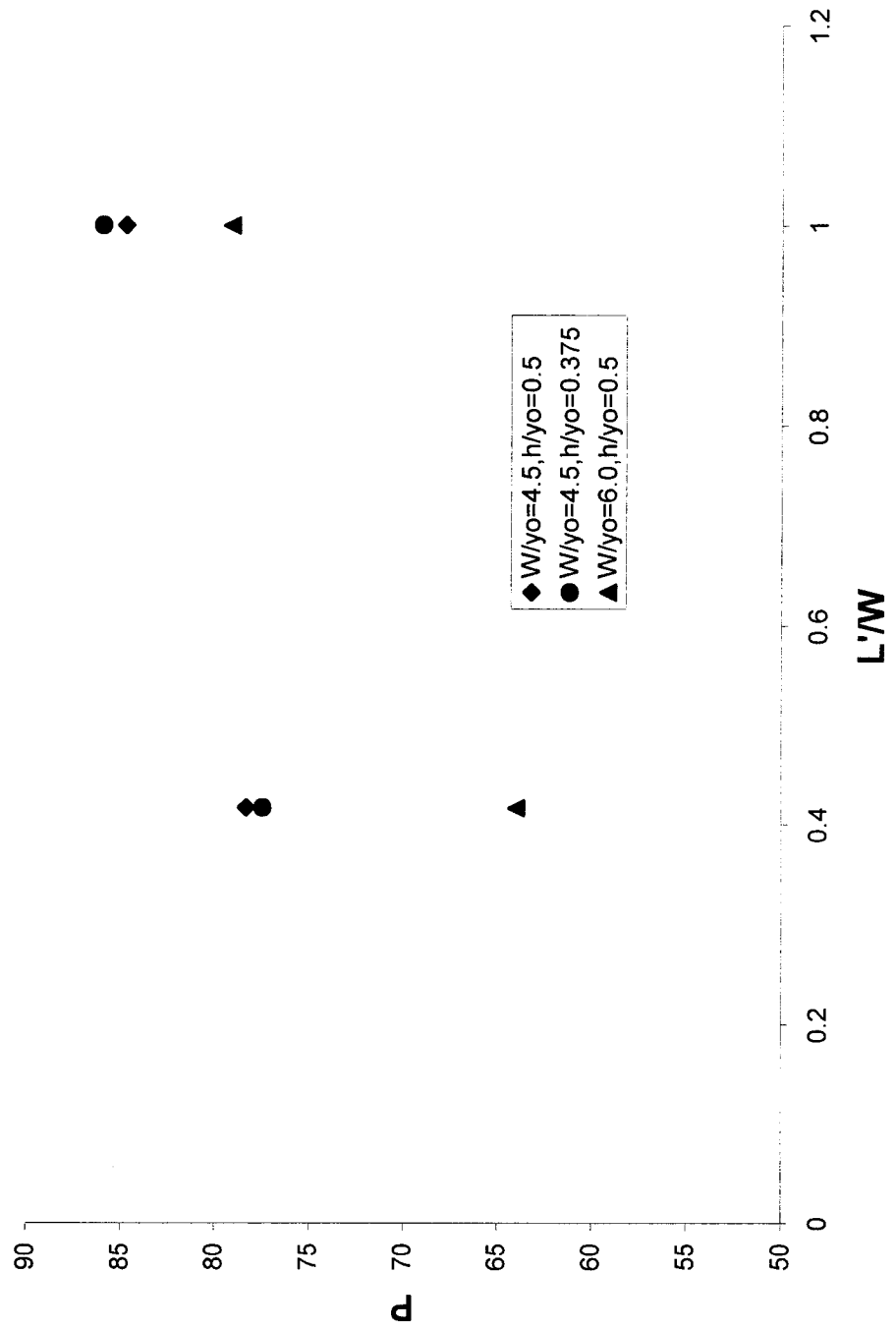


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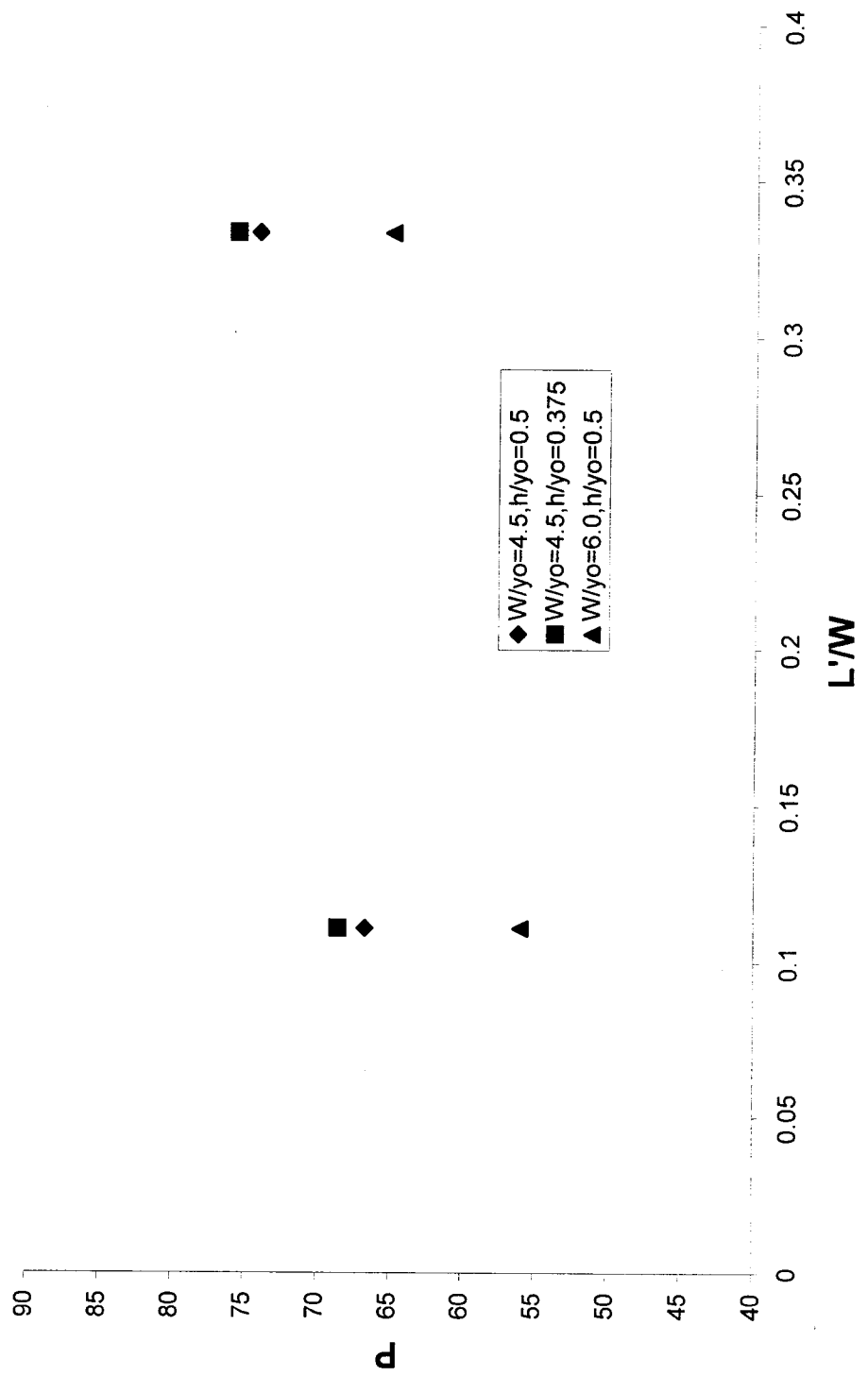


Figure A.3.7: P vs.  $L'/W$  (135-degree bend,  $\theta=40$ -degrees)

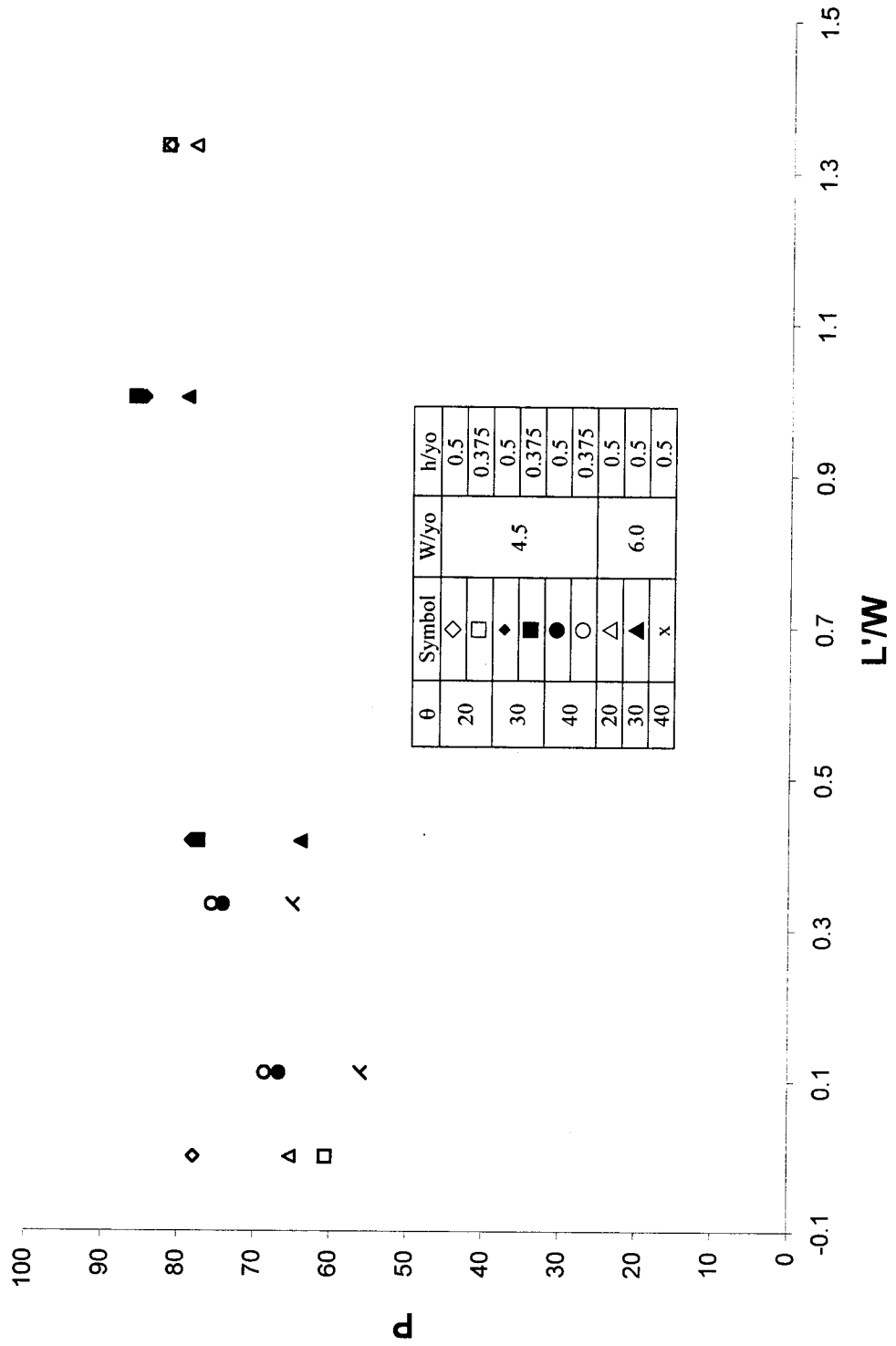


Figure A.3.8: P vs. L'/W (135-degree bend, All tests)

**Appendix A.4**  
**Bend Comparison Figures**

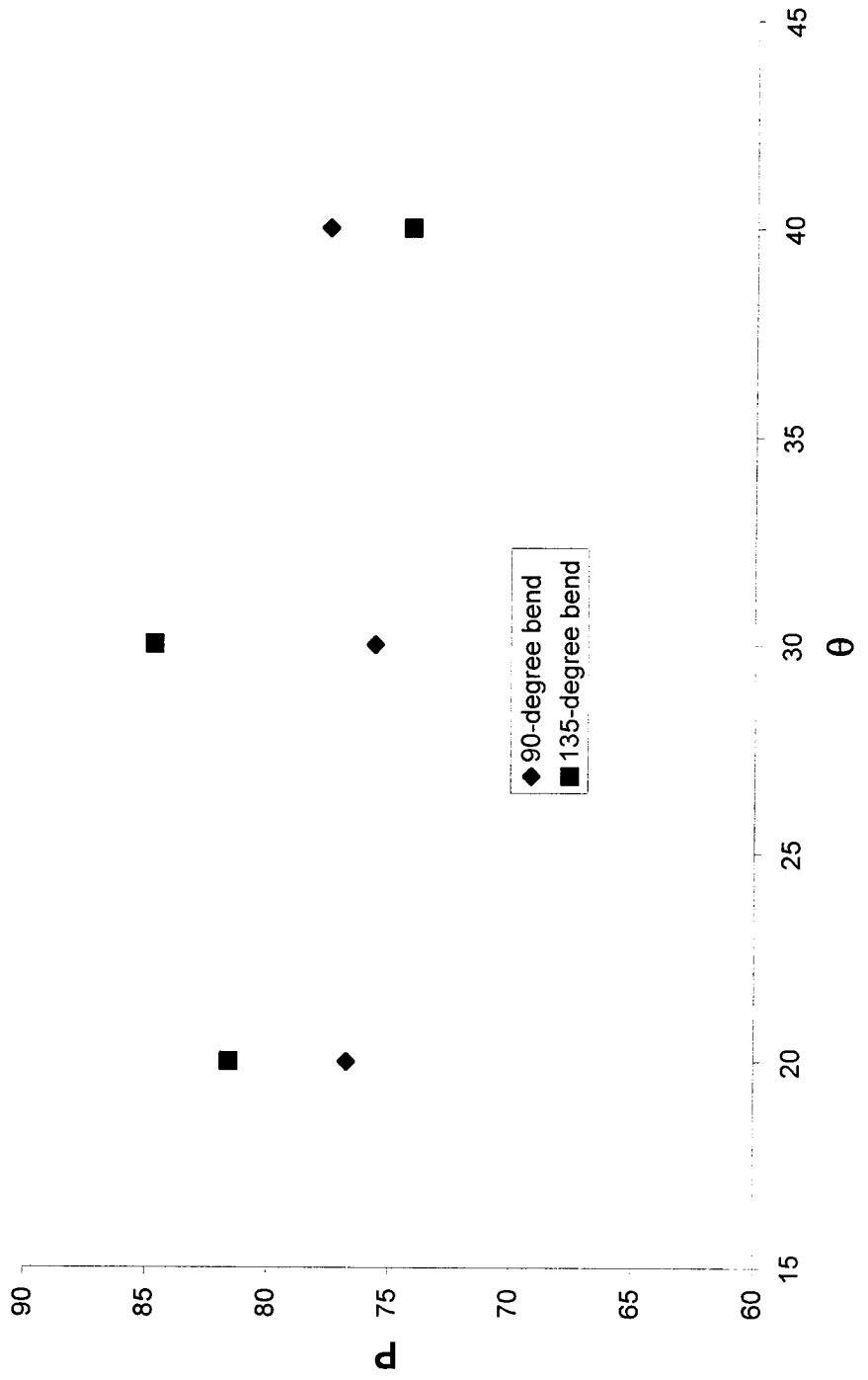


Figure A.4.1:  $P$  vs.  $\theta$  (Barb group B,  $W/y_0=4.5$ ,  $h/y_0=0.5$ )

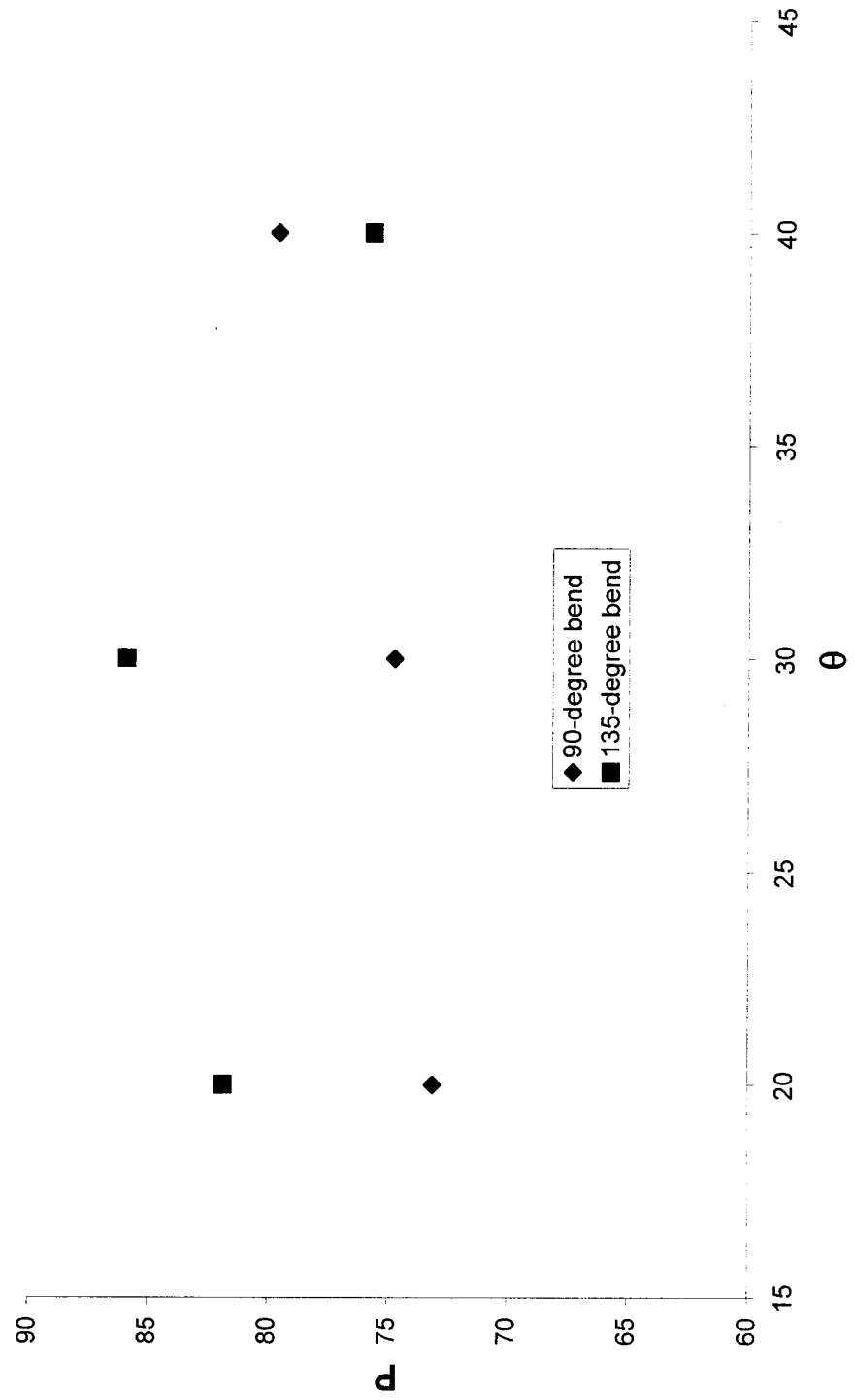
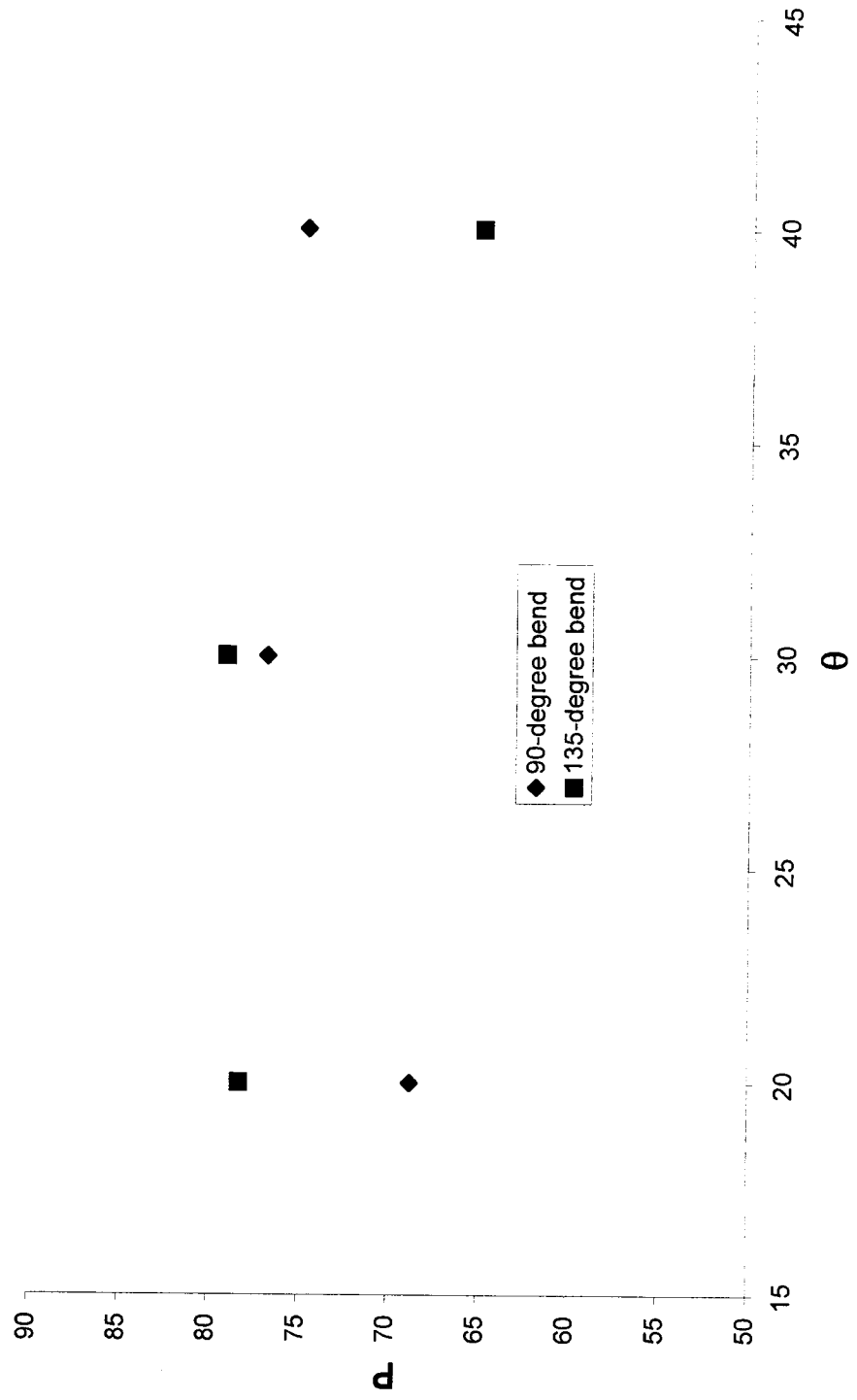
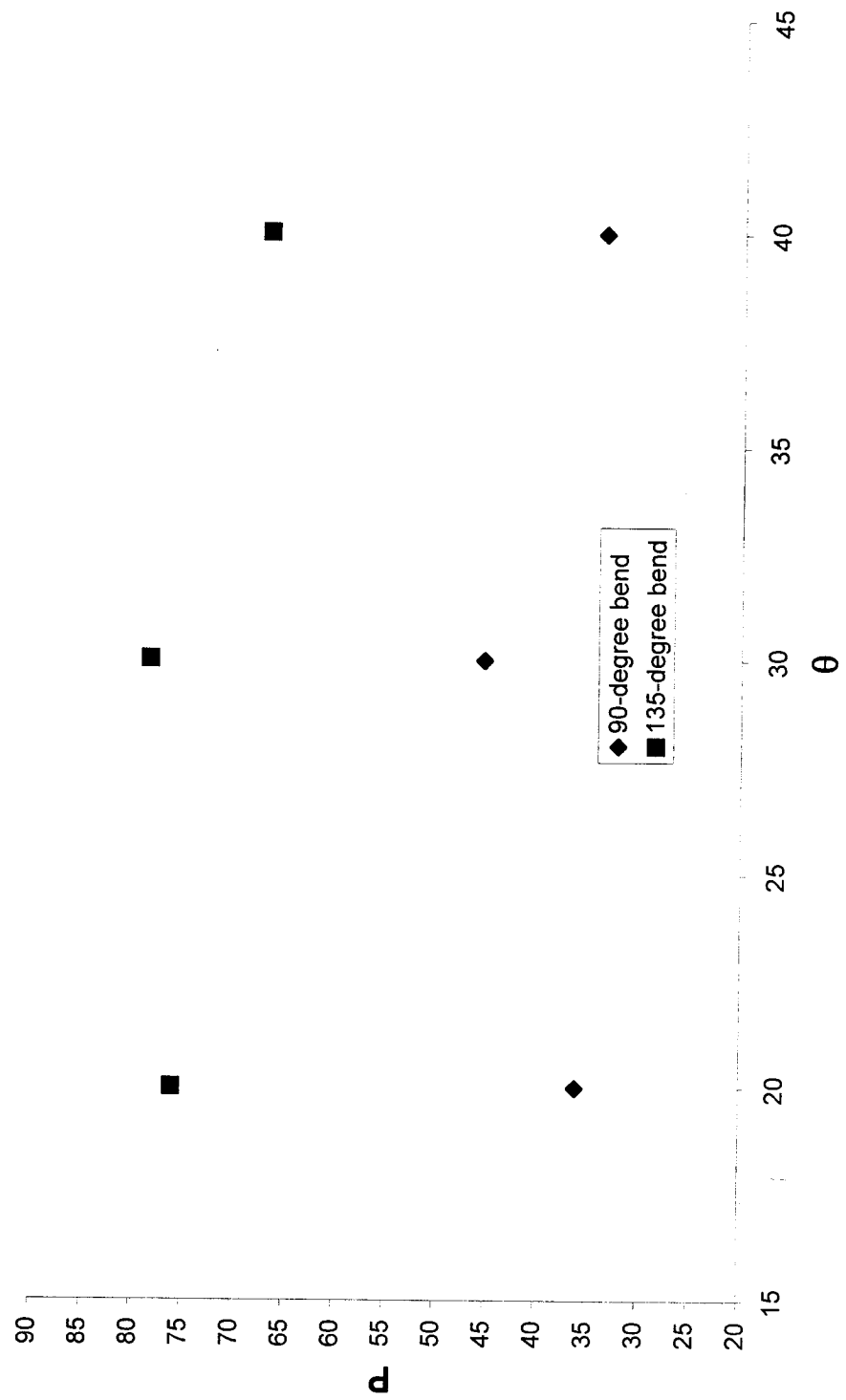


Figure A.4.2:  $P$  vs.  $\theta$  (Barb group B,  $W/y_0=4.5$ ,  $h/y_0=0.375$ )



**Figure A.4.3:**  $P$  vs.  $\theta$  (Barb group B,  $W/y_0=6.0$ ,  $h/y_0=0.5$ )



**Figure A.4.4:** P vs.  $\theta$  (Barb group C,  $W/y_0=4.5$ ,  $h/y_0=0.5$ )

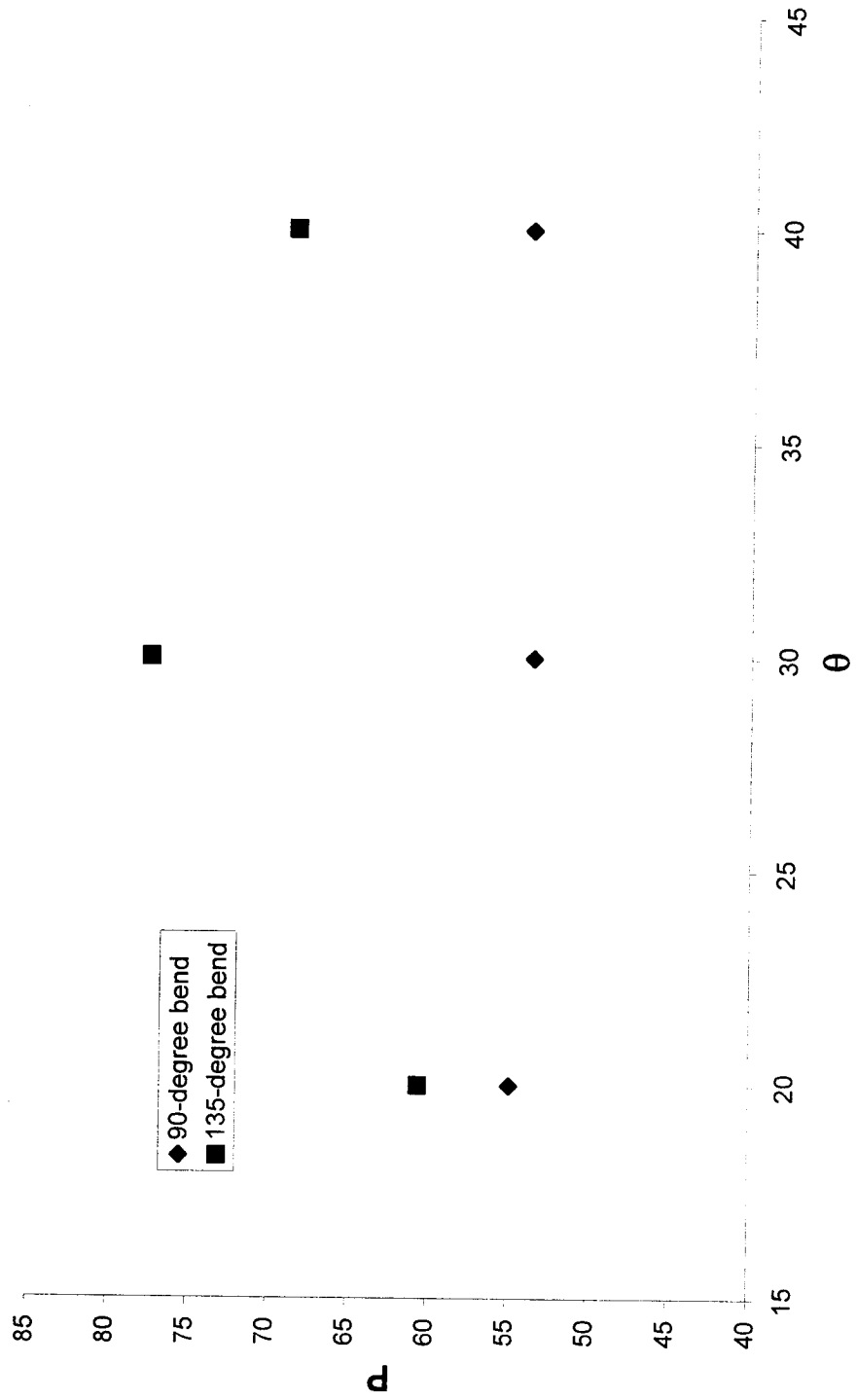


Figure A.4.5:  $P$  vs.  $\theta$  (Barb group C,  $W/y_0=4.5$ ,  $h/y_0=0.375$ )

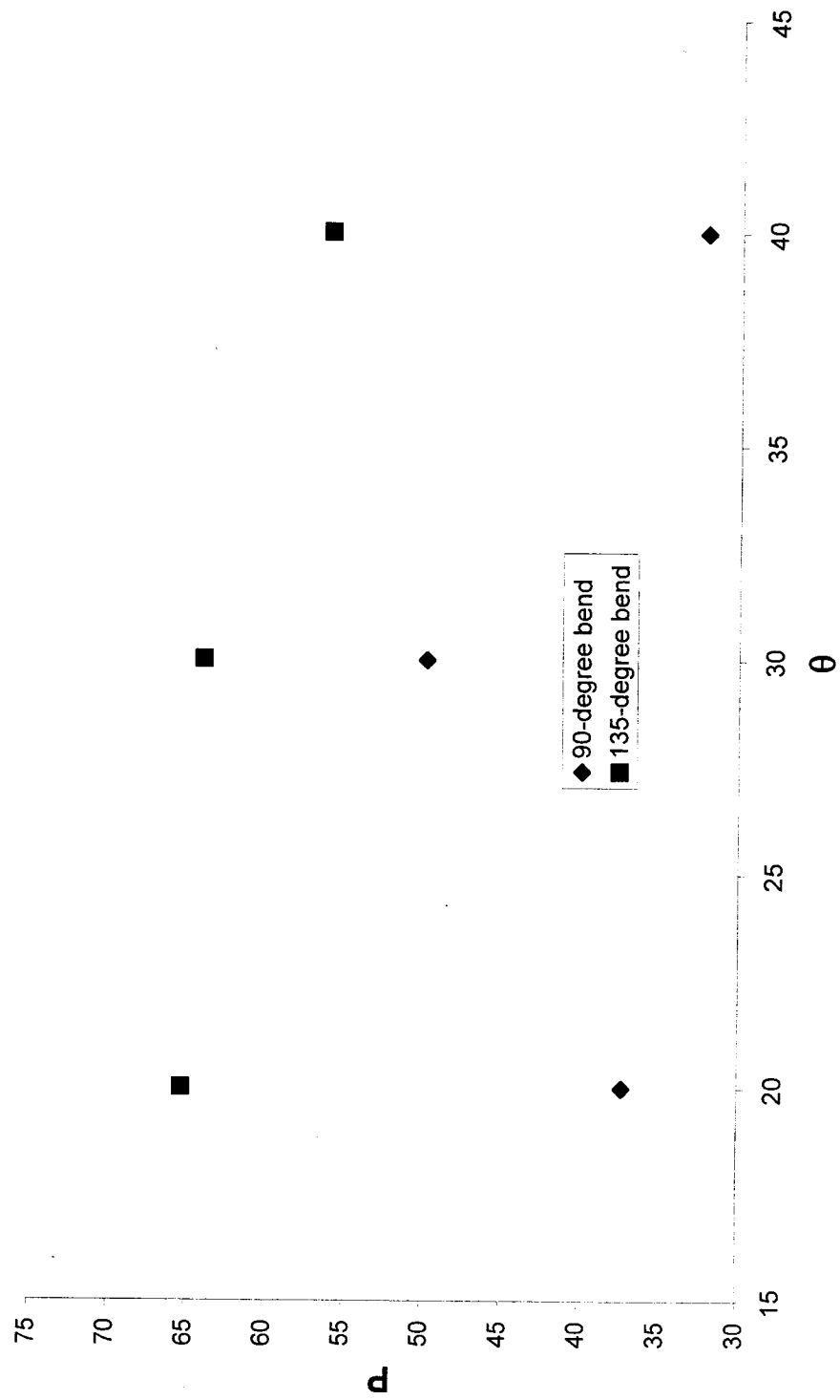
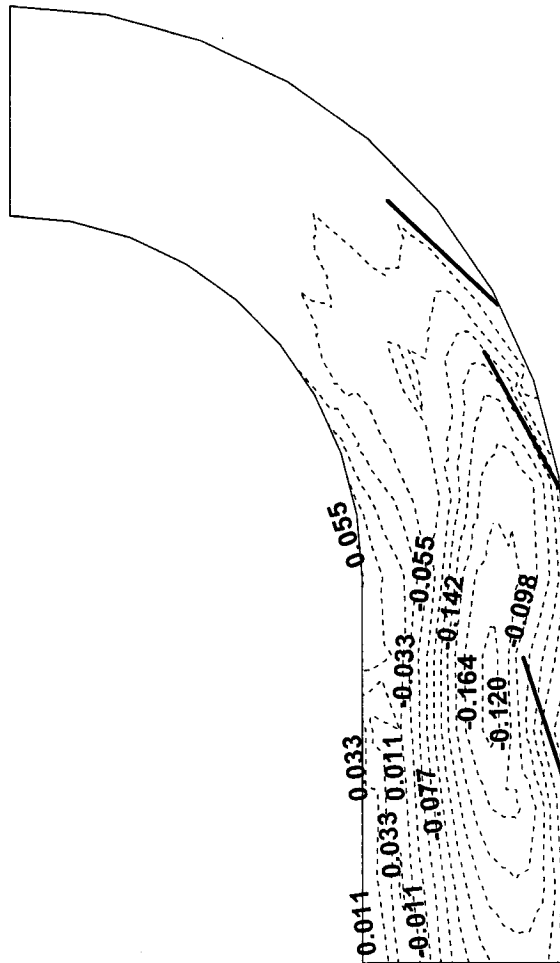


Figure A.4.6: P vs.  $\theta$  (Barb group C,  $W/y_0=6.0$ ,  $h/y_0=0.5$ )

## **Appendix A.5**

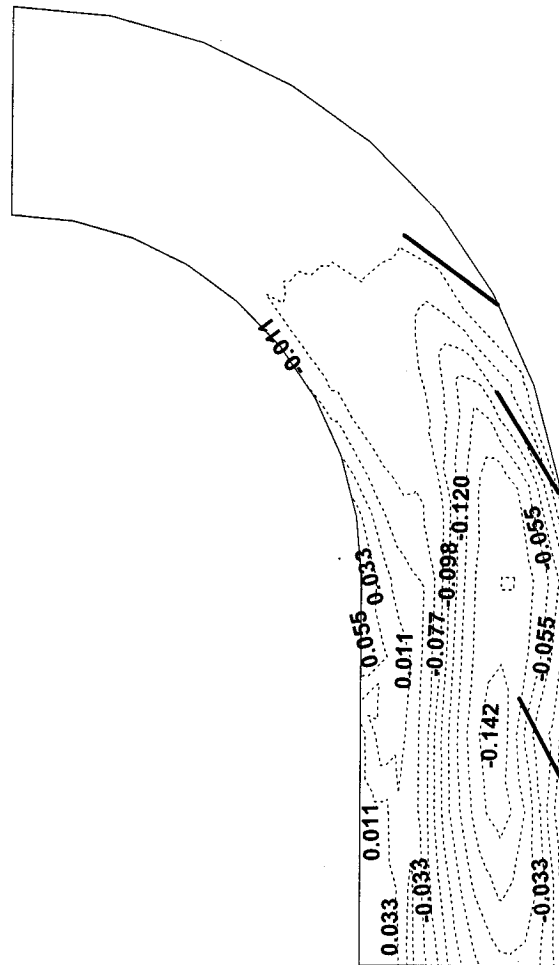
### **Bed Contours**



**Figure A.5.1:** 90-degree bend, Barb group A,  $\theta=20$ -degrees,  $W/y_o=4.5$ ,  $h/y_o=0.5$



**Figure A.5.2:** 90-degree bend, Barb group A,  $\theta=20$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$



**Figure A.5.3:** 90-degree bend, Barb group A,  $\theta=30$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$



Figure A.5.4: 90-degree bend, Barb group A,  $\theta=30$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$

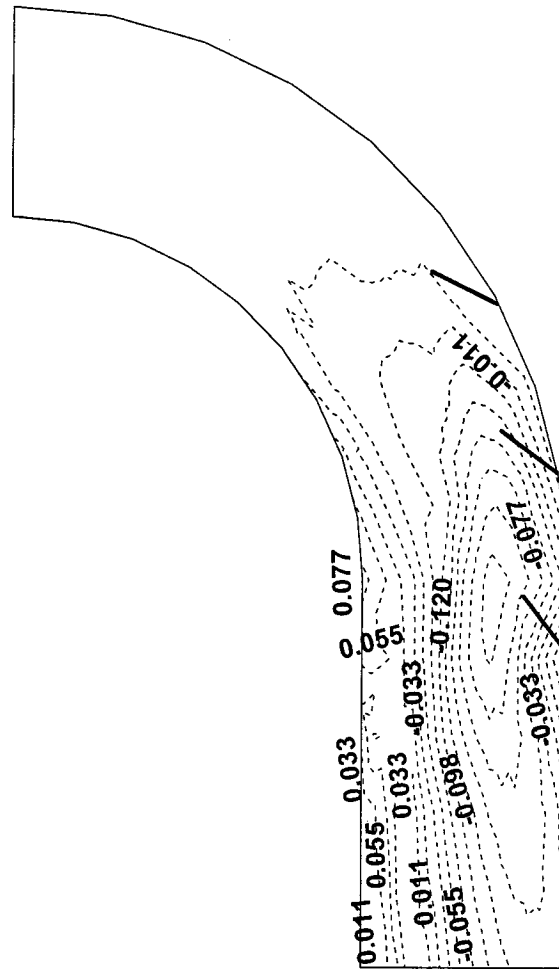
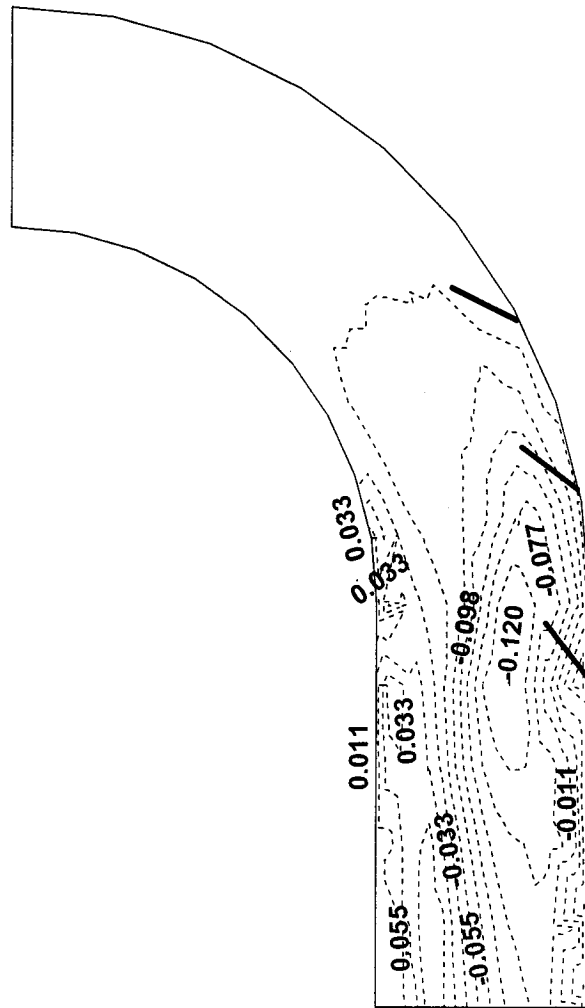
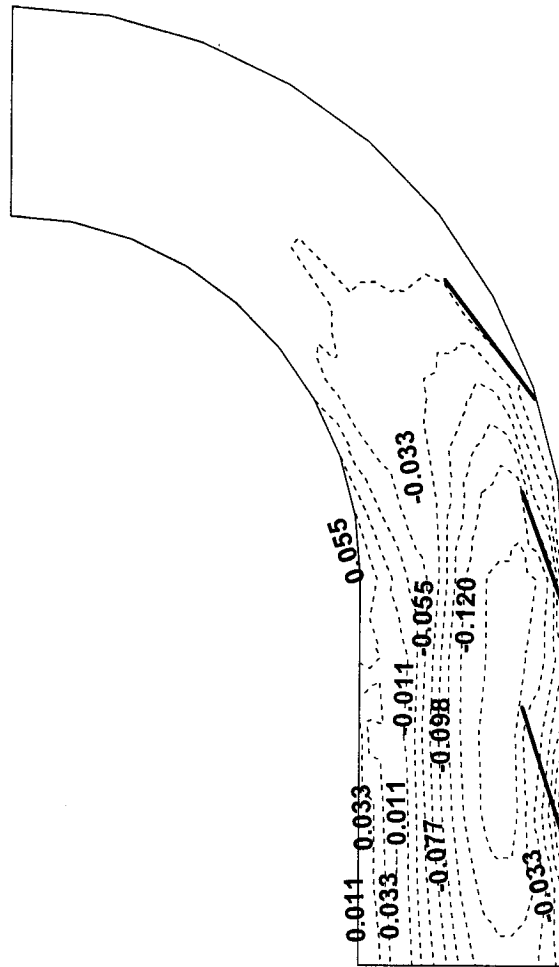


Figure A.5.5: 90-degree bend, Barb group A,  $\theta=40$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$



**Figure A.5.6:** 90-degree bend, Barb group A,  $\theta=40$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$



**Figure A.5.7:** 90-degree bend, Barb group B,  $\theta=20$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$

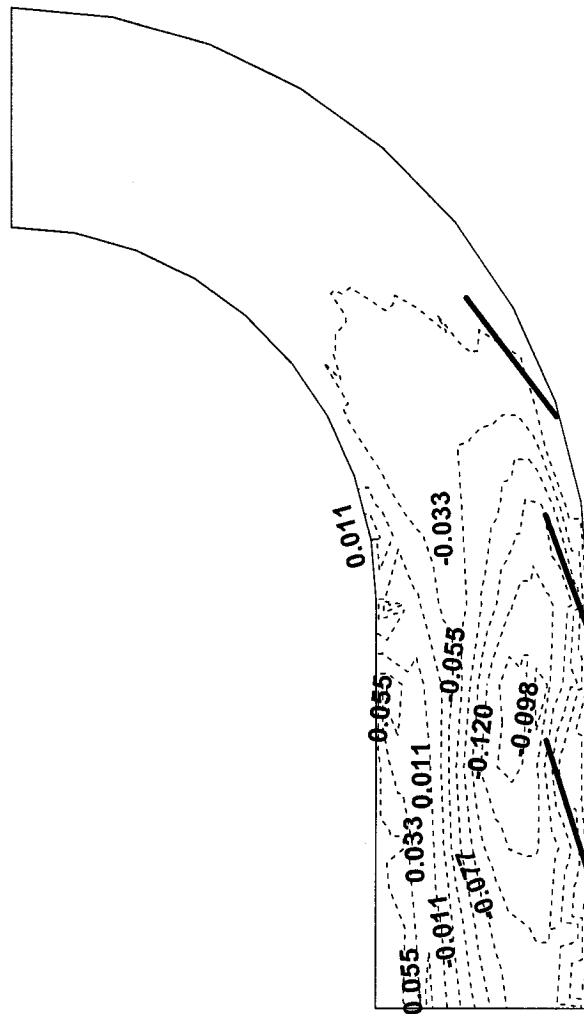
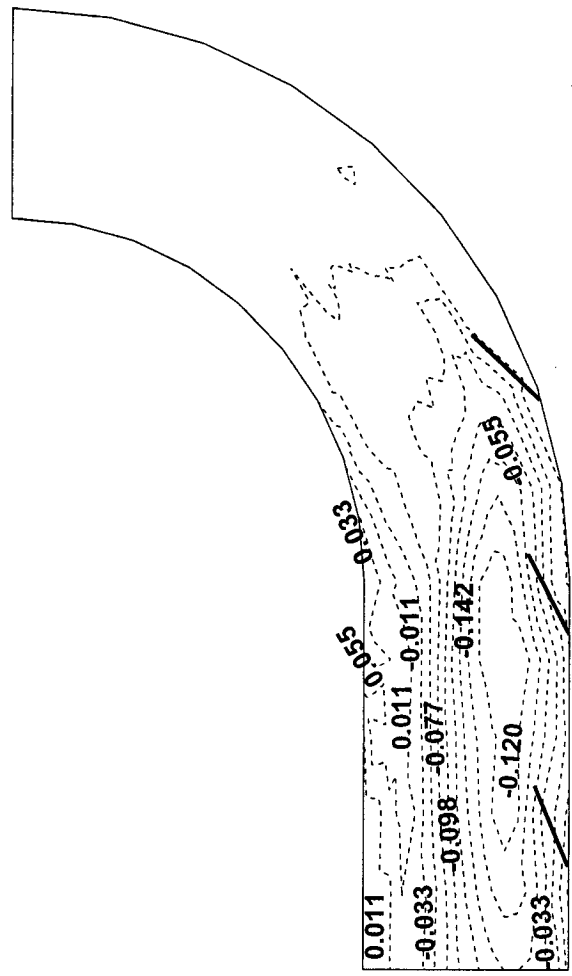
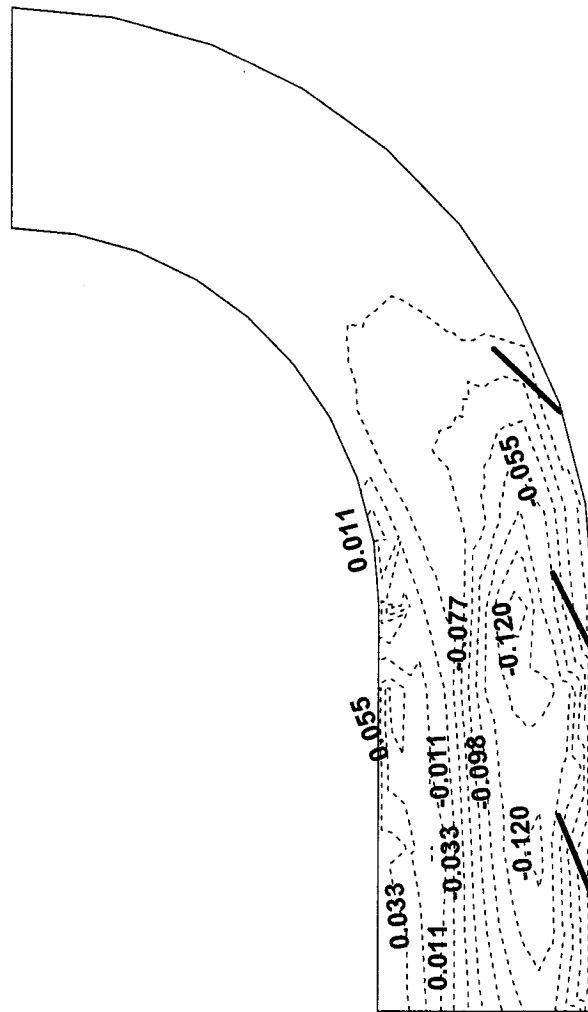


Figure A.5.8: 90-degree bend, Barb group B,  $\theta=20$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$

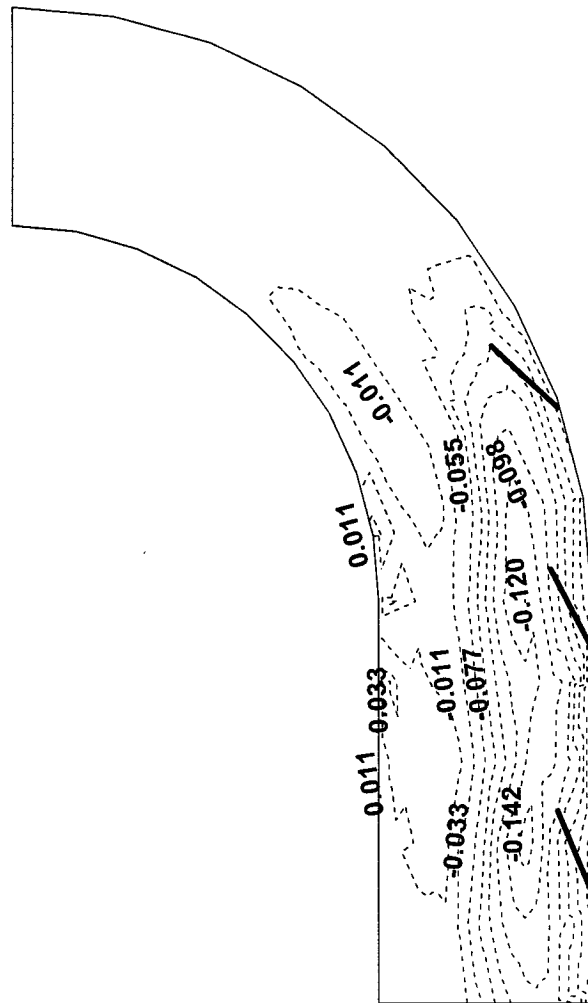




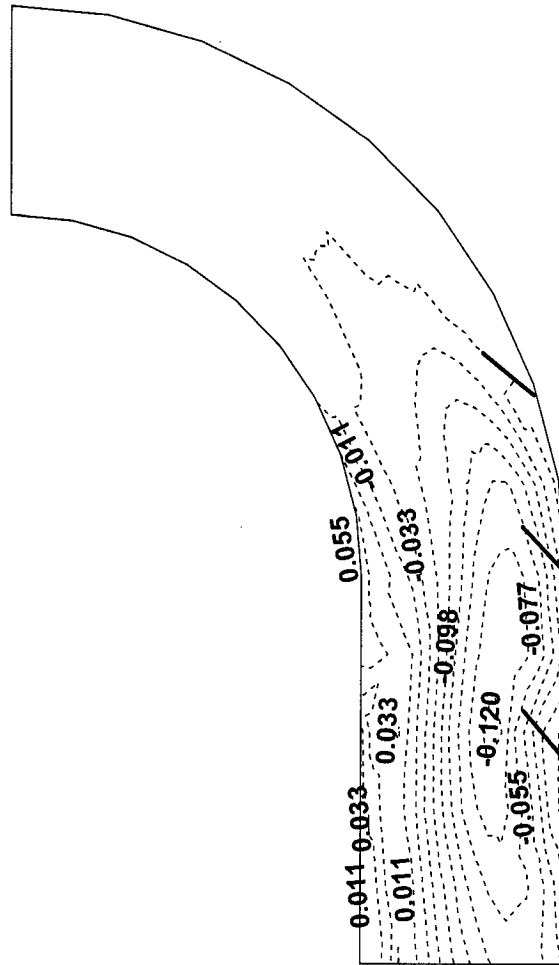
**Figure A.5.10:** 90-degree bend, Barb group B,  $\theta=30$ -degrees,  $W/y_o=4.5$ ,  $h/y_o=0.5$



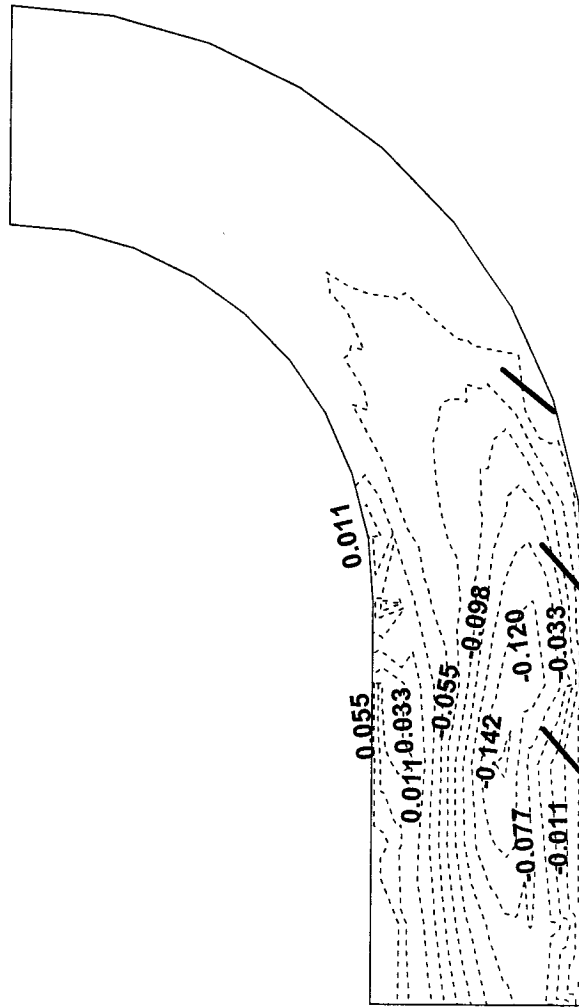
**Figure A.5.11:** 90-degree bend, Barb group B,  $\theta=30$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$



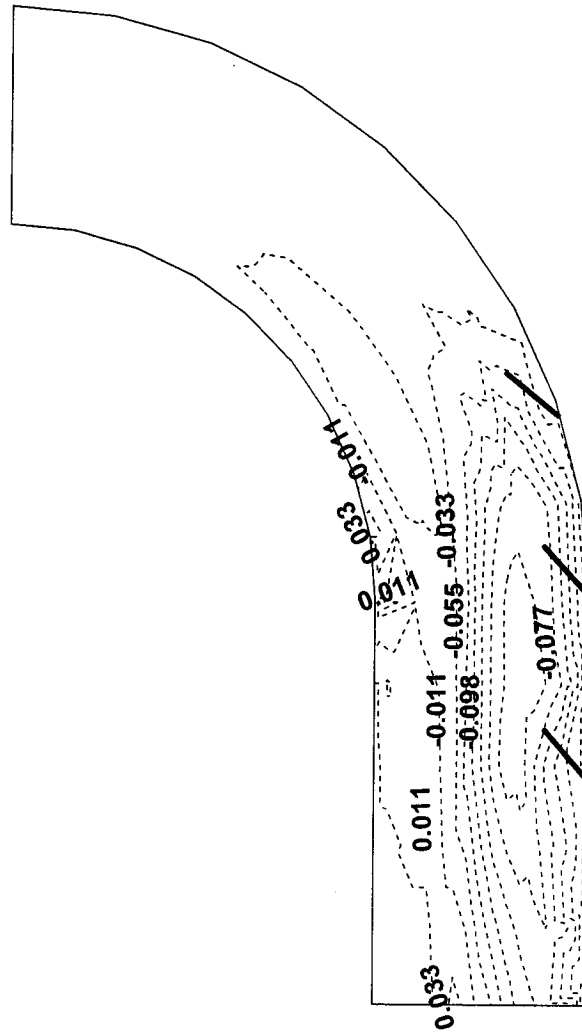
**Figure A.5.12:** 90-degree bend, Barb group B,  $\theta=30$ -degrees,  $W/y_0=6.0$ ,  $h/y_0=0.5$



**Figure A.5.13:** 90-degree bend, Barb group B,  $\theta=40$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$



**Figure A.5.14:** 90-degree bend, Barb group B,  $\theta=40$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$



**Figure A.5.15:** 90-degree bend, Barb group B,  $\theta=40$ -degrees,  $W/y_0=6.0$ ,  $h/y_0=0.5$

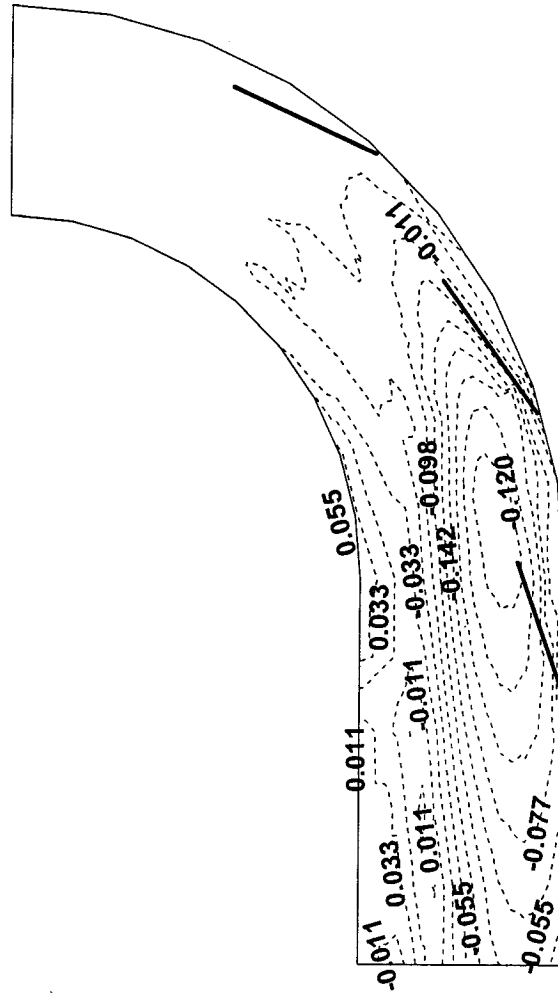
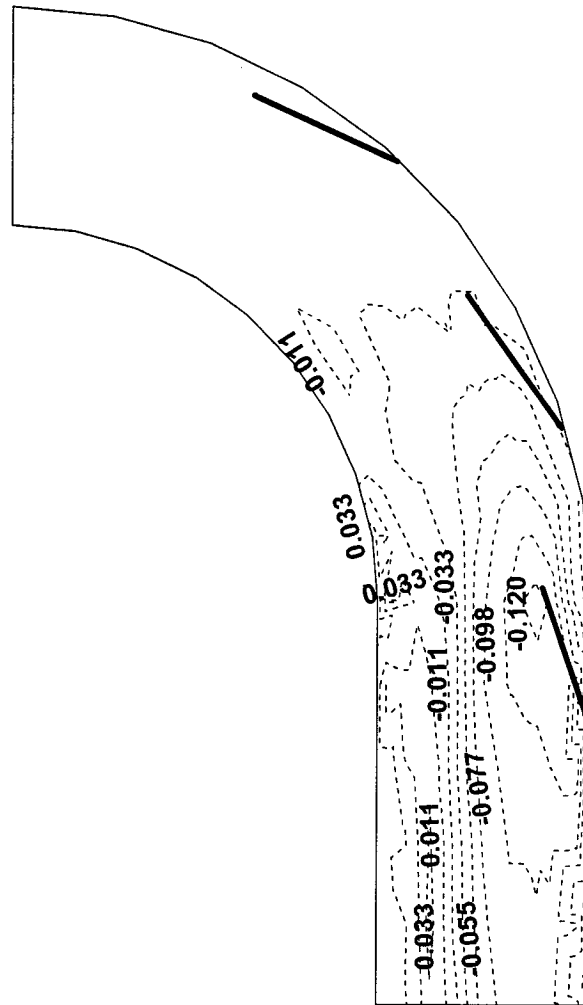
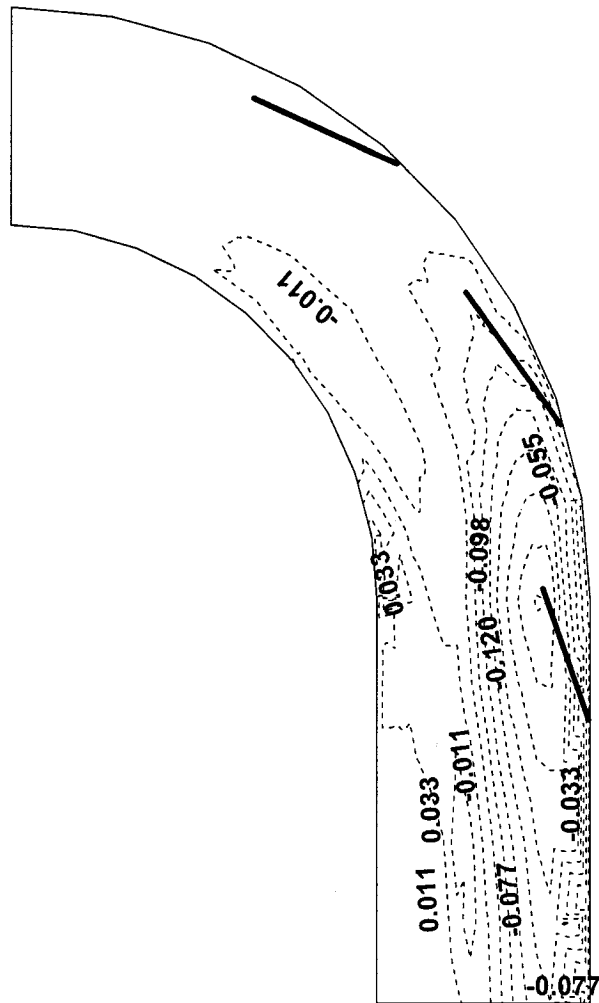


Figure A.5.16: 90-degree bend, Barb group C,  $\theta=20$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$



**Figure A.5.17:** 90-degree bend, Barb group C,  $\theta=20$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$



**Figure A.5.18:** 90-degree bend, Barb group C,  $\theta=20$ -degrees,  $W/y_0=6.0$ ,  $h/y_0=0.5$

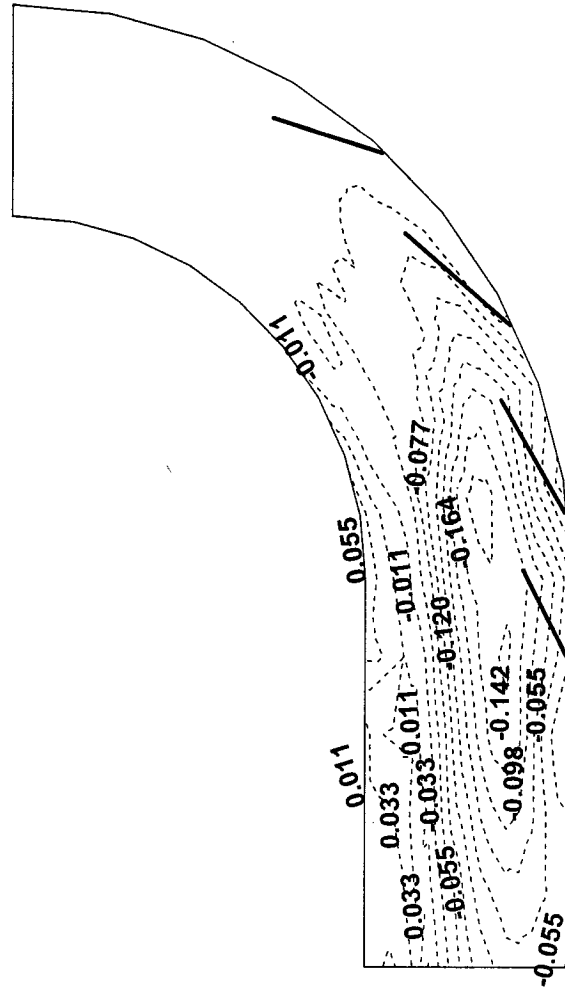
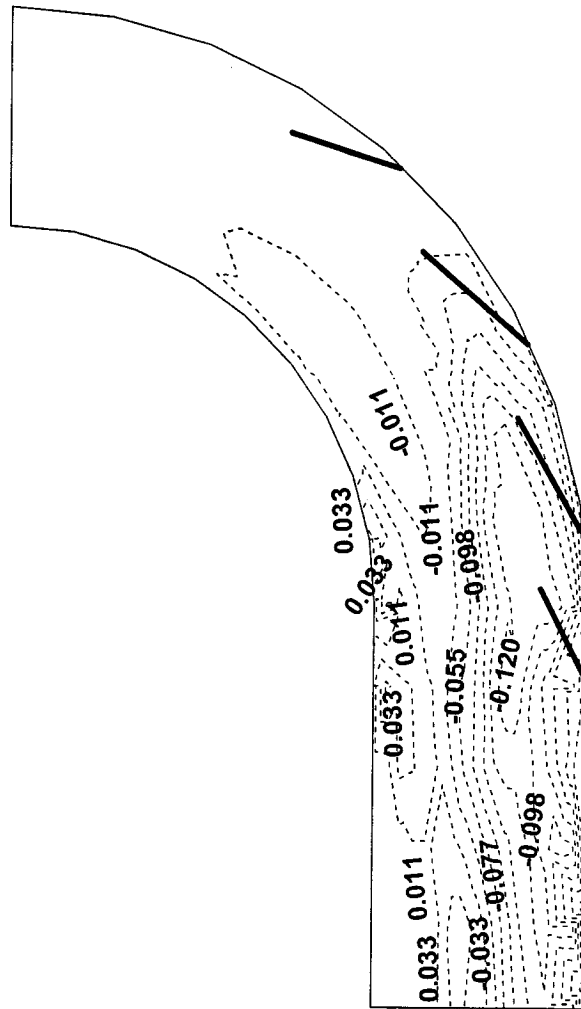
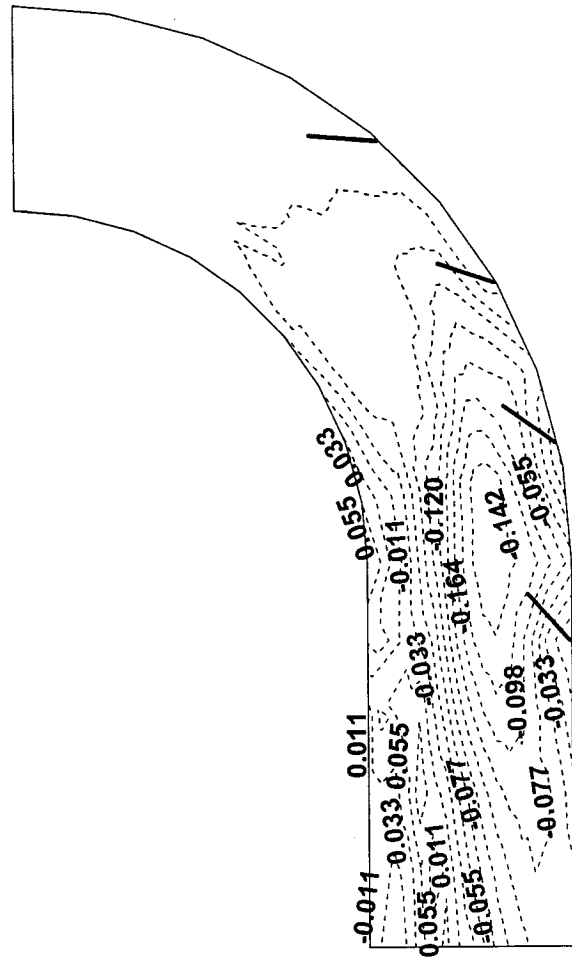


Figure A.5.19: 90-degree bend, Barb group C,  $\theta=30$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$



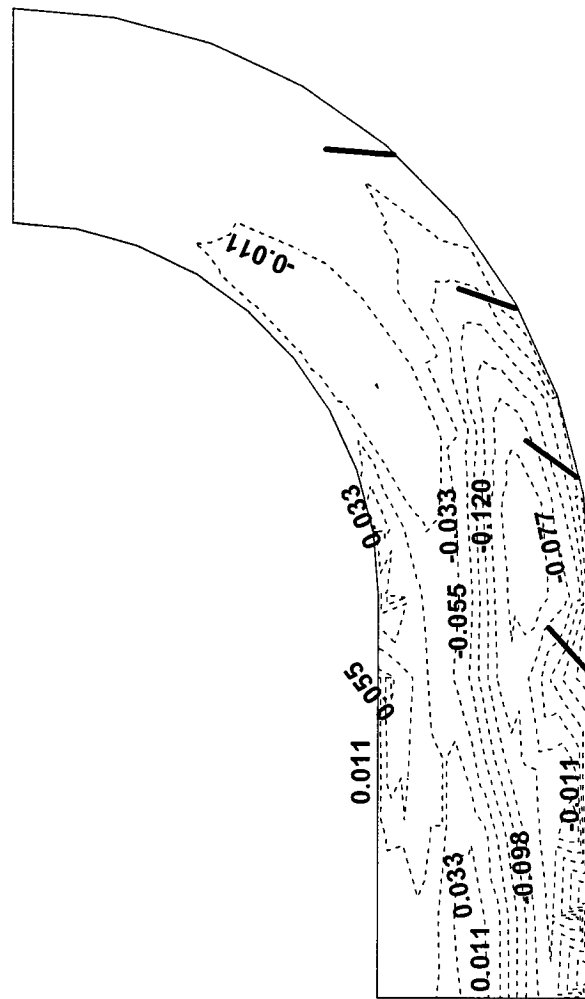


**Figure A.5.21:** 90-degree bend, Barb group C,  $\theta=30$ -degrees,  $W/y_0=6.0$ ,  $h/y_0=0.5$

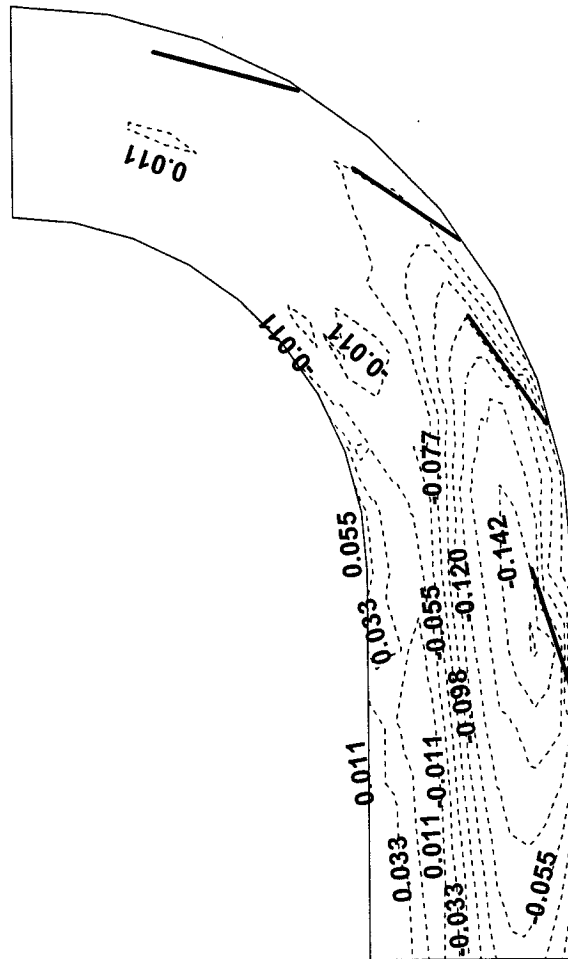


**Figure A.5.22:** 90-degree bend, Barb group C,  $\theta=40$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$

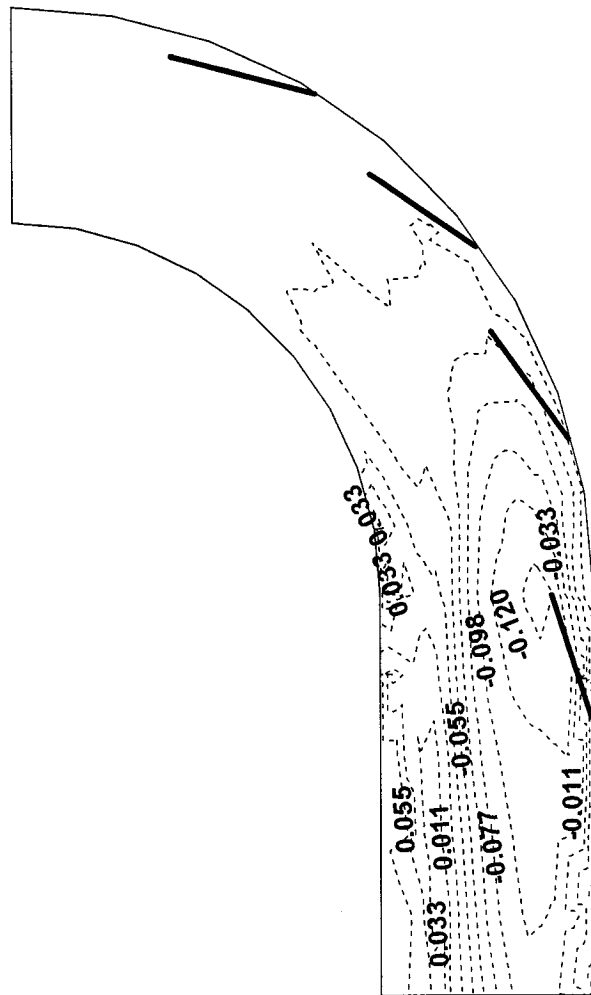




**Figure A.5.24:** 90-degree bend, Barb group C,  $\theta=40$ -degrees,  $W/y_0=6.0$ ,  $h/y_0=0.5$



**Figure A.5.25:** 90-degree bend, Barb group D,  $\theta=20$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$



**Figure A.5.26:** 90-degree bend, Barb group D,  $\theta=20$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$

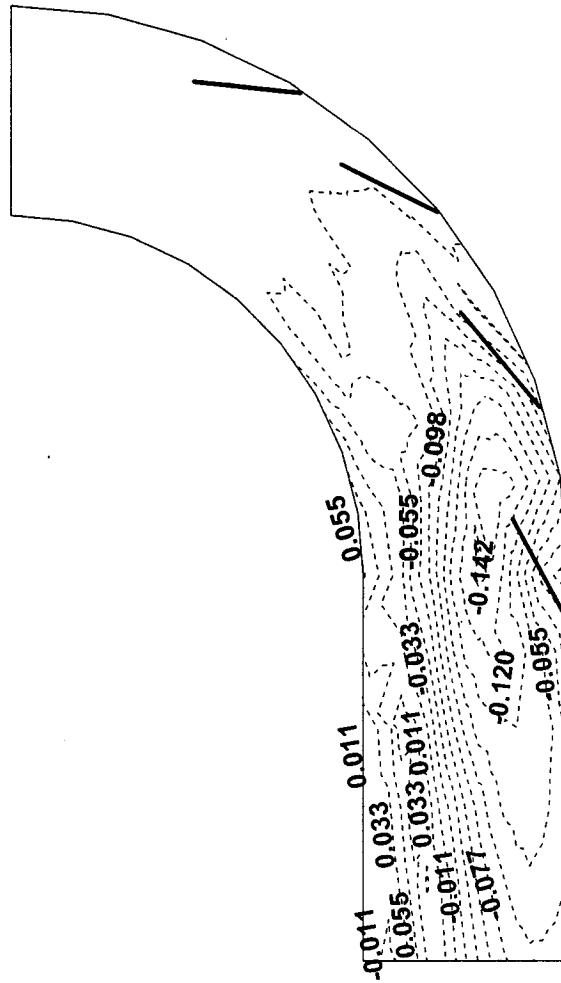
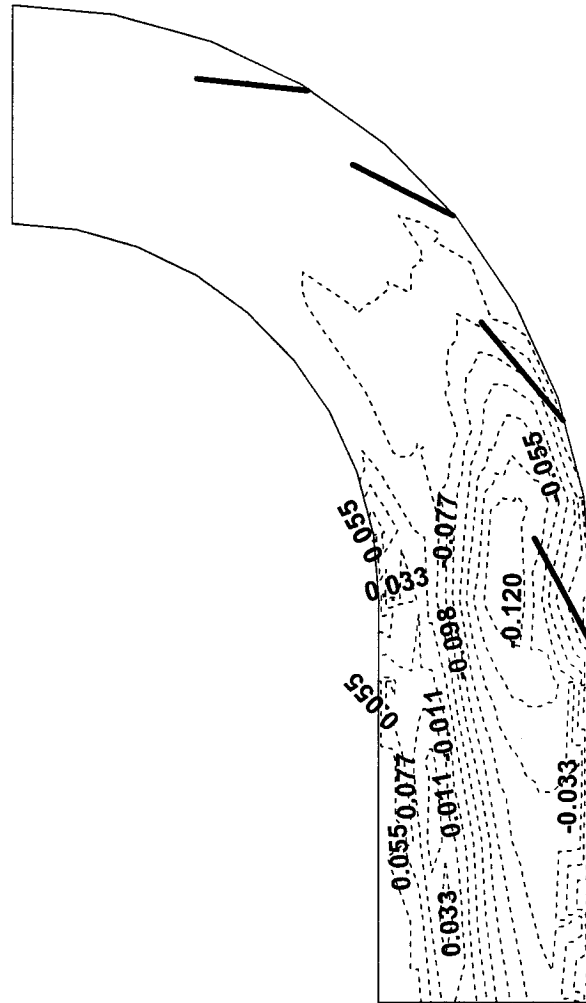


Figure A.5.27: 90-degree bend, Barb group D,  $\theta=30$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$



**Figure A.5.28:** 90-degree bend, Barb group D,  $\theta=30$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$



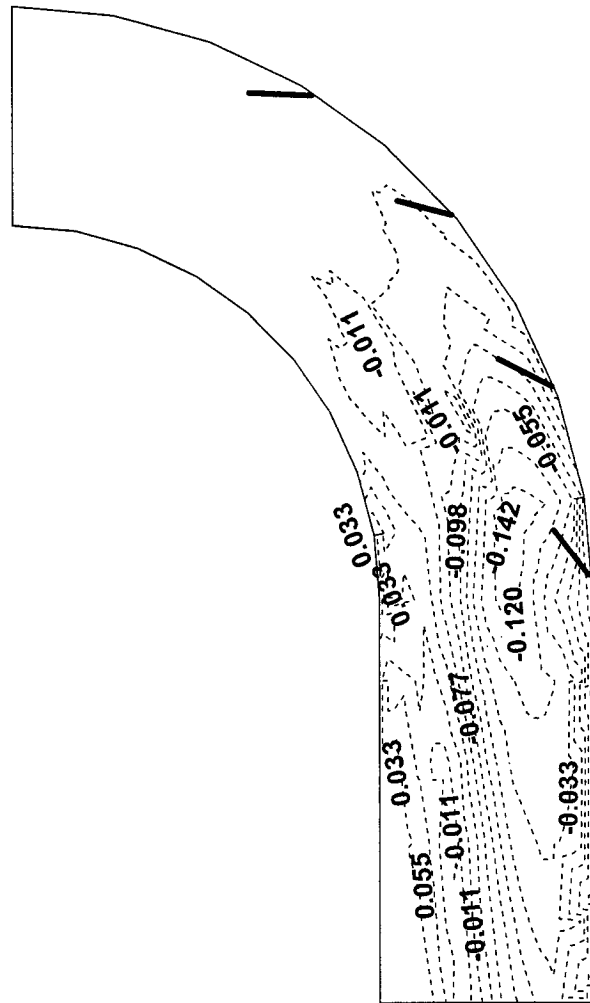
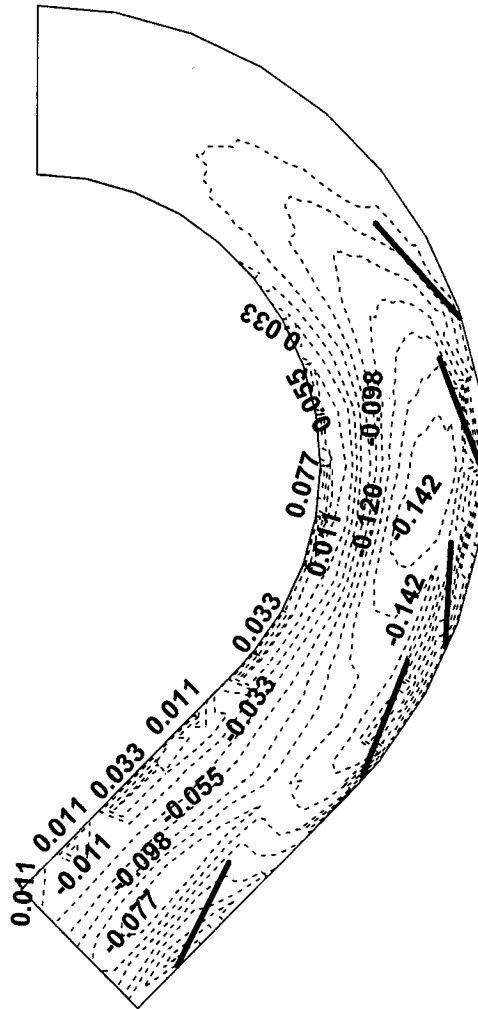


Figure A.5.30: 90-degree bend, Barb group D,  $\theta=40$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$



**Figure A.5.31:** 135-degree bend, Barb group B,  $\theta=20$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$

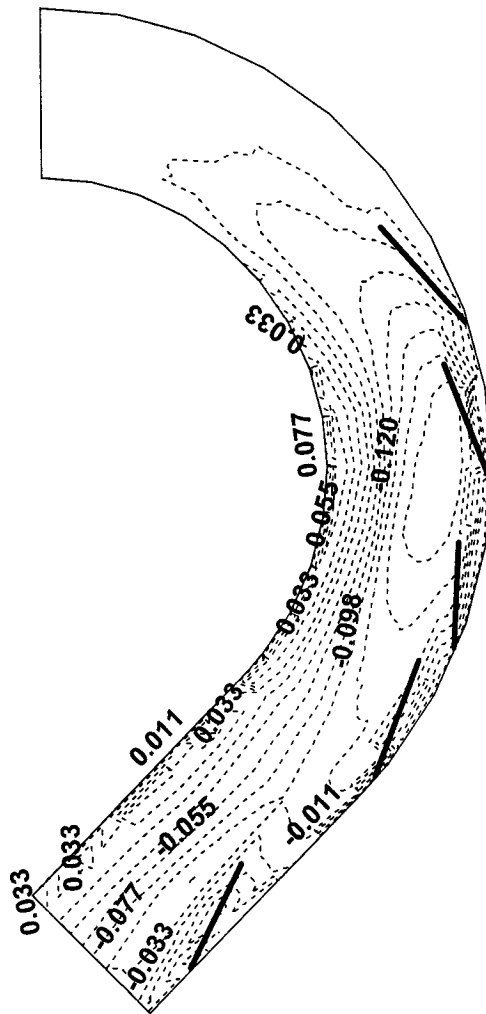
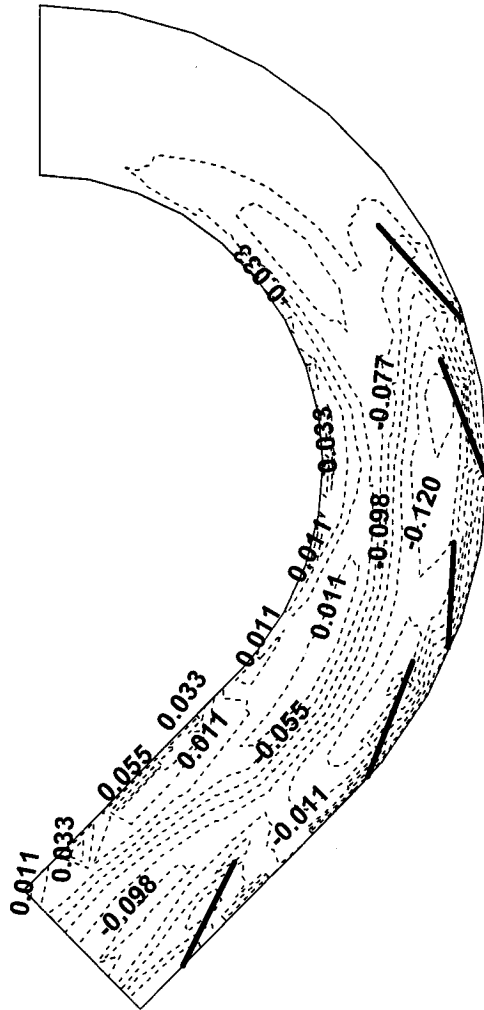
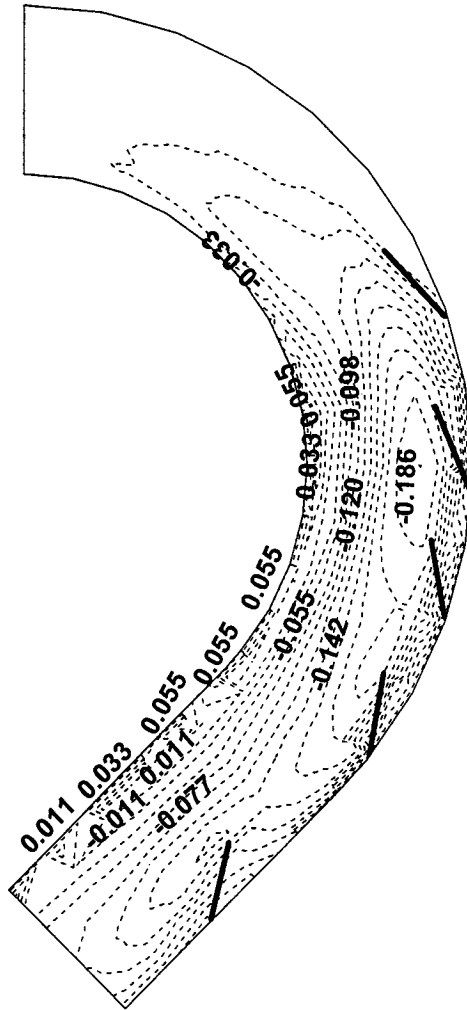


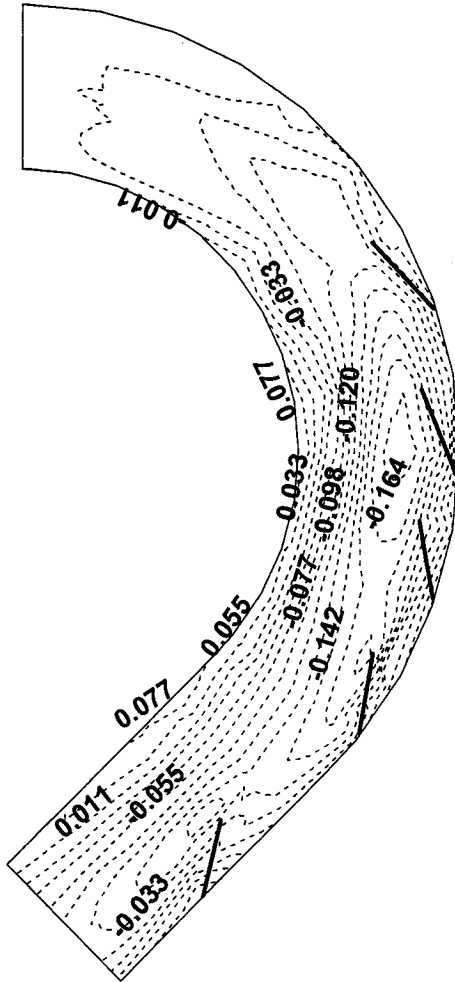
Figure A.5.32: 135-degree bend, Barb group B,  $\theta=20$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$



**Figure A.5.33:** 135-degree bend, Barb group B,  $\theta=20$ -degrees,  $W/y_0=6.0$ ,  $h/y_0=0.5$



**Figure A.5.34:** 135-degree bend, Barb group B,  $\theta=30$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$



**Figure A.5.35:** 135-degree bend, Barb group B,  $\theta=30$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$

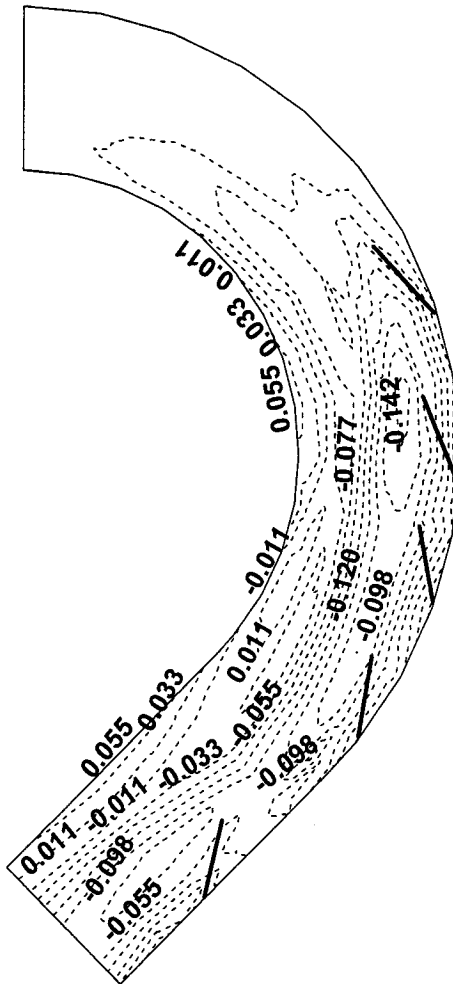
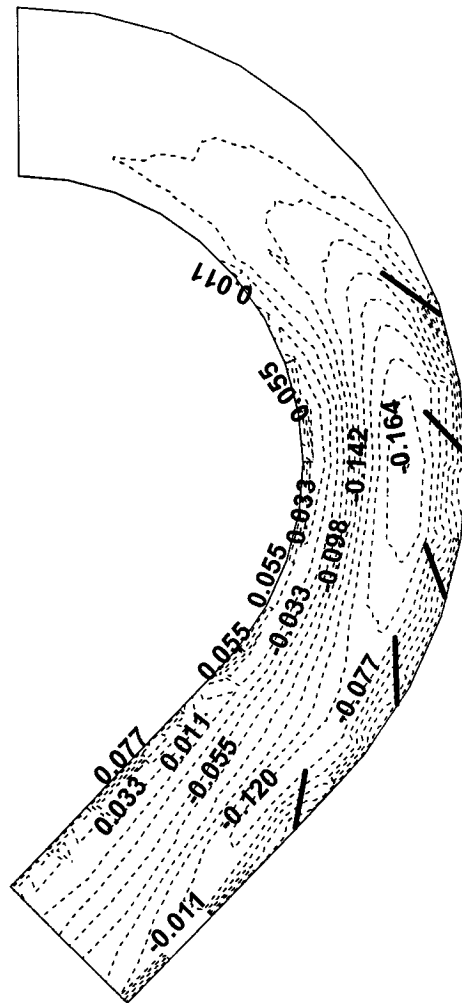


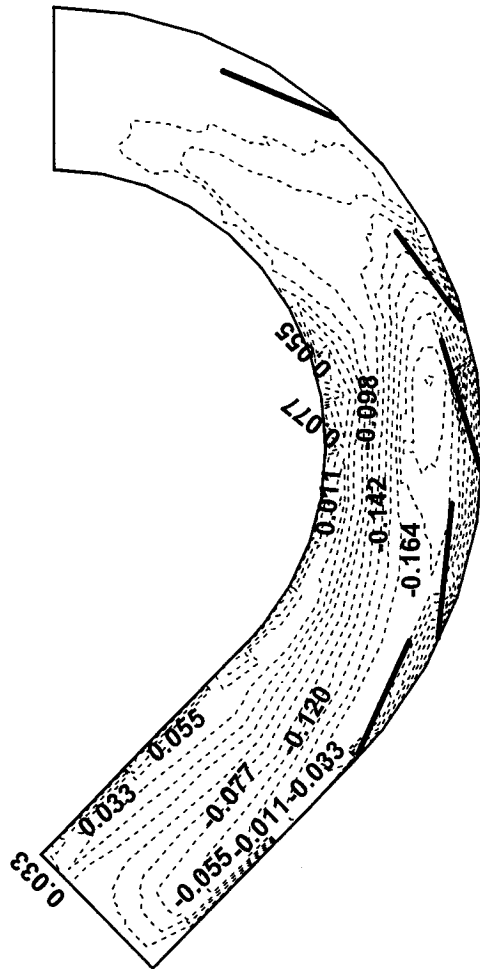
Figure A.5.36: 135-degree bend, Barb group B,  $\theta=30$ -degrees,  $W/y_0=6.0$ ,  $h/y_0=0.5$





**Figure A.5.38:** 135-degree bend, Barb group B,  $\theta=40$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$





**Figure A.5.40:** 135-degree bend, Barb group C,  $\theta=20$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$

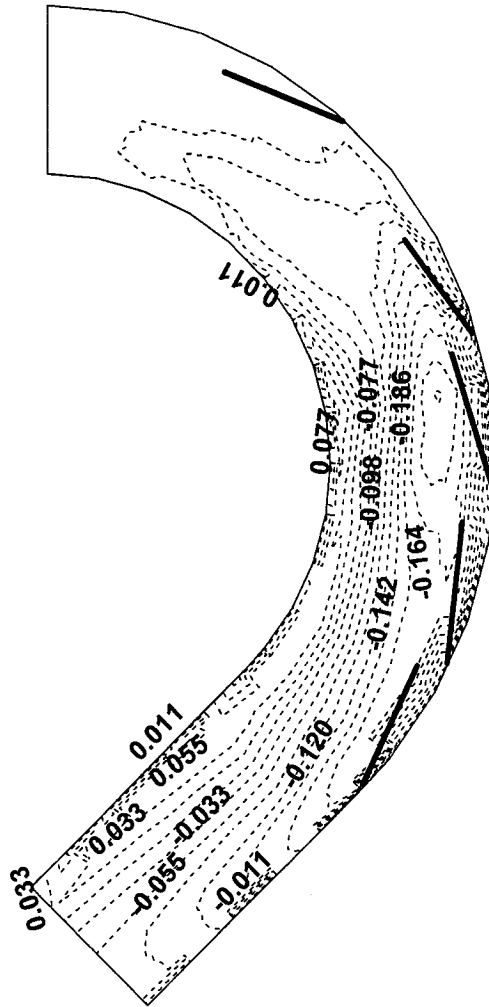
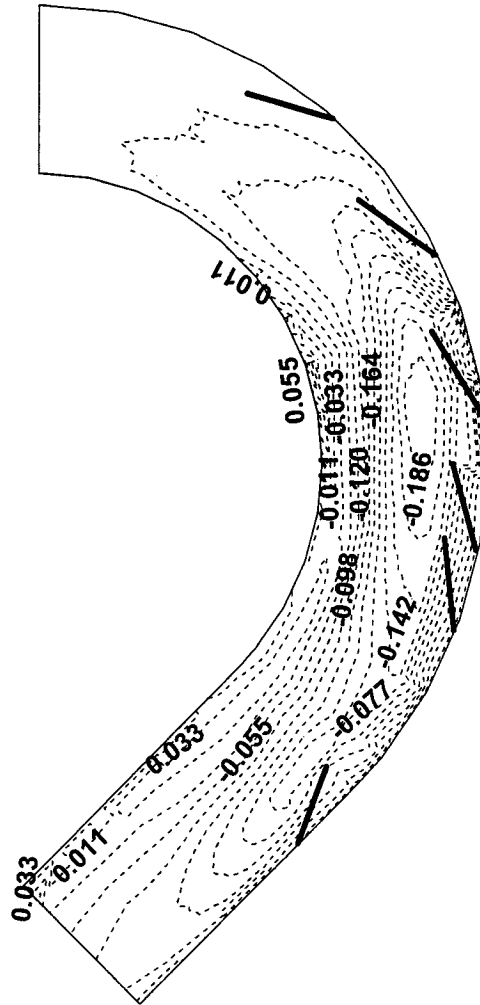
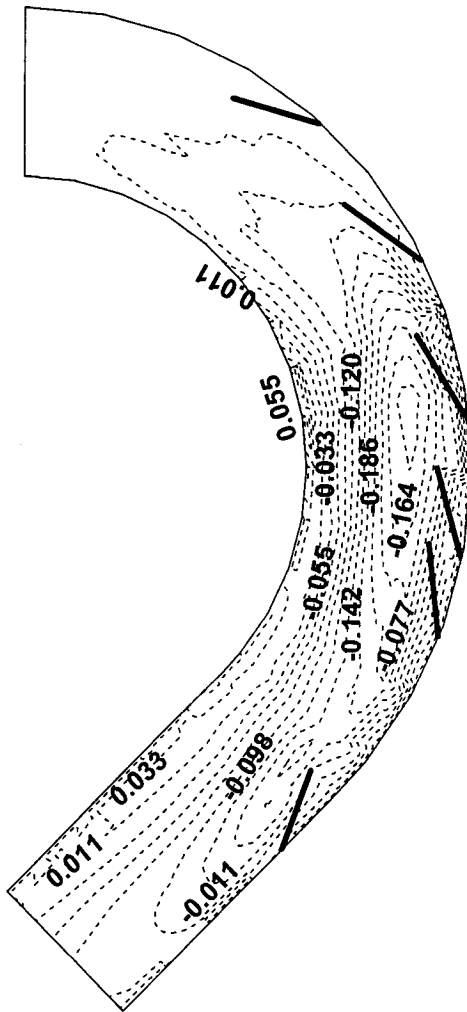


Figure A.5.41: 135-degree bend, Barb group C,  $\theta=20$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$

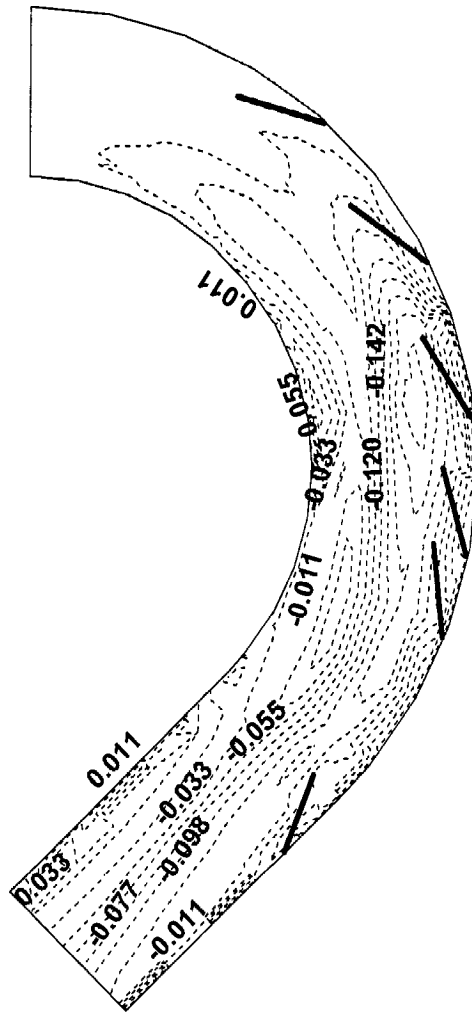




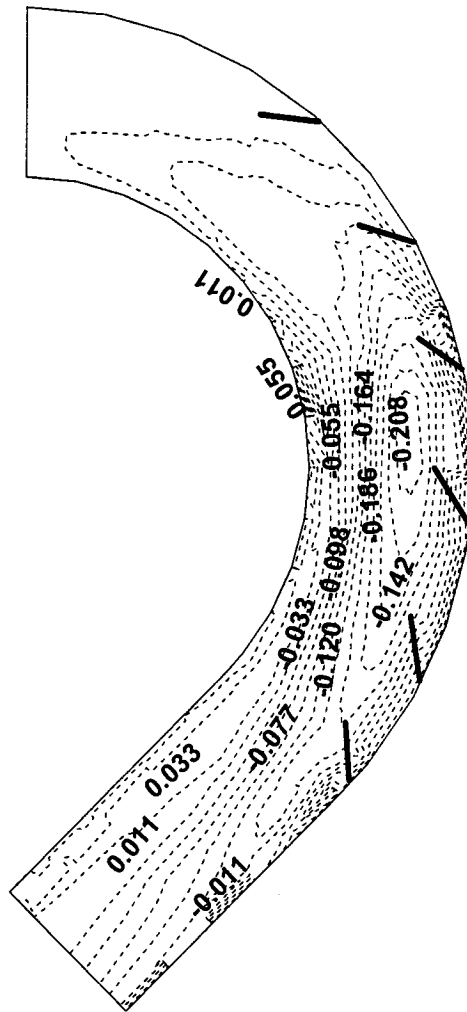
**Figure A.5.43:** 135-degree bend, Barb group C,  $\theta=30$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$



**Figure A.5.44:** 135-degree bend, Barb group C,  $\theta=30$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$



**Figure A.5.45:** 135-degree bend, Barb group C,  $\theta=30$ -degrees,  $W/y_0=6.0$ ,  $h/y_0=0.5$



**Figure A.5.46:** 135-degree bend, Barb group C,  $\theta=40$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.5$

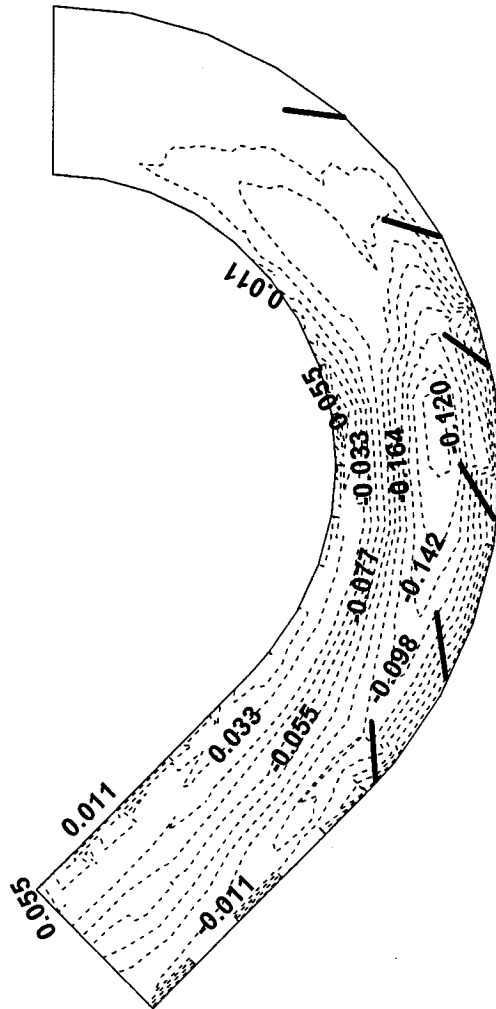
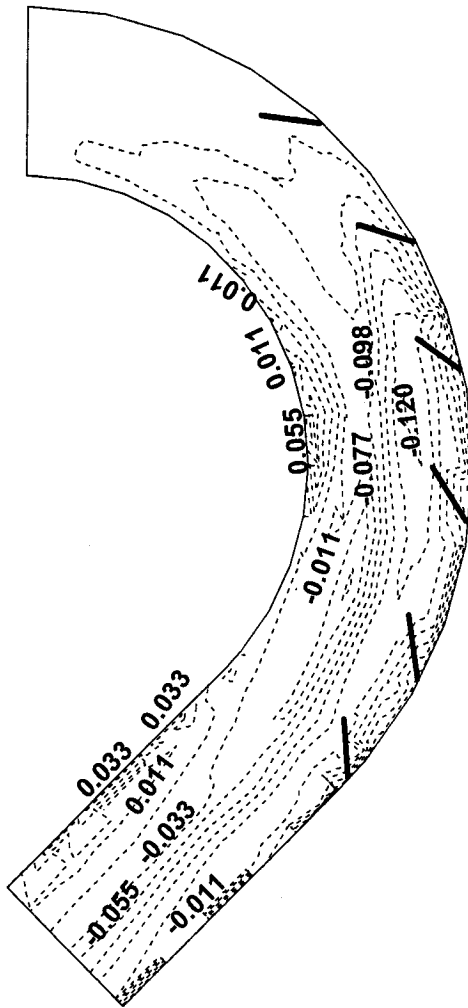


Figure A.5.47: 135-degree bend, Barb group C,  $\theta=40$ -degrees,  $W/y_0=4.5$ ,  $h/y_0=0.375$



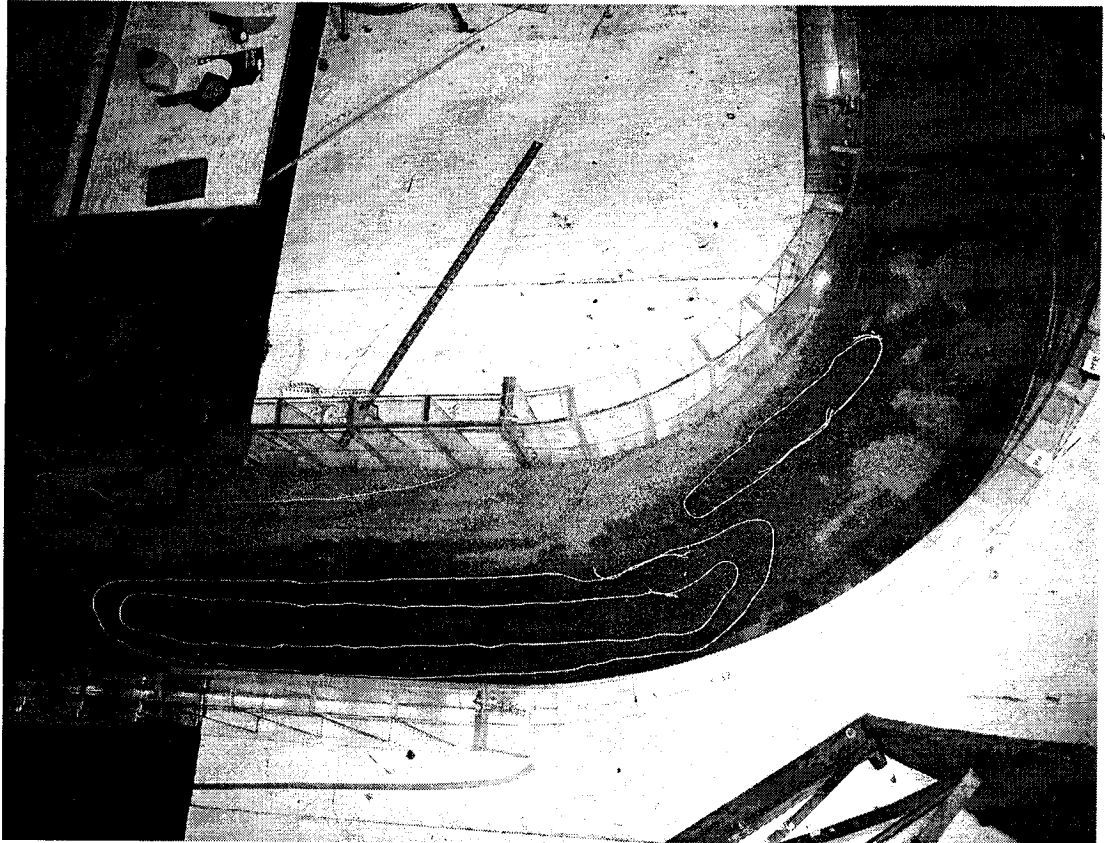
**Figure A.5.48:** 135-degree bend, Barb group C,  $\theta=40$ -degrees,  $W/y_0=6.0$ ,  $h/y_0=0.5$

## **Appendix A.6**

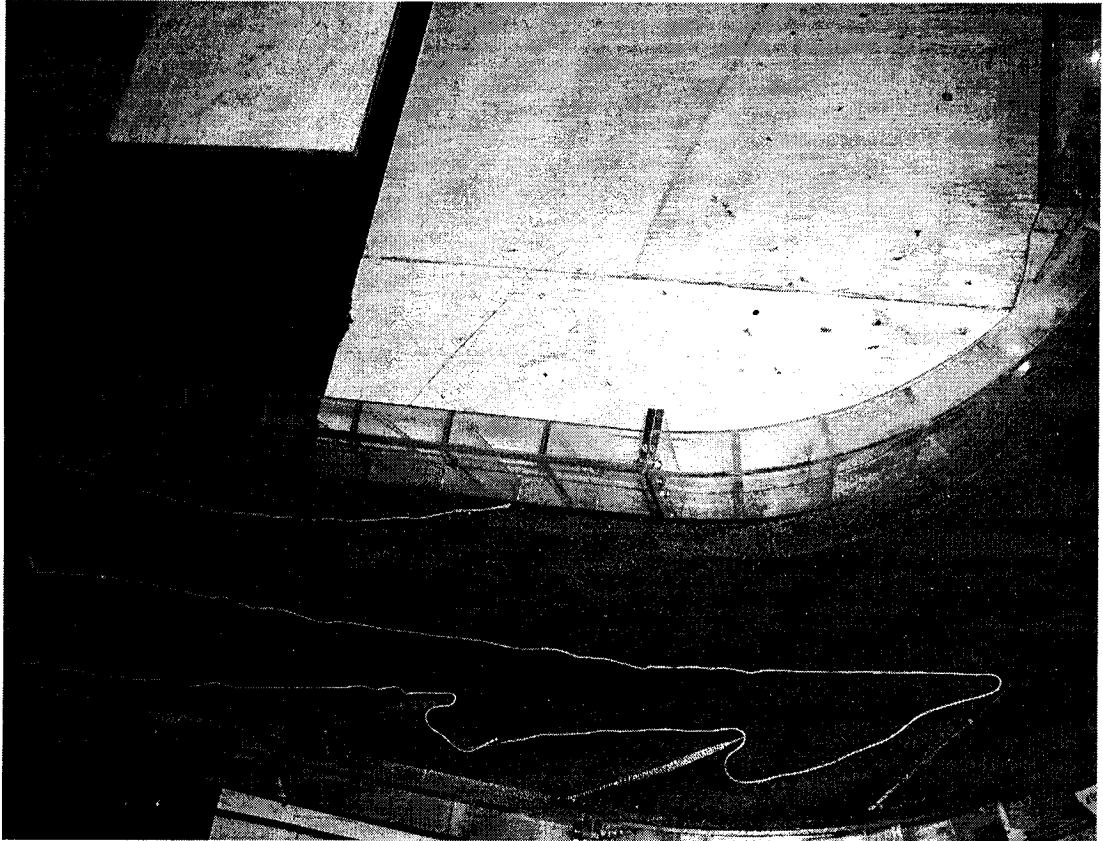
### **Photographs**



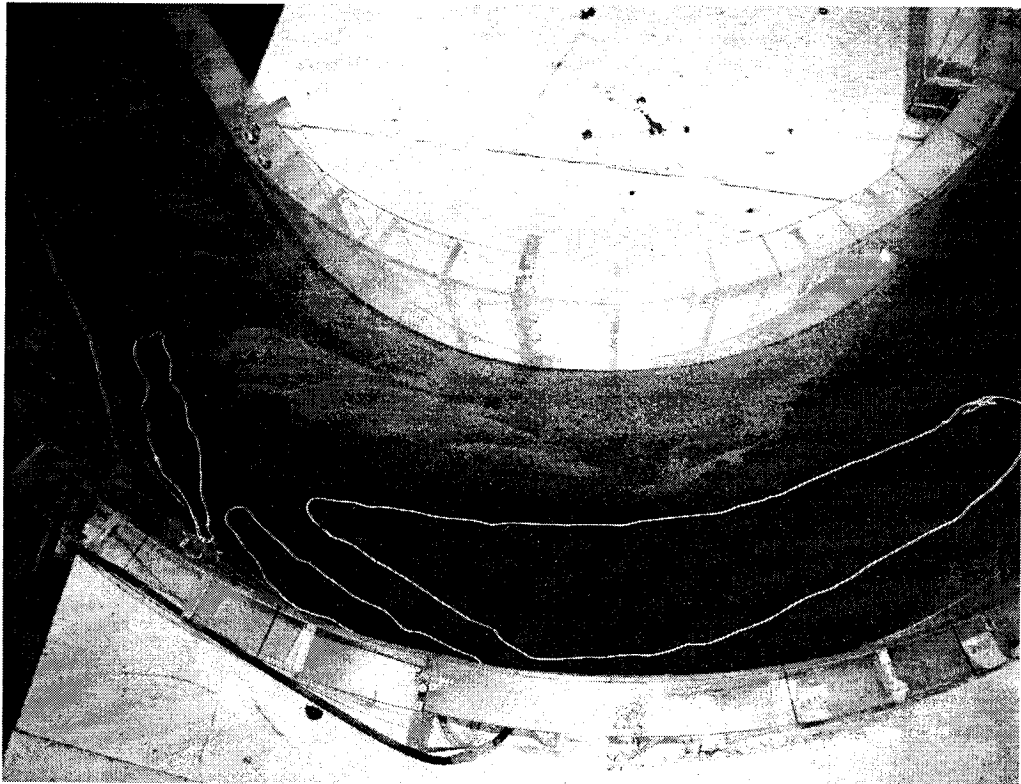
**Photograph A.6.1:** 90-degree bend 'reference' bed contour



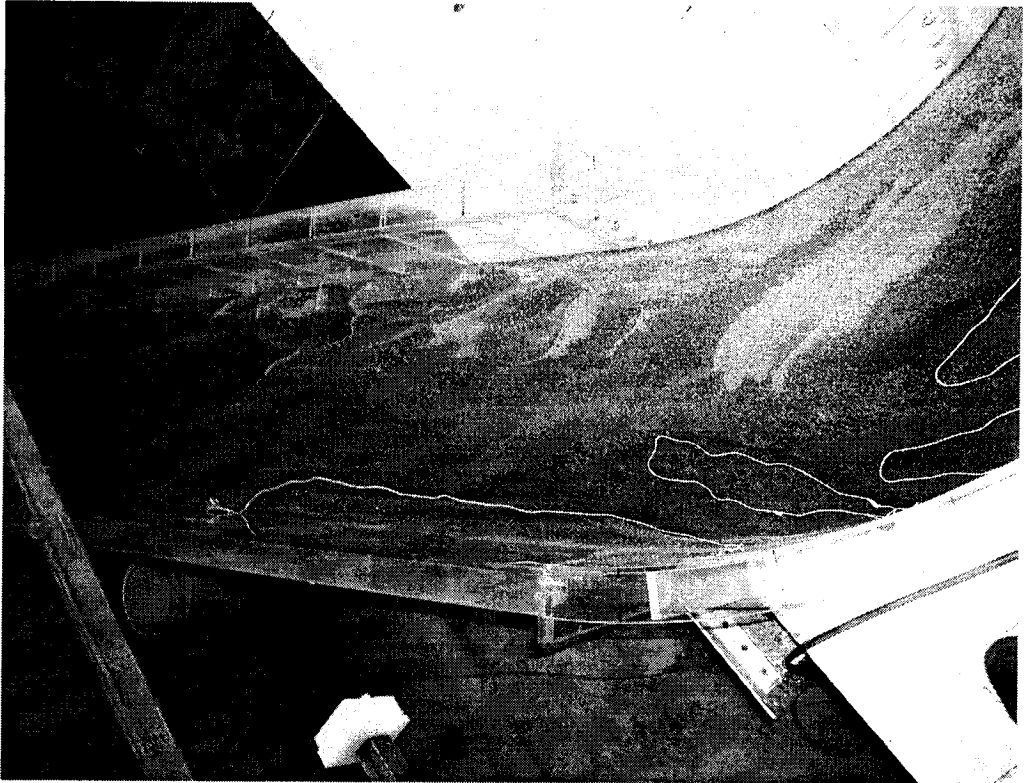
**Photograph A.6.2:** 90-degree bend 'reference' bed contour



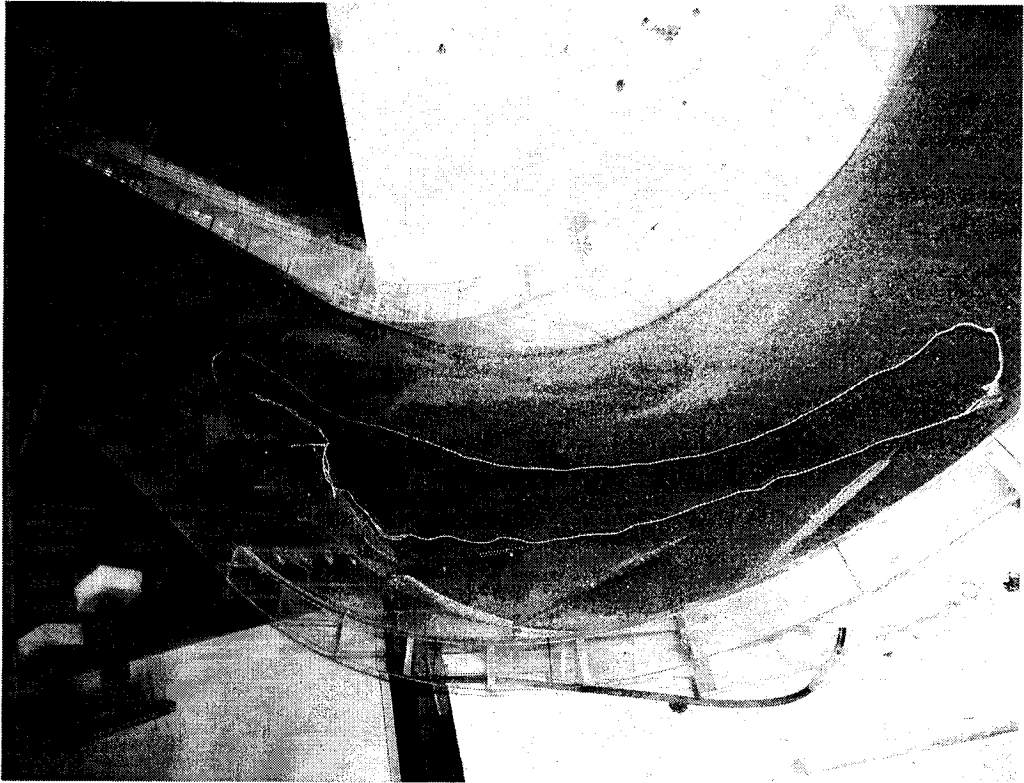
**Photograph A.6.3:** 90-degree bend, Barb group B,  $W/y_0=4.5$ ,  $h/y_0=0.5$



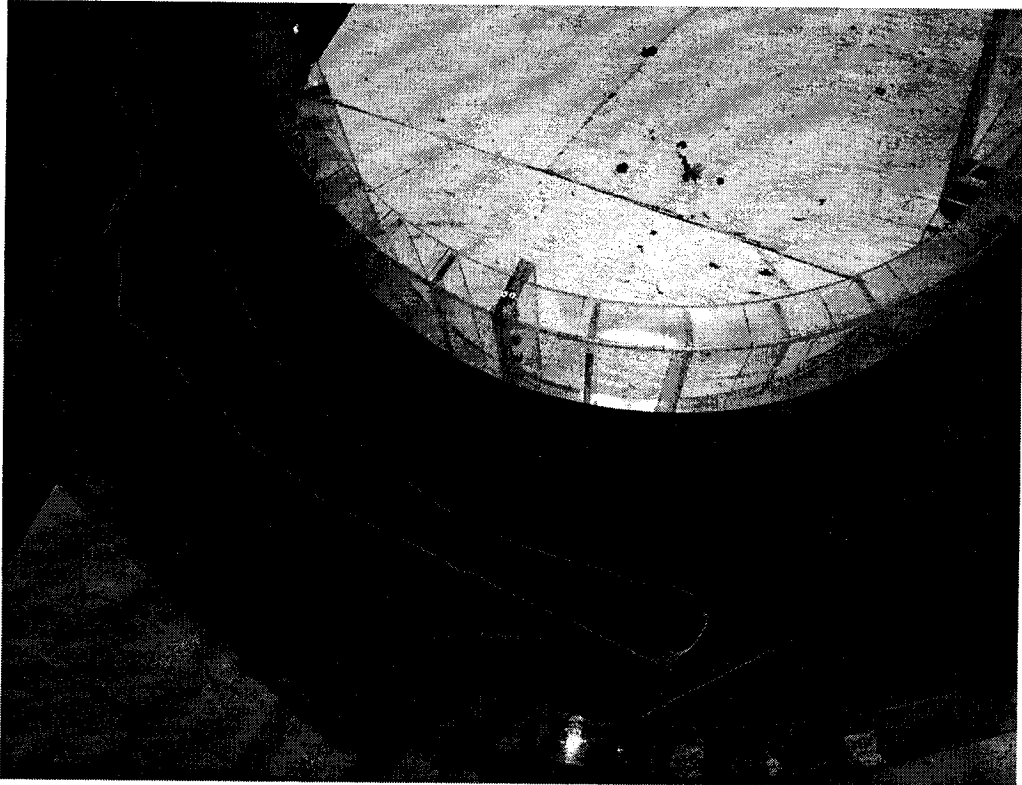
**Photograph A.6.4:** 135-degree bend 'reference' bed contour



**Photograph A.6.5:** 135-degree bend 'reference' bed contour



**Photograph A.6.6:** 135-degree bend, Barb group B,  $W/y_0=4.5$ ,  $h/y_0=0.5$



**Photograph A.6.7:** 135-degree bend, Barb group B,  $W/y_o=4.5$ ,  $h/y_o=0.5$