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<b>16. Abstract</b> <p>Extensive accumulation of large woody debris at bridge piers poses a chronic and sometimes quite severe problem at several bridge crossings in Indiana. This study, involving both laboratory and field components, examines the factors contributing to the initiation and development of such debris piles. The laboratory study, performed in a rectangular channel with a single model pier (and in some cases with an upstream vertical cylinder modeling a debris deflector, as well as a model sand bar) and both dowels and twigs as model logs, considered the effects of velocity and depth. The experiments point to a stronger than might be expected effect of local depth, with the potential for debris accumulation generally greater when the local depth is smaller. The field study consisted of video monitoring and recording of debris-transporting events at two sites, the SR59 south crossing of the Eel River (in operation since 9/2001), and the SR63 southbound bridge over the Big Vermillion River (in operation since 4/2003). Results (images) during significant flow events have only been obtained at the SR59 site during the 2001/2002 season, and some qualitative conclusions can be drawn regarding the initiation and growth of debris piles in relation to significant flow events.</p>					
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Final Report

**Debris Accumulation at Bridge Crossings:  
Laboratory and Field Studies**

FHWA/IN/JTRP-2003/10

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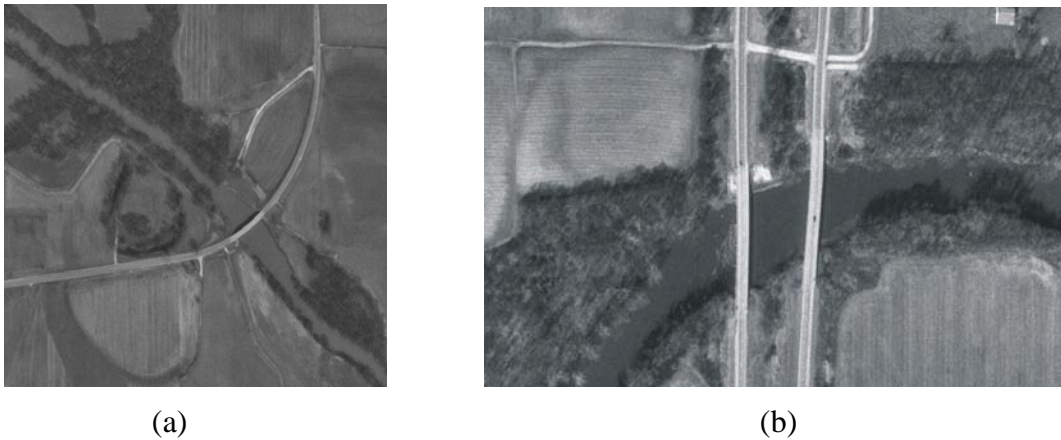
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# 1. Introduction and Review

The study of large woody debris (LWD) in river systems has attracted much recent research attention. This renewed interest has been motivated primarily with regards to the geomorphological and ecological implications (e.g., Abbe and Montgomery, 1996), but aspects relevant to bridge hydraulics have also been investigated, including the early works of Brice et al. (1978) and Chang and Shen (1979), and the more recent study of Diehl (1997). Within the context of bridge crossings, the main concerns raised by LWD are an increased potential of upstream flooding due to greater backwater effects, and the possible aggravation of local scour, either directly around piers or indirectly around abutments or banks.



**Figure 1.1: Sites with chronic and severe debris accumulation problems in Indiana, a) SR59 south crossing of the Eel River (flowing from top left to bottom right), b) SR63 crossing of the Vermilion River (flowing from bottom left to bottom right)— source: USGS aerial photographs from <http://TerraServer-USA.com>**

The SR 59 south crossing of the Eel River in Clay County provides an example of a bridge crossing in Indiana where substantial debris accumulation poses a chronic problem (Fig. 1.1a). In Fig. 1.2a, the site is shown when it has been cleared of debris, while Fig. 1.2b shows the extensive debris accumulation that can be expected after only a single season. Apparent in the photograph of the cleared site, the mid-stream sand bar or ‘island’ that has developed around one of the piers, may be relevant, as will be argued later. Under high-flow conditions, this bar is submergent. Another Indiana site where debris accumulation requires continual attention from INDOT is the southbound SR63 bridge over the Vermilion River (Fig. 1.1b, where the debris pile can even be seen). This occurs at the beginning of pronounced bend in the river, towards the inner bank. At both of these sites, the problem is of such magnitude that INDOT has installed structures, debris deflectors (the vertical cylindrical structures in Fig.1.2), intended to guide the woody debris through the bridge opening between the piers.

The present work aims to elucidate the major factors contributing to debris accumulation at bridge crossings in Indiana, as a step towards alleviating the problem at

existing sites, or if possible, entirely avoiding the problem at future sites by judicious design. It included both a laboratory as well as a field component. In the former, simplified idealized cases were investigated systematically under controlled conditions to identify mechanisms or at least correlations between flow or channel parameters and the potential for debris accumulation. The field component, basically video monitoring of sites where debris is known to accumulate, complements the laboratory study in observing the actual rather than the idealized phenomena, and so may be of more immediate interest to the bridge engineer.



a)



b)

**Figure 1.2: View of the Eel River site from bridge deck in upstream direction: a) site cleared of debris (April 2001), b) site with debris pile after a major flow event (August 2000); cylindrical vertical structures are debris deflectors**

## **1.1. Literature Review**

### **Large woody debris (LWD) in river systems**

Stream banks are often heavily wooded, and LWD may be ‘continuously’ introduced into river channels during a major rainfall/flow event, through various mechanisms. In a study site in Oregon, Lienkamper and Swanson (1987) found that the majority of trees entering the stream channels were growing in areas not susceptible to bank erosion, and therefore attributed their introduction to wind effects. In contrast, studying a Tennessee river basin, Diehl and Bryan (1993) concluded that bank instability seemed to be main cause of woody debris introduction;. Diehl and Bryan (1993) also remarked that ‘evaluation of debris potential based on inspection of a single site probably would have yielded a poor estimate of availability and potential for transport of debris’. Nakamura and Swanson (1994) studied the distribution of woody debris in a mountain stream in Oregon, and suggested that channel and sinuosity play an important role in the distribution of woody debris. They note that one of the predominant storage site for woody debris is on the outside of bends.

### **Laboratory studies involving woody debris**

A number of recent laboratory studies have reported on the effect of LWD on stream geomorphology as well as on aquatic habitat. Cherry and Beschta (1989) examined the effect of a single fixed model log in various geometric configurations on local scour. In addition to proposing a simple theoretical model of forces on a log, Braudrick and Grant (2000) performed experiments examining the entrainment or incipient motion of logs resting on a rough bed by flow. Braudrick and Grant (2001) studied the transport and deposition of woody debris in model channels with a variety of channel patterns, such as alternate bars, mid-channel bars, and meander bends. They observed that, in general agreement with field studies, woody debris tended to deposit on the outside of bends, heads of islands, and bar cross-overs. They remarked that ‘Often pieces would roll up onto bars, where depth was very shallow’. The experiments were nevertheless carried out under quite a small scale, with mean depths of less than 2 cm, and also Froude numbers exceeding unity. Wallerstein et al. (2001) discuss the theoretical aspects of laboratory/field scaling and requirements for similitude, and studied experimentally the hydrodynamic forces on a single log, as well as the resulting scour when a single log falls perpendicular to a stream.

### **Large woody debris and bridges**

The effect of LWD on local scour around bridge piers was already briefly discussed in Laursen and Toch (1956), who concluded that debris may aggravate pier scour by effectively making the pier wider. They nevertheless cautioned that the overall effects were difficult to evaluate because the permeability and the position of the debris pile could be equally as important as the overall size of the pile. In a FHWA-sponsored study focusing on debris problems, Chang and Shen (1979) reported on a statistical analysis of debris hazards to bridges, based on case histories provided by state highway personnel. They summarized some observations by field engineers regarding floating debris in rivers, including

- seldom was debris observed floating in large masses,
- in fairly straight streams, floating debris tends to move in the thalweg at the rising stage, and towards the banks on the receding stage,
- a great deal of debris usually enters the river during the first big flood of the season.

They also recommended further studies into the development of debris deflectors as well as debris traps and basins.

The study by Diehl (1997), also sponsored by the FHWA, and based on information taken from a national survey, as well as from field studies conducted in 11 states and the District of Columbia, gives a comprehensive review of issues related to debris accumulation at bridges. Diehl emphasizes the concept of a design log length, which is a length 'above which logs are insufficiently abundant ... to produce drift accumulations equal to their length', which would provide a criterion for determining the length of a bridge span. According to Diehl, the main factors entering into the design log length are the width of the upstream channel and the maximum length of sturdy logs. Diehl further found that, in Indiana, 25% of selected bridges had spans less than the design log length.

## ***1.2. Organization of the report***

The project was divided into two parts—a laboratory study and a field study. Chapter 2 discusses the issues associated with a laboratory study and the design of the experiments. The results of the experiments are presented in Chapter 3. Details of the implementation of the field study are described in Chapter 4, and field observations are presented in Chapter 5.

## 2. Laboratory studies

Field phenomena occur in a complex and uncontrolled environment, e.g., channel geometry or hydraulic conditions, which makes it difficult, in a study of debris accumulation, to identify essential mechanisms or relationships. In this regard, laboratory studies hold a distinct advantage, in that idealized, i.e., simplified, cases can be investigated under well-defined repeatable conditions, thereby facilitating interpretation of results. Evidently, not all details can be simulated faithfully in the laboratory, and issues of scaling between laboratory and prototype (field) need to be considered carefully. The design of the present laboratory studies was influenced substantially by the features of a particular site, namely, the SR 59 south crossing of the Eel River, but no attempt was made to implement any strict scaling between a specific field site and the laboratory model. Rather, the goal was to seek general relationships between debris accumulation at bridge piers and channel hydraulics that might be applicable to a wide range of site conditions.

### 2.1. *Characterization of laboratory debris and debris piles*

Before any discussion of scaling issues, an even more basic question arises regarding the appropriate characterization of debris and debris piles. What are the physical characteristics of LWD relevant to the problem of accumulation at bridge crossings? Further, single logs are usually of no concern to bridge hydraulics, except when they serve as the initial element of a pile, so how many logs together constitute a pile? A third question surrounds the stability of a pile, however defined. A pile may form ‘initially’, but within a ‘short’ time, the pile may disaggregate, such that the initially trapped logs disperse quickly into the flow.

In their flume study of transport and deposition of LWD in streams, Braudrick and Grant (2001) argue that the two main characteristics relevant at least for wood deposition are length and diameter. As an alternative to the diameter, the buoyant depth, which includes the effect of wood density, was also proposed. This has been implicitly or explicitly assumed in several flume studies that have sought to simulate LWD in channels (Cherry and Beschta, 1989; Braudrick and Grant, 2000, 2001; Wallerstein et al., 2001), in which the model for the LWD has generally consisted of cylindrical wood dowels of varying densities, sometimes with a circular disk attached at an end to model a rootwad. The choice of cylindrical dowels makes for simple characterization, because the length and diameter are simply defined. The importance of shape (except for the presence or absence of a rootwad) and surface texture or roughness of the log is thought to be negligible or at least secondary.

While cylindrical dowels (without model rootwads) have also been used in the present study, most of the experiments were conducted with natural twigs (Fig. 2.1). The effect of shape as such will not be examined in any detail, but, as will be seen, shape does materially affect the likelihood that a model log will be trapped by a bridge pier, and hence it was decided to use a more ‘realistic’ model log element. The more complicated shape does however bring with it the difficulty of defining its ‘length’ and ‘diameter’. For the twigs shown in Fig. 2.1, length and diameter were determined by image analysis as the longest straight-line distance between points on the twig, and the fiber width, i.e.,

the average diameter of the largest circle that could be inscribed at each point on the ‘skeleton’ of the twig respectively,. The mean length and diameter defined in this manner were 4.55 in (11.6 cm) and 0.23 in (0.58 cm).



**Figure 2.1: Twigs used as model logs**

The problem also arises of defining what constitutes a laboratory (or even a field) debris pile. Two different approaches to a definition have been taken in the present study. A fixed number of logs, say,  $N_L^*$ , is specified, and if the actual number of logs,  $N_L$ , trapped at the pier is equal or greater than  $N_L^*$ , then a pile is deemed to have developed. The specific value chosen for  $N_L^*$  will necessarily depend on the details of the experiment. An alternate approach recognizes that  $N_L$  will be variable, and can be treated as ‘random’. As such, the distribution of  $N_L$  motivates a ‘probabilistic’ approach. This avoids a somewhat arbitrary specification of a fixed limiting value,  $N_L^*$ , and instead, observed distributions of  $N_L$  will form a more flexible basis of the data analysis and interpretation. Some preliminary experiments in this project used the first approach, but most of the results to be discussed used the second.

Observations of the motion of model logs in the laboratory indicate that they may sometimes contact with the bridge pier, but are immediately deflected. At other times, individual logs may be initially trapped for a period of time at the pier, but eventually escape downstream. As a consequence, piles may initially develop and grow to a significant size, but dissipate within a short time. Since such events undoubtedly occur also in the field, the *stability* of debris piles needs to be considered. Evidently, in the field, a debris pile will be of engineering concern mainly if it is sufficiently long-lived, particularly if it continually grows. Similar to the first approach taken in defining a debris pile,

a simple approach to defining a *stable* debris pile consists of defining a time period,  $T^*$ , such that if a debris pile survives longer than  $T^*$ , it is considered a stable pile.

## 2.2. *Scaling issues*

Although strict similitude between laboratory and field was not sought in this study, scaling issues still need to be considered. A general discussion of the topic may be found in elementary hydraulics texts. Within the specific context of LWD in streams, Wallerstein et al. (2001) discussed distorted Froude scaling in connection with estimating drag forces as well as local scour effects associated with LWD.

Traditionally, laboratory experiments involving open-channel flows have chosen Froude scaling, i.e., the Froude number,  $Fr$ , in the model and in the field should be the same, where  $Fr \equiv V / \sqrt{gH}$  ( $V$  is a cross-sectionally averaged velocity,  $g$  the acceleration due to gravity, and  $H$  a characteristic flow depth). This emphasis reflects a focus on free-surface phenomena and on the location of the water surface elevation, which is the central concern in open-channel-flow problems. Reynolds-number scaling, which requires the same Reynolds number,  $Re \equiv VL/\nu$  ( $L$  is a characteristic length scale, and  $\nu$  the fluid kinematic viscosity), in both model and prototype, cannot in practice be satisfied at the same time as Froude-number scaling. The neglect of strict Reynolds number similitude is usually justified provided that  $Re$  is sufficiently large that the flow is fully turbulent, because, in that case, flow parameters are relatively insensitive to variations in  $Re$ .

The appropriate scaling for phenomena related to the development of a debris pile is not clear. On the one hand, woody debris floats on the free surface, and so the Froude number should retain its relevance. On the other hand, the importance of the flow details around the debris as well as the bridge pier suggests that the Reynolds numbers (based on the pier width or say the debris diameter) may play a more important role than in typical open-channel-flow studies. In the present work, strict similitude between laboratory and the field was not sought for specific field conditions, and so the experiments covered a range of Froude numbers corresponding approximately to those that might conceivably be encountered in the field. This permits studies in which the average velocity is kept the same, which implies that a Reynolds number based on the log diameter would remain the same (though of course much smaller than in the field), but the depth (and hence the Froude number) varies.

In addition to dynamic similitude involving parameters such as  $Fr$  and  $Re$ , strict similitude also requires geometric similitude, i.e., length-scale ratios in model and prototype should be the same. Again, because of conflicting requirements and the given size of the available facilities, this will not necessarily be adhered to strictly. While depths are reasonably close to being geometrically similar, the size of the logs, particularly length, may be quite far from geometric similarity. Because the laboratory study was intended to focus on single-pier accumulations, the model debris length needed to be substantially smaller than one half of the channel width. These choices undoubtedly make interpretation less straightforward, but an interpretation in terms of dimensionless quantities, such as a log-length/depth ratio, should circumvent some of these difficulties.

## **2.3. Experimental apparatus and procedure**

### **The laboratory channel**

The laboratory experiments were performed in a tiltable, recirculating flume that is rectangular in cross-section, 40 cm wide and 15 m long. The bottom and sidewalls are constructed entirely of transparent acrylic. Flow is driven by a Peerless horizontal centrifugal pump (Model 1040 AMBF) capable of delivering a head of 41.55 ft at a discharge of 800 gpm. This is connected to an ABB variable speed drive, Model ACH401, by which means the discharge can be conveniently varied in a continuous manner. The discharge is measured with an electromagnetic flow meter (Omega-Mag FMG-700) with stated accuracy of 1% for discharges larger than 0.29 cfs; in some cases, the discharges studied were less than this value, and the accuracy may be less in those cases.

### **The model pier**

Because of the straight laboratory channel, the design of experiments was based primarily on the characteristics of the SR59 Eel River site, which is contained within a fairly straight reach (Fig. 1.1). Laursen and Toch (1956) concluded, in a study mainly of local scour around bridge piers but including some discussion of flow with debris, that, in laboratory simulations, it was sufficient to model a single pier. Moreover, the situation at the Eel River site is also primarily one of single-pier debris accumulation, at least initially, and so it was decided to model only a single pier in the laboratory channel. At both field sites, the ratio of pier-to-pier span to the pier width exceeds 32. To achieve reasonable pier Reynolds numbers, however, a convenient choice of a model pier width was ½" (1.25 cm), resulting in a ratio of channel width,  $B$ , to pier width,  $b$ , of 32. The plan shape of the model pier is rectangular with rounded nose and rear, with a total length of 4.125" (10.5 cm).

### **Experimental procedure**

The laboratory study was aimed at understanding better the trapping of LWD at bridge piers. As such, the variety of quite complicated natural phenomena needed to be simplified in order to see more clearly the relationship between flow parameters and debris trapping. According to Chang and Shen (1979), logs in the field primarily travel as individual logs rather than in groups or 'rafts'. Chang and Shen (1979) (see also Braudrick and Grant, 2001) also noted that logs tend to travel along the thalweg of the stream, and assume an orientation approximately parallel to the flow direction.

The basic experimental procedure was motivated by these observations. Model logs were introduced manually one at a time at a section approximately 20 ft upstream of the section with the model pier, with their longest dimension approximately aligned with the flow. No attempt was made to introduce the single logs in a manner that was strictly periodic in time. Examination of video recordings of the arrival of logs at a section just upstream of the pier reveal that, on average, the time between arrival of successive logs was typically greater than 1 sec, though occasionally two model logs arrived almost simultaneously. In a single *run*, a total of approximately 70 logs were thus introduced. After all logs have been introduced, it was ascertained to what extent if any a stable number of model logs had accumulated. This procedure was repeated for a number of

times,  $N$  (see below for specific values), which constituted a single *experiment*, so as to determine the fraction of total runs in which a stable debris pile develops or a certain number of model logs had accumulated. This is taken as an estimate of the likelihood or probability that a stable debris pile will occur for those specific hydraulic conditions.

In the first series of experiments (Series A and B), a pile has been defined as any grouping of logs at the pier that consists of 3 or more logs, i.e.,  $N_L^* = 3$ . The choice of  $N_L^* = 3$  is somewhat arbitrary, but  $N_L^* < 3$  seems unreasonable, while  $N_L^* > 3$  would be equally arbitrary. In the Series A and B experiments, a stable pile was also simply defined as any debris pile that survived more than  $\approx 15$  minutes. It had been previously found that, as a rule, a pile surviving more than  $\approx 15$  minutes would last indefinitely. Under Froude number scaling, with the chosen length ratio (based on the ratio of pier-widths), 15 minutes in the laboratory would correspond to  $\approx 100$  minutes in the field. In the Series A experiments, the number of runs in an experiment was chosen to be  $N = 16$ . In subsequent experiments,  $N$  was increased to 50 in order to obtain better statistical estimates. Quite often, an experiment was performed over two days, with 25 runs being done on one day, and the remainder on another day. It was observed that statistics based on the first 25 runs could quite different from the second 25 runs. This suggests that even 50 runs may still be marginal as far as statistically stable results were concerned. Quantitative differences were also sometimes seen in results from experiments under similar conditions conducted by different individuals, though the same qualitative trends were noted.

The later series of experiments (Series C, D, and E) refined the definitions of a debris pile. Instead of simply determining whether or not there were three or more logs trapped at the pier, in these later experiments, the actual number of trapped logs surviving more than 15 minutes is counted. In this manner, more information from experiment is retained, a simplistic definition of a debris pile is avoided, and a distribution of surviving logs can be estimated. A similar refinement could also be made in the definition of the stability of a debris pile, but this was not pursued.

In many runs, a video recording was made with a standard video cassette recorder (VCR), using a camera mounted  $\approx 3$  ft above the channel, with lens looking directly downwards in the vicinity of the pier nose. This permitted visualization of the movement of model logs and the trapping process.

## **2.4. Experimental design**

The first series (A and B) of experiments were essentially exploratory in nature. They were undertaken to determine whether a debris pile could in fact develop around the laboratory pier using the model logs, to determine parameter ranges of interest, and to test the experimental procedures. Initial experiments were also performed with a debris deflector (simulated with a  $\frac{1}{4}$ " threaded rod, installed directly upstream of the pier).

The results of the first series of experiments set the direction for the later series C and D. In particular, the observed surprisingly strong effect of the depth (together with the known features of the field sites) prompted a more systematic study, and a refinement

of data gathered and its analysis. Two depths were chosen for extensive study, namely,  $H \gg 5$  cm and  $H \gg 15$  cm.

A possible criticism can be leveled at comparisons between experiments in which the flow is approximately uniform (and so the depth is everywhere the same) in that extraneous effects due to the flow upstream of the pier, such as the strength of secondary currents, may influence results. Further, it was pointed out in Chapter 1 that the mid-stream sand bar may play a significant role in debris accumulation (see Fig. 1.2, and also Diehl, 1997). For both these reasons, experiments with a model sand bar (Series D) were motivated. Results for cases with and without a sand bar, but under the *same* upstream flow conditions, could then be more meaningfully compared. The model sand bar had rather exaggerated dimensions, and no attempt was made to make it geometrically similar to that at the Eel River site. It is  $\approx 40$  cm in total length, and  $\approx 11$  cm in total height, with streamlined upstream and side faces but a vertical downstream face. Pictures of the model sand bar are given in Chapter 3 (Fig. 3.8).

Experiments with two types of countermeasures were also conducted. One type of countermeasures is modeled after those in the field, namely, a vertical cylinder installed as a debris ‘deflector’ directly upstream of the model pier. As seen in Fig. 1.2b, the effectiveness of such debris deflectors in the field has been dubious. An alternative countermeasure consisting of submerged groin-like structures was also tested. Both countermeasures were tested with and without a model sandbar present, and at two different velocities but essentially at constant depth.

Table 2.1 summarizes the range of parameter values in the laboratory study and compares them with values typically encountered in the field (heavily based on characteristics at the Eel River site) under debris-transporting conditions.

**Table 2.1: Parameter ranges in the field and in the laboratory**

<b>Parameter</b>	<b>field</b>	<b>laboratory</b>
pier width, $b$ (ft)	2	1/24
bridge span, $B_{sp}$ (ft)	66	2/3
depth, $H$ (ft)	10-20	1/6 – 1/2
velocity, $V$ (ft/s)	4-7	1/2 – 1
log length, $L$ (ft)	< 65 ft	1/3
log diameter, $d$ (ft)	< 3 ft	1/48
Froude number, $Fr \circ V / \sqrt{gH}$	0.2 – 0.4	0.2 – 0.4
Reynolds number, $Re_b \circ Vb/n$ ( $\cdot 10^3$ )	1000	3
Log length/depth, $L/H$	< 3	< 2
Log length/span, $L/B_{sp}$	< 1	1/4

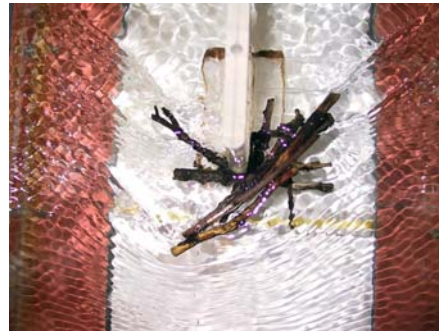
### 3. Results of laboratory experiments

#### 3.1. Series A experiments

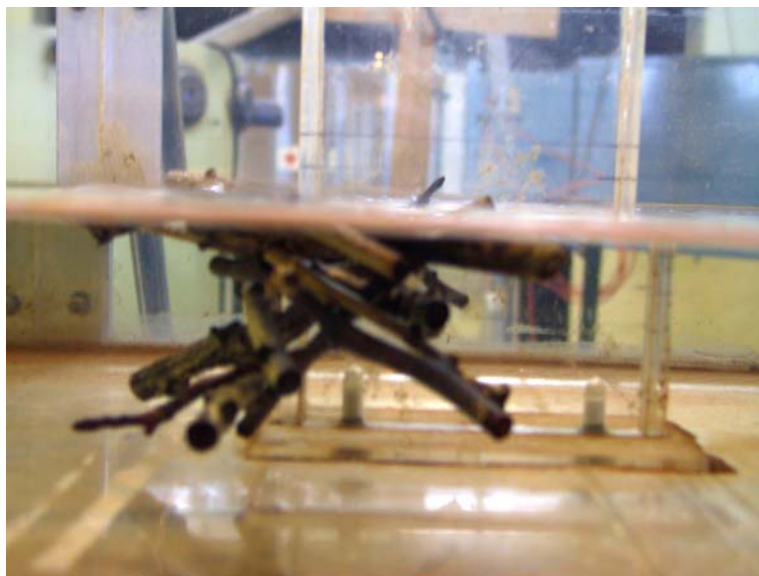
A photograph of model logs (twigs), upstream of the pier section, traveling downstream (to the left) is shown in Fig. 3.1a. In spite of the logs being introduced with their longest dimension approximately aligned with the flow, they do not necessarily maintain this orientation, often rotating significantly as they move downstream. A plan-view photograph of a stable debris pile at the model pier is shown in Fig. 3.1b, while the corresponding side view is given in Fig. 3.1c. In this particular run (at a shallow depth of  $\gg 5$  cm), the pile consists of a large number (at least 6) of twigs, with the pile almost extending through the entire water column, and at least one twig touching the bottom of the channel.



(a)



(b)



(c)

**Figure 3.1: a) model logs (twigs) moving downstream (to the left) in laboratory channel, b) top view of debris pile at model pier, c) side view of debris pile at model pier**

Table 3.1 shows the results of the first series of experiments performed, in which only 16 runs with twigs as model logs were made at three different flow depths and velocities (the intermediate case at the highest velocity was omitted). In this table, the fraction of runs in which a debris pile has occurred is given, e.g., for a depth of 0.5 ft and a velocity of 0.7 fps, a debris pile developed in 3 out of 16 runs. Except at the smallest depth, a debris pile seems less likely to occur at higher velocities. Indeed, initial experiments with velocities greater than 1 fps were unsuccessful in creating large stable debris piles. More surprisingly, a debris pile seems *more* likely to develop at smaller depths, except possibly at the lowest velocities,. A more definitive conclusion is however not possible because of the relatively small samples involved, and the small differences between the results for the different cases.

**Table 3.1: Series A experiments—fraction of runs developing debris piles**

depth ( $H$ ) velocity ( $V$ )	0.5 ft (15 cm)	0.33 ft (10 cm)	0.21 ft (5 cm)
0.98 fps (30 cm/s)	1/16		3/16
0.82 fps (25 cm/s)	0/16	1/16	4/16
0.70 fps (21 cm/s)	3/16	2/16	3/16

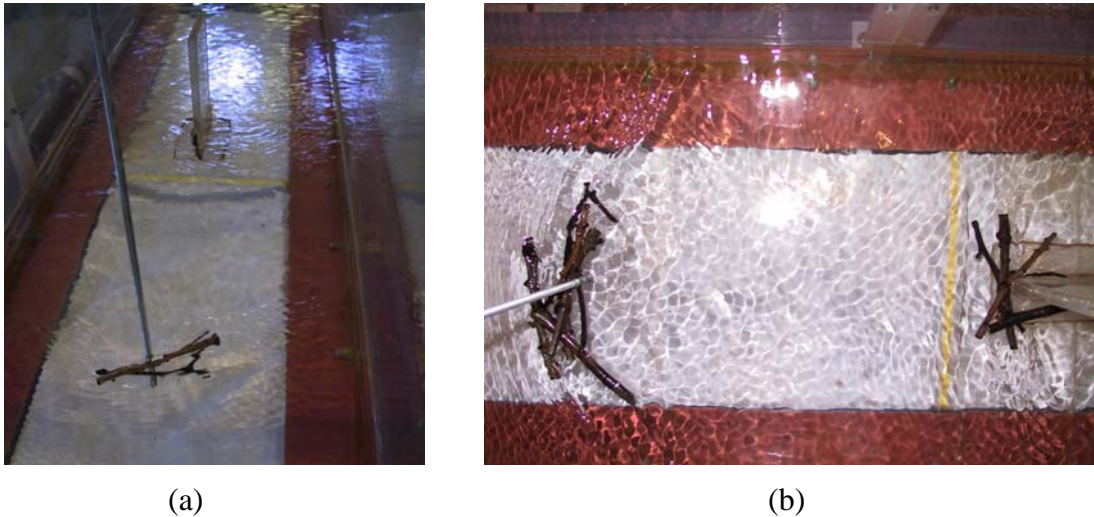
### 3.2. Series B experiments

#### Case without deflector

The second series of experiments explored in greater detail the rather surprising effect of depth, by keeping constant the velocity at the intermediate value of 0.8 fps (24 cm/s) and looking at the two extreme depths (0.5 ft and 0.21 ft). The experience during the Series A experiments also suggested that, for more reliable statistics, the number of runs needed to be increased, and so, for Series B, this was increased from 16 to 50 runs. The results are given in Table 3.2. The difference between the fraction of runs in which debris piles develop under the shallow-flow and under the deep-flow cases is still small, and the statistical significance of the difference can be questioned. A standard statistical hypothesis test can be performed to determine whether the two proportions might actually be equal (i.e., the null hypothesis, in spite of the small difference in the observed values) against the alternative hypothesis that the small depth does give a higher proportion of runs with debris piles. The resulting  $p$ -value is 0.16, which means that there is a 16% probability that the potential for debris-pile development are actually the same for both depths, and the observed small difference arises due solely to statistical sampling error. Although 16% is fairly small, statistical significance has traditionally been based on smaller values such as 10% or 5%. This may be misleading, in that, the difference might have been larger if a more stringent definition of a debris pile had been used, e.g., if 6 logs instead of 3 were necessary in order to qualify as a debris pile. This motivated the more refined approach taken up in later experiments.

### Case with deflector

The first experiments with a deflector were also conducted in this series. The deflector, modeled with a ¼” threaded rod, was installed 15” (corresponding, assuming a length scale ratio of 48, to 60-ft in the field) directly upstream of the pier. Fig. 3.2 shows photographs taken of cases with the deflector in place, again all at the smaller depth. In the large majority of the runs, the debris pile developed at the deflector, with very few or no logs at the pier. Logs can however still be trapped at the pier (Fig. 3.2b).



**Figure 3.2:** Case with deflector installed upstream of model pier, a) perspective view looking downstream, b) top view with debris accumulated at both deflector and pier

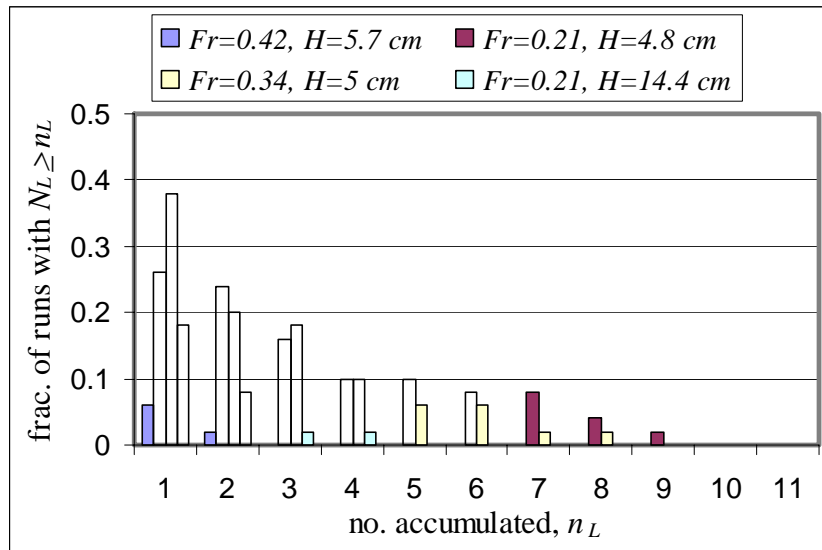
**Table 3.2:** Series B experiments–fraction of runs developing debris piles (constant velocity, 0.8 fps or 24 cm/s) with and without a deflector installed upstream of the pier

depth (H)	without deflector	with deflector
0.50 ft ( $Fr = 0.21$ )	9/50	19/50
0.21 ft ( $Fr = 0.32$ )	11/50	26/50

The results shown in Table 3.2 show a pronounced difference between the corresponding cases with and without a deflector, with the deflector apparently enhancing the trapping of the debris. The source of this difference is not definite, though the cylindrical geometry and the ‘rougher’ surface texture (due to its being threaded) of the model deflector are likely contributing factors. The effect of the depth receives rather stronger support in the case with a deflector, with a larger observed difference between the two depths studied (the  $p$ -value here is 0.08, and so the difference would be significant at the 10% level).

### 3.3. Series C experiments

These experiments continued along the lines of the Series B experiments, but refined the definition of a debris pile and pile stability. Consider first the results shown in Fig. 3.3 of experiments with wooden dowels in which, on the one hand, the depths were kept approximately constant,  $\gg 5$  cm, but the velocity and hence the Froude number,  $Fr$ , was varied, and on the other hand,  $Fr$ , was kept constant at 0.21 and two depths were studied. On the  $x$ -axis is plotted the number of model logs,  $n_L$ , remaining at the pier after 15 minutes, whereas on the  $y$ -axis is plotted the fraction of runs in which the actual number of logs,  $N_L$ , is greater than or equal to  $n_L$ . Fig. 3.3 therefore is interpreted as showing the distribution of  $N_L$  treated as a discrete random variable. At the highest velocity ( $Fr = 0.42$ , corresponding to a velocity of 1 fps or 31 cm/s), very few logs were trapped (only in 0.06 of the 50-runs was any log trapped, and in none of the runs were there more than 2 trapped logs observed). According to the definition of a debris pile used previously, i.e., a minimum of 3 trapped logs, no debris pile developed for this particular case. As might be intuitively expected, if the depth is kept (approximately) constant and the velocity (and hence  $Fr$ ) decreases, then the likelihood of debris (dowels) accumulating at the model pier increases. Interestingly, both of the lower-velocity cases resulted in quite similar distributions, so it seems that, below a certain velocity, the velocity may cease to have any significant influence. This was already seen though less clearly in the results of the Series A experiments. Whether this is a scaling effect which would not occur at full scale is less clear.



**Figure 3.3: Distribution of experimental runs with number of logs accumulated for cases where dowels are used as model logs**

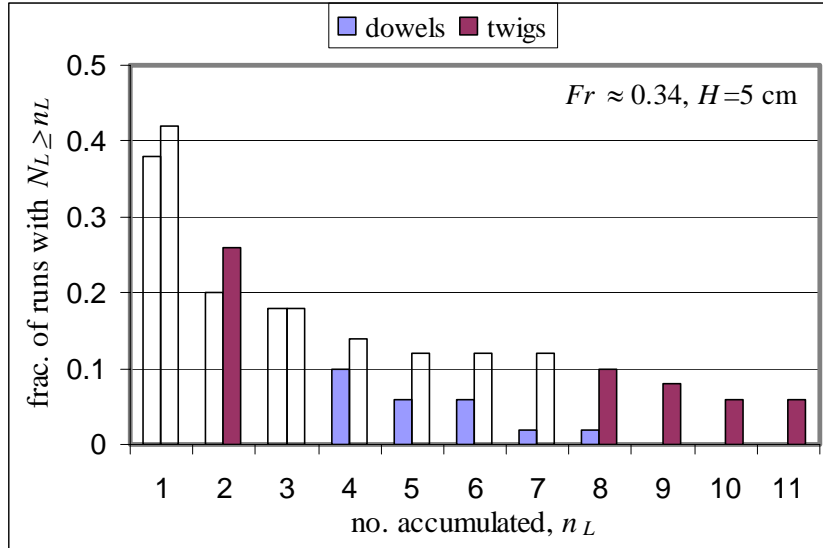
The comparison of the two cases at constant  $Fr$ , but different depths points to more dramatic differences. Whereas at the larger depth, no run with 5 or more trapped dowels was observed, at the smaller depth, this occurred in 10% of the runs, i.e., 5 runs. The more refined approach to defining a debris pile permits more flexibility in data ana-

lysis in that the implications of different criteria for a debris pile can be investigated. For example, if 3 or more logs trapped at the pier is taken to constitute a debris pile, Fig. 3.3 shows that a debris pile occurred in 0.16 of the runs for the smaller depth, but in only 0.02 of the runs for the larger depth. On the other hand, if 5 or more trapped logs is the basis of the definition of a pile, then the corresponding values are 0.1 and 0. The  $p$ -values in testing the hypothesis that the proportions are the same versus the alternate hypothesis that the proportion in the smaller-depth case is higher than in the larger-depth case can be estimated for both definitions, and in fact are both found to be statistically significant at the 5% level.

The above analysis supports the argument that the depth may have a stronger influence than might be otherwise thought, but it should be emphasized that this conclusion holds for the specific case where  $Fr$  is maintained constant (and more specifically at a value of  $Fr \gg 0.2$ ). Varying depth at constant  $Fr$  implies that the velocity is changed. In the small-depth case, the average velocity was 14 cm/s whereas in the large-depth case, it was 25 cm/s. Whether the observed effect should be attributed primarily to change in depth or change in velocity remains to be explored, and will be taken up below again.

Fig. 3.4 shows results illustrating the difference when using dowels and using twigs as model logs for given velocity (the case with the twigs had a slightly higher velocity, 25.5 cm/s, compared to 24 cm/s for the case with the dowels) and depth. For small values of  $n_L (\leq 3)$ , the distribution are perhaps surprisingly similar; only at larger values of  $n_L$  do they diverge. Thus, the primary difference between dowels and twigs would seem to be that larger accumulations are much more stable when twigs are used, and so are observed more frequently. Since the flow conditions are for practical purposes identical, the difference is attributed to the difference in physical characteristics, namely the much rougher surface texture of the twigs as well as their occasional branching features.

For those cases where twigs are used as model logs, the effect of velocity at constant depth or of depth at constant velocity is considered in Figs. 3.5 and 3.6. In the former case, the distributions at both depths studied exhibit quite similar features (Fig. 3.5). As seen before with the dowel as model logs, there is a strong effect of velocity, in that the case with the lowest velocity ( $V \gg 15$  cm/s, i.e.,  $Fr \gg 0.13$  or  $0.21$  depending on the depth) shows a markedly higher probability of occurrence of large debris accumulation. Somewhat different from the case of the dowel logs, the difference between the cases of the two highest velocities is relatively small. The effect of depth can be inferred from Fig. 3.5, but is made clearer in Fig. 3.6, where the results for two depths are compared at each of three velocities. At the lowest velocity ( $V \gg 15$  cm/s), the effect of depth is weak or non-existent. At the two other higher velocities ( $V \gg 24$  cm/s or  $V \gg 30$  cm/s), however, a much more noticeable effect of depth is observed. If as before a debris pile is defined as containing 3 or more logs, then the differences seen in Fig. 3.6 between the two depths at the two higher velocities would be statistically significant at the 5% level. The variation of the probability of occurrence of a debris pile with either velocity or depth is therefore not necessarily monotonic. Above a certain velocity, the effect of velocity may be negligible, while below a certain velocity, the effect of depth may be negligible.



**Figure 3.4: Comparison of distribution of accumulated model logs in cases with dowels and with twigs as model logs**

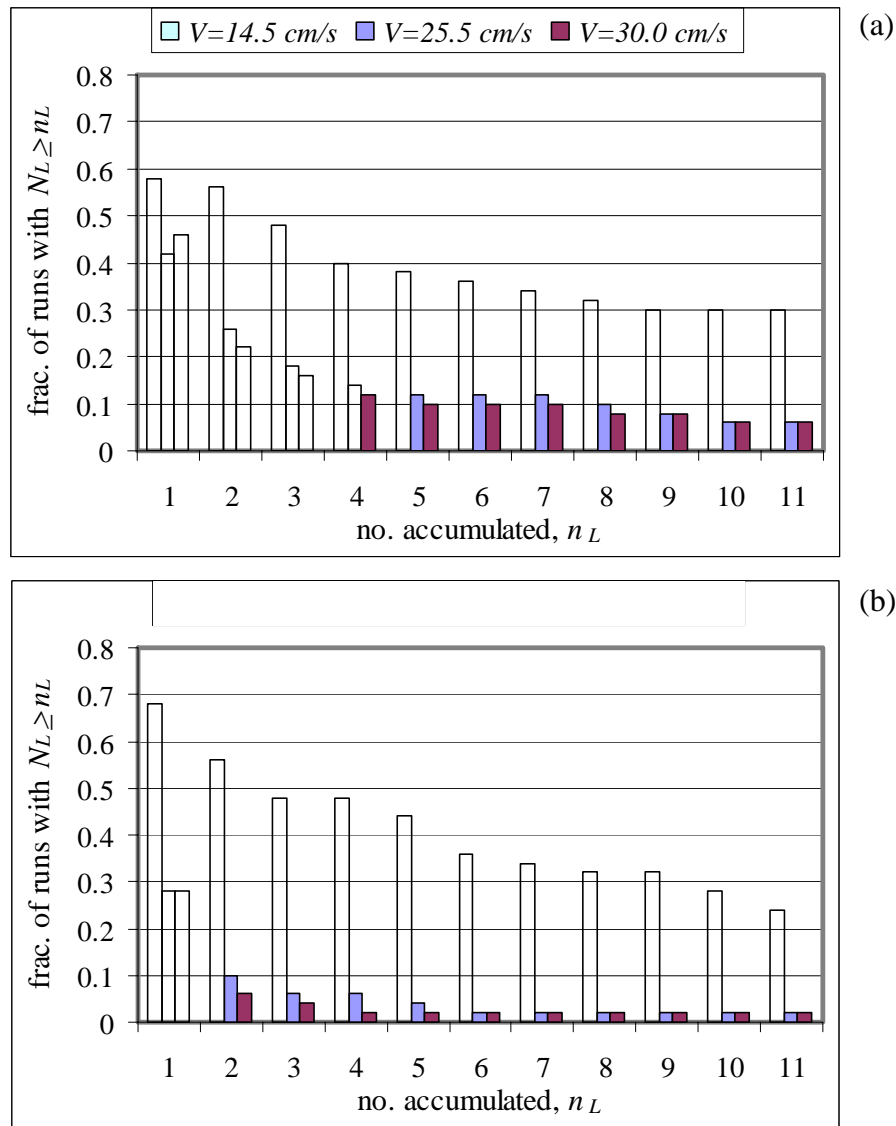
The above has focused on the *qualitative* dependence of the likelihood of debris piles on changes in velocity or depth. Caution must be exercised in applying the results *quantitatively* to the field scale. As argued in Sec. 2.3, any such application must be formulated in terms of dimensionless parameters, such as the Froude number, rather than in terms of absolute values of velocities or depths. Fig. 3.7 compares the effect of depth at constant  $Fr$  when twigs are used as model logs. Strictly speaking, to be consistent, the depth should be also expressed in dimensionless terms. The more obvious choices of a normalizing length scale, such as the pier width, or an average log diameter or length, are constant in the experiment. Thus, for qualitative purposes at least, it is immaterial whether the depth is used in dimensional or dimensionless form. As seen before when dowels were used as model logs, the effect of depth is evident even at constant  $Fr$ .

### 3.4. Series D experiments

This series of experiments was performed with a simulated sand bar in the channel. The submerged sandbar is shown in Fig. 3.8. Two experiments were performed with this configuration, with a single depth corresponding to the larger depth of the previous experiments, i.e.,  $H \gg 15$  cm, but at two upstream velocities, again approximately equal to those used in the previous experiments without the simulated sand bar. In the case with  $V = 15.9$  cm/s, only 25 runs were performed, in part because such a large proportion of the runs resulted in large debris accumulations. As discussed in Sec. 2.3, this configuration avoids possible extraneous upstream effects because the upstream flow should be for practical purposes identical—only the section with the pier on top of the bar and its immediate vicinity differs from the cases studied without the sand bar.

The results in Fig. 3.9 indicate that the presence of the sand bar will likely enhance the likelihood of larger debris accumulation at the model pier for *both* upstream velocities. As done previously, the statistical significance of the difference in proportion can be analyzed by computing the  $p$ -values for the hypothesis that the proportion of runs with a

debris pile are the same with and without the sand bar against the alternate hypothesis that the case with the sand bar yields a larger proportion. These will depend on the number,  $N_L^*$ , of trapped logs deemed to constitute a debris pile. Except for the lowest and the highest values of  $N_L^*$ , i.e.,  $N_L^* < 3$  or  $N_L^* > 8$ , the observed difference is found to be statistically significant at least at the 10% level, and for some values even at the 5% level. What is also noteworthy is that, at the section where the pier is situated, the local velocity is substantially larger, approximately 25%, because of the blockage due to the presence of the sand bar. As has been seen above, the tendency for debris accumulation may be substantially reduced at larger velocities. That, in spite of the velocity effect, the presence of the sand bar resulted in greater probability of a debris pile further supports the view that the sand bar plays a key role in debris accumulation.



**Figure 3.5: Effect of velocity on distribution of number of accumulated logs at approximately constant depth, a)  $H \gg 5$  cm, b)  $H \gg 15$  cm**

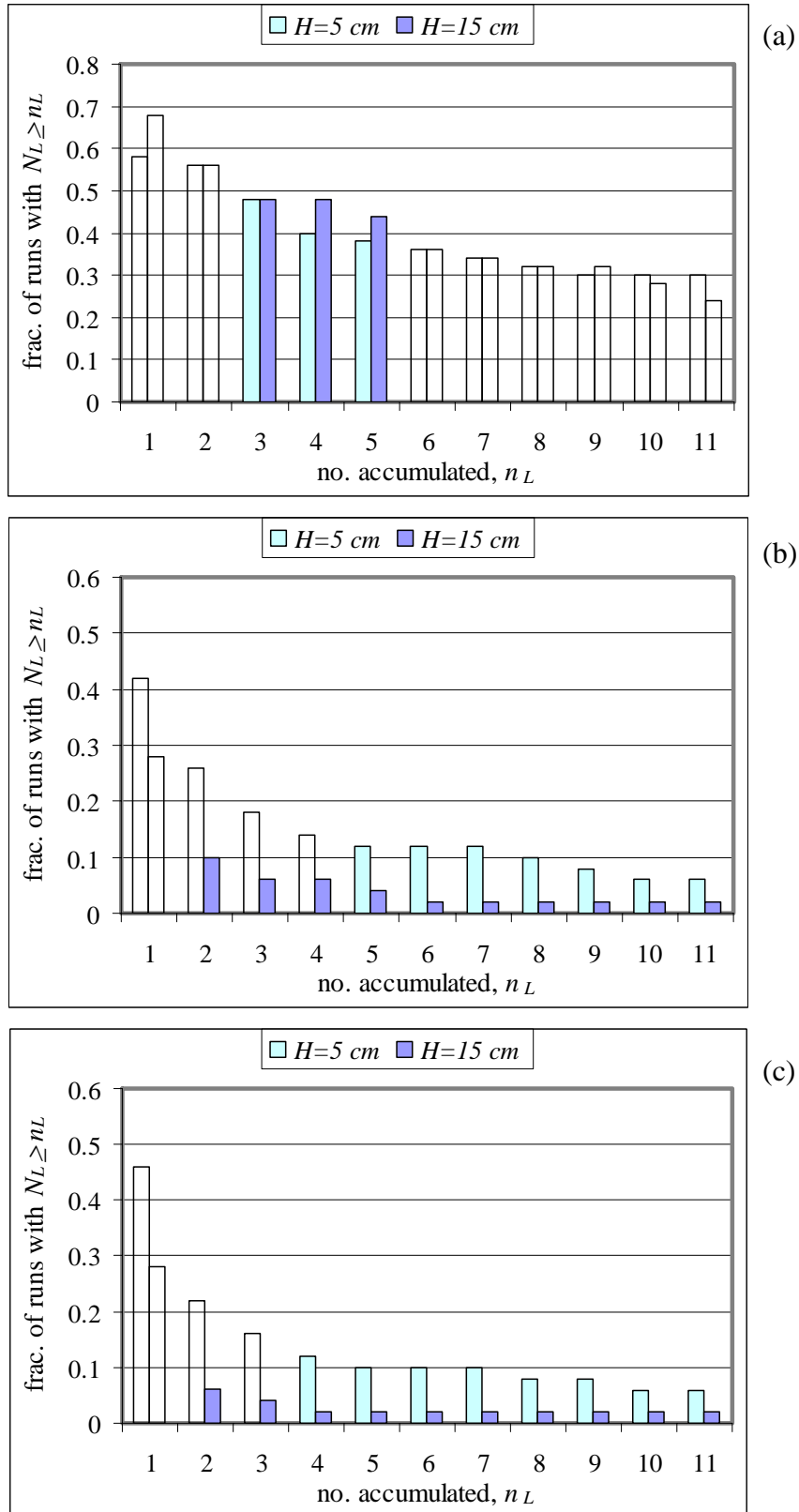
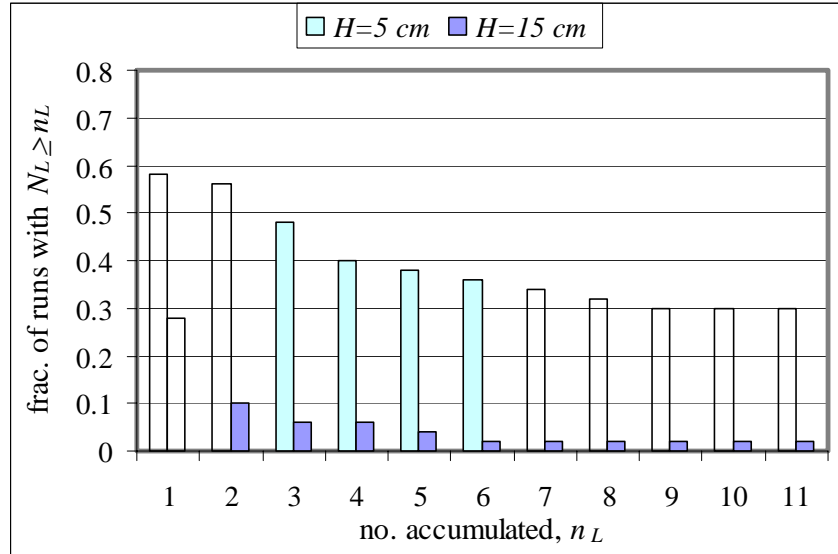
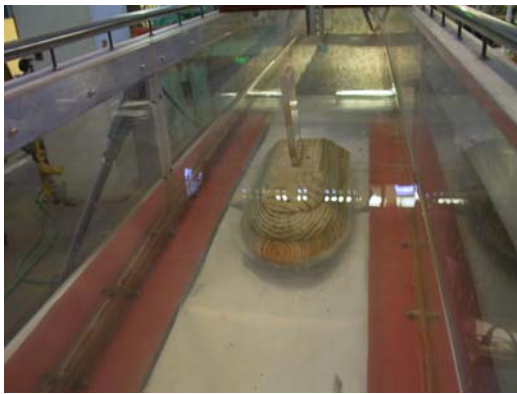


Figure 3.6: Distribution of accumulated logs at two different depths with velocity approximately constant, a)  $V \gg 15\text{ cm/s}$ , b)  $V \gg 25\text{ cm/s}$ , c)  $V \gg 30\text{ cm/s}$



**Figure 3.7: Effect of depth on distribution of runs with debris piles at constant Froude number,  $Fr \gg 0.2$ .**



(a)



(b)

**Figure 3.8: Model sand bar in laboratory channel, a) perspective view downstream, b) sideview (flow is from right to left)**

While the above suggests the strong influence of the sand bar, the precise mechanism or even the physical characteristic of the sand bar most important in this regard is less certain. From our previous experiments, smaller depths would seem to be associated with debris accumulation. In the neighborhood of the pier installed on the simulated sand bar, the depth is certainly locally smaller than in the main flow, and so the hypothesis can be advanced that this aspect of the sand bar is decisive. Without a precise physical mechanism causing this tendency, the hypothesis is primarily based on a statistical correlation, and so should be viewed cautiously.

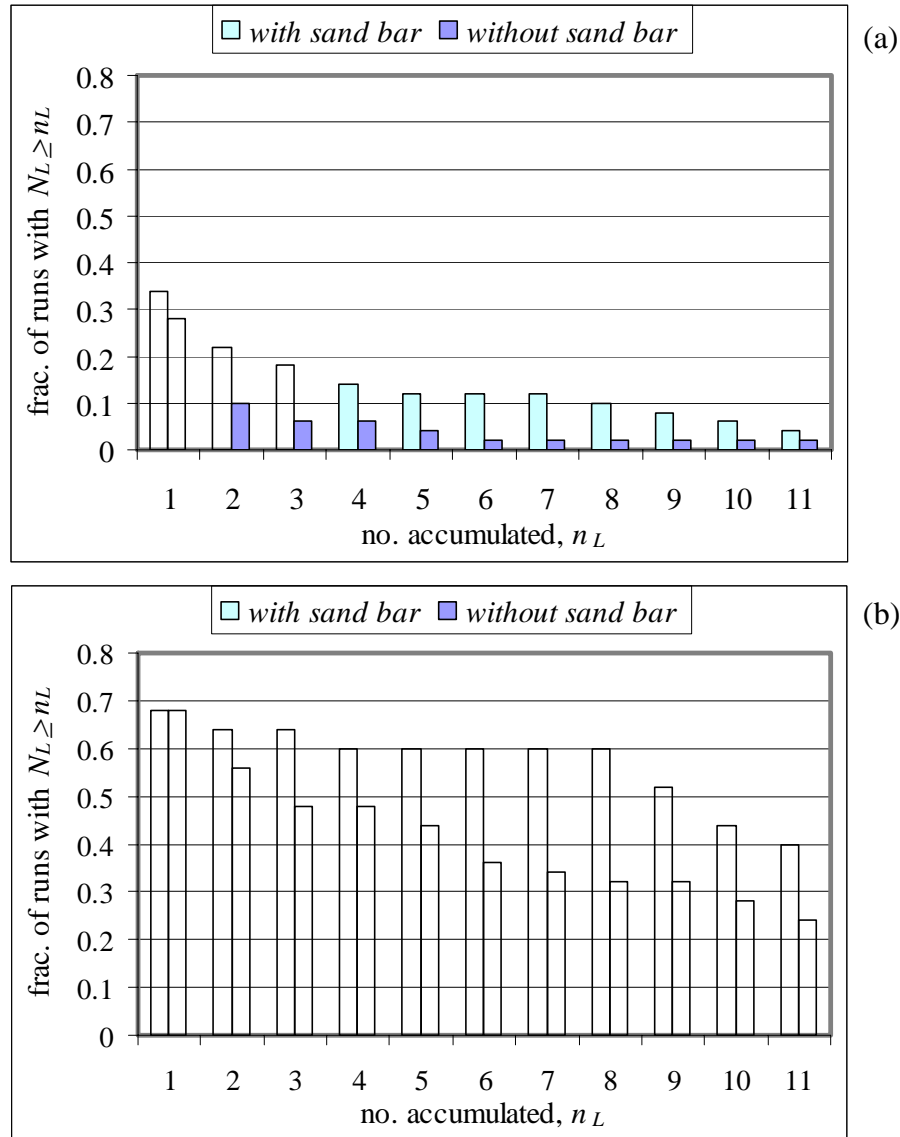
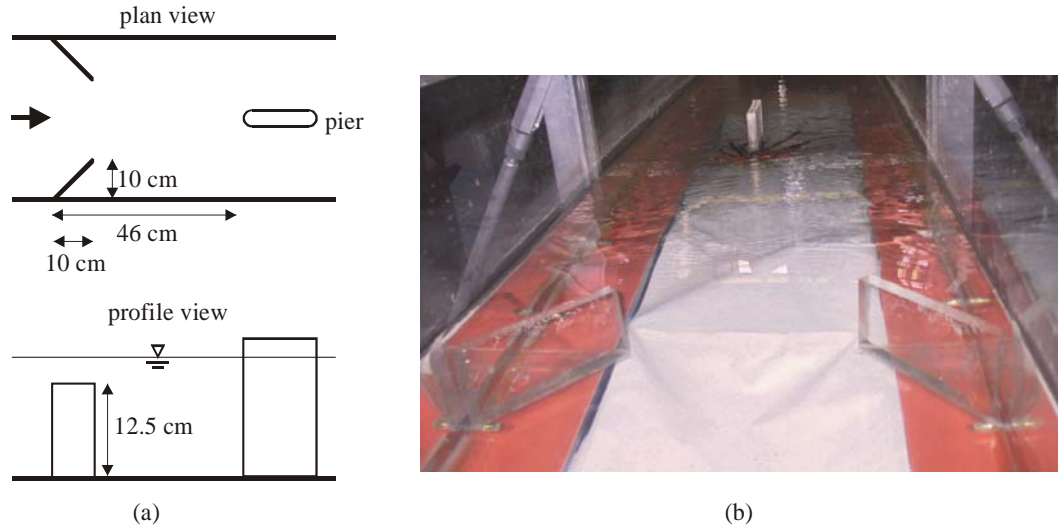


Figure 3.9: Effect of the presence of a sand bar on the distribution of runs with a debris pile, a)  $V \approx 25$  cm/s, b)  $V \approx 15$  cm/s.

### 3.5. Series E experiments

This series of experiments studied the effect of two types of countermeasures for minimizing debris accumulations. Upstream depths were kept constant at  $H = 16.2$  cm. The first type of countermeasure was the vertical cylinder ‘deflector’, similar to that studied in the Series B experiments, except that the model deflector was constructed of smooth acrylic tubing rather than of a threaded rod. The second type of countermeasure consists of submerged groin-like structures, extending from the channel wall into the flow at a  $45^\circ$  angle (Fig. 3.10). Because it is submerged, it would not, as the cylindrical deflector tended to do, act to collect debris. It was designed primarily to generate a flow field that might be less conducive to debris accumulation, namely higher velocities throughout the water column in the pier region, with possibly streamlines diverging away

from the pier region. Approximate dimensions used in the study are shown in Fig. 3.10. Due to time constraints, the study of this alternative countermeasure was exploratory, and no attempt was made to vary the dimensions or orientation to find an optimal configuration. These countermeasures were tested in various settings, with and without the model sandbar, singly and together, and at two different approach velocities.

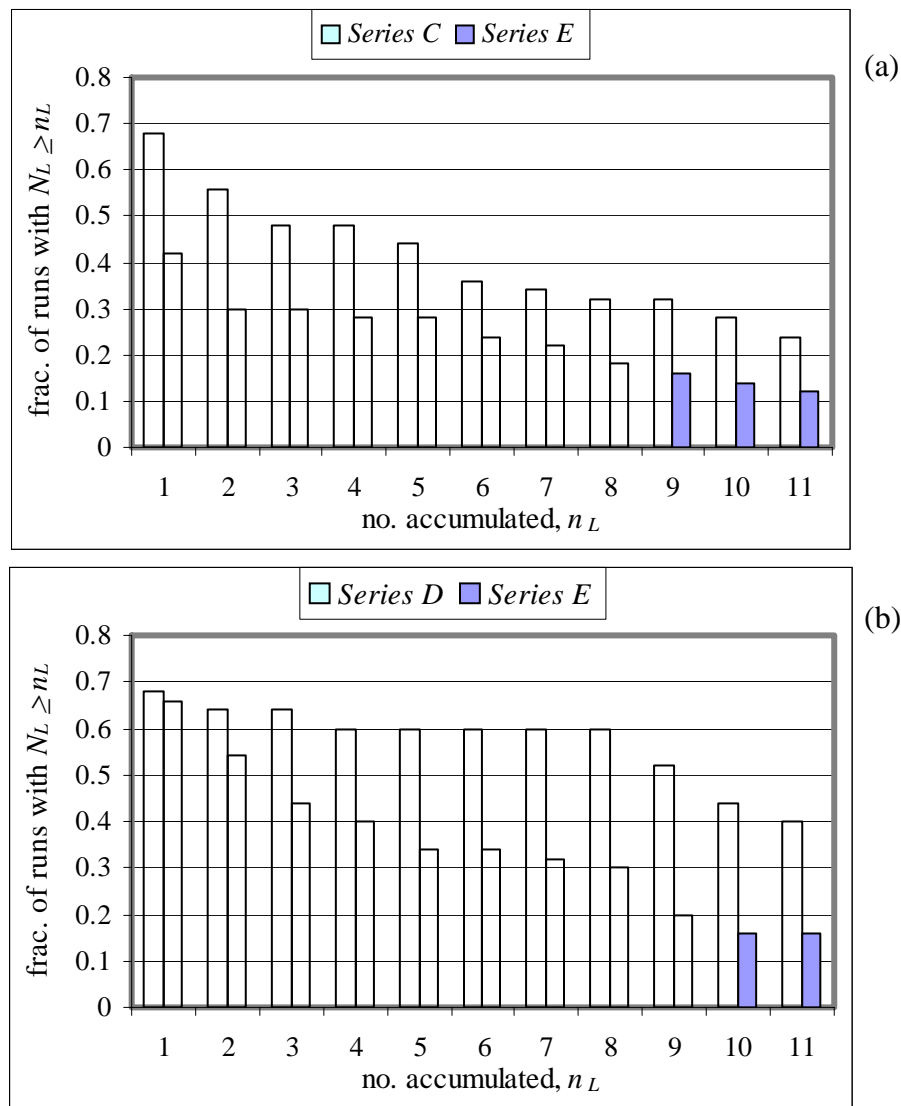


**Figure 3.10:** a) Plan and profile schematic representations of submerged groin-like structures as an alternative debris countermeasure, b) downstream view of groin-like structures in laboratory channel.

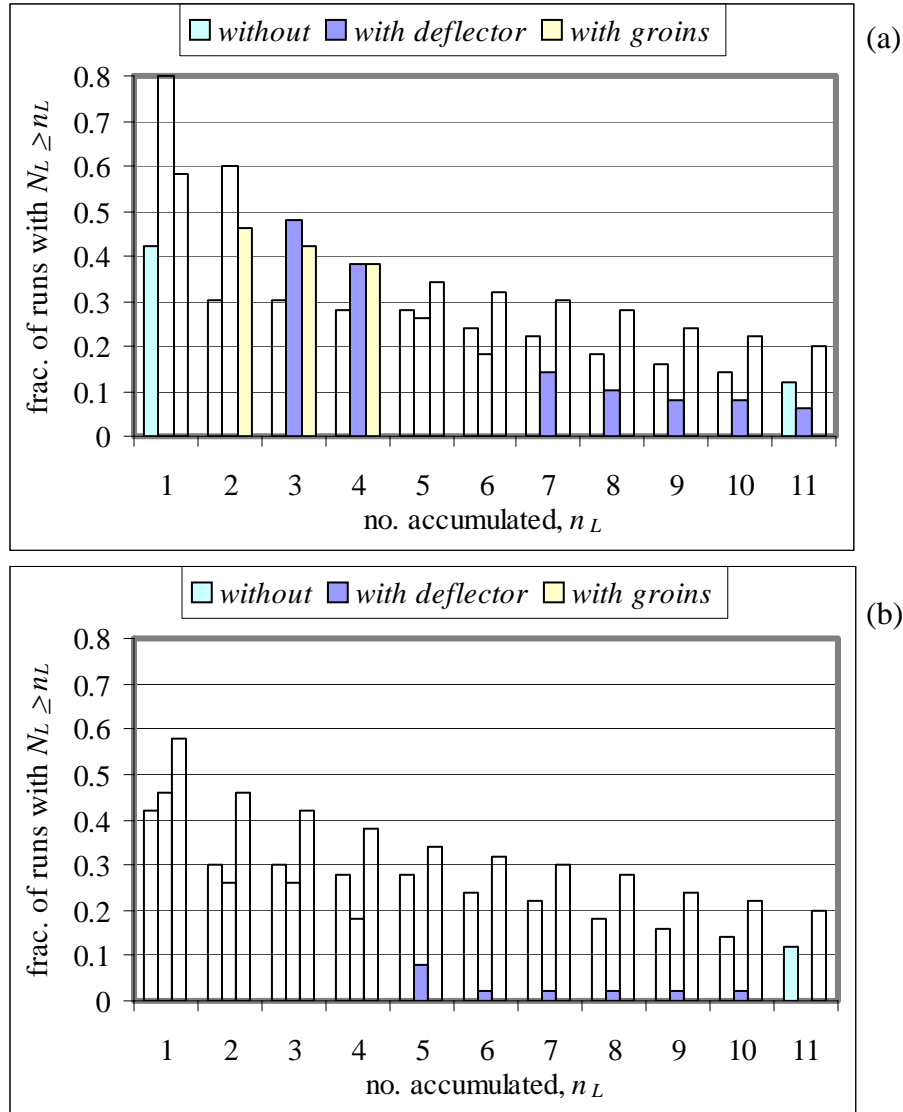
Before results involving deflectors are presented, results from series E are compared to those from earlier series with comparable experimental conditions. Fig. 3.11 shows two cases, one with and one without the model sandbar. Differences are evident, and may be indicative of the statistical variation that is associated with such results when experiments are performed by different individuals. For the case without the sandbar, the discharges were the same, but there was a slight difference in depth (the Series E flow was 1.3 cm deeper), while for the case with the sandbar, the number of runs in Series D was 25 compared to 50 in Series E. Qualitative trends tend to be preserved, however—in both cases, there is evidence that the presence of the sandbar is associated with greater likelihood of debris accumulation. For greater consistency, comparisons in the following will therefore be made only between Series E experiments since these were performed by the same individual.

Results for experiments without a model sandbar at a discharge of 0.32 cfs (this corresponds to an upstream velocity of 0.46 fps and an upstream Froude number,  $Fr = 0.11$ ) are shown in Fig. 3.12. For experiments with deflectors, a further problem arises in defining the number of logs accumulated in that a model log may be trapped either at a pier or at the deflector. The total number of logs accumulated, whether at the deflector or the pier, might be considered the most relevant statistic because logs trapped at either deflector or pier can develop into a debris pile and are therefore cause for concern. Nevertheless, it should be recognized that this total number may lead to quite different distribution than, for example, the number accumulated at the pier; Fig. 3.12 shows both. When the total number is used, a much larger likelihood of debris

accumulation is observed for small  $n_L$  because logs can be trapped at either deflector or pier. For large  $n_L$ , however, the presence of the deflector does tend to reduce the likelihood of debris accumulation. The results of Fig. 3.12a also indicate that the effectiveness of the alternative countermeasure is rather doubtful since the likelihood of debris accumulation is uniformly higher for all  $n_L$  compared to the reference case. Fig. 3.12b replots the results of Fig. 3.12a, except that, in the case with deflector installed, the number of logs accumulated at the pier (not the total number) is considered. In this representation, it is clearer that the presence of the deflector reduces the debris accumulation at the pier, though this may be due to accumulation at the deflector.



**Figure 3.11: Comparison of Series E experimental results with results from earlier series, a) case with no sandbar, b) case with sandbar (in both cases, discharge was 0.32 cfs, and upstream depth was  $\approx 16$  cm)**



**Fig. 3.12: Effect of countermeasures on likelihood of debris accumulation, a) considering total number of logs accumulated at both pier and deflector, b) the same as in (a), but considering only the number of logs accumulated at the pier.**

The case where a sandbar is present is examined in Fig. 3.13 at two different discharges (and hence two different approach velocities). Here, in the case where a deflector is present, the total number of logs accumulated is taken. At the higher discharge ( $Q = 0.5$  cfs,  $V = 22$  cm/s), the submerged groin structures appear to be quite effective, inhibiting to a large degree even small debris accumulations. At the lower discharge ( $Q = 0.32$  cfs,  $V = 14$  cm/s), however, the effectiveness is considerably reduced, consistent with the result already seen in the absence of a sandbar. The results would suggest that the presence of a deflector does reduce the likelihood of debris accumulation, though possibly not sufficient to prevent the development of large piles. For the lower-discharge case with deflector installed, a debris pile developed with over 30 logs at the deflector in one run, while in another run, a debris pile with over 20 logs accumulated *at the pier*.

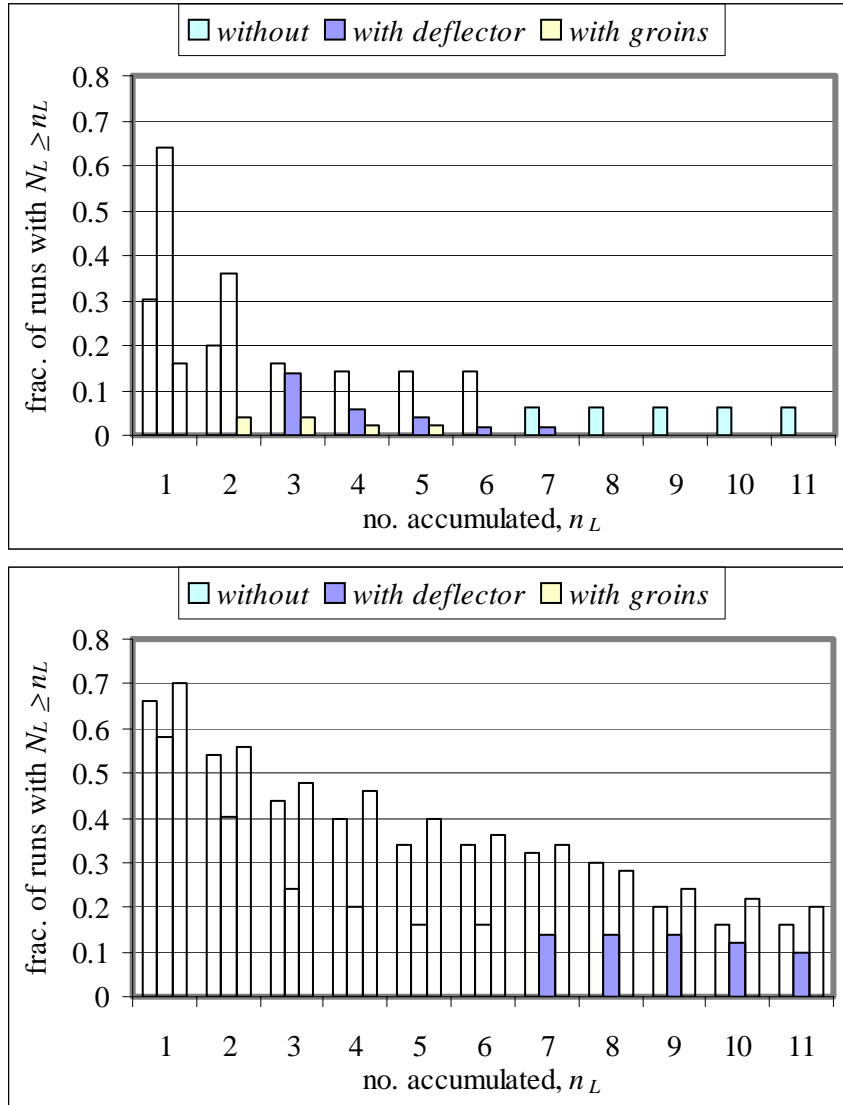


Fig. 3.13: Case when a model sandbar is present, a) higher discharge case, ( $Q = 0.5$  cfs,  $V = 22$  cm/s), b) lower discharge case ( $Q = 0.32$  cfs,  $V = 14$  cm/s)

### 3.6. Discussion and implications

#### The depth effect

While a strong effect of the approach velocity is intuitively plausible, the rather marked effect of depth is rather surprising. Precise mechanisms explaining the depth effect have not been determined with any degree of confidence. Several possible contributing factors may be mentioned. In many, though not all, of the cases with a locally shallow depth, particularly when the debris pile is quite large, it was observed that one or more of the model logs was actually resting on the channel bottom (recall Fig. 3.1c). This would tend to stabilize the pile in a manner that would not be available if the depth was larger than the length of the log. This suggests that the ratio of log length to local depth may be an important parameter for characterizing debris accumulation

potential. Secondary currents may also play a role in enhancing debris trapping, though the link to depth is rather obscure. It is known that horseshoe vortices arise in the flow around a bridge pier. These are strongest near the channel bottom, and are thought to be the cause of local scour around bridge piers. For shallows flows, the secondary currents generated by these vortices could possibly affect log movement. Preliminary experiments investigating this effect were inconclusive at best. It had originally been thought that, if secondary currents played any significant role in promoting debris accumulation, then the submerged groin structures would disrupt any such secondary currents. The somewhat disappointing results with these groin structures add to the suspicion that near-bed secondary currents are not a major factor in debris accumulation.

### **The effectiveness of countermeasures**

Two types of countermeasures were studied: a vertical cylinder similar to those installed in the field, and a submerged groin-like structure. To the extent that the probability of occurrence of a debris pile of any given size was reduced in the presence of the vertical cylinder deflector, it might be argued that this structure achieved some measure of success. Nevertheless, quite large accumulations were possible both at the deflector and the pier, and so its effectiveness in the field might be questionable, as in fact has been the field experience. Although the groin-like structures seemed effective at the higher discharges in preventing large accumulations, its effectiveness at lower discharges was disappointing. It may be that the dimensions of the system as tested were not optimal, and additional study might result in a more broadly applicable countermeasure.

### **Relationship between laboratory and field observations**

Because the design of experiments has been strongly influenced by observations and features of the Eel River site, the relationship of the laboratory study to the field is fairly clear. The apparent importance of depth may also be relevant to the interpretation of observations at the SR63-Vermilion River site. As will be discussed in the next chapter, this crossing occurs at a pronounced bend in the river, and there appears to be a ‘terrace’ or ‘shoulder’ or point bar on the inside of the bend. The bulk of the debris accumulation is observed (and in fact where the debris deflectors there have been installed) on this shoulder. This shoulder has a locally smaller depth than the thalweg which runs closer to the outside of the bend, and so it would appear that, at this site also, debris accumulation occurs where the depth is locally small.

### **Practical implications for bridge hydraulics and debris deflectors**

If substantiated, the depth effect observed in the laboratory has a direct implication for the choice of a bridge crossing site, and for the hydraulic/sedimentation design of the site. In particular, caution is advised in siting a crossing where channel geometry exhibits substantial areas in the main channel where locally ‘small’ depths may occur (always assuming that woody debris is known to be delivered to the site). Further, design practices that may aggravate non-uniform sediment deposition in the main channel leading again to locally shallow areas, should be questioned. This was apparently the case at the SR59 site where the channel was apparently deliberately widened presumably

to increase the bridge clear opening but likely accelerated the growth of the island bar, the presence of which may have aggravated the problem of debris accumulation.

## Summary

Laboratory experiments have been carried out to investigate under controlled but idealized conditions the accumulation of debris at bridge piers. With dowels and twigs as model logs, the effects of velocity and depth have been separately studied, and problems of scaling and similitude have been discussed. The expected strong effect of velocity (at constant depth) was observed, in which smaller velocities were more conducive to debris accumulation. More surprisingly, a strong effect of depth (either at constant velocity or at constant Froude number) was also noted. Smaller depths were associated with greater potential for debris accumulation. This was further studied by considering a model pier located atop a model sand bar, and the same tendency was found, even though the upstream conditions are identical, *and the velocity at the pier section was larger*. Experiments with vertical cylindrical debris deflectors showed that these did result in a small reduction in the likelihood of larger debris accumulation. As is found in the field, however, some large debris piles did develop even in the presence of debris deflectors. Submerged groin-like structures were also examined as a possible alternative counter-measure, but did not prove to be broadly applicable, and would require further refinement before they could be recommended for practical implementation.

## **4. Field study: design and implementation**

As noted in the literature review, a number of field studies related to large woody debris in rivers have been carried out. Invariably, these have consisted mainly of site visits before or after a debris pile has been established. Similarly, anecdotal evidence of debris movement in streams has been reported, but a systematic study of debris movement and accumulation during a large-flow event has so far been rare or not available. The present study undertook a field study with the aim of obtaining information that might provide a sounder empirical basis for developing guidelines for design and construction practice and possibly for the design of countermeasures. Because information was desired regarding the process of debris accumulation in the field, it was decided that continuous video monitoring and recording at a site would be the approach taken. This chapter discusses the main issues that needed to be addressed in such a field study, and describes the equipment used and its installation.

### **4.1. Site selection and details**

The limited budget for equipment implied that at most two sites could be adequately instrumented. With a view to selecting sites for the study, several sites in the Crawfordsville INDOT district were visited. These sites were selected from a list provided by the district (Larry Vaughn, personal communication) as having a history of problems with debris accumulation. During this single visit (in August 2000), most of these sites were found to have fairly minor accumulation, possibly due to prior clearing and the timing of the visit, i.e., before the first major flow events after the summer season. Two sites were chosen as candidates for monitoring, namely the SR59 south crossing of the Eel River and the SR63 southbound crossing of the Vermilion River. A major reason for this choice was the presence of a type of debris deflectors at both of these sites. This implied, on the one hand, that the debris accumulation problems at these sites were sufficiently severe as to cause the district to undertake significant countermeasures. On the other hand, installing the equipment at these sites would permit a better assessment of the effect of these deflectors, and possibly stimulate ideas for their improvement.

#### **SR59 south crossing of the Eel River (structure 59-11-6778)**

As was hinted at in the discussion of the laboratory study, much of the attention in this project centered on the Eel River site, since it seemed to present a simpler situation for initial study. An aerial photograph of the Eel River site has already been given in Fig. 1.1a. At this site, the current bridge is a two-lane highway, constructed circa 1985, replacing an old 2-span steel-truss bridge, originally located just upstream of the current bridge. The spur leading to the old bridge can still be seen in the aerial photograph (Fig. 1.1a). Whereas the old bridge had span lengths of 150 ft and 197 ft, the new bridge is a 5-span continuous pre-stressed concrete I-beam bridge with each span having a length of 66 ft. Anecdotal evidence suggests that debris accumulation was not a significant problem before the construction of the new bridge. This was most likely due to the much longer span of the old bridge, but, as will be argued, other factors may also have contributed.

The reach where the crossing is situated is relatively straight. The aerial photograph shows an upstream channel that is heavily wooded on both banks, though the woods do not extend far away from the banks. In the database of Indiana bridge crossings compiled by Hopkins and Robinson (1997), the percentage of channel bank covered by woody vegetation is given for this site as less than 25%. This however applies only to the region within two bridge lengths upstream and downstream of the bridge. Even farther upstream than is shown in Fig. 1.1, the woods do thin noticeably, and it is not clear to what extent the debris accumulated at the site is generated locally or much farther upstream.

The upstream channel varies somewhat in width (120 ft to 160 ft, estimated from the aerial photograph), but, as the bridge crossing is approached, it widens significantly to over 300 ft. In the Hopkins and Robinson database, the upstream channel width,  $B_{up}$ , is given as 160 ft, while the width at the bridge,  $B_{br}$ , is given as 300 ft. Here again, Hopkins and Robinson defined the upstream section as one is that is approximately two bridge lengths upstream of the bridge section, and so this section may not necessarily be representative of the stream farther upstream.  $B_{up}$  is of some significance since it figures in the design log length,  $L_d$ , of Diehl (1997), which can be estimated as

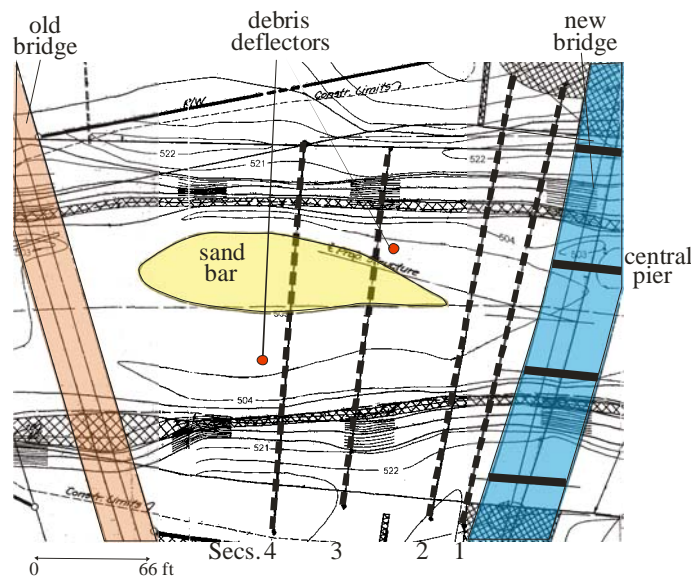
$$L_d = 9 + (B_{up} / 4)$$

with both  $L_d$  and  $B_{up}$  in meters. Thus, if the smallest width,  $\approx 120$  ft, is chosen for  $B_{up}$ , since the smallest width acts to trap the largest logs, then  $L_d$  is evaluated to be  $\approx 60$  ft, whereas if the largest width, namely 160 ft, is taken, then  $L_d$  is estimated to be  $\approx 70$  ft. The pier-to-pier span length (66 ft) would exceed  $L_d$  based on the former estimate, thereby satisfying the design criterion based on  $L_d$ , but would not satisfy the criterion if the latter estimate of  $L_d$  is used.

The significant expansion as the bridge is approached requires further discussion. The photograph of the site cleared of debris (Fig. 1.2a) shows clearly a sand bar in midstream, which, from the results of the laboratory study, may play an important role in debris accumulation. Fig. 4.1 shows the site bridge plan prepared for the bridge construction, and a nascent sand bar can be seen, indicating that the beginning of a sand bar already existed before the bridge construction. Apparently, because the expansion results in lower velocities, and hence lower sediment-transporting capacities, sediment deposition has occurred, such that the nascent sand bar has grown substantially, at the same time, possibly also being slightly displaced downstream.

Fig. 4.2 shows a series of sections that were surveyed in August 2002 for the present project—the locations of these sections are shown in Fig. 4.1, with the distance being measured from the left looking downstream. The lateral distances are not measured from a common reference, but the approximate location of the ‘central pier’ is given for reference in each cross-section. Further, the low chord of the bridge is at 529.3 ft above mean sea level (M.S.L.), while the stage of a typical debris-transporting flow would be  $\approx 521$  ft M.S.L. (comparable to 521.7 ft which is given as the average high water level on the bridge plan).

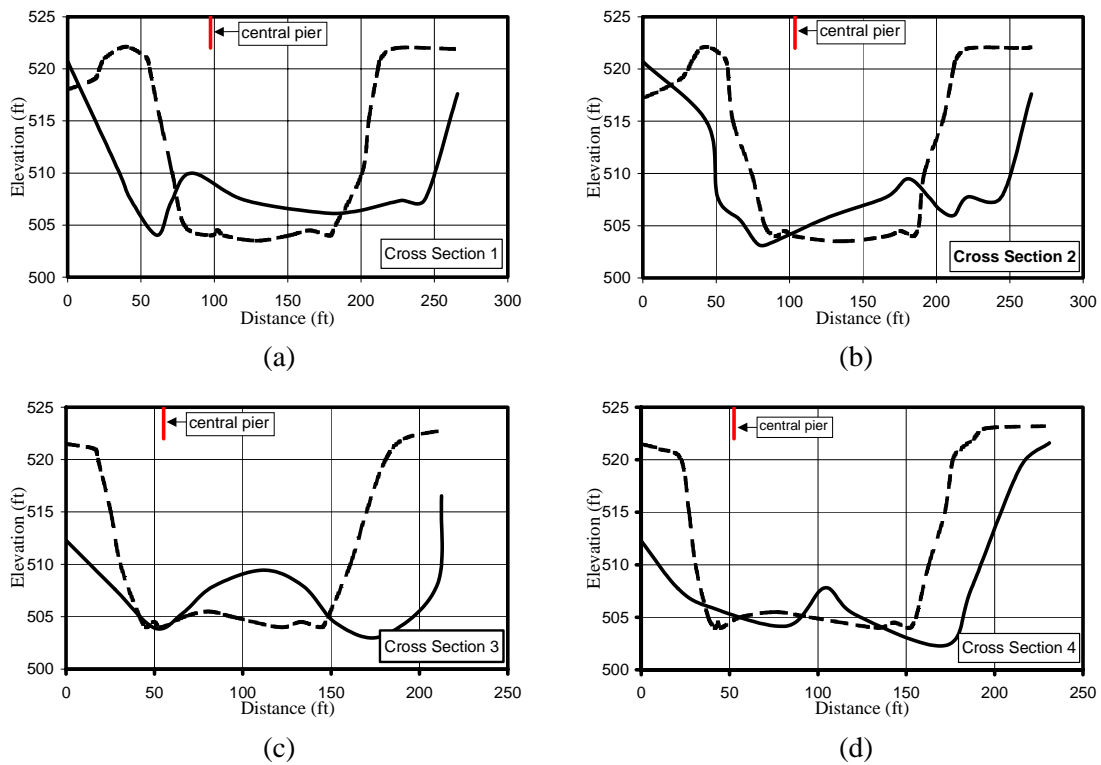
The surveyed cross-sections (Cross-section 1 is closest to the bridge) are compared with the corresponding cross-sections taken from the design bridge plan, dated 1985. Two differences between the 1985 bridge-plan and the current (2002) cross-sections are striking: i) the width of the channel, and ii) the growth of the sand bar. The first indicates that, if there had been an expansion before the bridge construction, it was likely to have been much less pronounced than is the current expansion. The widening seems to have been a deliberate design choice to increase the bridge clear opening and therefore to be able to pass a larger discharge at a lower water surface elevation, since instructions on the plans are given to clear an area substantially wider than the original channel. The second shows a marked growth, in excess of 4 ft, in the highest elevation of the sand bar, and also a displacement of the highest points somewhat to the left (looking downstream) and downstream. Lateral as well as longitudinal (in the streamwise direction) growth may have also occurred, but a definite conclusion is more difficult to arrive at, due to the possible displacement. The sediment deposition that caused this sand bar growth is, as was noted above, mainly attributed to the lower velocities resulting from the expansion, but it may also have been enhanced by the debris accumulation. In this way, sedimentation and debris accumulation may be self-reinforcing processes.



**Figure 4.1: (1985 design) Bridge plan, showing topography of stream at that time, nascent sand bar, old and new bridges, debris deflectors, and sections surveyed in 2002 (heavy dashed lines)**

The two debris deflectors, also seen in Fig. 4.1 (also in Fig. 1.1), are cylindrical (14-inch diameter) concrete-filled steel piles, axis vertical, driven into the stream bed. Each is placed approximately aligned with a pier, one  $\approx 90$  ft upstream of one pier (the central pier in Fig. 4.1) and one  $\approx 150$  ft upstream of an adjacent pier. The origin of the design of these deflectors is not clearly documented, though some version seems to have been used in Alaska (Merril Dougherty, INDOT Hydraulic Engineer, personal communication,), and may be related to the similar debris deflectors at the Chena River Flood Control Intake Structure in Alaska shown in Chang and Shen (1979).

The USGS stream gaging station on the Eel River nearest to the site is located near Bowling Green (station 03360000), which is over 20 (river) miles upstream from the site. The drainage area for the Bowling Green station is 830 sq. miles (USGS web site). In the Engineer's report filed with the Dept. of Natural Resources for the construction of the bridge, a drainage area of 1060 sq. miles was estimated for the site. Discharge at the site will therefore be somewhat higher than that measured at the Bowling Green station. The 100-year discharge at the site was estimated to be 46,000 cfs for the purposes of the floodway analysis (Engineer's report, 1985). The mean and median annual peak flow (based on data from 1933-2000) are respectively 13669 cfs and 12800 cfs. Because debris accumulation occurs frequently, the return period for a debris-transporting flow is likely less than one year, and as will be seen in Chapter 5, discharges as low as 4000 cfs may transport significant debris.



**Figure 4.2: Comparison of cross-sections surveyed in 2002 (solid line) and those obtained from the 1985 design bridge plan (dashed line), a) Sec. 1, b) Sec. 2, c) Sec. 3, d) Sec. 4.**

### **SR63 southbound crossing of the Vermilion River (structure 63-83-1496 BSBL)**

The other field site, SR63 southbound crossing of the Vermilion River, has received somewhat less attention than the Eel River site, both because the salient site features are less directly related to the laboratory study and because the field installation was completed at a later date (in April 2003 compared to December 2001). An aerial photograph of this site has already been given in Fig. 1.1b. The two-lane southbound crossing is an older structure, originally built in 1957, but was widened at about the same time that

the parallel northbound crossing was built, circa 1967. It is an 8-span continuous steel beam structure, with spans of 32 ft, 57.5 ft, 4@69 ft, 57.5 ft, and 32 ft.

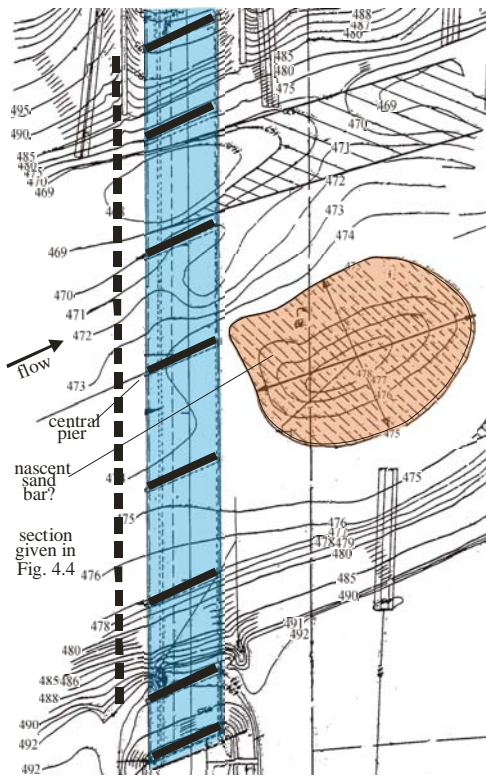
A prominent feature of this site, and one of the reasons for the interest in it, is the noticeable bend. The southbound crossing (on the left in Fig. 1.1b) is at the beginning of the bend. The radius of curvature of the bend has been estimated from the aerial photograph to vary from a minimum of  $\approx 1000$  ft to  $\approx 1500$  ft at the beginning of the bend. There is also an expansion, less dramatic than that previously seen at the Eel River site, in the immediate vicinity of the bridge crossing. At the bridge, the width is estimated to be  $\approx 250$  ft, whereas the upstream width may vary from  $\approx 160$  ft to 200 ft. For comparison, the Hopkins and Robinson database lists the upstream width as 220 ft and the bridge width as 250 ft. Even if the smallest width is assumed, the Diehl criterion for a design log length is either marginally or not all satisfied by the 69 ft span, and certainly not by the 59.5 ft span (which however does not lie in the thalweg of the channel).

Fig. 4.3, adapted from the 1968 bridge plan for the new bridge, shows the stream topography in the immediate vicinity of the southbound bridge. Interestingly, a bar-like feature is found, though, *downstream* of the southbound bridge, along with specific instructions that this should be cleared to elev. 475 ft (similar instructions were also given to clear the thalweg region just downstream of the bridge to elev. 469 ft, hatched in Fig. 4.3). A cross-section just upstream of the southbound bridge (the location is shown in Fig. 4.3 as a heavy dashed line) based on the 1968 design plan is shown in Fig. 4.4a (as before, this is plotted looking downstream). It is fairly typical of cross-sections at a river bend, with the thalweg region towards the outer part of the bend (left looking downstream). A quite steep outer bank may be indicative of some stream instability. A feature of particular interest is a very mild ‘shoulder’ or terrace beginning at  $\approx 175$  ft in Fig. 4.4a, which is  $\approx 5$  ft higher than the channel thalweg elevation. This is the region, towards the *inner* part of the bend, where debris accumulation predominantly occurs. As noted in the literature review, laboratory and field observations indicate that, in the absence of bridges and piers, debris deposition and hence accumulation tend to occur towards the *outer* part of a bend. On the other hand, the observations at the Vermilion R. site are consistent with the hypothesis suggested by the present laboratory study that a smaller local depth increases the potential for debris accumulation. Figs. 4.4b,c are upstream cross-sections based on flood analyses (Flood Insurance Study, Cayuga, 1980), performed in 1979, i.e.,  $\approx 10$  years after the bridge construction. Secs. 1 and 2 are  $\approx 150$  ft and 2200 ft upstream of the southbound bridge. The beginning of a shoulder feature might possibly be seen at a lateral coordinate of  $\approx 2250$  ft (note that this coordinate system is not the same as that used in Fig. 4.4a) in Sec. 1, closer to the bridge and the bend. Well away from the bridge and the bend, Sec. 2 (Fig. 4.4c) exhibits little if any sign of a shoulder feature. It may be relevant that the Vermilion R. also goes through another bend, similar to that at the SR63 crossing, about 4000 ft upstream of the SR63 crossing.

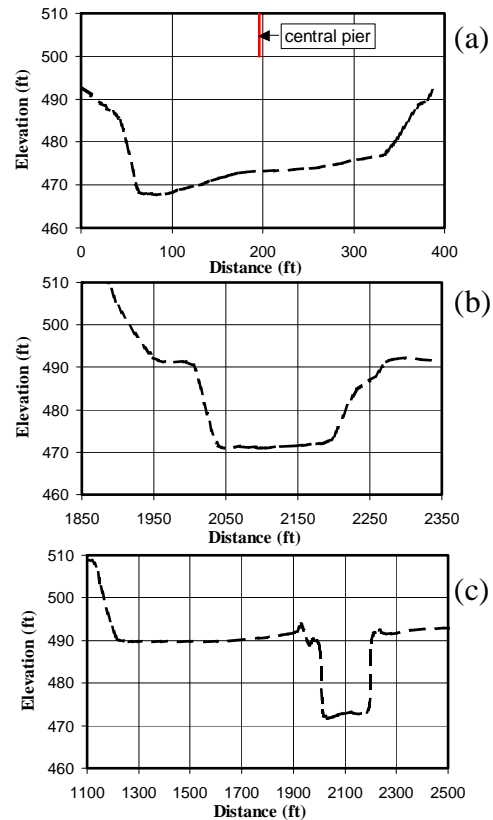
More recent bathymetric information is available from underwater bridge inspection studies carried out in 1994 and 2001 (Fig. 4.5, adapted from Collins Engineers, 1994, 2001). The thalweg is found close to the northern (left looking downstream) outer bank, while an island feature is developing towards the opposite inner bank. As was the case at the Eel River site, substantial sediment deposition seems to have also occurred in this reach even during the seven years between 1994 and 2001. Fig. 4.6 shows the bed

elevations at three upstream sections obtained during bridge inspection studies in those years. This prompts the speculation of a positive correlation between aggrading reaches and the potential for debris accumulation.

Flood levels were estimated in a flood study (Flood Insurance Study, Cayuga, 1980) for the community of Cayuga, which is  $\approx 1$  mile upstream of the SR63 crossing. The 10-yr flood ( $Q_{10} = 29,000$  cfs) stage is predicted to reach an elevation of  $\approx 493.4$  ft, while the predicted 100-yr flood ( $Q_{10} = 57,000$  cfs) stage is at an elevation of 500.4 ft. As is also the case for the Eel River site, return periods for debris-transporting flows are likely less than one year for this site, and so the stages of interest will be less.

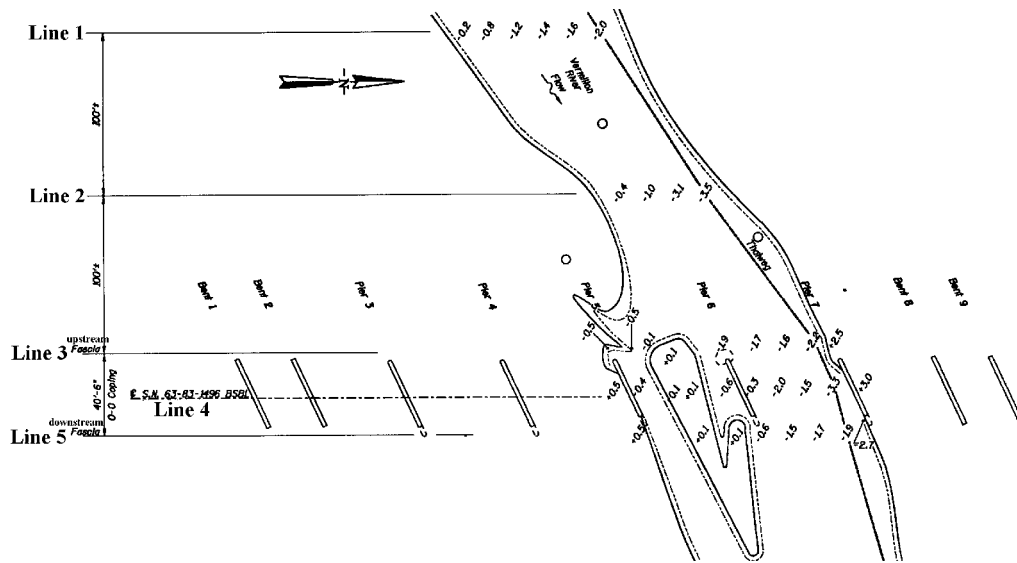


**Fig. 4.3: Channel geometry in the vicinity of the SR63 southbound crossing of the Big Vermilion River, taken from the bridge plans developed for the construction of new (northbound) bridge**



**Fig. 4.4: Cross-sections upstream of the SR63 southbound crossing of the Big Vermilion River, a) section shown in Fig. 4.3 taken from bridge plans, b) and c) sections at 150 ft and 2200 ft upstream, taken from flood study**

Three debris deflectors, identical in form to those at Eel River, are also installed in a similar geometric arrangement, all in the ‘shoulder’ region of the cross-sections, at the Vermilion R. site. As in the case at the Eel River site, the performance of the debris deflectors as deflectors is doubtful in view of the chronic debris accumulation.



**Figure 4.5: Bathymetric data for the Vermilion River at the SR63 southbound bridge obtained during underwater bridge inspection study (adapted from Collins Engineers, 2001)**

## **4.2. The field equipment**

The field study was aimed at obtaining information regarding the movement and accumulation of debris at bridge piers during high-flow events. Both the random nature of the flow events and the distance of the sites from Purdue University (more than one hour by car) motivated an approach in which the sites would be remotely monitored using video cameras. Considerations in the choice of camera systems included

- low-light capabilities in order to have as much useable time as possible, i.e., permitting use in the late evening and early morning, and major flow events are often associated with cloudy low-light conditions, together with auto-iris so as to be able to adapt to highly variable light conditions,
- ruggedness to withstand possibly severe outdoor environment, and
- remotely controlled pan/tilt/zoom capabilities for cameras mounted on bridge piers.

Video images were to be recorded so as to permit viewing and analysis at any arbitrary subsequent time. The two available technologies considered were standard (or even time-lapse) video cassette recording, or digital video recording in which the video images are directly converted into digital format before being written onto a hard disk. Although the former was somewhat less costly, the latter was favored because of its flexibility of operation, and the convenience of the digital format for later analysis. The choice of a digital recorder with finite storage capacity, typically > 30 GB, left open the question of archiving the stored images. Due to the large size of the video image files, remote transfer was not feasible, either because it was too slow over phone lines, or too costly over a dedicated line. Digital audio tape (DAT) archiving, which does require periodic site visits, was chosen as the most cost-effective means of mass storage. Remote access over a phone line was however desired for spot checks to verify correct operation of the sys-

tem, and to prepare for tape archiving. This was accomplished through the commercial remote access software, pcAnywhere. An important ancillary ‘equipment’ is a climate-controlled fiberglass enclosure to house temperature- and humidity-sensitive data acquisition electronics and computer.

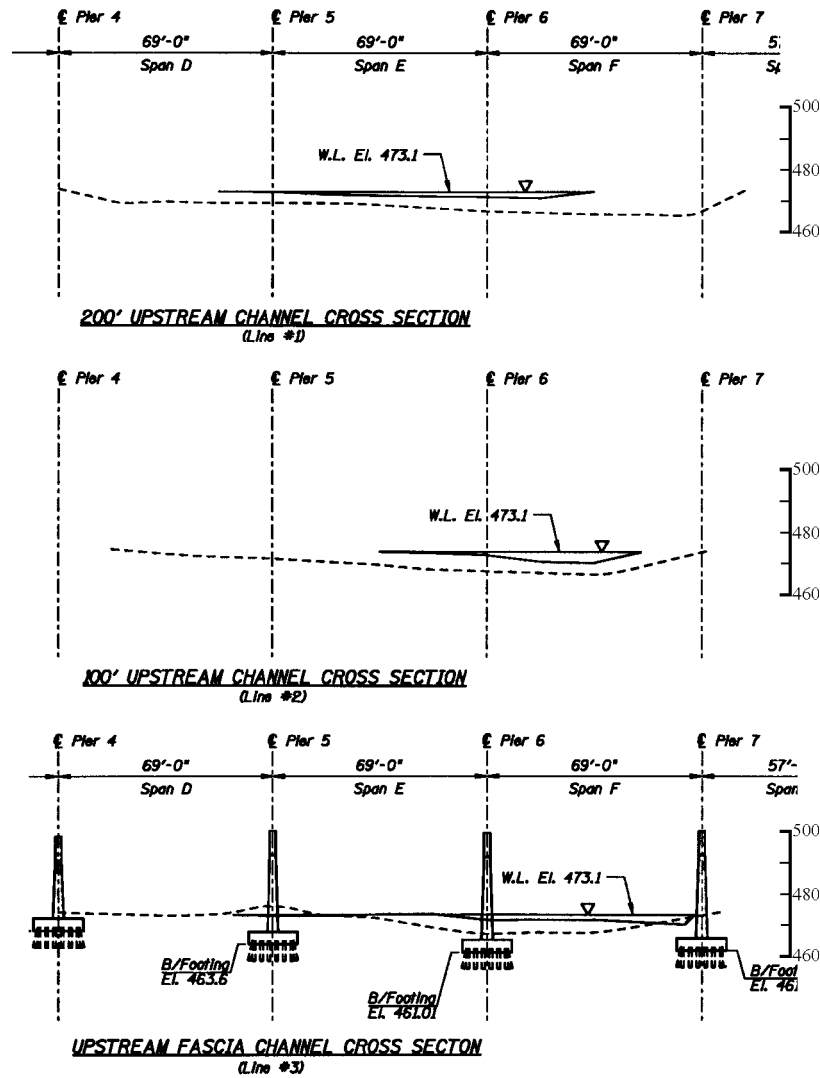


Figure 4.6: Bed elevations at three upstream sections at the SR63 Vermilion River site (adapted from Collins Engineers, 2001)

### The SR59 Eel River site details

The Eel River site was instrumented first for various reasons, including its more dramatic debris accumulation, its simpler channel geometry more directly related to the laboratory study, and a greater ease in setting up the instrumentation because of specific site layout. Two cameras were used, one installed on a bridge pier with a view upstream, and the second, installed on the bank atop the fiberglass enclosure, from the side, almost

at right angles to the bridge camera, with a view of the bridge and the piers (Fig. 4.7). The cameras were Wattec (Model WAT-902HS) with black and white 1/2" CCD, specially chosen for its high sensitivity (according to specifications, with a minimum required illumination of 0.00015 lux), fitted with Tokina 6-15 mm auto iris lens with motorized zoom and focusing. The cameras were housed in a Pelco outdoor camera housing with mount. The camera on the bridge pier, because it was not easily accessible once it was mounted, also had a pan/tilt/zoom mechanism that could be controlled from a unit within the instru



(a)



(b)



(c)

**Figure 4.7: SR59 site, a) camera (circled) mounted on top of fiberglass enclosure located on right bank, b) camera mounted on bridge pier, c) plan view indicating location and view of cameras (arrows) and location of debris deflectors (solid circles)**

ment enclosure. The digital recorder was obtained from Integral Technologies (Model DVX4000 D30), and included a dedicated Pentium 3 level data acquisition computer with a 30 GB hard drive, an internal modem and watchdog timer. The DAT3 tape drive

(Seagate Scorpion 24) for archiving data was acquired separately. The instrument enclosure, a PlasticFab (Model 3A) corrosion-resistant fully insulated shelter with integral floor, flat roof and NEMA panel and light switch, is equipped with a 500-W heater with thermostat, to which was added an air-conditioning unit. Because security was of some concern, the enclosure also had a stainless-steel padlockable handle. A stage recorder (to determine the water surface elevation at the bridge) was fashioned from an ultrasonic sensor (Greyline Instruments, Model LIT25) and accompanying electronics, together with a battery-powered data logger (Hobo H08).

## **The Vermilion River site details**

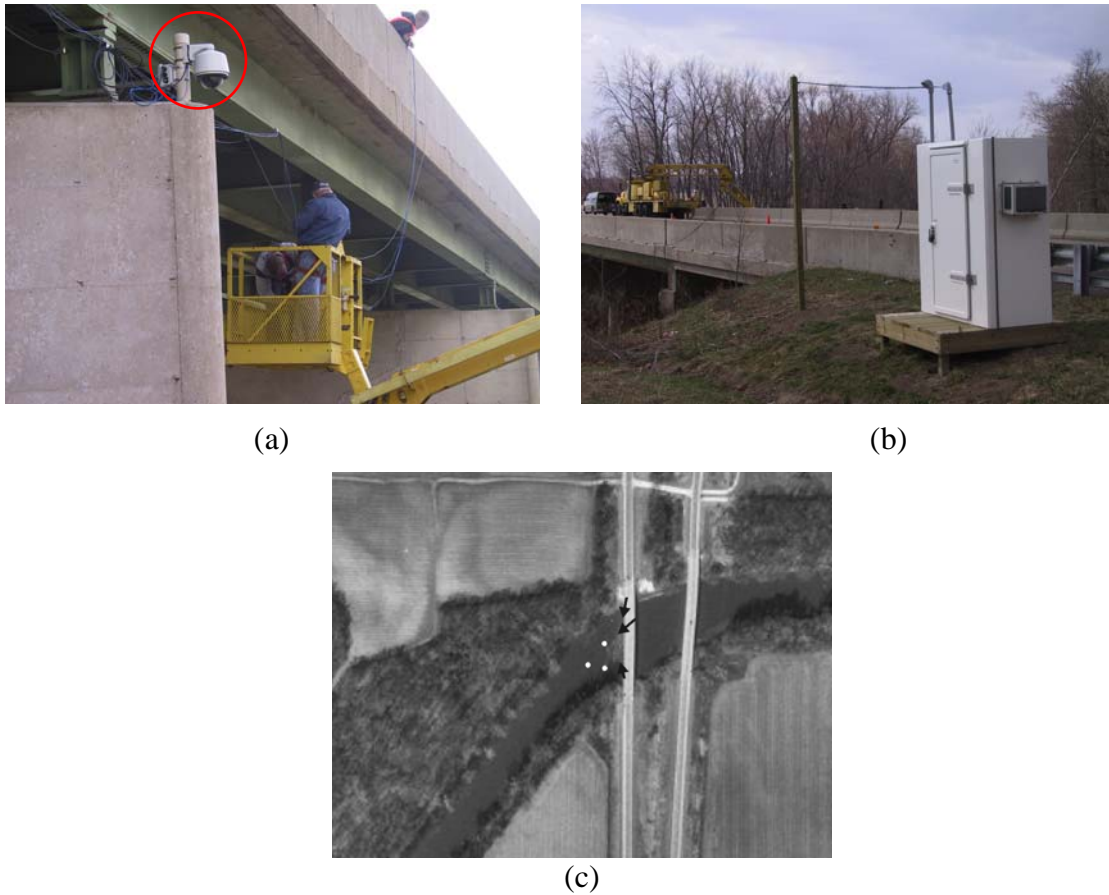
The approach to instrumenting the Vermilion River site was basically the same as was done for the Eel River site. With however the experience gained from the Eel River site, as well as the different physical characteristics of the southbound bridge over the Vermilion River, a number of changes were made. The SR63 southbound bridge is longer than the SR59 bridge (455 ft to 334 ft) with 8 spans rather than 5, and is somewhat higher. As noted before, the two-camera setup at the Eel River site had the disadvantage that the view of one side of the channel from a camera on one bank could be blocked by accumulated debris. The major change in the equipment setup from the previous setup was therefore that a three-camera arrangement was preferred. Due to the length of the bridge, it was also decided that all three cameras would be mounted on piers. The experience with the Watec cameras was not entirely happy, and so Pelco Spectra III dome cameras (Model SD53TC, with a workable light sensitivity of 0.05 lux) with integrated auto-iris mechanism were chosen. The video recorder was also upgraded (Integral Technology DVX XP440 D), and included a 40 GB (rather than a 30 GB) hard drive.

One of the cameras, during the equipment installation, is shown mounted atop a bridge pier in Fig. 4.8a. Because all cameras were pier-mounted, there was greater flexibility in siting the instrument enclosure, and it was found more convenient to locate it on the downstream side of the southbound bridge (Fig. 4.8b), closer to utilities. The approximate location of the cameras, their viewing angles, and the debris deflectors are shown in Fig. 4.8c. A stage recorder was not installed at this site.

## **Notes on installation and operation of equipment**

Due to limited resources in personnel and equipment, the installation was typically accomplished in steps. From information derived from site inspections, the placement of the cameras and instrument enclosure was decided. This took into consideration not only camera angles and the flow and channel features of interest, but also ease of access to telephone and power utilities. A wooden deck had to be constructed for the one-piece fiberglass instrument enclosure, which was installed with the help of INDOT, who provided the transport and the use of a bucket truck for putting it in place. The cameras were then mounted. The cameras on the bridge piers were mounted with the aid of INDOT through the use of the REACHALL truck and personnel. Cables connecting the cameras (and possibly other equipment, such as the stage recorder) to the associated electronics and computer were then routed. Some thought and care should be given to supporting or fastening the cables, e.g., higher-level power cables should be separated

from lower-level signal/video cables. Voltage drops and possible signal losses in long cable runs may need to be considered. Security against vandalism should also be considered not only in running cables, but also in the placement of equipment in general.



**Figure 4.8: SR63 site, a) one of the dome cameras (circled) mounted on a pier at the SR63 site, b) instrument enclosure on the other (downstream) side of the southbound bridge, c) approximate location of debris deflectors (solid circles) and angle of view of pier-mounted cameras (arrows)**

The SR59 cameras have been in combined operation since 11/01 (the bank-located camera, installed first, has been operating since 9/01), while the SR63 installation was completed in 4/03. The practical experience reported here has therefore been gained with the SR59 setup. As will be seen later, initial video images recorded by the bank-mounted camera were quite grainy, and may be due to a sub-optimal choice of settings. Raindrops on the window in front of the camera lens also presented problems for obtaining clear images. The Watec cameras coupled with the Tokina lens and auto-iris proved to be possibly too sensitive. A minor problem related to a camera housing without this window is however caused by birds who perched in front of the camera lens, and even apparently have made a nest nearby. Some problems have been encountered with electrical power and telephone access, the precise cause of which has not been entirely determined.

One of the prime advantages of a digital system of recording video images is the flexibility afforded by remote computer control. The DVX system software permitted remote (by telephone) setting of such operational parameters as the degree of image compression performed, the timing of video recording, and the frame rate. Even remote backup could be performed through the software although this capability was not exploited since it was deemed too costly compared to the alternative approach to be described below. Various strategies, i.e., combinations of image compression, recording timing, and frame rates, both static as well as dynamic, i.e., changing with conditions, were considered and tested in the initial phase of the project. The objective was to obtain the highest quality recorded images that would provide the longest recording time within the constraint of fixed mass storage size, e.g., 30 GB.

Various levels of compression were available from low to extra high. For still images, the differences between the various levels were difficult to detect visually; for moving images, differences were more evident, but even at a medium setting, the resulting images were difficult to distinguish from those obtained at an extra-high setting. Thus, it was eventually decided to fix the degree of compression at the high setting for all subsequent recording at the SR59 setting. Because of the additional camera at the SR63 site, which increases the amount of stored information, a reduced (medium) setting has so far been chosen. Similarly, recording could be done continuously, during any arbitrary specified time interval, e.g., from 6:00 am to 7:00 pm, or could be automatically triggered by an external event, an 'alarm', such as a measured water surface elevation signal provided by the stage recorder. The stage recorder was not found to be always reliable. In the end, the simplest strategy of a fixed frame rate (1 frame every 10 seconds) using a specified image compression quality during a specified period of time during the day (5:00 am-8:00 pm at the SR59 site, and 5:00 am to 10:00 pm for the SR63 site) was eventually chosen, but may be changed in the future.

The above choices of settings permitted recording for a period of  $\approx 1$  month, after which a tape archiving operation was performed before old images were overwritten, and then the tape(s) were retrieved manually. Although an integrated tape drive unit could have been acquired, an add-on tape drive was preferred for reasons of cost and compatibility with available equipment. This however required telephone access to the computer operating system, which in any case was necessary for controlling, and storing data from, the stage recorder. Because this was not allowed by the DVX recording software, this was resolved by using the commercial remote-access software, pcAnywhere (Symantec).

## **Summary on video monitoring installation**

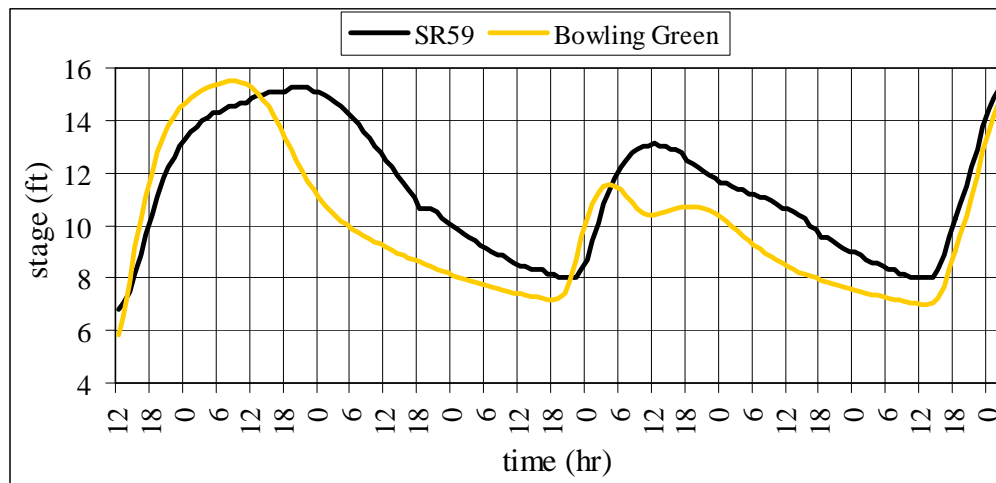
In general, the installation at SR59 has proved successful to the extent that reasonably clean video images of flow events transporting debris are being reliably recorded, and it is expected that the SR63 installation will prove in this regard equally or more successful. Unfortunately, as will be seen in the next chapter, interesting events frequently occur in the late night or very early morning, when video monitoring is not possible, with the result that it was not always possible to monitor continuously changes in scenery.

## 5. Field study: observations

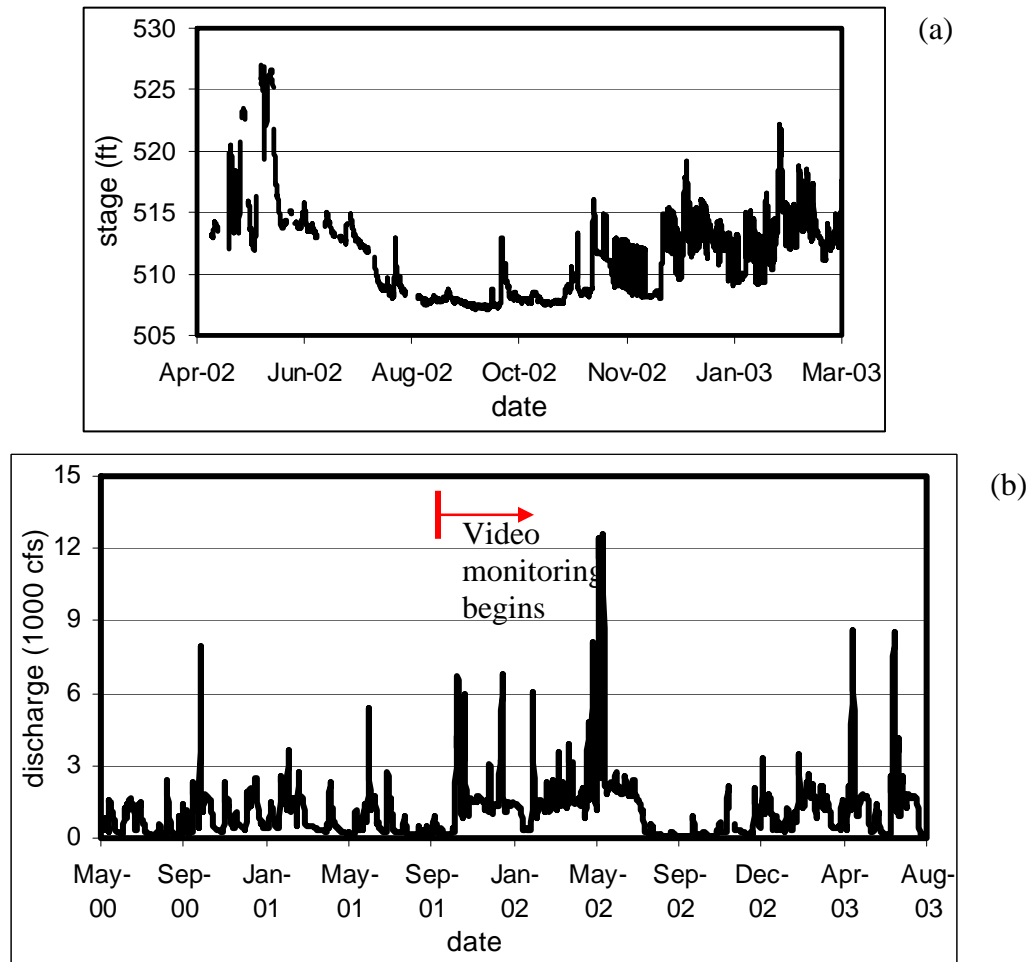
The results of the field study consist mainly of video images recorded at the sites. The SR63 site has only recently (4/03) begun operation, and so far images from that site have not yet proved instructive, in part because some debris had already accumulated prior to the start of operation of the monitoring program. This chapter will therefore focus on the results from the SR59 site.

### 5.1. Flow events at the SR59 site

As noted in the previous chapter, the nearest USGS stream gaging site is located at Bowling Green, over 20 river miles upstream of the site. The stage recorder at the site, in continuous operation since 4/02 (it was installed earlier but needed further adjustment), can on occasion operate erratically (as will be seen shortly), and so the Bowling Green data will be used where necessary as an alternative consistent surrogate for the purposes of identifying major flow events that might be transporting large woody debris through the research site. Fig. 5.1 compares the stage measured at the site by the stage recorder with the stage at the USGS Bowling Green station (elevation datums are different, so the stages are not absolutely comparable). The stage at the SR59 site is generally consistent with that reported at Bowling Green, with there being a small difference in phase, the peak in the SR59 site occurring  $\approx 6$  hours after the peak at the Bowling Green site.



the bridge low concrete is at 529.3 ft and the average high-water elevation is 521.7 ft (bridge plan, file: 59-11-6778). The discharge reported at the USGS Bowling Green station is shown in Fig. 5.2b, and, unlike the stage record, covers the entire project period. Video monitoring began with one camera on 9/12/01, and with both cameras on 11/20/01. A large flow event is indeed recorded at Bowling Green in May 02, which is consistent with the stage measured at the SR59 site; on the other hand, while there is discharge peak occurring in Feb. 03, it is not particularly large, and so the peak in stage at that time is likely overstated in Fig. 5.2a.



**Figure 5.2:** a) stage recorded at SR59 site, b) discharge at Bowling Green USGS stream gaging site, over the course of the project period

## 5.2. Overview of flow events and debris accumulation

An overview of flow and other relevant events and the effects on debris accumulation over the entire course of the debris-transporting season is first given. Before the installation of the video cameras, during the first visit to the site (8/17/2000), there was some amount of debris already at the site (Fig. 5.3), some at the deflector closer to the bridge, the bulk however at the bridge piers. Fig. 5.2b suggests that there were only relatively minor flow events from May to August 2000, and so it is not clear whether the observed debris accumulation were associated with these events or were perhaps the resi-

dual from before this period. On the subsequent site visit, on 10/26/2000, a dramatic increase in accumulation is evident (Fig. 5.4), both upstream of the deflectors as well as at the bridge. In the intervening time between the two visits, one large event (discharge,  $Q \gg 8000$  cfs, where discharge will always refer to that reported at the Bowling Green station, with a water surface elevation at the SR59 site estimated to be  $Z \gg 525$  ft) can be identified at the Bowling Green station about two weeks before the second site visit. Two relatively minor events ( $Q \gg 2000$  cfs, with estimated  $Z \gg 515$  ft at the SR59) did also occur in that period, but the very substantial increase in the observed debris accumulation is most likely attributable to the *single* larger flow event. It is somewhat debatable whether the presence of already existing accumulation might have caused or at least exacerbated the ultimate accumulation.



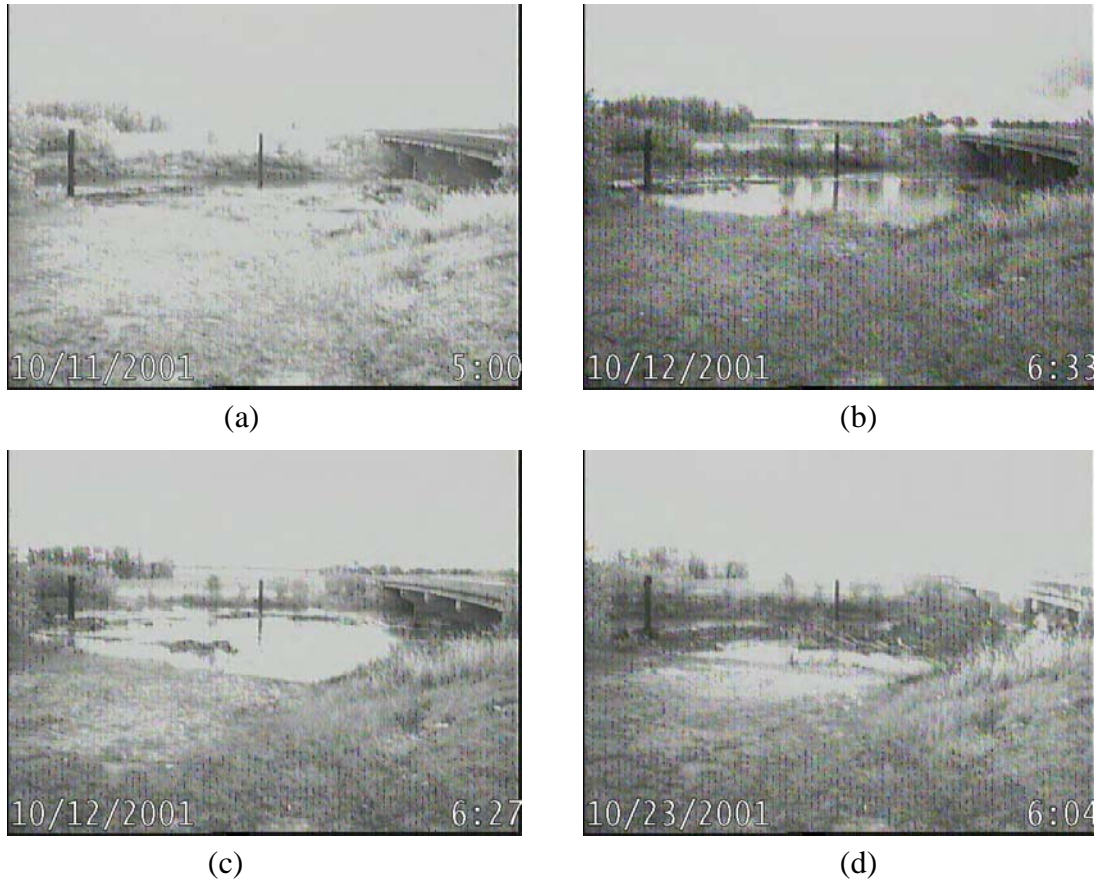
**Figure 5.3: Site photographs from Aug. 17, 2000: a) view upstream of debris deflectors, b) view of piers on the left side (looking downstream) of river**



**Figure 5.4: Site photographs from Oct. 26, 2000: a) view upstream of deflectors (same as Fig. 1.2a), b) view of piers on left side (looking downstream) of river**

The site was cleared of debris in April-May 2001. Between this time and the beginning of the video monitoring in September 2001, one significant event ( $Q \gg 5000$  cfs on 6/7/01) and two minor events ( $Q \gg 2000$  cfs, the later on 7/4/01) occurred, but no significant accumulation was observed at the SR59 site. This permits of two interprê-

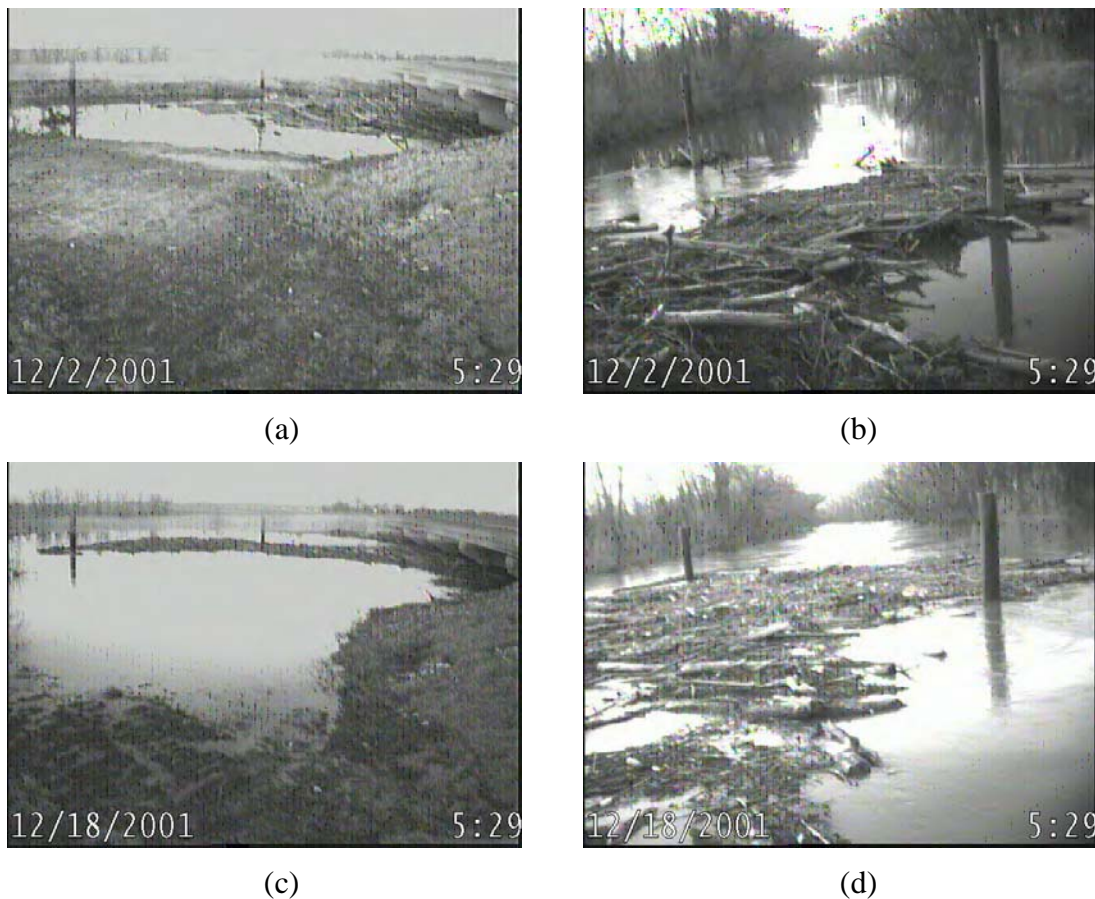
ations: on the one hand, none of these flows events might have been of sufficient magnitude to mobilize a ready supply of woody debris, or the supply of woody debris had been exhausted by events prior to the debris clearing, and hence even fairly large events could not cause any significant debris accumulation.



**Figure 5.5: Images of the first recorded significant flow event at the SR59 site, a) before the event, sand bar unsubmerged, b) during the event but before any significant debris movement, c) the first significant debris movement, d) after the event, sand bar again unsubmerged**

The first significant flow event after the installation of the first camera began on 10/12/01 (at Bowling Green, so there may be a slight lag at the SR59 site). Fig. 5.5a shows an image recorded by the bank camera on the previous evening (note the time stamp, 5:00 pm, at the lower right corner), while Fig. 5.5b shows the same scene the following morning at 6:33 am. Unfortunately, the quality of these initial images is rather poor, possibly due to less than optimal settings on the recorder. On 10/11, the stage was sufficiently low that the sand bar can be seen; in contrast, by the following morning, the stage had risen (by an estimated  $\approx 2.5$  ft) significantly, and the debris deflectors were partially submerged. Some debris can be seen at the more upstream deflector in *both* of these pictures, and so the arrival of significant amounts of debris has not yet occurred. Indeed, not until later in the evening, at about 6:27 pm, is large woody debris observed in any large quantity. The discharge (at Bowling Green) returned to below 1000 cfs on 10/21 and remained below that level until 10/24. An image from 10/23 (Fig. 5.5d) shows

significant debris at the site; in comparison with Fig. 5.5a, the debris pile is seen to extend from the piers in a thick mat to the nearer debris deflector, only thinning out towards the more upstream deflector.

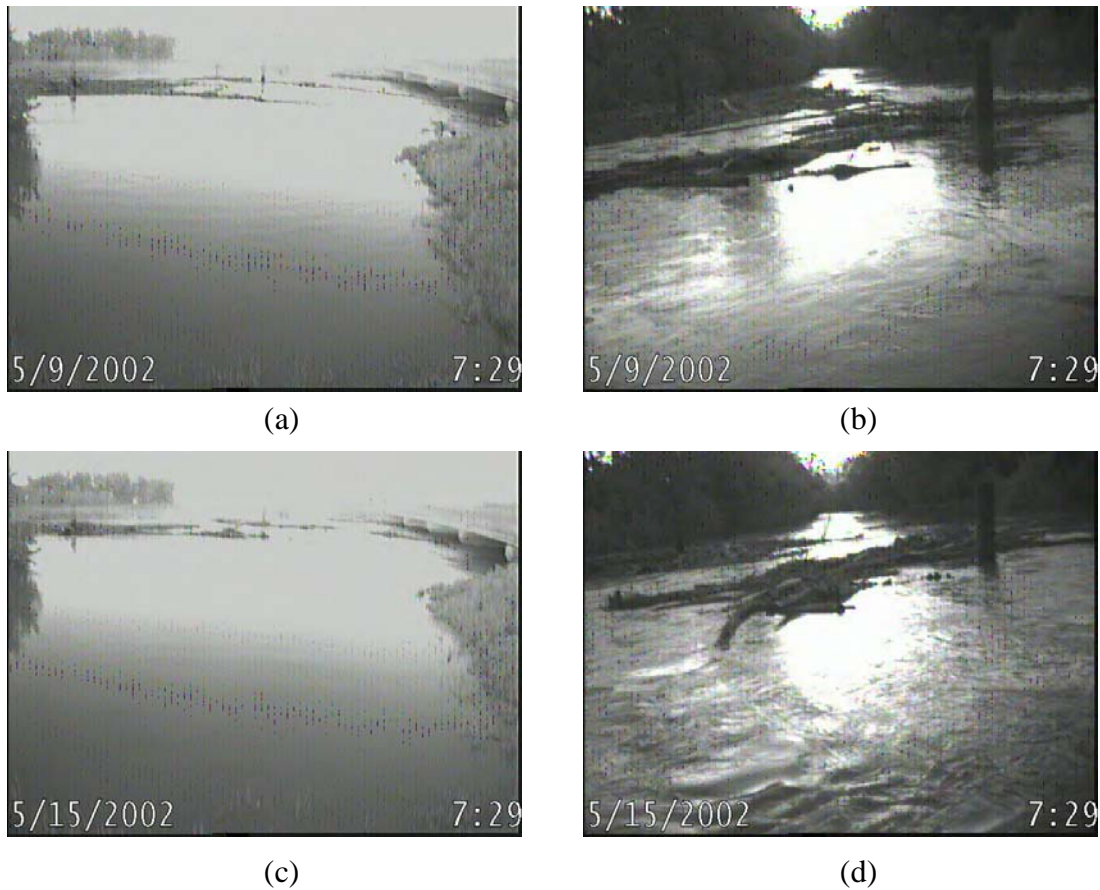


**Figure 5.6: Images from two-camera system, (a) and (b) (relatively) low-water at site on 12/2/21, (c) and (d) high water at the site on 12/18/21, from the bank camera and bridge camera respectively**

The first large (a smaller one preceded it on 11/30) flow event after both cameras (the one on the bank and the one on the bridge) had been installed started  $\approx$  12/15 and continued through until 12/19. Fig. 5.6 shows images from the two cameras, before the event and at the height of the event, all in the evening, 5:29 pm. In Fig. 5.6a and b, the already established debris pile is shown before the event, and in Fig. 5.6c and d, when the stage has increased by an estimated 5 ft, the streamwise extent of the pile is seen to grow noticeably. Whereas Fig. 5.6b gives the impression of debris actually being trapped at the brideward deflector, Fig. 5.6d suggests that the debris is actually trapped by the pier downstream rather than by the deflector.

Several significant events occurred after the December events, but two large flow events occurred in close succession in May 2002. This is seen in Fig. 5.2 not only in the Bowling Green discharge ( $Q > 12,000$  cfs) time series but also in the stage time series at the site, where the stage likely exceeded 525 ft (for comparison, recall that the 100-year design discharge stage is 528.1 ft and the ‘average’ high-water is 521.7 ft according to

the bridge plan). Fig. 5.7 shows that the stage is noticeably higher than in previous figures (the water surface almost reached the top of the levee on which the instrument enclosure was placed), but also that the debris accumulation seems to have been reduced, compared to the images seen previously. Although this could imply that the disaggregation process observed in the laboratory may also be significant in the field at least under certain circumstances, care should be exercised in interpreting what is seen at the water surface—as will be seen shortly, it is possible that some large part of the pile may be submerged. Indeed, underwater bridge inspection has often found submerged accumulated debris at piers (Collins Engineers, 1994, 2001).

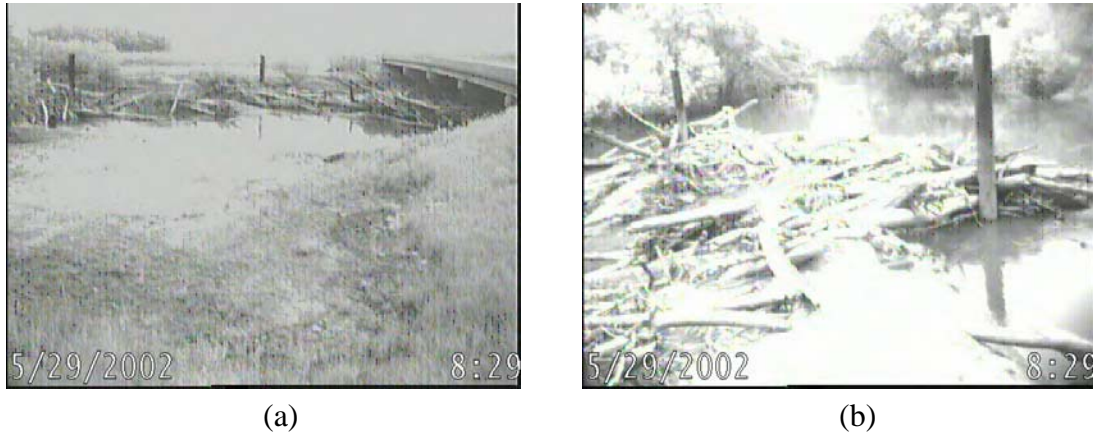


**Figure 5.7: Side- and upstream-views during the two largest flow events during the project period, (a) and (b) on 5/9/2002, (c) and (d) on 5/15/2002.**

On the afternoon of May 29, work began to clear the debris from the site. Images recorded that morning are shown in Fig. 5.8. It is evident that large amounts of debris remain at the site, and the images in Fig. 5.7 may be misleading. The stage is still relatively high ( $\approx 514$  ft compared to a typical low-water elevation of 507 ft), and the sand bar is still submerged.

After the clearing of the site and the dry summer season, there have been a few ‘minor’ events, but no large event, either in the fall of 2002 or in the winter of 2002/2003, comparable to those occurring in the preceding year. Not surprisingly, no sizeable debris accumulation has occurred at the site. There have been two large events in 2003 (Fig.

5.2), one in May and one in July, of size comparable to previous events, i.e.,  $Q > 8000$  cfs at Bowling Green. The May 2003 is of particular interest, because site inspection indicated that no substantial debris pile developed as a result of this event, and it had been assumed that no significant debris transport had occurred. Examination of the video record however showed otherwise.



**Figure 5.8: Scene at the SR59 the morning before the site was cleared, a) side view, b) upstream view**



**Figure 5.9: Logs taken from debris pile: a) 42-ft log being measured, b) longest log (length, 67.7 ft) measured**

### **5.3. Debris characteristics**

The clearing of the debris from the site provided an opportunity to determine rough debris characteristics, since debris was taken from the piers and placed temporarily on the bank. The main interest was to determine whether there were logs that exceeded the design log length of Diehl or indeed the bridge span. Hence, the sample of 12 taken was far from being random, and was biased towards the larger specimens. Measurements were taken with a simple tape measure. The longest specimen (Fig. 5.9b) measured 67.7 ft and was 5 ft in diameter at its base tapering down to 1 ft diameter at its tip. Another specimen was estimated to be greater than 62 ft in length, but could not be entirely measured because it was partially submerged. The other logs that were measured ranged

from 18 ft to 55 ft. As can be seen from Fig. 5.9, a substantial amount of debris was still in place at the bridge, so there is some possibility that even larger specimens were buried in the debris pile. Evidently, log lengths in the Eel River may attain magnitudes approaching or even exceeding the span between piers (66-ft).

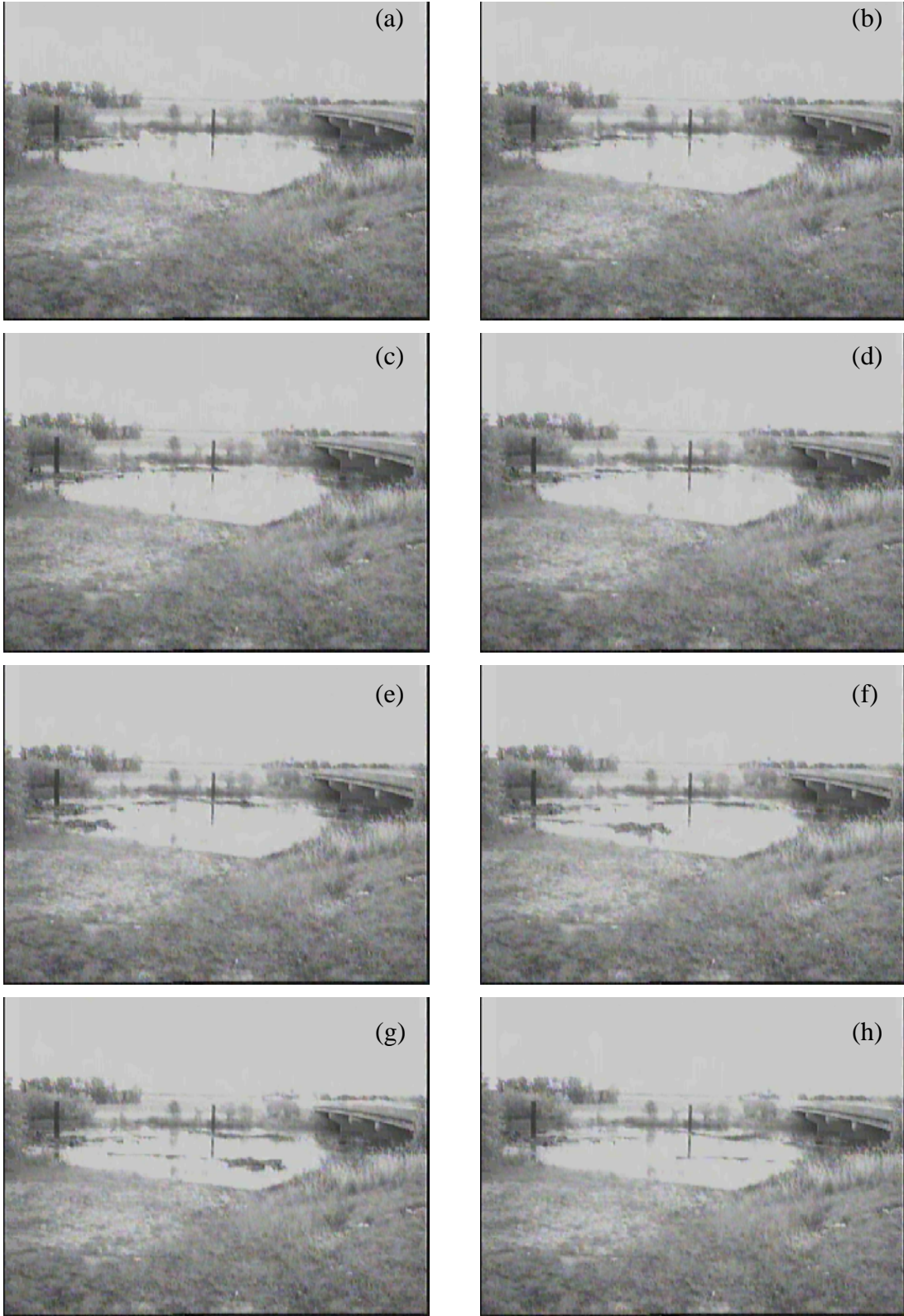
#### **5.4. Details of individual events**

In Sec. 5.2, an outline of the flow and debris accumulation events over the course of the project period (2000-2003) was given, describing broadly the overall picture of debris accumulation at the site. In this section, individual events are examined in greater detail in order to gain a better understanding of the processes involved in debris accumulation. This will necessarily be very selective, since an event lasting several days could be represented by more than a gigabyte of archived video images. Attention is focused on two events, one during October 2001 and the other during May 2003. The former is of interest since the site was essentially clear of debris preceding it as it was the first major event of the 2001-2002 'season', and a stable debris did indeed develop from it. The latter is instructive in that it illustrates a disaggregation event, in which a fairly sizable debris pile had initially formed, but later was broken up by the flow.

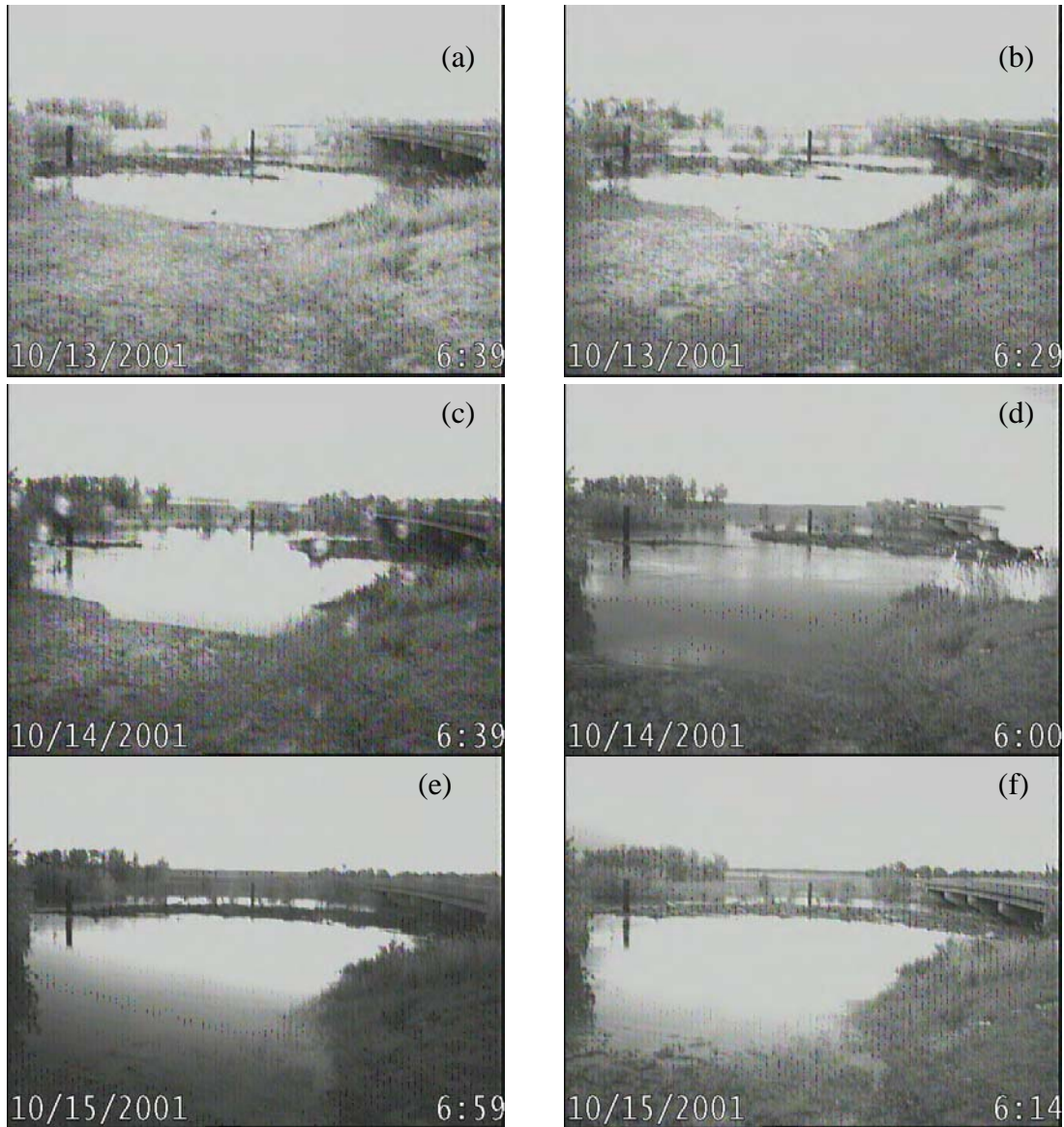
##### **Oct. 12-Oct. 15, 2001 event**

As noted before, although the stage had already risen substantially from its low-water elevation the previous day, noticeable amounts of large woody debris were only observed towards the evening of Oct. 12. The first appearance of much debris is shown in a sequence of images in Fig. 5.10 taken every 10 seconds. The motion is particularly evident in the last four images, where a fairly large (estimated length of 38 ft) partially submerged specimen traverses the field of view near the bottom of the frame. Its velocity, estimated from the image sequence, is 1.5 ft/s. The uncertainty associated with this velocity estimate is large, and this value seems rather small for flood conditions, but it may be that the specimen is traveling near the bank and as a result travels much more slowly than it would in mid-channel, even possibly dragging along the bed. These specimens all seemed to be aligned with their longest dimension parallel to the flow, as has been reported in the literature (e.g., Chang and Shen, 1979). They also all seem to pass successfully under the bridge section without being trapped. After this brief (6-7 minutes) burst of activity, no further movement of large amounts of woody debris was observed that evening, though video monitoring was very soon thereafter (15 minutes) not possible because of low light conditions.

Unfortunately, as noted previously in Sec. 5.2, by the time ( $\approx$  6:30 am) that video monitoring was feasible the next morning, a debris pile seemed to have already become established (Fig. 5.11a), and for the rest of that day (10/13), very little debris activity was observed (Fig. 5.11b). The Bowling Green discharge on 10/12 and 10/13 was only  $\approx$  4000 cfs, i.e., not the maximum for that event, which was  $\approx$  6600 cfs,



**Figure 5.10: Sequence of images showing movement of floating woody debris (a)-(h) images are taken every 10 seconds**



**Figure 5.11: a) and b) debris pile established at deflectors, c) debris pile now found at bridge, d), e), and f) slow growth of debris pile over two days**

occurring on 10/14 and 10/15. In Fig. 5.11a,b, it seems that a substantial part or even the larger part of the pile is trapped at the deflectors.

The earliest image on the following day (Fig. 5.11c) however showed a markedly different picture, with the bridgeward deflector almost bare of debris, and practically all of the debris at the bridge. It would seem that the debris pile at the deflector became unstable, and then became re-established later at the bridge. Whether this is associated with the increase in discharge, evidenced by the increase in stage, is uncertain. Thereafter, the pile grows fairly slowly by accretion of isolated single logs, usually rather small in size, with the ultimate result as shown previously in Fig. 5.5d.

## May 10-May 13, 2003 event

The May 2003 event highlights the usefulness of continuous video recording of a large flow event. Images taken on the morning of July 5, 2003, i.e., after the event in question and prior to the next large flow event, are shown in Fig. 5.12. These show a site that is essentially clear of debris, except for a large single log. The sand bar is evident because of the low water level. Such a scene would suggest that no significant debris-transporting event had occurred since the previous clearing at the end of May 2002. This would seem plausible except for the May 2003 flow event, which was comparable in magnitude to previous debris transporting events.



**Figure 5.12: Images taken at the SR59 site on 5 July 2003.**

The USGS Bowling Green gaging station recorded a daily discharge of 2020 cfs on 5/10 and peaking on 5/12 at 8,500 cfs, dropping to 3439 cfs and to 1900 cfs on 5/13 and 5/14. The first significant wave of debris observed at the SR59 site occurred on the evening of 5/10 after 5:00 pm. By the next morning, a small debris pile has developed at the brideward deflector, and possibly at the bridge. A particularly intense period of debris transport occurred at about 8:00 am on 5/11 as is seen in Fig. 5.13. The debris piles at both the brideward deflector as well as at the bridge continued to grow, so that by 12:50 p.m., they were both of significant size. Shortly thereafter however the pile at the deflector became unstable, and quickly disaggregated. The pile at the pier remained, but eventually this also became unstable, so that by noon on 5/13, there is little sign of any debris at either pier or deflector. Thus, the lack of debris at the site on 7/5 (Fig. 5.12) can be quite misleading, because substantial debris transport may have taken place and debris piles may have developed at the site. These piles may however have become unstable, and did not survive the duration of the flow event.

### **5.4. General observations and discussion**

Several broad observations may be drawn from the admittedly very limited data so far obtained (essentially from a single site). It should also be kept in mind that only what is happening on the surface is accessible to the present video monitoring.

- **Debris motion and size**
  - As noted previously in the literature, logs do tend to travel singly rather than in a clump or large aggregations of material.

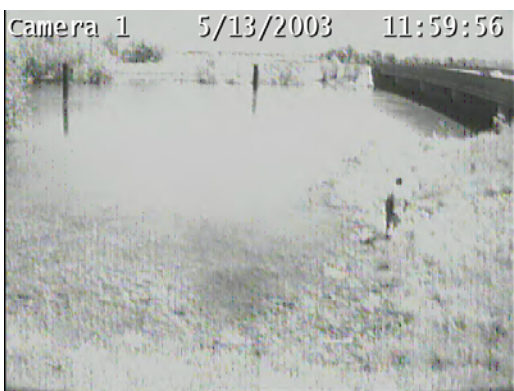
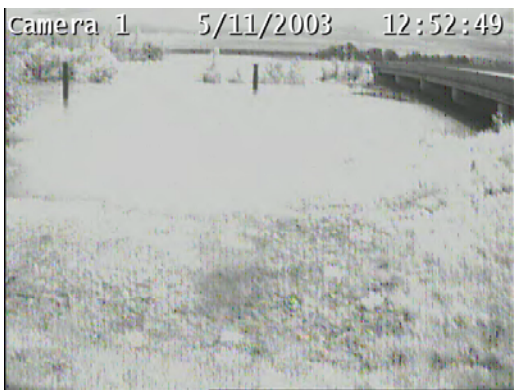
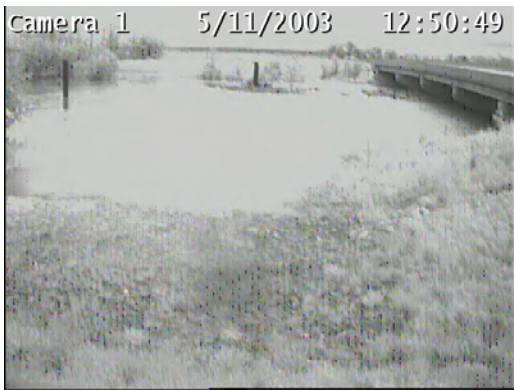


Figure 5.13: Debris transport, accumulation and disaggregation during the May 2003 flow event

- Because of the sand bar, the thalweg of the channel in the immediate vicinity of the bridge is somewhat ambiguous. Upstream of the bridge, where the thalweg is more clearly defined, debris does tend to travel in midstream which can be assumed to be approximately the location of the thalweg.
  - Whether a log tends to be oriented lengthwise in the streamwise direction is less clear. Certainly an orientation closer to being perpendicular to the streamwise direction is common, if not necessarily the most probable. This may also be a function of size, with longer logs taking a more streamwise orientation, and shorter debris open to a wider range of orientations.
  - The overwhelming majority of debris are small in size (e.g., less than 15 ft in length); very long logs of the order of the pier-to-pier span are exceedingly rare. It should however be noted that size obtained from the video monitoring may be especially prone to be underestimated, since logs may be partially submerged in their travel.
- **Debris occurrence**
    - The delivery of debris to the site seems to occur in bursts, rather than continuously, even during a flow event of extended duration (e.g., several days). Periods of short-duration (1-2 hours) intense transport activity may be followed by much longer periods where transport activity, while not entirely absent, is much reduced. One possible explanation for this is that the debris is *not* generated in the vicinity of the site, and the bursts result from different travel times from different contributing areas.
    - A debris pile may already become established during the first ‘large’ event of the season. A ‘large’ event may actually be quite moderate in terms of discharge and stage with a return period of less than a year, and with a discharge that is substantially lower than the mean annual maximum discharge.
    - The pile can be initiated and indeed grow to a considerable size well ahead of the peak of the event. After the hydrograph peak, debris transport is greatly reduced. This is consistent with the explanation of contributions from different areas, since the occurrence of the hydrograph peak is presumably associated with all areas of the watershed contributing to the flow as well as to the debris accumulation.
    - The differing results of May 2003 and the July 2003 events at the Eel River site show that similar hydrological events (peak discharge,  $\approx 8500$  cfs, duration,  $\approx 3$  days, at Bowling Green) can lead to quite different outcomes as to debris accumulation. In the first case, a debris pile had formed, and grew to a size of some note, but eventually was dispersed at the height of the event. In the second case, a very large and stable debris accumulation resulted, covering the region between the two debris deflectors and even extending upstream past the more upstream deflector. A precise explanation for the different outcomes is still to be formulated. A first-flush-type supply-limited model does not seem entirely adequate. The possibility that the hydrologic event may generate its own debris supply, either through wind effects, or bank instabilities, is more plausible, but

in that case the similarities in the hydrologic events would tend to generate similar amounts of debris with equal likelihood of a debris pile forming.

- **The debris deflectors**

- The installed ‘deflectors’, when bare, do not display any significant deflecting ability in that debris traveling in the direction of the deflectors are more likely to hit the deflector rather than to veer away from the deflector. The deflector exhibits more of a tendency to trap debris. After a small pile has become established at the deflector, some deflecting ability is noted, but the inertia of the moving debris is usually sufficient that it still brushes the pile, frequently becoming trapped.
- While the debris deflectors act to trap woody debris, the piles, at least when small, seem prone to become unstable. The pile previously trapped at the deflectors may then have a greater likelihood of being re-trapped and re-established at the pier because of the size of moving pile. From this point of view, the deflectors might be said to have a contribute to rather than minimizing debris accumulation at piers. Nevertheless, it is not clear what would have happened in the absence of deflectors, since the debris trapped at the deflectors could have been trapped at the piers in the absence of deflectors.

## **5.5 Summary**

In the field study, two sites have been instrumented for video monitoring and recording of phenomena related to debris accumulation at bridge crossings. This chapter has focused on observations at the SR59 south crossing of the Eel River because that has been in operation since September 2001, and has recorded two (and a half) seasons of debris-transporting flow events. The other site, at SR63 southbound crossing of the Vermilion River, has only been operating since April 2003. At the Eel River site, hydrological information from an upstream USGS gaging station, and later from a local stage recorder has been combined with the recorded video images to interpret the events surrounding debris accumulation. In particular, the movement of debris and their occurrence within the flow hydrograph and the effect of the debris deflectors were discussed.

## 6. Conclusions and recommendations for implementation

The project was aimed at gaining a better understanding of the physical processes involved in single-pier debris accumulation at bridge crossings, and to study the performance of possible countermeasures, such as debris deflectors. Both laboratory and field studies were undertaken in the course of the project.

Although scale effects cannot be ruled out, single-pier debris accumulations were reproduced under laboratory conditions in a range of Froude numbers comparable to those in the field. Experiments were conducted with dowels or twigs as model logs and with a single pier alone, or in combination with a model sand bar, or model countermeasures (deflector or groin-like structure). As might be expected, stable debris piles were more likely to form at lower velocities. A possibly important and less expected result is an apparent effect of flow depth on the potential for debris accumulation. Stable debris piles were more likely to develop in shallower flows or flow regions. This may be due at least in part to a stabilizing effect of logs resting on the bed. In spite of the empirically based correlation, any causal relationship remains associated with a large degree of uncertainty until the precise mechanisms for this effect or correlation are clarified.

The laboratory study of the effect of a model deflector indicated that it did reduce accumulation at the pier, but did so only by itself trapping debris. An alternative countermeasure that was studied, namely groin-like submerged structures extending from the channel walls into the flow, gave rather mixed results with regards to performance. Under conditions conducive to large debris accumulation, it did not significantly reduce either the likelihood or even the amount of debris accumulation.

The field observations have so far mainly been limited to a single site for two debris-transporting seasons, although another site has recently been instrumented. Several notable qualitative features regarding debris transport and debris accumulation can be tentatively put forward. The transport of debris occurs rather intermittently with long periods of comparative inactivity punctuated by short periods of intense activity, generally on the rising limb of the hydrograph. A debris pile is initiated early in the flow event, and can grow to a substantial size well ahead of the hydrograph peak. Hydrologic events, similar in peak discharge and overall duration, may lead to quite different outcomes as to debris accumulation.

The currently installed deflectors were observed to trap rather than to deflect debris, the inertia of the moving debris generally being sufficient to overcome any weak deflecting effect. This is consistent with the results of the laboratory study, which indicated that debris piles often developed at the model deflector. Unlike the laboratory experiments, the field observations indicated that debris piles at the deflectors were prone to become unstable. This difference between the laboratory and field study may be due to the more complicated channel geometry and hence more complicated flow. In such an eventuality, the debris pile could become re-established at the bridge. In the presence of a significant debris pile already established at the bridge, the current (or even alternative) deflectors are likely to be ineffective.

## Recommendations for implementation

While the laboratory and field studies conducted so far have led to a number of interesting observations, definitive results that could be implemented on a broad scale with reasonable promise of success are not available at this time. The studies so far have looked primarily at a situation like that found at the Eel River crossing, and a broader study examining a larger sample of cases with more varied conditions would be necessary for more definite results. Tentative recommendations may however be formulated:

- Hydraulic design of new bridges
  - Stream geomorphology (and sedimentation processes) should be given greater consideration if large woody debris poses a potential problem. Both sites under study have experienced significant sediment deposition, leading to regions with locally shallow areas of the flow. Laboratory evidence suggests that this may aggravate the problem of debris accumulation. Design choices or construction practices, such as deliberate channel widening to obtain a larger clear opening, that might promote sediment deposition at a bridge crossing should seriously consider possible adverse consequences for debris accumulation.
- Countermeasures for existing bridges
  - The currently installed deflectors have certainly not been successful in alleviating the problem of debris accumulation at the study sites. It might even be argued that in some cases they may even contribute to the problem by trapping debris, which, when the debris pile becomes unstable, may be more likely by their size to become re-established at the bridge. Whether a strong case can be made for removing the deflectors is less clear. It is more likely that the effect of the deflectors is generally negligible, and so the cost of removal cannot be justified solely on the basis of its effect on debris accumulation.
  - If sedimentation and stream aggradation play an essential role in debris-accumulation problems, and if regulatory issues do not present any obstacles, dredging the channel might be considered an option. By itself, dredging would not provide a long-term solution, because sedimentation will likely still occur since the underlying causes of deposition have not been addressed. A combination of dredging and bank revetment/redevelopment might be considered. To inhibit sedimentation, reduction of the bridge clear opening seems unavoidable, and the negative consequences for bridge hydraulics (flooding, local scour, etc.) will need careful consideration. A somewhat related option is a more ‘efficient’ channelization of the stream. This may not necessarily involve dredging, but would still likely involve a reduction of the bridge clear opening, or some notable increase in flow velocity, with the related issues noted above. In view of the remaining uncertainties surrounding the relationship between depth and debris accumulation, as well as the cost of any dredging and/or channelization project, and the attendant issues of other adverse side effects, a strong case for this option is difficult to make.
  - Based on the field observations, there would seem to be some advantages to having countermeasures which do not trap debris immediately upstream of the

bridge. This prompted the consideration of groin-like structures which would be entirely submerged during debris-transporting events, and so would not trap debris traveling at the water surface. The results of limited laboratory tests carried out with a single (certainly not optimized) configuration were disappointing in that a more definite improvement in reducing debris accumulation was not achieved. Whether the basic concept is sound, and the detailed configuration needs to be refined and optimized, can only be assessed with additional work. At the present stage of development, such structures cannot be recommended for implementation.

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