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EFFECTS OF THE UPSTREAM LENGTH OF A FIXED ICE COVER ON LOCAL SCOUR AT A BRIDGE PIER

by

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LIST OF SYMBOLS

The symbols listed here are ones used in equations, figures, tables and the text of this thesis.

B	width of flume
b	pier width
d	grain diameter
d_{16}	grain diameter where 16% of the particles are finer
d_{50}	median grain size
d_{84}	grain diameter where 84% of the particles are finer
F	fall velocity parameter
F_b	pier Froude number
F_r	Froude number
g	acceleration due to gravity
K_1	correction factor for pier nose shape
L	length of ice cover upstream of pier
L_f	length of flume test section upstream of pier
Q	flow rate
Q_v	flow rate as calculated using Marsh-McBirney current meter velocities
R_{c*}	critical Reynolds number
S	slope of flume
T	water temperature
U_*	shear velocity
V	average velocity

V_{ave}	average of Marsh-McBirney current meter velocities
V_{avep}	average velocity at pier using flow rate from venturi meter
V_c	critical velocity of the sediment in open water conditions
V_p	average velocity at pier
V_x	velocity in the downstream direction as measured by ADV
V_1	upstream velocity as measured by Marsh-McBirney current meter
V_2	downstream velocity as measured by Marsh-McBirney current meter
y_p	depth of flow at the pier
y_s	depth of scour
Y_1	depth of flow upstream of the ice cover
γ	specific gravity of water
γ_s	specific gravity of a sediment
ν	kinematic viscosity
σ_g	sediment uniformity
ω	fall velocity

ABSTRACT

Previous research and field observations have shown that the presence of a fixed ice cover at a bridge pier affects the scour mechanism and provides a greater local scour depth. However, there is no previous research regarding the significance of the upstream length of the ice cover. An experimental study was performed at Fritz Engineering Laboratory at Lehigh University to observe the effects of the upstream length of a fixed ice cover on local scour at a bridge pier. Nineteen tests were conducted utilizing two flow rates, two pressure head conditions, and four upstream cover lengths. The test set up included a 15 foot long flume test section, a 1.25 inch diameter cylindrical bridge pier, and sediment with a median diameter of 0.42 mm. The clear water scour condition was tested.

The results of this preliminary study showed a difference in scour depth between the 100% covered case and the 50%, 25%, and 10% covered cases. The longest cover case provided the smallest scour depth for all flow rates and pressure head ratios tested. It is surmised that a change in cover length does provide a change in the flow and scour mechanisms, affecting the depth of scour that occurs. Three different flow regimes related to the cover length are proposed: a short length similar to a submerged bridge deck; a long length which allows for fully developed flow; and a transition length, where the flow is not yet developed but the cover is too long to be comparable to a submerged bridge deck.

INTRODUCTION

Problem Statement

In rivers, the effect of an ice cover on local scour at a bridge pier is an area that warrants a significant amount of research. Although research has been performed and equations have been developed to help design for scour at piers and abutments in a variety of open water situations, the situation of an ice cover, and especially a pressure flow, is one that has only recently started to garner interest. The pressure flow situation caused by an ice cover does have an impact on scour at a bridge pier. In fact, case studies such as the Fort Peck Reach of the Missouri River (Zabilansky, et al. 2002) or the bridge failure on the White River at White River Junction, VT (Zabilansky, 1996) demonstrate that the impact of an ice cover can be significant.

The U.S. Department of Transportation addresses pressure flow in its Hydraulic Engineering Circular No. 18 (HEC-18), *Evaluating Scour at Bridges* (Richardson and Davis, 2001), based on the situation when "the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure" as per Figure 1. HEC-18 also provides a design methodology for pressure flow situations, offering an equation for bed vertical contraction scour. The scour depth obtained from this equation would then be added to the local pier scour, computed as if no pressure flow existed, to obtain the total local pier scour depth. However, HEC-18 does acknowledge limitations in the method, as the next step is to "Use engineering judgment to evaluate the local pressure flow pier scour" (Richardson and Davis, 2001). This section in HEC-18 on pressure flow has no mention of an ice cover providing the pressure situation; it only addresses an overtopped bridge deck.

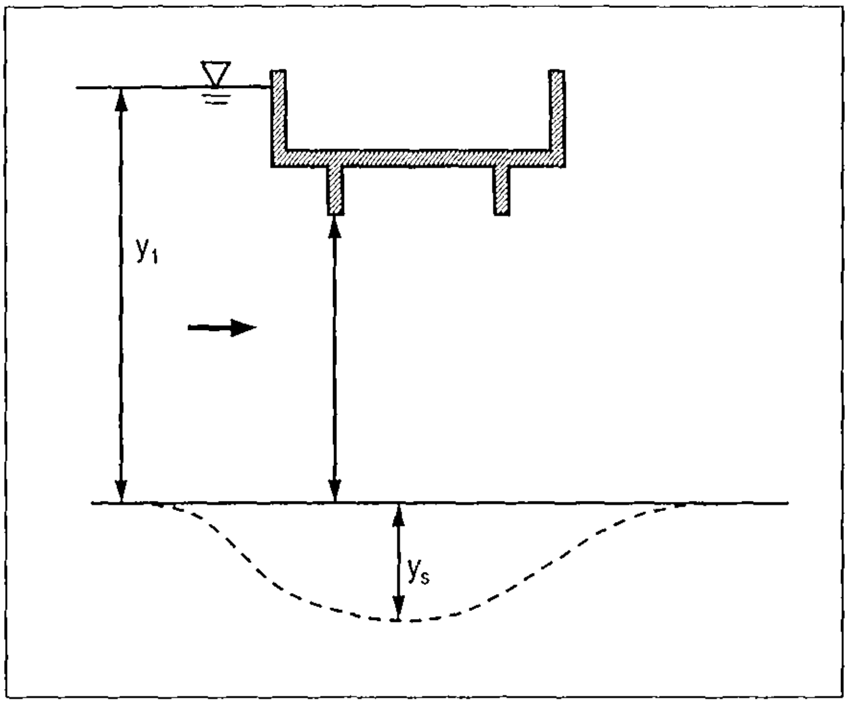

Figure 1: Pressure Flow Due to a Submerged Bridge Deck (Richardson and Davis, 2001)

The limitations of the current design methodology are further pointed out in a memorandum to HEC-18 from John Matthews of the Virginia Department of Transportation Hydraulics section. In his memorandum, entitled "The applicability of the Pressure Scour Equation in HEC-18," Matthews questions the threshold where the pressure scour equation stated in HEC-18 becomes applicable:

> Unfortunately there is no documentation within HEC-18 or the published paper about at what point the application of the equation becomes appropriate. Citing the just touching case, there are no guideline[s] for at what point there is enough vertical contraction to warrant computation of pressure scour. Also there is no discussion about the relative nature of the contraction. 4 feet of vertical contraction when the flow depth is 10 feet is significant, 4 feet of contraction when the flow depth is 40 feet is less significant (Matthews, 2002).

From Matthews' statement, one can ascertain that the ratio between the depth of the pressure flow and the length of the pressure-inducing cover may have some effect on the scour mechanism or the depth of scour that occurs. However, prior to this study, no research has been performed to determine if the ratio between the upstream length of the pressure flow and the depth of flow has any significant effect on the depth of local scour at a bridge pier. This ratio may be important in ice cover cases, where the ratio of pressure flow length to flow depth is significantly larger than in cases of bridge deck overtopping.

Although case studies have shown that ice covers do affect the scour mechanism and can cause catastrophic damage, little reference is made to this situation in design methodologies such as HEC-18. In addition, little study has been done on the effect of ice covers on the local scour at bridge piers, and no studies have been performed regarding the variable length that may occur in the upstream ice cover and how this may affect the scour depth.

In this study, the effect of upstream ice cover length on the local scour at a cylindrical bridge pier under clear water conditions has been observed in a laboratory flume. Four different cover lengths were tested, using two different flow rates with two upstream depths for each flow. The equilibrium scour depth for these different cases were observed and compared to determine if the length of the ice cover and the ratio between cover length and flow depth are significant parameters in the study of local scour.

Scour Theory

Scour is defined as "erosion of streambed or bank material due to flowing water" and is the "most common cause of bridge failures" according to background information provided by HEC-18. Additionally, HEC-18 also defines scour as "the engineering term for the erosion caused by water of the soil surrounding a bridge foundation (piers and abutments)." Scour can be divided into three major categories: general, contraction, and local scour.

General scour can occur naturally with or without the presence of a bridge pier, abutment, or other man-made structure. The processes that are usually involved in causing general scour can encompass a wide range of temporal and spatial scales. The temporal scale can be classified as long-term or short-term. Long-term scour occurs over the course of several years, and includes bank degradation and lateral bank erosion. Short-term scour is caused by a flood or a series of floods over a short period of time. Spatially, general scour can occur over the entire river or in a small section of the stream reach, and can be caused by a variety of factors such as geomorphic instability, earth flows and slides, and land use changes (Melville, 2000).

Contraction scour can be caused by the presence of a bridge. This type of scour "occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction of the stream channel or by a bridge" (Richardson and Davis, 2001). A decrease in the flow area, based on continuity, causes an increase in velocity. This increase in flow velocity also provides an increase in tractive forces, which causes more bed material to be removed from the bed than under the previously unobstructed flow conditions.

Although most of the literature focuses on horizontal flow contraction, HEC-18 does acknowledge that ice formations or jams and pressure flows are forms of contraction flow which are vertical, not horizontal. Equations for estimating vertical contraction scour are not available. It is important to realize that, in the conditions of an ice cover reaching far upstream of a bridge pier, a contraction does occur and may contribute to the depth of scour.

Local scour is "caused by the interference of the piers and abutments with the flow" (Melville, 2000). Because of the obstruction, the flow is accelerated and creates vortices that remove the sediment around the obstruction. Local scour is a process that is dependent on several factors, such as flow intensity, flow shallowness, sediment coarseness, sediment nonuniformity, foundation shape, foundation alignment, approach channel geometry, time, and Froude number. Because the focus of this study is on the effect of ice cover on local scour at bridge piers, the following discussion is restricted to bridge piers only.

The formation of vortices is the basic mechanism that causes local scour at a bridge pier. Figure 2 shows an illustration of the flow patterns that develop at a cylindrical pier. These flow patterns include the downflow in front of the pier, the horseshoe vortices at the pier base, and the downstream wake vortices.

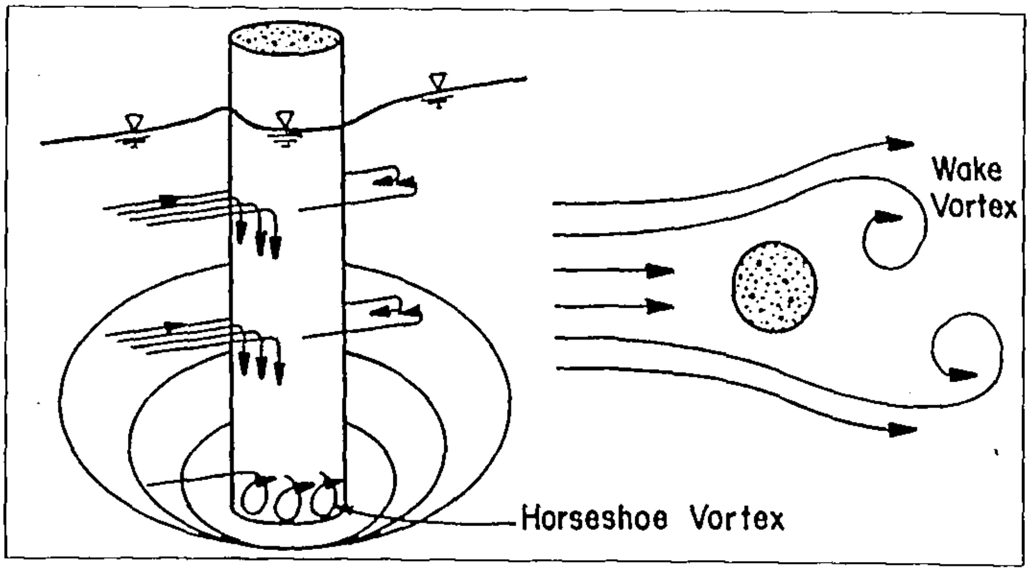

Figure 2: Flow Patterns and Vortices Due to a Bridge Pier (Richardson and Davis, 2001)

The downflow is caused by the deceleration of the flow ahead of the pier. Stagnation pressures form on the face of the pier, which cause a downward pressure gradient and forces the flow down. This downflow erodes away the sediment directly in front of the pier (Melville, 2000).

The horseshoe vortices are formed by the acceleration of flow around the pier. The vortices are able to remove the sediment from the base of the pier, and transport the sediment away from the base at a greater rate than sediment transport into the pier region, thus creating a scour hole. The strength of the horseshoe vortices decrease as the scour hole deepens, which allows for an equilibrium condition to occur (Richardson and Davis, 2001).

Downstream of the pier, wake vortices occur from flow separation around the sides of the pier. These, like horseshoe vortices, remove sediment from around the base of the pier. The wake vortices transport the sediment downstream, but diminish in strength quickly and deposit the sediment downstream of the pier (Melville, 2000).

Local scour can be affected by several factors, as previously stated. The following is a brief discussion of each of the factors as well as their applicability to this study.

Clear Water Scour vs. Live Bed Scour (Flow Intensity)

Clear water and live bed scour are the two conditions for scour based on the velocity of the flow. Clear water scour is the condition where "there is no movement of the bed material in the flow upstream" of the bridge pier (Richardson and Davis, 2001). In terms of velocity, clear water scour "occurs for velocities up to the threshold velocity for general bed movement" (Melville, 2000). In other words, clear water scour is the condition that occurs when the velocity of the flow is less than the critical velocity of the sediment in an open channel condition, $V/V_c < 1$.

When the live bed scour condition is present, movement of the bed material upstream of the pier does occur (Richardson and Davis, 2001). The sediment is moved downstream by the flow, and bedforms occur. The average velocity of the flow is greater than the critical velocity of the sediment, or $V/V_c \geq 1$.

For both clear water and live bed scour, an equilibrium scour depth is eventually reached when the scour depth does not change with time. However, the determination of these equilibrium values, especially that of live bed scour, can be difficult. Clear water scour steadily increases until it reaches its maximum, equilibrium scour depth. This process occurs over a longer period of time than live bed scour, and this maximum is often greater than the live bed equilibrium scour depth. Although the live bed scour occurs at a much faster rate, the equilibrium value is more difficult to determine. Since, in live bed scour, there is upstream sediment being transported downstream, the scour depth fluctuates as bed forms move past the pier. Thus, the scour depth oscillates around an equilibrium value. Figure 3 illustrates the depth of scour vs. time for both clear water and live bed scour.

In this study, the focus will be solely on clear water scour. This limit is imposed for two reasons. First, it is difficult to pin down an equilibrium value for the scour depth using live bed scour. Second, the facilities available for this study cannot allow for the recirculation of sediment through the system needed for live bed scour. Finally, clear water scour is a logical starting point to perform this research.

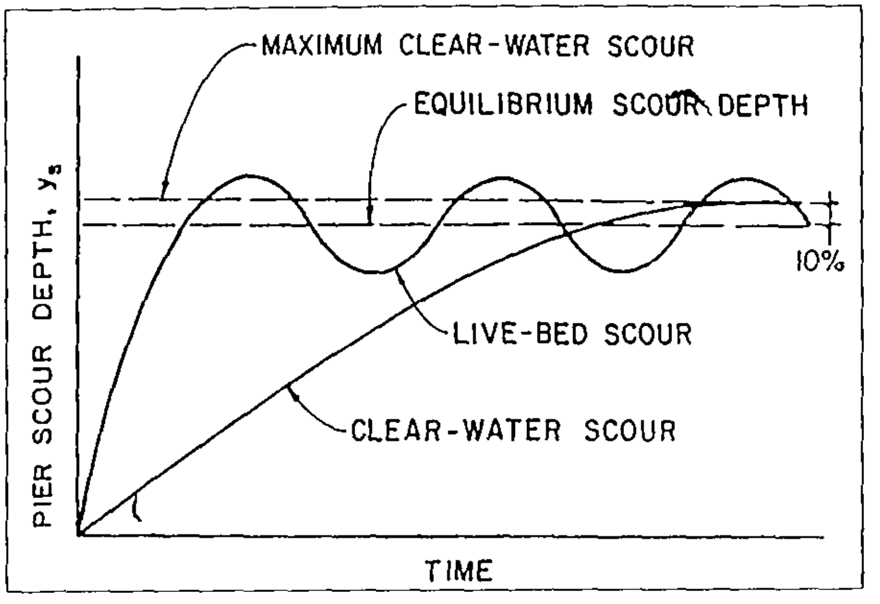

Figure 3: Clear Water and Live Bed Scour vs. Time (Richardson and Davis, 2001)

Flow Shallowness

Flow shallowness is defined as the ratio of depth of flow at the pier to the pier width, or y_p/b. In this study, the depth of flow at the pier is $y_p = 5$ inches and the width of the pier is $b = 1.25$ inches, giving a ratio of 4. Thus, the situation is classified as a deep flow with a narrow pier, as a narrow pier is defined by Melville (2000) as $b/y_p < 0.7$. In this situation, "the scour depth increases proportionately with foundation size and is independent of y." This is because the transverse size of the pier is related to the strength of the horseshoe vortices and the downflow. It is appropriate to use a narrow pier in this type of study, as most bridge piers fall into the narrow category (Hains, 2004).

Sediment Coarseness

Sediment coarseness is defined as the ratio of pier width to median size of the bed sediment, or b/d_{50}. Unless the sediment is so large that the sediment coarseness ratio is less than 50, local scour depths will not be affected by this parameter (Melville, 2000). In this study, the pier width is 1.25" and the median sediment size is 0.42 mm, which yields a sediment coarseness ratio of approximately 76, above the limit of 50.

Sediment Nonuniformity

Scour depth increases as a sediment becomes less uniform. Sediment uniformity is defined as:

$$\sigma_g = \sqrt{\frac{d_{84}}{d_{16}}}$$

For the sediment gradation curve used in this study, which is shown in the chapter entitled "Experimental Work," the value for d_{84} = 0.57 mm and the value for d_{16} = 0.33 mm. This yields a uniformity factor of σ_g = 1.31. From Raudkivi (1991), the criterion for determining uniformity is $\sigma_g < 1.35$. Therefore, the sediment used in this study can be considered uniform, and sediment nonuniformity does not have any effect on the scour depth.

Foundation Shape, Alignment, and Approach Channel Geometry

Bridge foundations can be many different shapes and these shapes may impose a flow obstruction that can affect the depth of scour. Depending on the streamlining of the pier, the strength of the horseshoe vortices vary. A squared-off pier will have approximately a 10% greater scour depth than a cylindrical pier (Richardson and Davis, 2001).

The guidelines for scour design published in HEC-18 include correction factors for pier nose shape. In this study, the model bridge pier is a circular cylinder, and has a correction factor of $K_1 = 1$.

The pier alignment to flow strongly affects the depth of local scour for all pier shapes except for circular (Melville, 2000). As the angle between the pier and the flow direction increases, the scour depth is also increased. This is due to the increase in the projected size of the pier. In this study, a circular pier is used and pier alignment is not an issue.

Approach channel geometry is a parameter that describes the effect of the cross-sectional shape of the channel, the lateral distribution of flow velocity, the lateral distribution of channel boundary roughness, and the flow intensity in terms of the cross-sectional geometry on the local scour depth (Melville, 2000). This parameter is a factor when the approach channel is of a compound shape rather than rectangular. Since all of the experimental work performed here utilized a rectangular laboratory flume, consideration of the approach channel geometry is not necessary.

Time

In the process of clear water scour, the scour depth increases with time asymptotically until an equilibrium depth is reached as shown in Figure 3. This equilibrium depth indicates the worst-case scenario, and is the value obtained in this experiment. Although Melville (2000) recommends that "small-scale" laboratory experiments be allowed to run for several days, other experiments that have been performed (Abed, 1991; Olsson, 2000; Umbrell et al., 1998) have utilized testing times as little as three hours.

Pier Froude Number

The effect of the pier Froude number, $F_b = V/(gb)^{0.5}$, is a reflection of the geometric similitude of the pier, flow, and sediment characteristics. This is a factor because, while the pier and flow characteristics are often scaled models, the sediment used is the same size as found in the field. Experiments performed by Ettema et.al. (1998) showed that the scour depth could possibly increase with the pier Froude number. However, the data is insufficient to be able to confirm the influence of F_b. It is believed that laboratory flume studies are conservative, meaning deeper scour holes are predicted than may occur in the field, and thus, it is safe to neglect any influence that F_b may have (Hains, 2004). Therefore, the pier Froude number was not considered during the course of this study.

Previous Experimental Investigations

A literature search shows that the research performed to date regarding the effects of pressure flow on local scour is limited. When the search is further limited to testing conditions where there is a fixed ice cover, local scour at a bridge pier, and clear water scour, the information available becomes even more limited. The following is a synopsis of the previous experimental investigations that have dealt with local scour with a pressure flow condition.

Floating Cover Tests

Experiments examining the effect of an ice cover on local scour at a bridge pier differ from tests in open water in that the presence of the ice alters the approach velocity profile and shear stress distribution. In some experiments, the ice cover was allowed to float on the water surface and rise and fall with changes in flow rate and control depth. The presence of this cover increases the wetted perimeter and flow depth, decreases the mean flow velocity, causing the redistribution of velocities and shear stresses, and increases the friction factor (Batuca and Dargahi, 1986). However, the floating ice cover does not induce a pressure condition, as a fixed cover does. A diagram of the floating ice cover condition is shown in Figure 4.

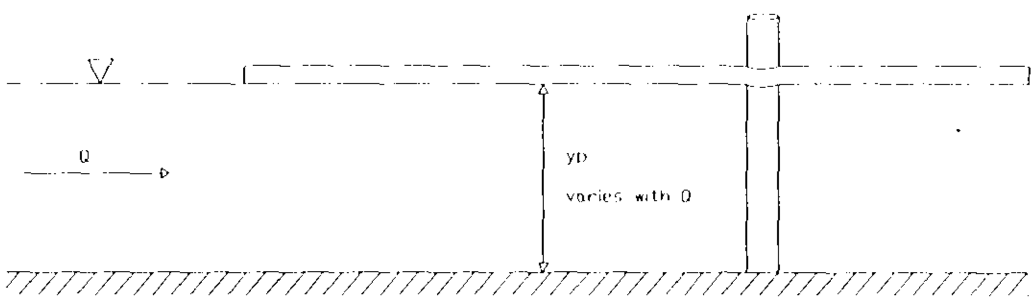

Figure 4: Floating Ice Cover Flow Condition

Batuca and Dargahi (1986) performed a laboratory study that compared the local scour around a cylindrical bridge pier in open water and floating cover conditions. All tests were for clear water scour. Sixteen tests were performed for the free surface condition, and 34 tests were performed for the ice cover condition. The equipment used was two concrete flumes, one sediment with a median diameter of d_{50} = 0.41 mm, five metallic cylinders to simulate bridge piers, and both a plywood and an aluminum cover.

The purpose of the study was to make visual observations regarding the differences between free surface and ice cover conditions, and there was no attempt to study flow structure or explain scour mechanism. The results of their study are as follows:

- The general size of the scour hole is greater in the ice covered condition.
- Scour depth increases with the mean flow velocity, and depths are greater for the ice covered condition.
- Scour depth decreases as flow depth increases, and depths are greater for the ice covered condition.
- For the same flow and bridge Froude number, the scour depths are greater for the ice covered condition.

Additionally, Batuca and Dargahi developed some empirical equations for scour depth for both free surface and ice covered conditions. These equations are limited by the clear water condition, the bed material size and distribution, the ratio of flow depth to bridge pier diameter, and the flow and pier Froude numbers. These limitations do not permit these equations to be used extensively for bridge pier design and scour estimation in the field.

Olsson (2000) also performed a comparison experiment between open water and floating ice cover conditions. However, Olsson also incorporated live bed scour into his experiment. Thirty tests were performed: twenty with the ice cover and ten with a free surface. Seven of the ice cover tests were performed with a rough cover, while the other thirteen were performed with a smooth cover. The equipment used was a Plexiglas flume, sediment with a median diameter of d_{50} = 0.42 mm, one smooth, transparent cylinder, and Styrofoam sheets to simulate ice. To simulate a rough cover, plastic pieces approximately 4 mm in diameter were glued to some of these Styrofoam sheets. Tests were run for a period of four hours.

The purpose of Olsson's study was to compare the local scour depth at a bridge pier in free surface conditions to scour depth in a floating ice cover condition. This analysis was performed for both clear water and live bed scour, and the equilibrium scour depth that was obtained was a time average. The results of the study were as follows:

- Covered tests for both clear water and live bed scour conditions gave larger scour depths than free surface conditions.
- The difference in scour depth between the ice covered condition and the free surface condition was more prominent in the live bed scour tests.
- The rough cover provided a greater scour depth than the smooth cover when all other variables were held constant.
- Ice covered and free flow conditions showed increased scour depths with increased velocity.

Pressure Flow Tests – Submerged Bridge Deck

Superstructure submergence, or bridge deck overtopping, is the condition where the water level in the channel becomes high enough to touch or completely cover the bridge deck. Studies have shown that the local scour depth at a bridge pier that occurs in the case of a submerged superstructure is greater than the scour depth that occurs at an unsubmerged bridge (Abed, 1991; Jones et al., 1993). The scour that occurs can be defined as having two components: the local scour that would have occurred due to the presence of the bridge pier and the scour caused by the pressure-inducing submergence (Melville, 2000). A diagram of a submerged bridge deck is shown in Figure 5.

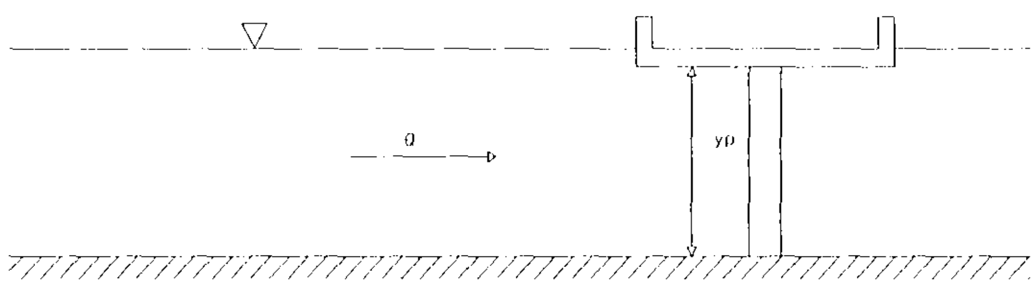

Figure 5: Submerged Bridge Deck Flow Condition

Abed (1991) presented the first study that analyzed the effect of pressure flow on local scour at a bridge pier due to submerged bridge decks. In her study, twenty five tests, fifteen with pressure flow and ten with a free flow condition, were conducted. All tests were performed for the clear water scour condition. Abed utilized a single 200 foot long flume with a 40 foot long test section filled with a pea-sized gravel with a median diameter of d_{50} = 3.2 mm. The bridge pier was a 1:10 scale model with a well-rounded Plexiglas nose. The bridge deck was also a 1:10 scale model, and was 8 feet wide across the flume by 6 feet long in the stream direction by 0.1 foot thick. One part of the deck was made of Plexiglas, and the other made of plywood. The scour depth measurements were taken at the conclusion of each test, and tests were run for a minimum of 3 hours. The purposes of this study were to determine the depth and width of local scour at bridge piers in a pressure flow, to develop equations to predict local scour in a pressure flow condition, and to provide factors to correct existing free flow equations for pressure flow.

Abed's study provided several conclusions regarding the effects that the pressure conditions had on local scour:

- The maximum local scour depth was larger in pressure flow than in free flow. In terms of Froude number, scour depth increased for pressure flow by a factor of 2 to 3 for a $F_r \geq 0.5$, a factor of 2 to 4 for $0.35 < F_r < 0.5$, and by up to a factor of 10 for $F_r < 0.35$.
- A decrease in flow depth at the pier in a pressure condition caused a greater scour depth for the same approach flow velocity and upstream flow depth.
- Maximum local scour width was increased by a factor of 2 to 3 in pressure flow.
- The pressure flow was found to cause disturbances, higher velocities, and stronger vortices in the flow. This may contribute to the increase in scour depth.

Abed developed equations from her experimental work and a combination of previously established equations and correction factors to estimate local scour depth at a bridge pier for a pressure flow. It should be noted that the length of the flow that is in the pressure condition was not considered, since only one bridge size, 6 feet long in the stream direction, was used in Abed's study.

Arneson (1997) performed a study which isolated the effect of the pressure flow die to a submerged bridge deck from the obstruction that the bridge pier causes. He accomplished this isolation by performing tests both with and without a pier. The purpose of his study was to define the processes that cause the pressure flow scour, using both live bed and clear water scour conditions. A secondary purpose was to develop new equations or methods of bridge scour evaluation to use in practice. In the course of his experimental work, Arneson utilized a 200 foot long flume and a 1:8 scale model bridge. A 1:8 scale model Plexiglas bridge pier was used in experiments that measured scour with a pier involved. Twenty eight tests were performed, 18 without a pier and 10 with a pier. Sediment sizes used ranged from 0.6 mm to 3.3 mm, and flow rates ranged from 8 cfs to 35 cfs.

In his study, Arneson used multiple linear regressions, partial residual analysis, and other statistical tests to obtain relationships to predict the amount of vertical deck scour and the amount of bridge pier scour under pressure flow conditions. One of these relationships is currently in HEC-18's method as the recommended relationship for bed vertical contraction scour. From his study, Arneson made the following conclusions:

- Relationships can be developed for the components of pressure flow scour with and without a pier. Three equations were developed, one for pressure flow scour without a bridge pier and two including the pier.
- Arneson's data was for a single pier width and alignment, so he recommends that future studies be performed to account for changes in these factors. He recommends that the K factors currently used in HEC-18 be used until more research is performed and adjustments to these factors are made.
- "The worst case condition of pier scour would occur just before the bridge transitions into pressure flow" (Arneson, 1997). Therefore, he recommends that the local scour at the pier can be calculated using open-

channel equations presently available in HEC-18 using the flow conditions that would occur just before the transition from open water to pressure flow.

Arneson's research does not take into account varying cover lengths in pressure flow, as may be present in a condition where an ice cover extends well upstream.

Jones and others (Jones, et. al., 1993; Umbrell, et. al., 1998; Jones, et. al., 1999) performed studies at the Federal Highway Administration facilities that focused on the separation between the vertical contraction scour that is caused by the submerged bridge deck and the local scour due to the presence of the pier. In the course of this work, three different conditions were tested: local scour at the bridge pier in free surface flow, scour due to a submerged bridge deck without the pier, and total scour with a submerged bridge deck and a pier. The 81 tests were for the condition of clear water scour.

The flume used was 21.3 m by 1.8 m by 0.6 m, with a test section of 2.4 m. Three different sediment sizes, 0.3 mm, 1.2mm, and 2.4mm were used. The model bridge deck was based on a two-lane roadway bridge and was of 1:15 scale. Each test was run for 3.5 hours and the equilibrium value of scour depth was extrapolated based on the work of Laursen (1963).

The conclusions of the FHWA study are as follows:

- The measured pressure flow pier scour components were nearly identical to the measured pier scour components in free surface conditions. Therefore, it is recommended that the local scour at a bridge pier and the vertical contraction scour due to a submerged bridge deck be measured separately and then added.

- The assumption that the bridge opening is enlarged by scour until the velocity through the bridge opening equals the critical velocity of the sediment tends to underestimate the laboratory scour predictions.
- A regression analysis provided an equation that predicts pressure flow contraction scour.
- These conclusions and the subsequent equation may not be applicable to live bed scour conditions.

The FHWA focused primarily on submerged bridge piers and did not address the issue of an ice cover. Although the FHWA study recommends that the pressure scour component and the local scour component at the bridge pier be separated, an attempt to separate the two components was not made in the course of this study. It is believed that separating the two scour components would not provide any additional insight to the current study.

Fixed Cover Tests

The fixed cover condition differs from a floating cover and a submerged bridge deck. Unlike a floating cover, the fixed cover is not free to change position with a change in channel flow rate. This causes a pressurized situation which is not present in a floating cover condition. Unlike a submerged bridge deck, the pressurized area of the channel can extend far upstream of the pier. A fixed ice cover condition can exist in the field when the ice cover attaches itself to the pier or the sides of the channel. A diagram of the fixed cover condition is shown in Figure 6.

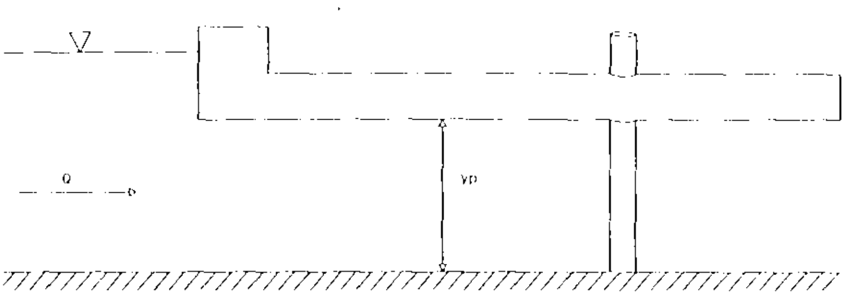

Figure 6: Fixed Ice Cover Flow Condition

Most recently, Hains (2004) performed a study that accounted for a fixed ice condition and the effect that this condition has on the local scour at a bridge pier. The purpose of his study was to obtain scour depth, scour pattern, and vertical velocity profiles under a fixed ice cover condition and to compare the results from the fixed ice cover conditions to the open water and floating cover conditions. This study was unique because the pressure flow condition of the fixed cover was the main focus.

In the course of Hain's experiment, 20 tests were performed, with 6 in the open water condition, 6 in the floating cover condition, and 8 in the fixed cover condition. For the covered conditions, both a smooth and a rough Styrofoam cover were tested. The flume used was a recirculating, tilting bed flume of dimensions 36.58 m long by 1.22 m wide by 0.61 m deep. The water temperature was maintained at 35° F. Uniform sand of median diameter d_{50} = 0.13 mm was used. The model bridge pier was a transparent cylinder with a diameter of 5.08 cm. The ice cover was fixed a distance of 12.19 m upstream of the pier and 3.05 m downstream and was set to maintain a flow depth of 22.86 cm. A majority of the tests were performed for the clear water condition; in five of the tests, live bed scour occurred. Tests ran for a period of 15 to 18 hours.

From the tests performed in this study, the following conclusions could be made concerning the effects of pressure flow on local scour at a bridge pier:

- The maximum velocity in the section of flow that is pressurized shifts its location toward the smoother boundary. The shift is more pronounced for larger values of upstream flow depth and for increasing velocities.
- Live-bed scour can occur under the smooth cover when the average velocity was less than the critical velocity for the sediment as derived for free surface conditions. For pressure flow conditions, the use of this critical velocity is not accurate.
- The pressurized flow condition accelerates rate of scour. This acceleration is greater for smaller upstream flow depths, all other factors held constant.
- Depths of scour for the smooth, fixed cover condition are similar to the depths of scour under the floating smooth cover condition. For both conditions, scour depth decreases with an increase in pressure head.

Hains included several recommendations for future study. One of these recommendations concerns the length of the ice cover upstream of the pier. In his study, the cover was present far enough upstream as to ensure the velocity profile approaching the pier was well established. This may not be the case in field conditions. If the cover is not long enough to induce uniform flow by the time the flow approaches the bridge pier, it may affect the scour mechanism at the pier.

Purpose, Objectives, and Scope of Work

From a review of literature pertaining to local scour at a bridge pier under pressure flow conditions due to an ice cover, it is clear that there have been no previous studies performed pertaining to the effect of the length of the ice cover on the scour depth at the pier. The purpose of this study is to observe the depth of local scour at a bridge pier under pressure conditions with different pressure-inducing ice cover lengths and make qualitative observations regarding any differences in the scour depths for these different cover conditions. By making these observations, a statement can be made regarding the impact that varying lengths of ice cover upstream of a bridge pier may have on the local scour at the pier itself. It is hoped that this study will serve as introductory work into this matter of length of pressure flow, and that the results of this work will lead to more in-depth studies.

The objectives of this study are as follows:

1. To collect data for equilibrium scour depth and scour depth vs. time for a fixed cover condition with four different lengths of cover upstream of the bridge pier in a laboratory flume.
2. To compare the scour depth results obtained from each of the different cover length conditions.
3. To collect velocity data from the two longest cover length conditions to create velocity profiles for flow underneath the cover and to compare these velocity profiles.

The scope of this study includes the following:

1. All of the tests are conducted for clear water scour.

2. All of the tests focus on local scour at a cylindrical bridge pier with a pressure-inducing cover attached that is meant to simulate an ice cover. The cover is fixed at one depth, which remains constant for all tests. Four different cover lengths are tested.

The scope of this study does not include the development of empirical equations to describe the depth of scour under pressure conditions, or the development of a scour hole profile at equilibrium. It is believed that further investigation should be performed before any equations are developed, and, if equations were developed from this study, they would be very limited in their applicability.

EXPERIMENTAL WORK

This chapter provides the details of the experimental work done in this study, including a description of the experimental setup, the instrumentation used, the testing matrix, and the general testing procedure.

Experimental Setup

Facilities

All of the work performed in this study utilized the tilting flume, shown in Figures 7 and 8, in the Fritz Engineering Laboratory at Lehigh University. The flume is 23 feet long, with a testing section of 15 feet, located between the 3 foot mark and the 18 foot mark. Prior to the 3 foot mark is a wooden entrance transition. Past the 18 foot, mark there is a 3 foot exit transition, followed by a 22 inch long lower section to allow for settling of sand and other objects that may move down the flume. The flume is 1.5 feet wide and 1.5 feet deep. The bottom 4.5 inches of the flume is covered with a sand bed, which makes the sand bed level with the entrance and exit transitions. The slope of the flume was kept constant at 0.0023 for the duration of the experimental work.

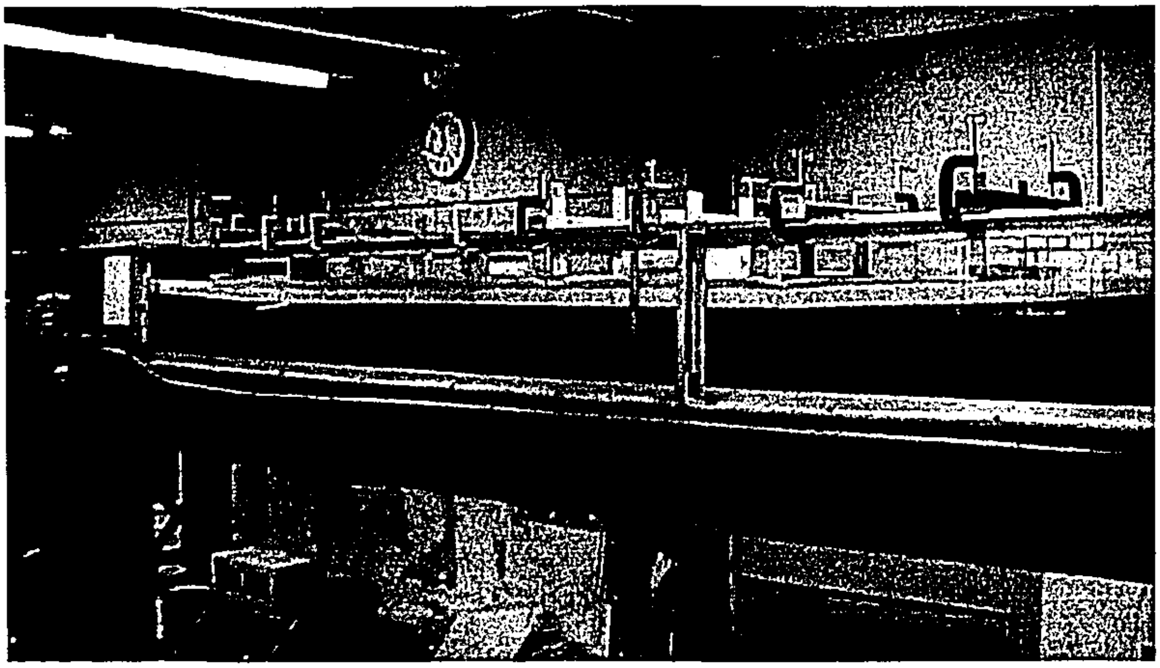

Figure 7: Tilting Flume, Fritz Engineering Laboratory, Lehigh University

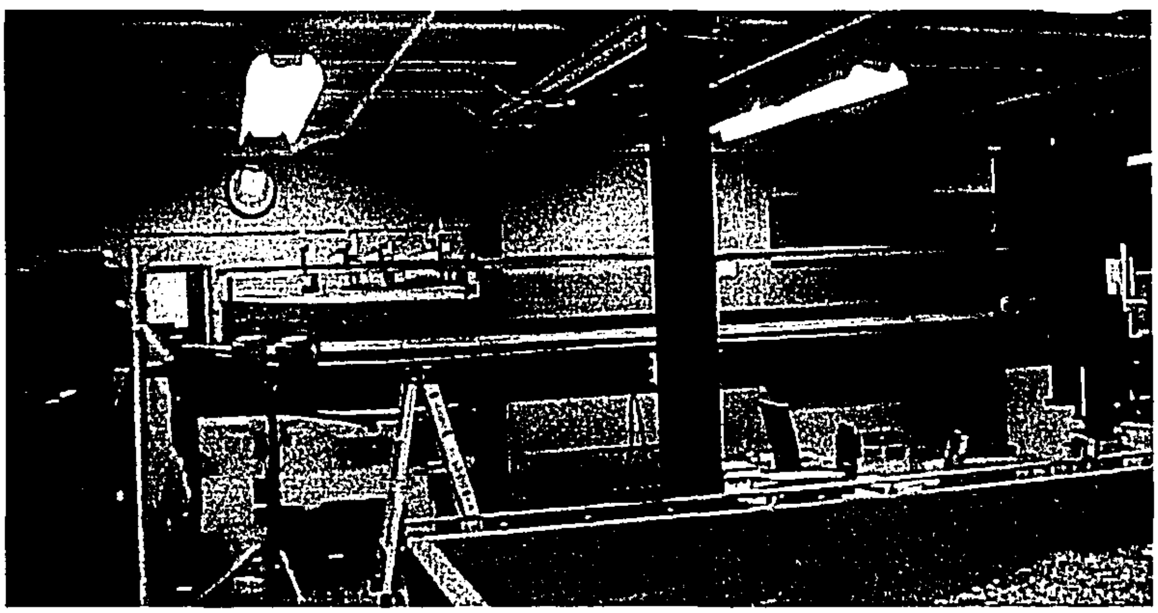

Figure 8: Tilting Flume, Fritz Engineering Laboratory, Lehigh University

A pump in the basement of Fritz Laboratory provides water from a sump underneath the building to a constant head tank, located on the second floor of the building. A series of pipes connects this head tank to the flume. Flow into the flume is regulated by a valve and measured by a venturi meter. Depth of flow in the flume is regulated by an adjustable tail gate at the downstream end of the flume. There is a sluice gate at the upstream end of the flume, which was left completely open and was not adjusted in the course of this experiment. A grate installed in the far upstream end up of the flume helps to straighten the flow and create turbulence. There is a second grate installed in the downstream end to prevent objects from flowing down into the sump.

All experimental work done for this study involves clear water scour. Care was taken during the course of the tests to ensure that sand was not moving to the end of the flume and possibly flow down into the sump.

The laboratory is not temperature regulated. Therefore, the temperature conditions that the water and ice would normally be under in the field were not present in this study. The temperature in the room was approximately room temperature, 70°F, and the water temperature was consistently 64° F.

Sediment Characteristics

The entire test section of 15 feet was filled with a 4.5 inch deep bed of uniform sand. The sand bed at this depth allowed the entire flume (bed, entrance transition, exit transition) to be at the same elevation. This depth also sufficiently allowed for scour to occur at the bridge pier without hitting the bottom of the flume.

A sieve analysis determined that the median grain diameter is $d_{50} = 0.42$ mm. The grain size distribution is shown in Figure 9.

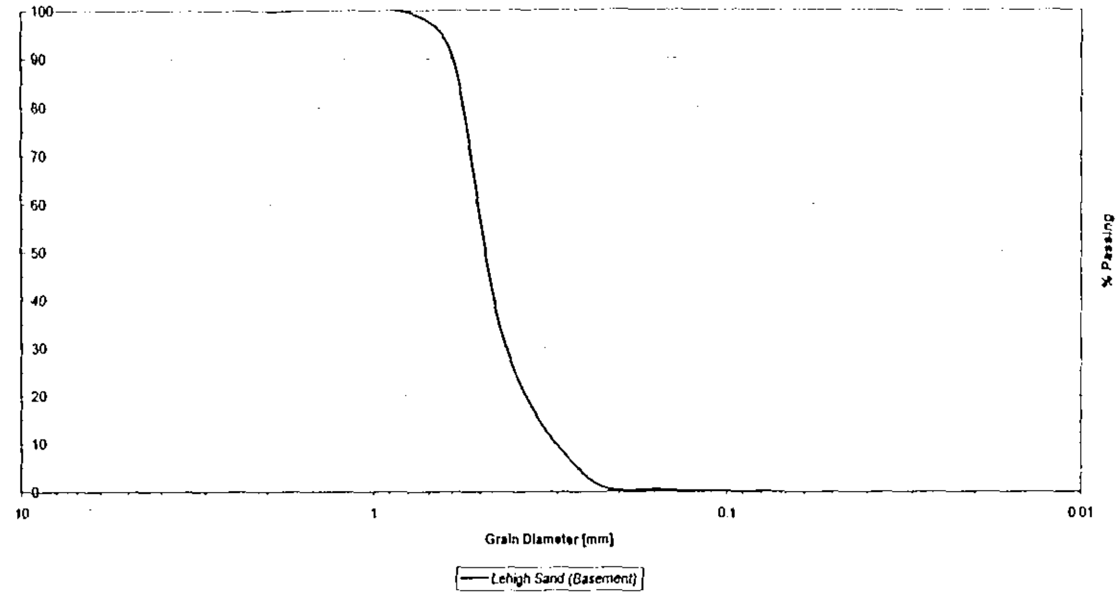

Figure 9: Grain Size Distribution, $d_{50} = 0.42$ mm

To plan for the appropriate flow rates to use in the course of testing, it was necessary to know the critical velocity of the sediment for the open water case. Since the focus of this work is on clear water scour only, the velocity of the water must be kept below the critical velocity so that live bed scour does not occur. The critical velocity for the open water condition can be determined in two ways: through the use of calculation methods such as Yang's criteria (Yang, 1973) or through in situ testing.

Three different empirical or graphical methods were used to determine the critical velocity for a sand grain with a mean diameter of 0.42 mm, including Shields diagram (Shields, 1936) with Yang's Criteria (Yang, 1973), the Hjulstrom diagram (Hjulstrom, 1935), and the Vanoni diagram (Vanoni, 1977).

In the first method, the Shields diagram, Figure 10, is used to determine the critical Reynolds number:

$$Re_* = U_* d/\upsilon$$

where U* = the shear velocity, d= d_{50} = the median grain diameter, and υ = the kinematic viscosity. This Reynolds number is obtained by calculating the Shields Diagram third parameter, as shown in Figure 10, and reading the corresponding Reynolds number off the Shields diagram. For a median grain diameter of 0.42 mm, a water temperature of 64° F, a kinematic viscosity of 1.1131×10^{-5} ft^2/s, and a sediment specific gravity of 2.65, the third parameter is equal to 10.6, which yields a Re* of approximately 6.7.

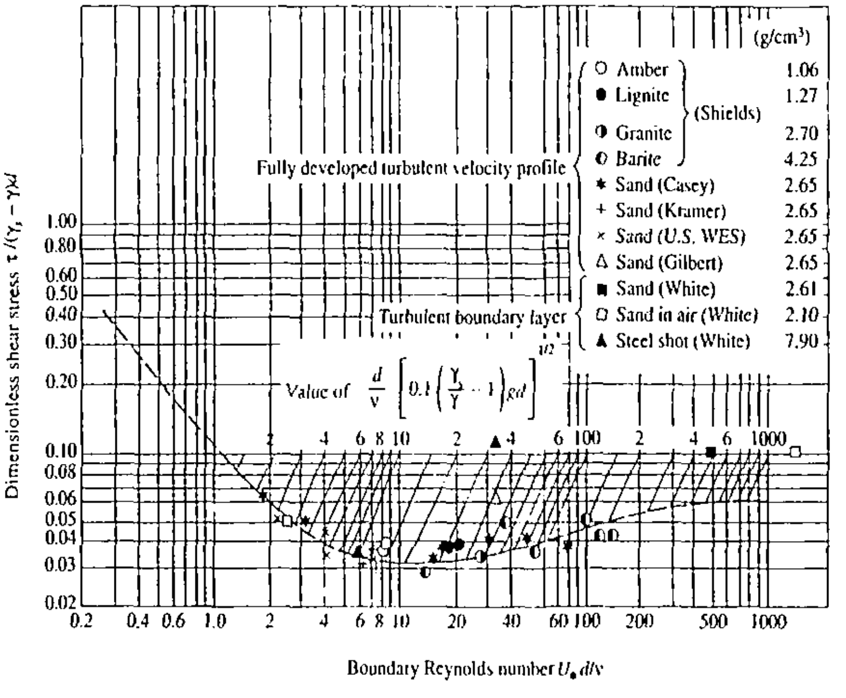

Figure 10: Shields Diagram (Yang, 1996)

The Reynolds number is used as part of Yang's criteria to determine the dimensionless parameter of critical velocity over fall velocity, V_c/ω. From Yang's diagram, shown in Figure 11, V_c/ω is approximately 4.

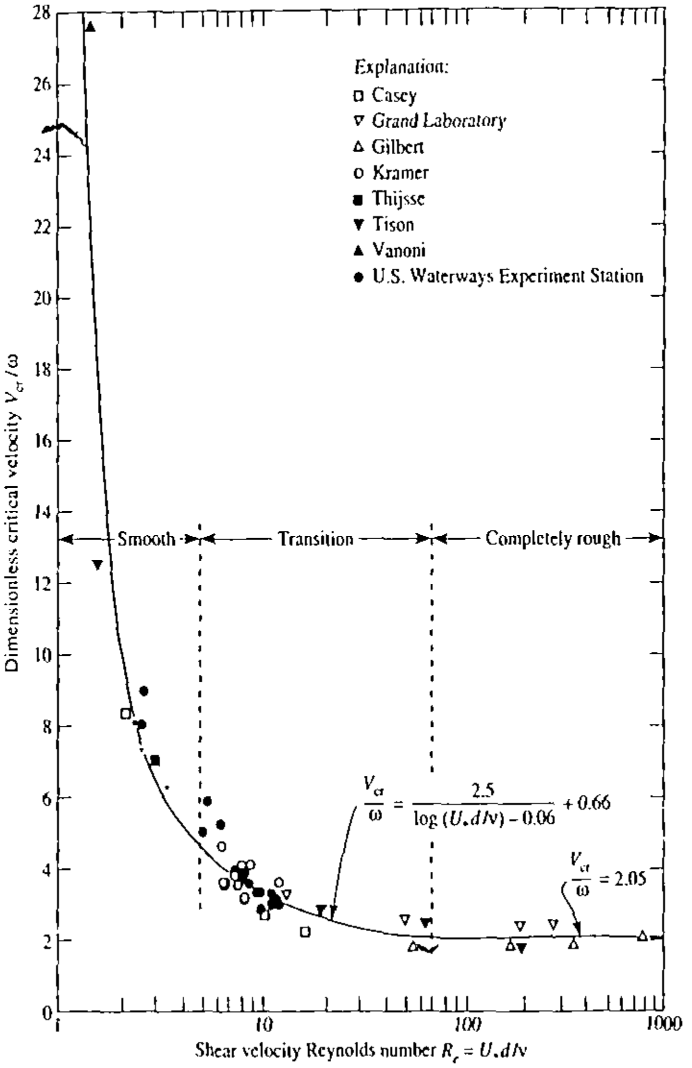

Figure 11: Yang's Criteria for Critical Velocity (Yang, 1996)

To determine the critical velocity, the fall velocity is calculated using Rubey's formula (Rubey, 1933):

$$\omega = F\left[dg\left(\frac{\gamma_s - \gamma}{\gamma}\right)\right]^{1/2}$$

where:

ω = fall velocity

$d = d_{50}$ = grain size diameter

g = acceleration due to gravity

γ, γ_s = specific weight of the water and the sediment, respectively and F is a parameter that, for sediment smaller than 1mm in water temperatures between 10° and 25° C, is:

$$F = \left[\frac{2}{3} + \frac{36v^2}{gd^3(\gamma_s/\gamma - 1)}\right]^{1/2} - \left[\frac{36v^2}{gd^3(\gamma_s/\gamma - 1)}\right]^{1/2}$$

where v is the kinematic viscosity. For the given parameters, F= 0.656, which gives a fall velocity of 0.177 ft/s. This yields a critical velocity of 0.71 ft/s.

The Hjulstrom and Vanoni diagrams are both graphical methods, shown in Figures 12 and 13, respectively. The Hjulstrom method yields a critical velocity range of 0.623 to 0.722 ft/s. The Vanoni method yields a critical velocity range of 0.5 to 0.88 ft/s, with a mean of 0.65 ft/s.

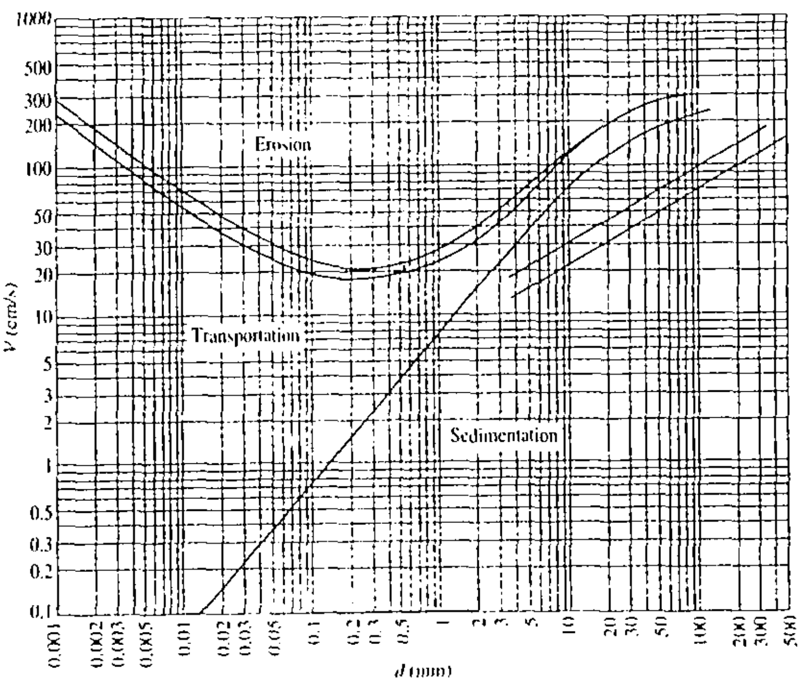

Figure 12: Erosion-Deposition Criteria for Uniform Particles (Hjulstrom, 1935)

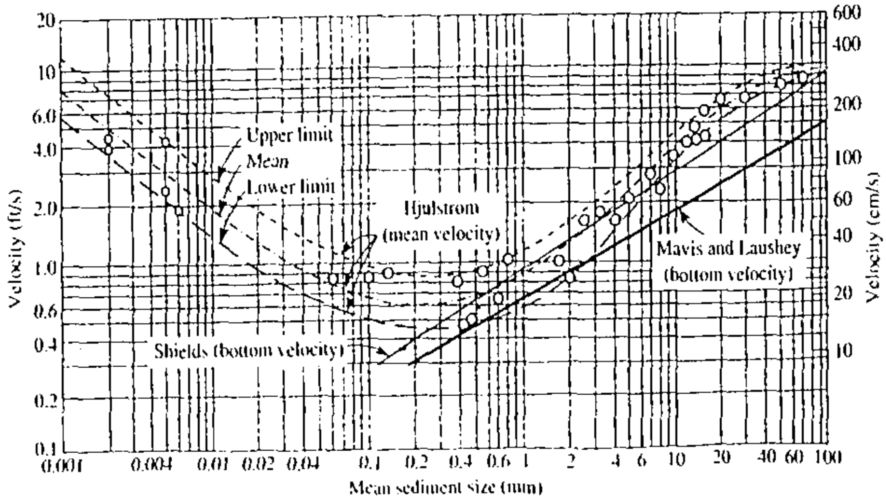

Figure 13: Critical Water Velocities for Sediment as a Function of Mean Grain Size (Vanoni, 1977)

An in-situ test was performed using the actual sediment, flume, and water conditions that were used in this study. The flume was filled with water, and then the flow rate was turned up to a value that could be clearly discerned by the venturi meter. The depth of flow was slowly lowered. When "incipient motion" occurred, the depth of flow was recorded, and the average critical velocity was calculated. "Incipient motion" was defined at the point where the movement of sediment on the bed could be visually observed. This visual observation took place at a point approximately 5.5 feet downstream from the start of the test section. The in-situ test was repeated for a series of three trials, yielding average critical velocities of 0.935 ft/s, 0.950 ft/s, and 0.935 ft/s. These values were averaged to obtain an average critical velocity of 0.94 ft/s.

The results of the critical velocity analysis are as follows:

Method	Critical Velocity
Shields w/ Yang	0.711 ft/s
Hjulstrom	0.623 - 0.722 ft/s
Vanoni	0.5 - 0.88ft/s
In Situ Test	0.94 ft/s

Table 1: Critical Velocity Analysis Results

These results aided in choosing the flow rates that were used in the course of this study. One flow rate was chosen to provide a velocity at the pier near to but not exceeding the critical velocity.

Test Setup

The pier used in this study is a cylindrical glass tube, with a 1.25 inch outer diameter. The tube is situated 14 feet from the flume entrance and 11 feet from the beginning of the test section and is centered in the cross-section.

A tape measure, graded in inches, was attached on the upstream side of the pier, inside of the cylinder. The use of the tape measure allows for the change of the elevation of sand directly in front of the pier to be observed.

A diagram of the important dimensions of the test set up is shown in Figure 14.

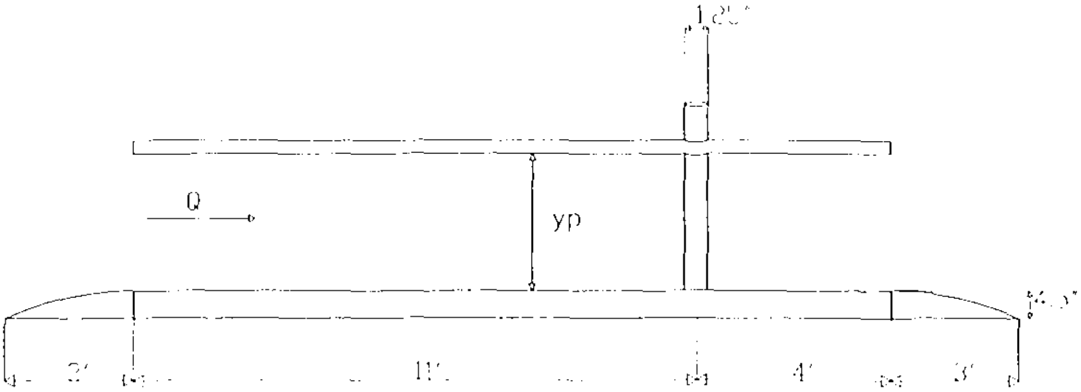

Figure 14: Test Set Up Dimensions

Simulated Ice Cover

To simulate an ice cover, sheets of Styrofoam insulation panels were used as a suitable substitute for ice due to its ability to float and its use in past work (Hains, 2004;

Olsson, 2000). Sheets were cut to the desired length and width to fit inside the flume and provide the appropriate cover lengths.

A hole was drilled in the appropriate length of Styrofoam to allow the bridge pier to fit through. Additionally, holes were drilled in the pieces designated for the 100% long and 50% long cases so that velocity profiles could be taken. Still wells were constructed over all of these holes using Styrofoam and rubber cement. These still wells contain water and prevent flow over the top of the Styrofoam. Additionally, for all length cases except for the 100% case, the cover thickness was increased at the upstream end of the Styrofoam cover to prevent significant flow overtopping in the higher head condition tests.

Cover Bracing System

All of the tests performed in this study were in a fixed cover condition. The Styrofoam cover was fixed at a point 5 inches above the sand bed and was not allowed to rise or fall with a change in flow rate. To accomplish this condition, a bracing system was installed consisting of a series of wooden braces and C-clamps as well as duct tape. Prior to the start of a test, the cover was placed on the surface of the water, which was at a constant depth of 5 inches above the sand bed and a very low flow rate. Duct tape was attached along the sides of the cover and the wall to hold the cover in place and prevent a significant amount of seepage through the gap. A range of four to eight braces, depending on the length of the cover, was placed on top of the cover and held down using C-clamps to prevent the cover from rising up when the flow rate was increased at the start of the experiment. The first brace and the last brace were raised up an extra ½ inch using wooden shims. This was to allow a "transition zone" into the pressure flow condition. The shims were not used on the 10% cover condition. The brace set up is shown in Figures 15 and 16.

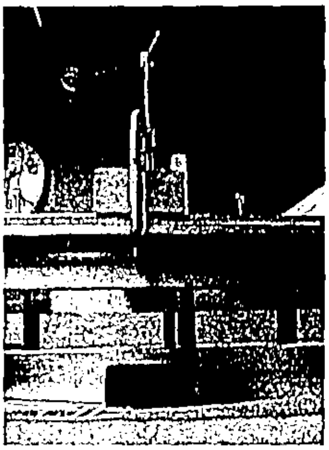

Figure 15: Brace Set Up

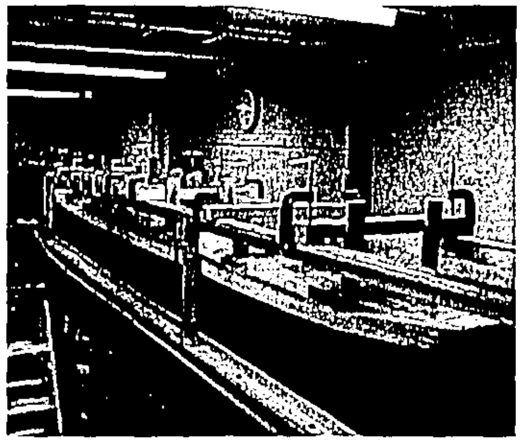

Figure 16: Brace Set Up

Instrumentation

Scour Depth Measurement

A small tape measure was installed on the inside of the bridge pier on its upstream side to observe the changes in the bed elevation at the pier due to scour mechanisms. To read the tape measure, a small dentist's mirror on a telescopic handle was used. The handle could be pulled out to its full length, and the mirror angle adjusted so that it could be visible from the top of the pier. A reading was taken by inserting the mirror down into the inside of the pier to see both the tape and the sand-water interface and record the elevation of that interface. A halogen lamp was set up adjacent to the flume and was turned on whenever a reading was taken. This aided the data acquisition by illuminating the flume, which allowed the sand-water interface to be seen clearly. Use of the halogen lamp was especially helpful when the water was cloudy.

No attempts were made to measure the shape and profile of the scour hole. However, after each test, the scour hole was photographed with a scale in order to estimate the length and width of the hole.

Velocity Profile Measurement

Having a velocity probe and a hole in the cover close to the pier would have an effect on the flow pattern at the pier and thus affect the scour mechanism. Therefore, velocity profiles were only taken for the 100% cover and 50% cover cases. For the 100% case, two velocity profiles were taken, one 5 feet from the bridge pier and one 2.5 feet from the bridge pier. For the 50% case, only one profile was taken, 2.5 feet from the bridge pier. These values were chosen because they are approximately the same ratio of distance (0.45 to 0.47) from the data acquisition point to the upstream end of the cover.

The velocity profiles were taken with a 2-dimensional SonTek Acoustic Doppler Velocimeter (ADV). The ADV was braced above a hole drilled in the cover that is centered over the flume and inserted through that hole into the water. It was initially positioned 1 cm above the sand bed and aligned with the flow. The software used in conjunction with the ADV controls how the ADV takes readings and records those readings. The program was set to collect one velocity data point per second. The program is allowed to collect data points for 30 seconds. The ADV is then raised by 1 cm, and another 30 seconds of data points are collected. This process continues until the ADV sensor reaches the underside of the cover. Mark points were inserted in the data at the start of every 30 second collection period. A second software program was used to properly format the data. All velocity profile data can be seen in Appendix B. The set up of the ADV system is shown in Figure 17.

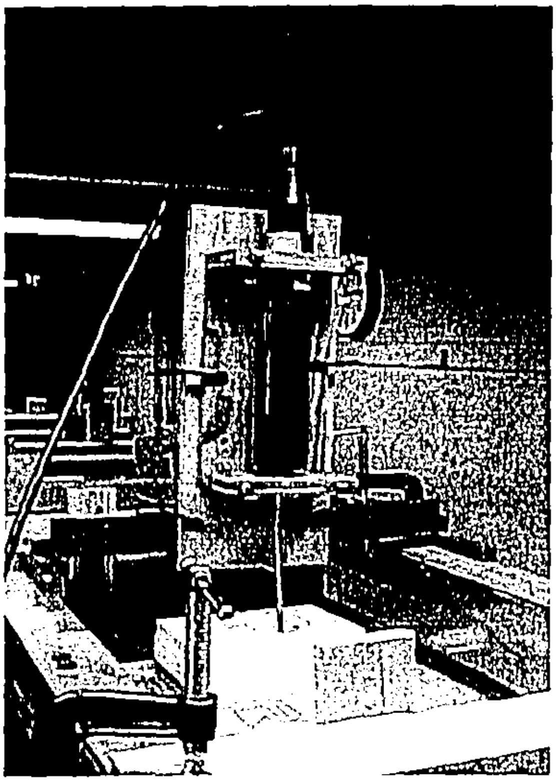

Figure 17: ADV Set Up

For all of the tests, velocity readings were taken to check the flow rate reading obtained from the venturi meter. These readings were taken with a Marsh-McBirney portable water current meter. The data from this segment of the experiment are included with the scour test data, which can be found in Appendix A. All velocity data were taken after the fourth hour of testing.

Testing Matrix

In this experimental work, the variables used in each test were the ice cover length, the flow rate, and the depth of flow upstream of the cover. Four different cover lengths, two flow rates, and two depths of flow were used. A summary is shown in Table 2:

Test #	Cover [ft]	Cover [%]	Q	Q [cfs]	y_1	y_1 [inches]
1	11	100	low	0.38	low	7
2	11	100	low	0.38	high	9
3	11	100	high	0.46	low	7
4	11	100	high	0.46	high	9
5	5.5	50	low	0.38	low	7
6	5.5	50	low	0.38	high	9
7	5.5	50	high	0.46	low	7
8	5.5	50	high	0.46	high	9
9	2.75	25	low	0.38	low	7
10	2.75	25	low	0.38	high	9
11	2.75	25	high	0.46	low	7
12	2.75	25	high	0.46	high	9
13	1.1	10	low	0.38	low	7
14	1.1	10	low	0.38	high	9
15	1.1	10	high	0.46	low	7
16	1.1	10	high	0.46	high	9

Table 2: Testing Matrix

The four lengths covered the range of possibilities that could occur in the field, from a long cover to a short one that is akin to a submerged bridge deck. The cover percentages are based on the longest test section of 11 feet upstream of the pier. One flow rate was chosen to provide a velocity near to but not exceeding the critical velocity underneath the cover; and a second flow rate was lower and would also provide clear water scour. The two upstream flow depths were chosen to provide an observable difference in scour depth, and also because of the limitations of the depth that could be provided in the flume.

General Procedure

Prior to the start of each test, the sand bed was leveled. The leveler was attached to the top lip of the flume and was pushed along to level all of the sand bed to the same elevation. A hand-held wooden block was used to level the sand around the pier. The leveling procedure was performed while there was a small flow rate in the flume, either right after the water was turned on to start a test or after the water was turned off when a test was over.

Once the bed was completely leveled, the flume was allowed to fill to a depth of 5 inches above the sand bed. The flow rate was kept very low to prevent any scour at the pier. When the flume was filled to a depth of 5 inches, the sand around the pier was leveled again with the hand held block if necessary. Then a reading was taken for the initial depth prior to scour. The Styrofoam cover was then placed in the flume on top of the water, duct tape was applied to the sides of the cover and the flume to prevent seepage between the two, and the bracing system was installed to hold the cover in place. To install the bracing system, all of the braces were placed on the cover, and C-clamps were tightened two at a time on each side of the flume. One half inch deep shims were used on the first and last braces for all tests except for the 10% cover tests. For the 10% cover tests, duct tape was also applied to the front of the dam at the far upstream end of the cover to prevent seepage and overtopping. Figures 18 through 21 show the four different cover conditions.

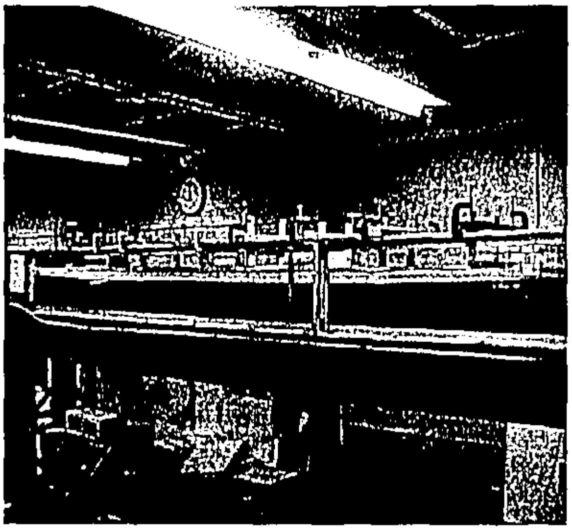

Figure 18: 100% Cover

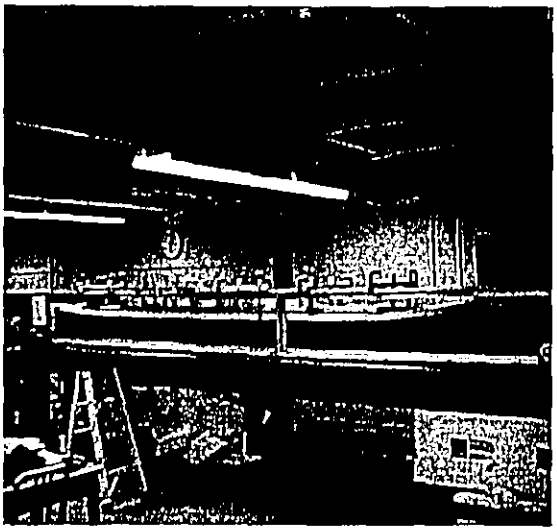

Figure 19: 50% Cover

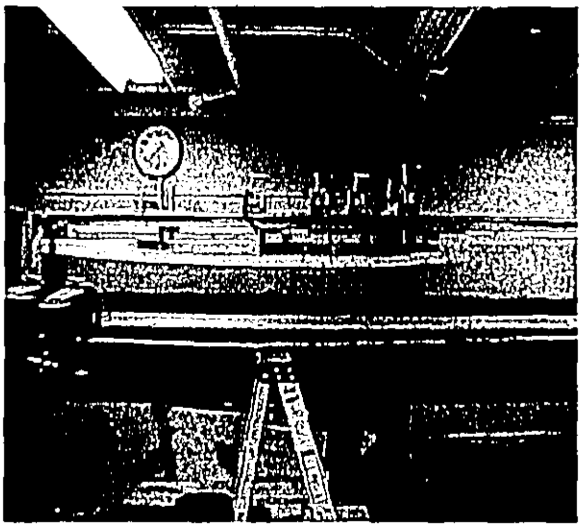

Figure 20: 25% Cover

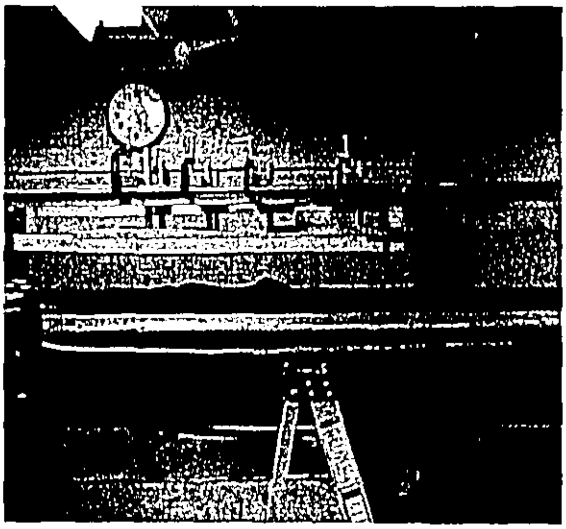

Figure 21: 10% Cover

When the cover was completely installed, the flow rate was adjusted to the desired value. This was accomplished by opening the valve controlling the flow rate and then checking the flow as determined by the venture meter. This was repeated until the desired flow rate was reached.

Next, the depth of flow upstream of the cover was adjusted to the desired level by raising the tail gate at the end of the flume and checking the depth of flow at the far upstream end of the flume and at the halfway point. When both of these readings were within ¼ inch of the desired water level, it was considered the appropriate depth. After this entire set up process was complete, another scour depth reading was taken. This reading was considered the reading at time equal to zero. Readings were taken at time equal to zero and prior to the installation of the cover so that any scour that occurred during the setup could be accounted for.

During the test, readings were taken every ten minutes for the first two hours of the testing period using the scour depth measurement procedure described above. After two hours, readings were taken once every hour. The tests were allowed to run until the scour depth reached equilibrium, which was defined for this study as nine hours or three consecutive hours in which the scour depth had not changed. If the data collection schedule was changed from the description above for any reason, it was noted in the test data, shown in Appendix A.

After the first four hours of testing, velocity profiles using the SonTek Acoustic Doppler Velocimeter for the 100% and 50% cases, as well as Marsh-McBirney current meter readings for all cases were taken, as per the procedure described above. These readings were taken to check the flow rate reading obtained from the manometer, as well as provide velocity profiles for some of the tests.

In addition, the temperature of the water was recorded for each test. Digital photographs of the flume and its changing scour conditions were taken occasionally throughout the course of the study. Once a test was completed and the cover removed, photographs were taken of the scour hole at the pier. Photographs were also taken of the scour hole with a scale, so that width and length of the scour hole could be observed.

Additionally, photographs were taken of any bed forms that may have occurred during the course of testing, such as dunes and entrance effects. Requests to view the photographs can be directed to the author of this report.

At the conclusion of each test, the valve was closed, the pump was shut off, and the cover was removed. The flume was allowed to completely drain prior to the start of the next test.

RESULTS AND DISCUSSION

The results of this study and a discussion are presented below. Nineteen tests were performed: sixteen as designated by the test matrix, two as a check for the results obtained in the prior tests, and one that resulted in live bed scour and was aborted. The full test results for the scour analysis are included in Appendix A. The full test results for the velocity profiles taken for the 100% covered cases and the 50% covered cases are shown in Appendix B.

Scour Depth Analysis

Equilibrium Scour Depth

All tests, except the aborted attempt, were allowed to run for 9 hours. This time was long enough for the scour depth to reach equilibrium based on observations of the rate of change of scour depth. Table 3 is a summary of the tests and their corresponding equilibrium scour depths, in which L is the length of the ice cover in feet, L_f is the length of the test section of the flume upstream of the pier equal to 11 feet, Q is the flow rate in cfs, y_1 is the depth of flow upstream of the ice cover in inches, and y_s is the equilibrium depth of scour in inches.

Test #	L/L_f [%]	Q [cfs]	y_1 [inches]	y_s [inches]
1	100	0.46	7	1.0000
2	100	0.46	9	1.4375
3A	100	0.38	7	0.8750
4	100	0.38	9	1.2500
5	50	0.46	7	1.8125
6	50	0.46	9	2.0625
7	50	0.38	7	1.8750
8	50	0.38	9	1.3125
9	25	0.38	9	0.9375
9A	25	0.38	9	1.4375
10	25	0.38	7	1.5625
11	25	0.46	7	1.8750
11A	25	0.46	7	1.8750
12	25	0.46	9	1.7500
13	10	0.38	7	1.8125
14	10	0.38	9	1.4375
15	10	0.46	7	1.7500
16	10	0.46	9	1.8125

Table 3: Summary of Measured Results

Test 3A is a repeat of Test 3, which was the aborted test. Tests 9A and 11A are repeats of Tests 9 and 11, respectively. Test 9 was repeated because it was believed that there may have been an error in the data; upon analyzing the data, the scour depth for Test 9 did not fit in with the rest of the data collected for all the other tests. Test 9A provided a scour depth that better followed the trend of the other data, and it is now believed that Test 9A is a better reflection of the scour mechanism that occurs under the given cover, flow rate, and pressure head conditions. Test 11A was performed to ensure that the testing procedure was repeatable and that the scour depths measured during this study were confirmable.

Photographs were taken of the scour hole that formed during all tests. A typical scour hole is shown in Figures 22 and 23.

Figure 22: Overhead View of Scour Hole

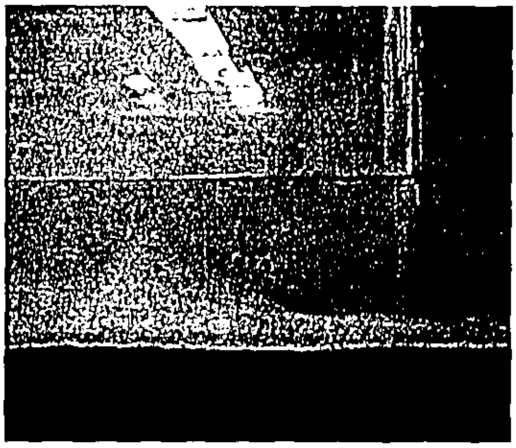

Figure 23: Side View of Scour Hole

Dimensionless Terms

To analyze the data, non-dimensional terms are used. In this analysis, four dimensionless parameters are used: L/y_p, y_s/b, y_1/y_p, and V_p/V_c. L/y_p is the dimensionless length of the ice cover, and is defined as the actual length of the ice cover over the depth of flow at the pier. The term y_s/b is the dimensionless scour depth, where y_s is the scour depth in inches and b is the diameter of the bridge pier (1.25 inches). The term y_1/y_p, the pressure head ratio, compares the flow depth upstream of the ice cover, y_1, to the flow depth at the pier, y_p. The term V_p/V_c, the flow intensity, describes the ratio of the average velocity under the fixed cover, V_p, to the critical average velocity of the sediment for open water conditions, V_c =0.94 ft/s. All the dimensionless terms for this study are shown in Table 4.

Test #	L [ft]	L/y_p	Q [cfs]	y_1 [inch]	V_p [ft/s]	y_s [inch]	y_s/b	V_p/V_c	y_1/y_p
1	11	26.4	0.46	7.0	0.736	1.0000	0.8	0.783	1.4
2	11	26.4	0.46	9.0	0.736	1.4375	1.15	0.783	1.8
3A	11	26.4	0.38	7.0	0.608	0.8750	0.7	0.647	1.4
4	11	26.4	0.38	9.0	0.608	1.2500	1	0.647	1.8
5	5.5	13.2	0.46	7.0	0.736	1.8125	1.45	0.783	1.4
6	5.5	13.2	0.46	9.0	0.736	2.0625	1.65	0.783	1.8
7	5.5	13.2	0.38	7.0	0.608	1.8750	1.5	0.647	1.4
8	5.5	13.2	0.38	9.0	0.608	1.3125	1.05	0.647	1.8
9	2.75	6.6	0.38	9.0	0.608	0.9375	0.75	0.647	1.8
9A	2.75	6.6	0.38	9.0	0.608	1.4375	1.15	0.647	1.8
10	2.75	6.6	0.38	7.0	0.608	1.5625	1.25	0.647	1.4
11	2.75	6.6	0.46	7.0	0.736	1.8750	1.5	0.783	1.4
11A	2.75	6.6	0.46	7.0	0.736	1.8750	1.5	0.783	1.4
12	2.75	6.6	0.46	9.0	0.736	1.7500	1.4	0.783	1.8
13	1.083	2.6	0.38	7.0	0.608	1.8125	1.45	0.647	1.4
14	1.083	2.6	0.38	9.0	0.608	1.4375	1.15	0.647	1.8
15	1.083	2.6	0.46	7.0	0.736	1.7500	1.4	0.783	1.4
16	1.083	2.6	0.46	9.0	0.736	1.8125	1.45	0.783	1.8

Table 4: Dimensionless Values and Results

Cover Length, L/y_p

The cover length is the most important parameter to consider in this study. Figure 24 shows the non-dimensional scour depth vs. the cover length.

It can be seen from this figure that the scour depths for the tests corresponding with $L/y_p = 26.4$, the highest value of L/y_p tested, are lower than the scour depths at all the other L/y_p values for the same flow intensity and pressure head ratio. This means that, for tests where the flume was covered for the full test section, the scour depths that occurred were shallower than when the cover was shorter. This indicates that there is a length of cover where the scour mechanism changes.

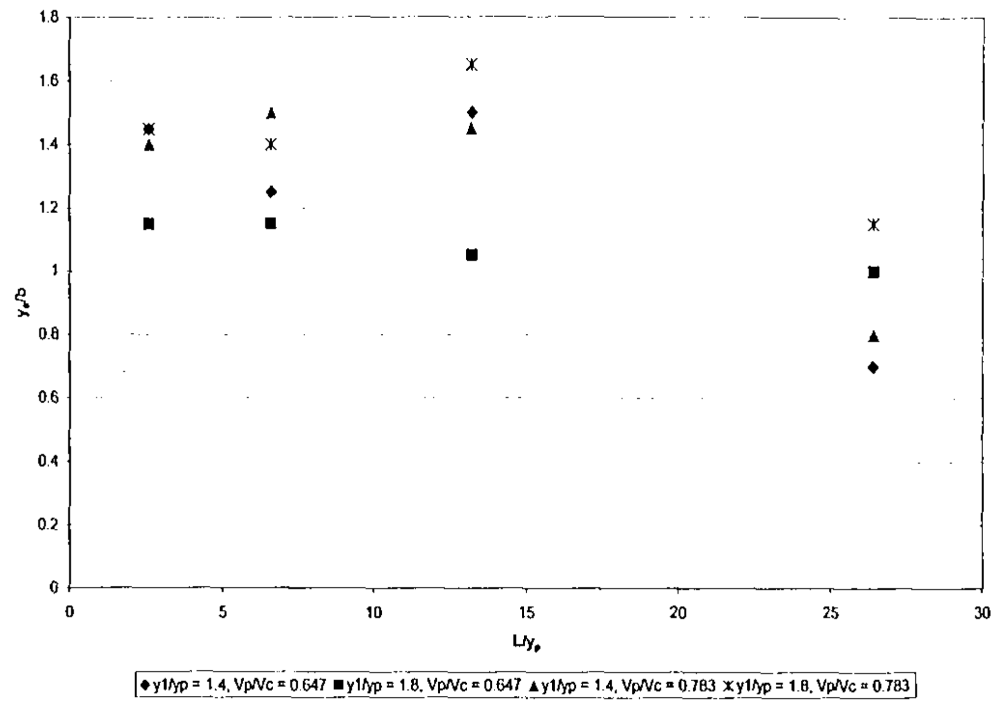

Figure 24: Scour Depth vs. Cover Length

The largest range of scour depth values occurs at $L/y_p = 13.2$. The ranges for the other L/y_p's measured are smaller and similar to each other. This again indicates that there may be a cover length where the scour mechanism changes. It is possible that at the smaller L/y_p values, the scour mechanism acts like that of a submerged bridge deck case. In the case of the larger L/y_p values, it is possible that the velocity profile has become fully developed and thus there is a corresponding scour mechanism. For intermediate L/y_p values, the scour mechanism does not correspond to either the submerged bridge deck case or a fully developed case; thus the scour depths that occur may be larger and more varied.

Flow Intensity and Pressure Head Ratios

Two different values of flow intensity, V_p/V_c, were used in this study, 0.647 and 0.783. The value of the flow intensity for the test aborted due to live bed scour was

0.936. Likewise, two different values of the pressure head ratio, y_1/y_p, were used, 1.4 and 1.8. Figures 25 and 26 show the comparison of y_s/b vs. L/y_p between the two flow intensity values used for each pressure head ratio.

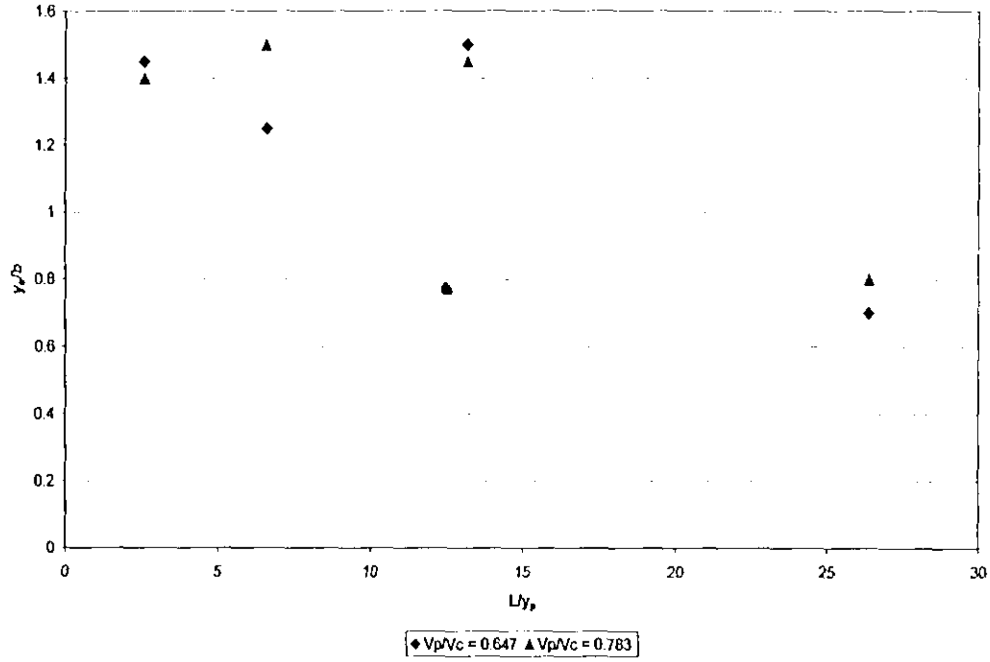

Figure 25: Scour Depth vs. Cover Length in Terms of Flow Intensity ($y_1/y_p = 1.4$)

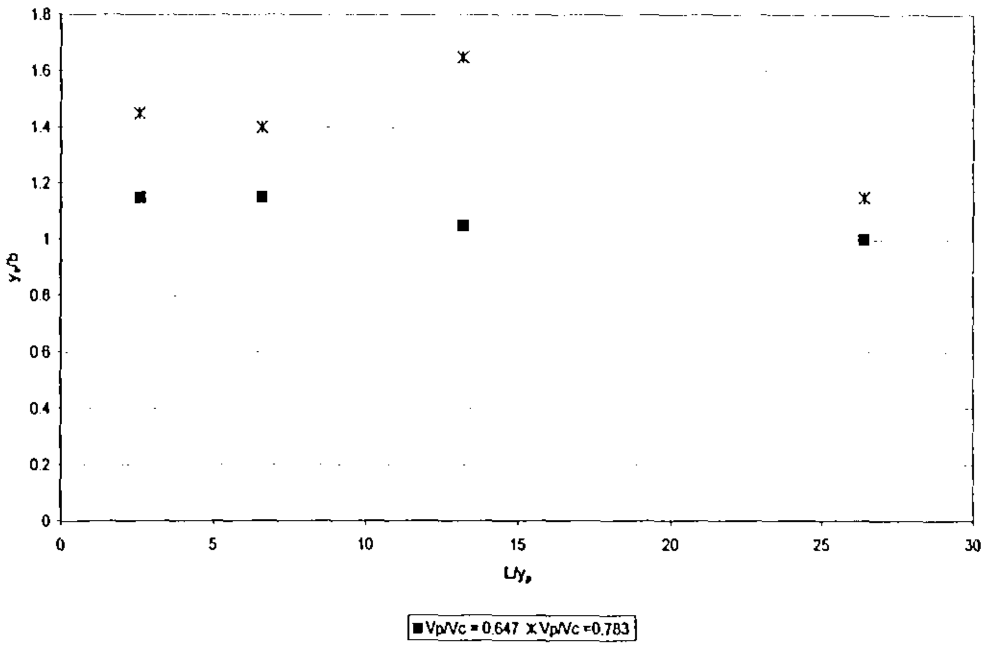

Figure 26: Scour Depth vs. Cover Length in Terms of Flow Intensity ($y_1/y_p = 1.8$)

In the case of $y_1/y_p = 1.8$, it is clear that, for the higher flow intensity of $V_p/V_c = 0.783$, a greater depth of scour occurs. For the lower value of $y_1/y_p = 1.4$, the higher flow intensity causes a significantly greater scour depth than the lower flow intensity for $L/y_p = 26.4$ and 6.6. For $L/y_p = 13.2$ and 1.083, the scour depths for both flow intensities are nearly identical. From this data, it is concluded, generally, that a higher flow intensity will lead to a higher depth of scour.

It should be noted that, during the tests run at $V_p/V_c = 0.783$, bedforms occurred downstream of the pier. A study of these bedforms was not included in the scope of this research; however, future investigations could measure these bedforms. A typical dune formation that occurred during the course of this study is shown in Figure 27.

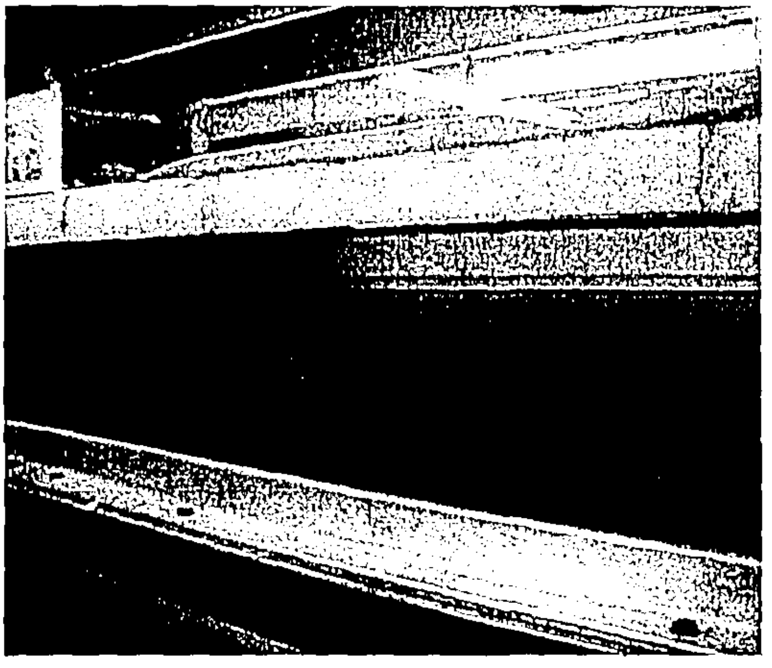

Figure 27: Bedforms

Figures 28 and 29 show the comparison of y_s/b vs. L/y_p between the two pressure head ratios for each flow intensity.

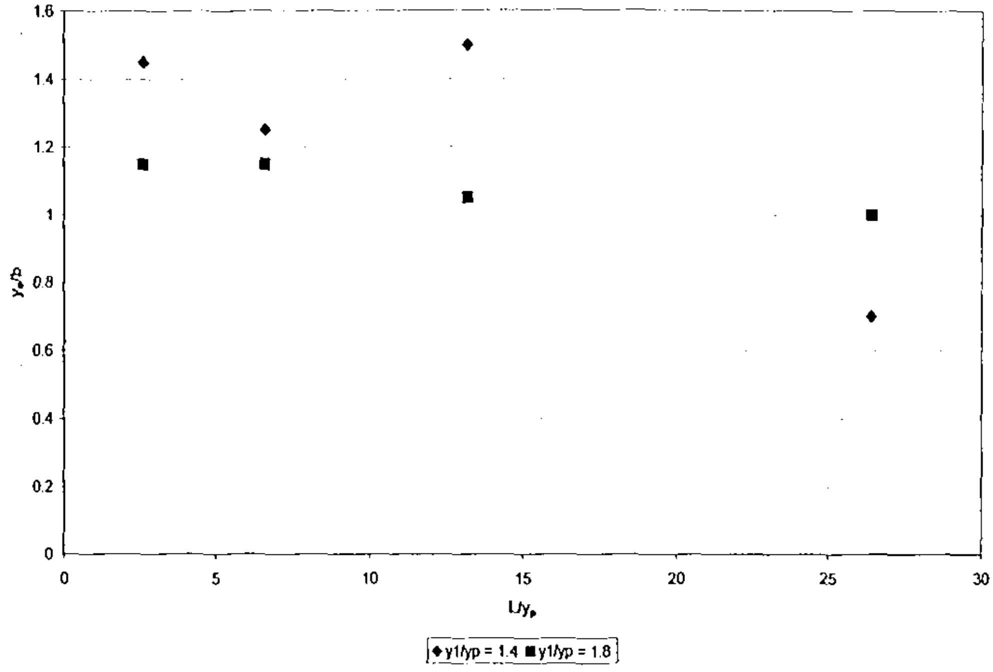

Figure 28: Scour Depth vs. Cover Length in Terms of Pressure Head Ratio ($V_p/V_c = 0.647$)

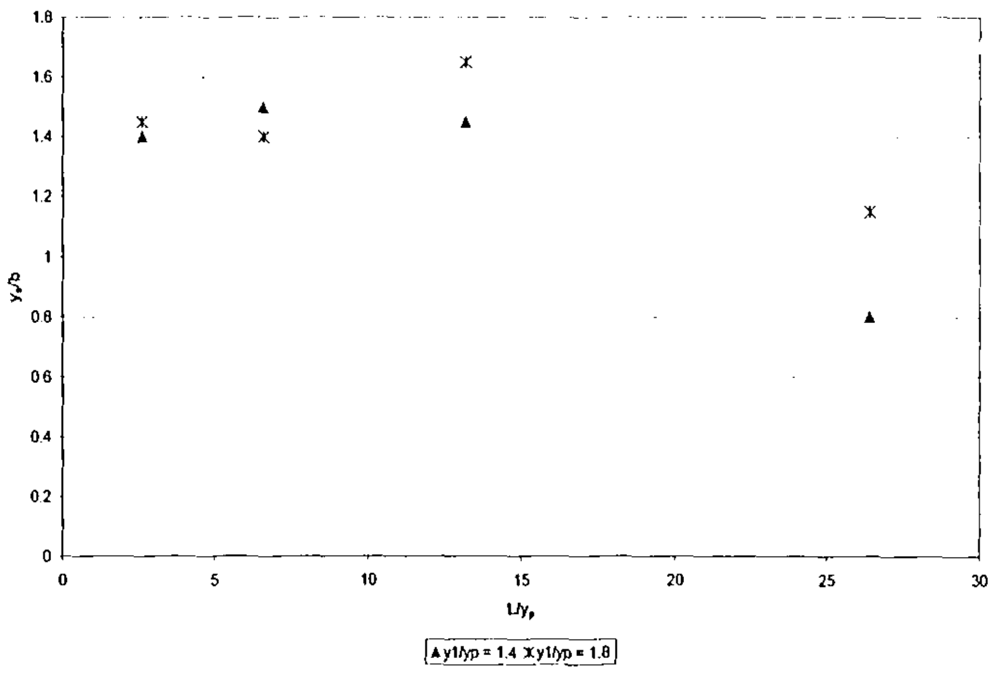

Figure 29: Scour Depth vs. Cover Length in Terms of Pressure Head Ratio ($V_p/V_c = 0.783$)

For the lower value of $V_p/V_c = 0.647$, the lower value of $y_1/y_p = 1.4$ provides a greater scour depth for all values of L/y_p except for the largest value of $L/y_p = 26.4$. For the higher value of $V_p/V_c = 0.783$, the higher value of $y_1/y_p = 1.8$ provides a greater scour depth for all values of L/y_p except for $L/y_p = 6.6$; however, for that case the scour depths are similar. This indicates that the scour mechanism may change in both a situation of changing flow intensity and a situation of changing cover length.

By analyzing the two pressure head ratios, the scour depth results obtained appear to be changing with flow intensity. For the lower flow intensity, an increase in the depth ratio shows a decrease in the equilibrium scour depth. However, for the higher flow intensity, the opposite occurred: an increase in depth ratio gives an increase in equilibrium scour depth. A hypothesis can be made that the depth of flow upstream of the ice cover may have an effect on the scour depth that occurs with respect to the flow intensity. In terms of the changing cover length, it is noted that, for the lower flow intensity, the higher scour rate occurs for the higher flow depth ratio at $L/y_p = 26.4$, opposite of the lower L/y_p cases for the same flow intensity and depth ratio. It is possible that for $L/y_p = 26.4$, the flow has reached fully developed conditions and the flow intensity is more important to the scour mechanism than the flow depth. However, it is recommended that more research be made into the effect of pressure head and cover length on the depth of scour before any definitive conclusions are made.

Comparison of 100% Cover Case with Previous Studies

In his study, Hains (2004) performed tests measuring the scour depth at a cylindrical bridge pier under a fixed ice cover condition. These tests can be compared to the tests performed in this study that were covered for 100% of the upstream flume length. Table 5 shows a comparison between the constants used in Hains' study vs. those used in this study.

	L [ft]	B [ft]	b [inch]	y_p [inch]	d_{50} [mm]	V_c [ft/s]
Hains	78.0	4	2.00	9.00	0.13	0.90
Miranda	11.0	1.5	1.25	5.00	0.42	0.94

Table 5: Comparison of Constants in Hains (2004) and Miranda (2004)

Hains performed six tests that utilized the 100% fixed cover case. He varied both the flow intensity and the flow depth ratio in his experiment. The results of his study are shown in Table 6.

Test	L/y_p	y_s [cm]	y_s/b	V_p [m/s]	V_p/V_c	y_1 [cm]	y_1/y_p
C1	53.3	7.983	1.571	22.40	0.8167	30.48	1.33
C2	53.3	8.255	1.625	25.48	0.9278	30.48	1.33
C3	53.3	7.303	1.438	25.48	0.9278	38.10	1.67
C4	53.3	7.983	1.571	22.40	0.8167	38.10	1.67
C5	53.3	8.255	1.625	23.56	0.8589	30.48	1.33
C6	53.3	8.096	1.594	23.56	0.8589	38.10	1.67

Table 6: Results of Hains (2004)

A comparison of Hains' data to the data collected in this study is shown in Figure 30. The graph shows a plots y_s/b vs. V_p/V_c in terms of y_1/y_p. The L/y_p values vary greatly between the two studies, since the flume used in Hains' study was more than seven times longer than the flume used in this study. The V_p/V_c values are also higher in Hains' study.

The scour depths obtained in Hains' study are all higher than those obtained in this study. This difference is probably due to the difference in the flow intensity values, as the values used in Hains' study are higher. It should be noted that for the two tests that correspond with $V_p/V_c = 0.9278$, live bed scour occurred. In this study, live bed scour occurred at $V_p/V_c = 0.9361$, which is consistent with Hains' test.

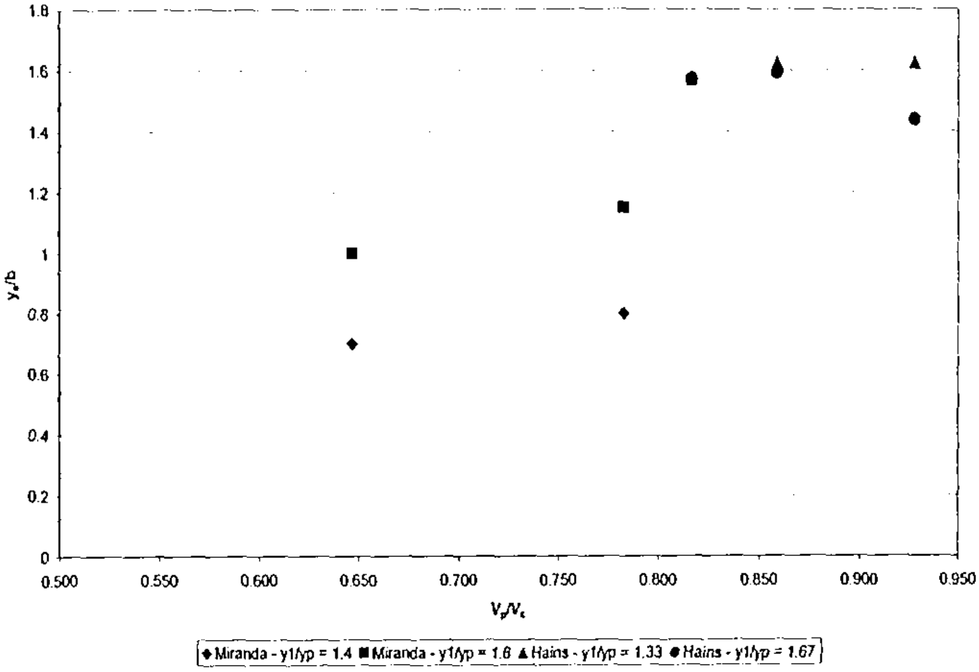

Figure 30: Comparison of Hains (2004) and Miranda (2004), Scour Depth vs. Flow Intensity

It was hoped that comparing these two studies would yield more consistent results, so that the difference in scour depths between fully developed, transition, and short L/y_p values would be further confirmed. Instead, comparing these two studies has shown an increase in scour depth with a doubling of the L/y_p value. It is possible that the 100% cover value of $L/y_p = 26.4$ used in this study is not long enough to provided fully developed flow as previously discussed, but is still in the transition range between a uniform pressure flow and a short submerged bridge deck situation.

Velocity Profile Analysis

The full test results for the velocity profiles taken for the 100% covered cases and the 50% covered cases are shown in Appendix B. The point velocity in the x-direction vs. y was found by collecting velocity data at one second intervals for thirty seconds and averaging those readings. Velocities were taken at 1.0 cm intervals over the flow depth beneath the cover. The point velocities were non-dimensionalized by dividing them by the critical velocity, $V_c = 0.94$ ft/s. These velocity profiles were taken in order to make some qualitative assessments about differences in the profiles as well as to confirm the flow rates used in this study as determined with the venturi meter.

100% Cover Cases

Three sets of data for the 100% cover ($L/y_p = 26.4$) case were taken in this study. All of the data were collected during tests where $V_p/V_c = 0.647$. Two locations were used for the test where $y_1/y_p = 1.4$, 5 feet and 2.5 feet from the bridge pier. One point was taken for the test where $y_1/y_p = 1.8$, 5 feet from the bridge pier. The dimensionless velocity profiles for the $L/y_p = 26.4$ cases are shown in Figure 31.

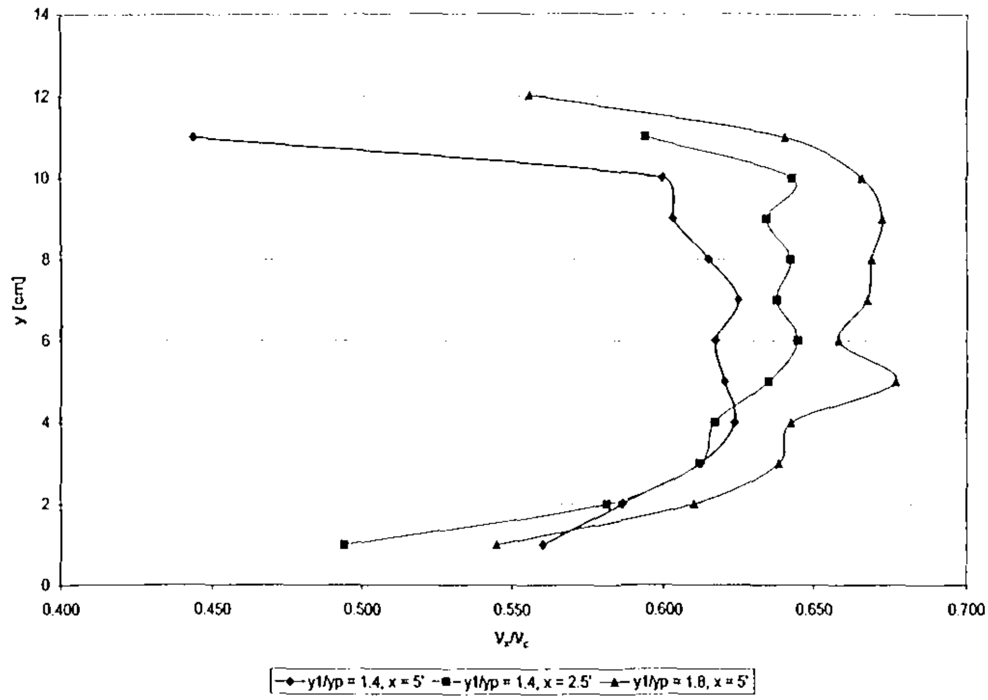

Figure 31: Velocity Profiles for L/y_p =26.4

For L/y_p = 26.4, it appears that the y_1/y_p = 1.8 has a slightly greater velocity profile. This may contribute to the higher scour depth for this flow depth ratio than at y_1/y_p = 1.4 for the same cover length and flow intensity. There is also a shift in the velocity profile for the y_1/y_p = 1.4 profiles as the data moves from the point farthest from the pier to the closer point. This may indicate that the flow mechanism is changing between the two points and that the distance provided in the flume may not be long enough to create fully developed flow.

50% Cover Cases

Four sets of data for the 50% cover (L/y_p = 13.2) case were taken. Data was collected for both V_p/V_c = 0.647 and V_p/V_c = 0.783, and for both flow depth ratios at these flow intensities. All data was taken at a point 2.5 feet from the bridge pier. The dimensionless velocity profiles for the L/y_p = 13.2 cases are shown in Figure 32.

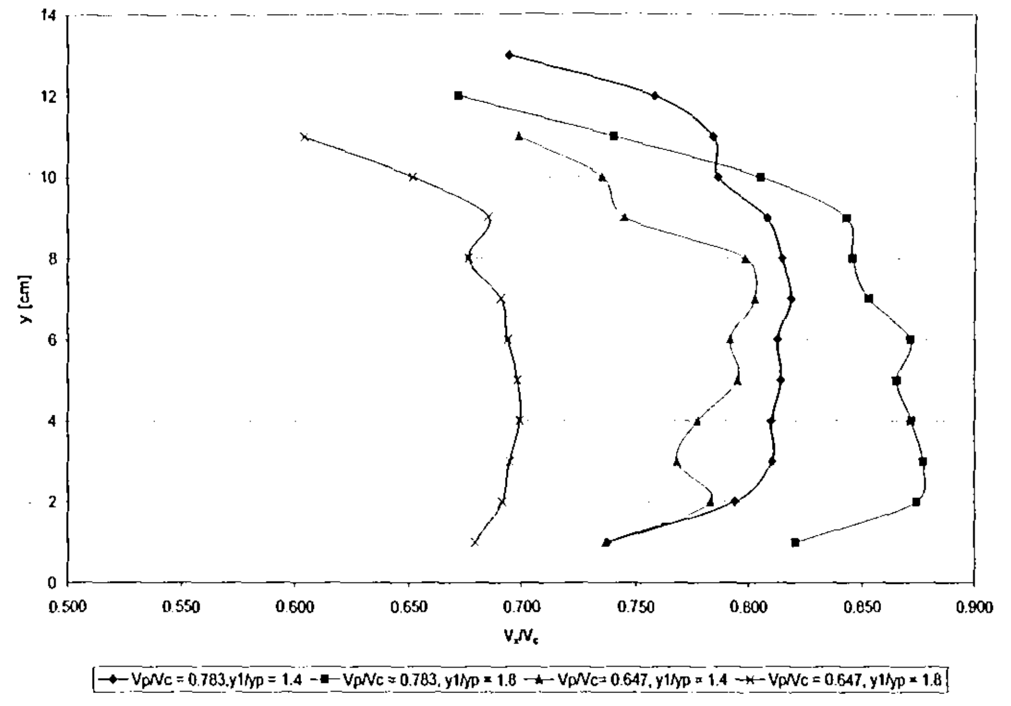

Figure 32: Velocity Profiles for $L/y_p = 13.2$

It can be seen that the maximum velocity for each profile is at a lower value of y for $y_1/y_p = 1.8$ at each flow intensity. Hains (2004) concluded that the location of this maximum velocity may have an impact on the scour depth that occurs; however, from the data collected in this study, no definitive statements can be made on this issue. It is believed that the effects of the flow depth ratio need to be investigated further.

Velocity Profiles to Determine Flow Rate

Each velocity profile was analyzed to determine its associated flow rate. The area beneath each profile was determined using the trapezoidal method. This yielded a factor with the units of area over time. This factor was multiplied by the width of the flume, B = 1.5 ft, in order to obtain the flow rate. It should be noted that the area under the velocity profile multiplied by the flume width creates inaccuracy, as the velocity across the width of the flume varies, with a maximum in the center. The flow rates obtained from the velocity profile analysis are shown in Table 7:

Test	Q – ADV [cfs]	Q –Venturi [cfs]
1	0.308	0.38
2	0.321	0.38
3	0.341	0.38
4	N/A	0.38
5	0.424	0.46
6	0.444	0.46
7	0.401	0.38
8	0.355	0.38

Table 7: Flow Rates as Obtained from ADV vs. Venturi Meter

It can be seen in Table 7 that, although the results are not identical, they are close. It is already known that an inaccuracy lies in not using a velocity profile in the z-direction, and slight discrepancies may exist in the initial velocity collected (i.e. the ADV sensors were not exactly aligned with the flume). Therefore, it is believed that the confirmation between the flow rate as determined by the ADV and as determined by the venturi is good and that the flow rates and velocities that have been used throughout this report are accurate.

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

In previous studies (Abed, 1991; Jones, et. al., 1993; Arneson 1997; Umbrell, et. al., 1998; Jones, et. al., 1999; Hains, 2004), the presence of a pressure flow condition on the local scour at a bridge pier was shown to alter the scour mechanism and increase the scour depth that occurs. However, none of these studies varied the upstream length of the pressure-inducing ice cover to determine if the cover length had an influence on the scour mechanism, although this investigation is recommended in both a memorandum to HEC-18 (Matthews, 2002) and in Hains' study (2004). To determine the effects of the upstream length of a fixed ice cover on local scour at a bridge pier, an experimental study was performed at the Fritz Engineering Laboratory at Lehigh University. Nineteen tests were performed using a 1.25 inch cylindrical bridge pier and sand with $d_{50} = 0.42$ mm. Two different discharges and two different pressure head conditions were used. Four different cover length conditions were studying, ranging from 100% to 10% of the flume test length.

Three major research objectives were accomplished during this study:

1. To collect data for equilibrium scour depth and scour depth vs. time for a fixed cover condition with four different lengths of cover upstream of the bridge pier in a laboratory flume.
2. To compare the scour depth results obtained from each of the different cover length conditions.
3. To collect velocity data from the two longest cover length conditions to create velocity profiles for flow underneath the cover and to compare these velocity profiles.

Summary of Conclusions

Effect of Cover Length

The dimensionless cover length, L/y_p, was varied in this study from a value of 26.4 to 1.083. The largest value of $L/y_p = 26.4$ yielded the shallowest values of scour depth for four conditions of varying flow intensity and flow depth ratio. The other values of L/y_p (13.2, 6.6, 1.083) all yielded scour depths that were similar to each other. The deepest scour depth occurred at $L/y_p = 13.2$ for the highest flow rate and pressure head. The effective length $L/y_p = 13.2$ also provided the widest range of scour depths.

It is surmised that a change in cover length does provide a change in the flow mechanism, thus affecting the depth of scour that occurs. Three different flow regimes are proposed: a short cover length, which acts like a submerged bridge deck; a long cover length, which allows the flow beneath it enough distance to become fully developed; and a transition length, where the flow is not yet developed but the cover is too long to be comparable to a submerged bridge deck. It was initially thought that the $L/y_p = 26.4$ was in the long cover condition, while the other three L/y_p's were either in the short or transition regime. However, in comparing the data collected in this study to the data from Hains' study (2004), it is now thought that $L/y_p = 26.4$ is also in the transition range. Hains' $L/y_p = 53.3$ and his higher values of V_p/V_c provided larger scour depths of scour than the current study.

Effect of Flow Intensity

In this study, two different flow intensities were used, $V_p/V_c = 0.647$ and $V_p/V_c = 0.783$. A third value of $V_p/V_c = 0.9361$ was attempted, and live bed scour occurred. It was shown that, in general, a higher flow intensity will provide a greater scour depth, no matter what the pressure head or cover length situation.

Effect of Pressure Head

In this study, two different pressure head ratios were used, $y_1/y_p = 1.4$ and $y_1/y_p = 1.8$. For the lower flow intensity value, the lower pressure head ratio provided the greatest scour depth in all cover length cases except $L/y_p = 26.4$. For the higher flow intensity value, the higher pressure head ratio provided the greatest scour depth for a majority of the cover length cases tested. The change in scour depth with head between $L/y_p = 13.2$ and 26.4 for the lower flow intensity may indicate a change in scour mechanism with changing pressure head with respect to effective cover length. The change in the dominant pressure head condition between the two flow intensities may also indicate a change in scour mechanism with changing pressure head with respect to flow intensity. It is clear that the effect of pressure head on the scour mechanism is dependent upon other factors such as flow intensity and effective cover length, and that these pressure head effects warrant continued research in order to further understand them.

Velocity Profiles

Velocity profiles were taken for two $L/y_p = 26.4$ tests and four $L/y_p = 13.2$ tests. For $L/y_p = 26.4$, two profiles were taken at a point 5 feet upstream of the pier and one profile was taken at a point 2.5 feet upstream of the pier. For $L/y_p = 13.2$, all four profiles were taken 2.5 feet upstream of the pier. The velocity profiles provided a sufficient way to check the flow rates that were indicated by the venturi meter. These profiles confirmed that the flow rates and thus the flow intensities that were used in the analysis of the scour depth data were accurate.

Other than confirming the flow rates that were used in the course of this study, the velocity profiles were not consistent enough to provide any meaningful relationships to this study. For the $L/y_p = 13.2$ velocity profiles, a shift in the maximum velocity point is discernable with the change in pressure head. However, not enough is known about the effects of pressure head on the scour depth to make any definitive conclusions from this profile shift.

Improvements to this Research

This study was meant to serve as an introductory study into the effects of upstream ice cover length. Since it was shown in this study that cover length does affect the scour mechanism, it is recommended that this research be repeated in order to gather more comprehensive data and develop further relationships between scour depth and cover length. However, improvements can be made to this experiment in order to obtain more significant data. Suggested improvements to this research are as follows:

- A longer flume should be used. This will allow for the testing of longer cover lengths so that a discernable break between fully developed flow and transition flow can be found. Second, a deeper flume should be used so that greater pressure heads can be studied. Third, a wider flume can allow for greater variation in pier widths if one wishes to examine that parameter.

- Facilities with more easily adjustable flow rates should be used. In the course of this experiment, the flow rate was found by opening and closing a valve and checking the deflection on the manometer. Since the flows used in this experiment were on the steep portion of a rating curve, it was difficult to consistently achieve the same flow rate. The ability to more accurately set the flow rate would remove this source of error.
- Temperature-regulated facilities should be used. In this study, the laboratory had no temperature regulation, and the water in the flume was consistently at 64° F. This does not adequately represent the temperature condition in the field where a fixed ice cover is present. If a temperature-regulated facility is used, actual field conditions can be more accurately portrayed.
- The width of the scour hole and the scour profile should be taken. In the field, there may be a series of bridge piers or a combination of bridge piers and abutments. It would be useful to determine if a fixed ice cover influences the scour hole width or the downstream bed enough so that other bridge structures might be affected.
- More velocity profiles should be taken. The velocity profiles taken here, although useful for confirming the flow rate, do not provide any significant data to make other conclusions. More velocity profiles need to be taken, both in terms of repeating profiles for the same test conditions and taking several profiles across the cross-section of the flume. Also, it would be useful to determine a way to take velocity profiles for the shorter cover lengths without being obtrusive to the flow pattern directly in front of the pier. Finally, more care should be taken in

obtaining the velocity profiles; it should be ensured that the readings are taken at the proper depth in the flume and that the sensor is properly aligned with the flow.

- In general, more data needs to be collected. This includes data from a greater variety of effective cover lengths, flow intensities, and pressure heads. Although a range of data was taken in this study, there are many blanks that need to be filled in, most importantly greater effective cover lengths and greater values of pressure head.
- Finally, tests need to be repeated. Several data points for scour depth need to be collected at each condition in terms of effective cover length, flow intensity, and pressure head. This is to confirm the data already collected and to form more solid relationships between effective cover length and scour depth.

By repeating the experiments performed in this study with these suggested improvements, it is possible that more conclusive data will be obtained. This data can be used to further understand the relationship between the cover length and the corresponding scour depth in hopes that some design criteria can be formed. Also, by repeating the experiments, it is possible that other relationships involving scour depth and other ways to improve the experiment will become highlighted.

Recommendations for Future Research

There are other factors that should be considered to better understand the scour mechanism that occurs under a fixed ice cover. These factors were outside of the scope of this study, and therefore are recommended as future research. These recommendations are as follows:

- Investigation of cover roughness should be performed. The cover roughness was investigated by Hains (2004) in some of his tests. He compared the differences between a smooth cover and one rough cover. It is recommended that different cover roughness be investigated and that more data regarding this factor are collected in general.
- A more extensive look at the effects of pressure head should be made. Pressure head does play a role in the scour mechanism that occurs; however, the actual effects are not clear. More data should be collected isolating the pressure head effects.
- Different grain sizes should be investigated, as only one was used in this study.
- At the higher flow intensity used in this study, bedforms occurred downstream of the pier. A study regarding the formation of these bed forms and how they may affect other structures downstream of the subject bridge pier is recommended.
- Tests should be performed utilizing different sizes, shapes, and alignments of bridge piers. Tests should also be performed for bridge abutments.
- The actual flow mechanism beneath the ice cover needs to be further investigated. This should be accomplished by taking many velocity profiles upstream of the pier. Also, if they can be taken in an unobtrusive way, velocity profiles should be taken in the vicinity of the bridge pier in order to see how the combination of the pier and the ice cover affect the flow mechanism.

In summary, it is shown in this study that the length of a fixed ice cover does have an effect on the local scour that occurs at a bridge pier. However, in order to form relationships between cover length and scour depth, more data for a greater variety of flow and pressure head situations need to be taken. Additionally, several improvements to this experimental work can be made, and more factors that were not within the scope of this study can be researched. It is believed that this topic is worth continued research in the future so that improvements to scour design procedures for ice covered situations can be made.

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APPENDIX A: SCOUR TEST RESULTS

This appendix contains the data that was collected during the course of performing this experiment. The results are presented in numerical order of tests, and the tests are numbered chronologically. Tests denoted with an "A" are repeated tests. The data presented for each test is as follows: a table of scour vs. time data, a table of parameters measured during the course of the experiment, any notes that were made during the course of the experiment, and a graph of the scour vs. time data.

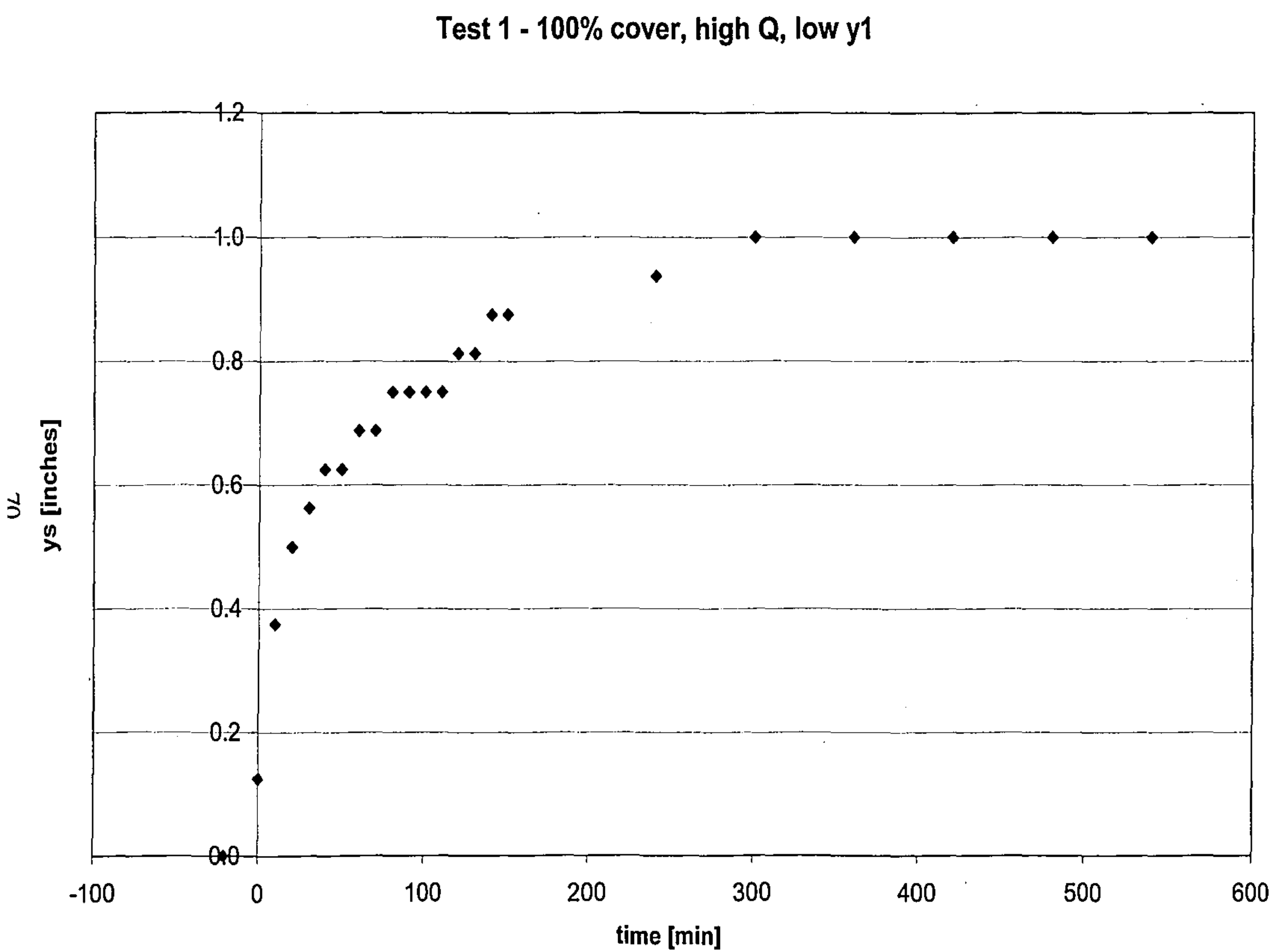
Photographs of all tests were taken during this study. These photographs are not presented in this report. Requests to view the photographs can be made to the author of this report.

Test No.: 1
Date: 12/10/2003

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
9:00	5 3/16	-0:21	-21	0.0000
9:21	5 1/16	0:00	0	0.1250
9:31	4 13/16	0:10	10	0.3750
9:41	4 11/16	0:20	20	0.5000
9:51	4 5/8	0:30	30	0.5625
10:01	4 9/16	0:40	40	0.6250
10:11	4 9/16	0:50	50	0.6250
10:21	4 1/2	1:00	60	0.6875
10:31	4 1/2	1:10	70	0.6875
10:41	4 7/16	1:20	80	0.7500
10:51	4 7/16	1:30	90	0.7500
11:01	4 7/16	1:40	100	0.7500
11:11	4 7/16	1:50	110	0.7500
11:21	4 3/8	2:00	120	0.8125
11:31	4 3/8	2:10	130	0.8125
11:41	4 5/16	2:20	140	0.8750
11:51	4 5/16	2:30	150	0.8750
13:21	4 1/4	4:00	240	0.9375
14:21	4 3/16	5:00	300	1.0000
15:21	4 3/16	6:00	360	1.0000
16:21	4 3/16	7:00	420	1.0000
17:21	4 3/16	8:00	480	1.0000
18:21	4 3/16	9:00	540	1.0000

$L =$	11	ft
$L_f =$	11	ft
$L/y_p =$	26.4	
$B =$	1.5	ft
$Q =$	0.46	cfs
$y_1 =$	7	inch
$y_p =$	5	inch
$V_{avep} =$	0.736	ft/s
$V_1 =$	0.492	ft/s
$V_2 =$	0.525	ft/s
$V_{ave} =$	0.5085	ft/s
$Q_v =$	0.445	cfs
$S =$	0.0023	
$T =$	64	°F

Notes: Setup time was 21 minutes


Test 1 - 100% cover, high Q, low y1
ys [inches]
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-100
0
100
200
300
400
500
600
time [min]

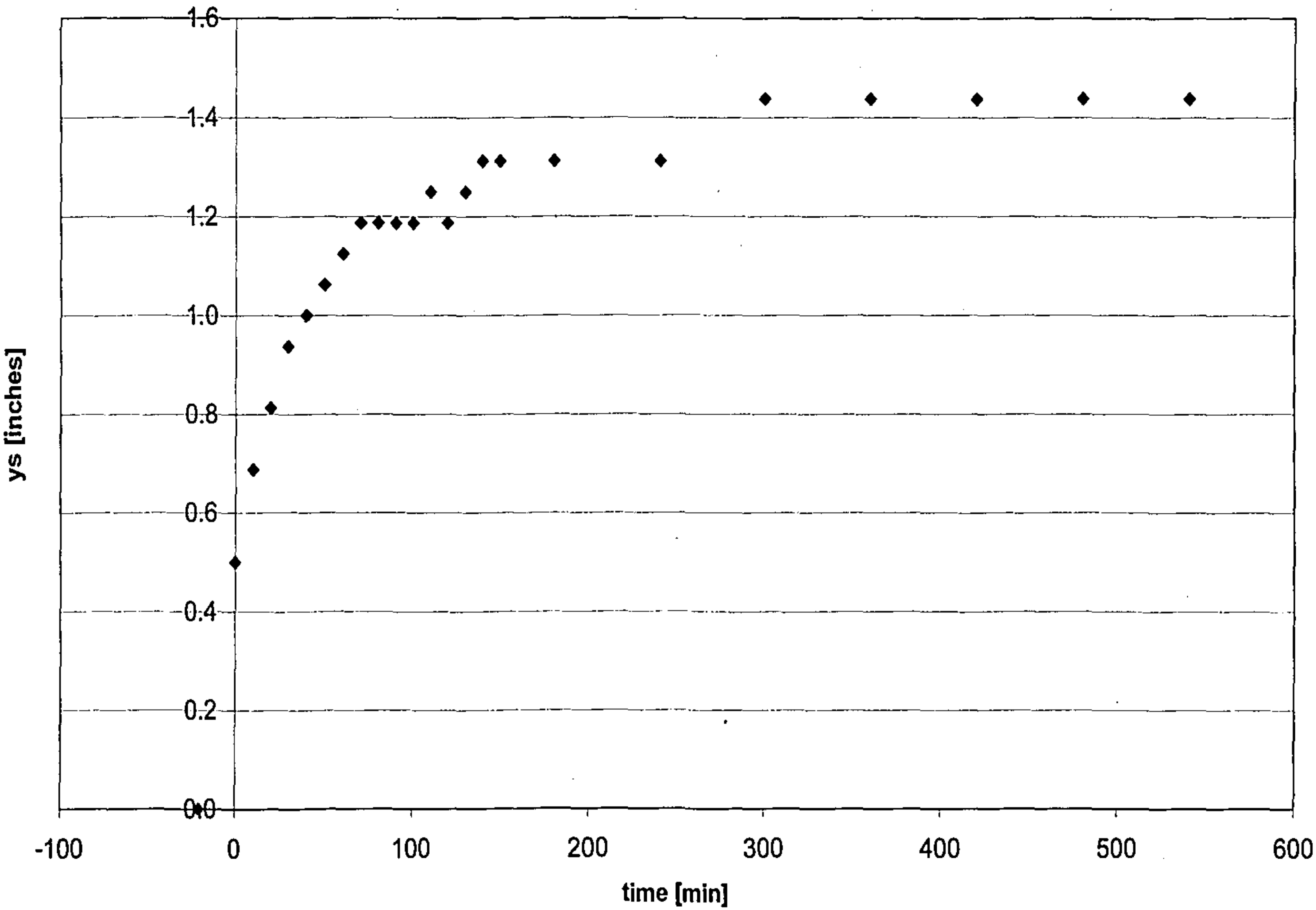
Test No.: 2
Date: 12/11/03

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
8:50	5 5/16	-0:21	-21	0.0000
9:11	4 13/16	0:00	0	0.5000
9:21	4 5/8	0:10	10	0.6875
9:31	4 1/2	0:20	20	0.8125
9:41	4 3/8	0:30	30	0.9375
9:51	4 5/16	0:40	40	1.0000
10:01	4 1/4	0:50	50	1.0625
10:11	4 3/16	1:00	60	1.1250
10:21	4 1/8	1:10	70	1.1875
10:31	4 1/8	1:20	80	1.1875
10:41	4 1/8	1:30	90	1.1875
10:51	4 1/8	1:40	100	1.1875
11:01	4 1/16	1:50	110	1.2500
11:11	4 1/8	2:00	120	1.1875
11:21	4 1/16	2:10	130	1.2500
11:31	4	2:20	140	1.3125
11:41	4	2:30	150	1.3125
12:11	4	3:00	180	1.3125
13:11	4	4:00	240	1.3125
14:11	3 7/8	5:00	300	1.4375
15:11	3 7/8	6:00	360	1.4375
16:11	3 7/8	7:00	420	1.4375
17:11	3 7/8	8:00	480	1.4375
18:11	3 7/8	9:00	540	1.4375

$L =$	11	ft
$L_f =$	11	ft
$L/y_p =$	26.4	
$B =$	1.5	ft
$Q =$	0.46	cfs
$y_1 =$	9	inch
$y_p =$	5	inch
$V_{avep} =$	0.736	ft/s
$V_1 =$	0.492	ft/s
$V_2 =$	0.459	ft/s
$V_{ave} =$	0.476	ft/s
$Q_v =$	0.54	cfs
$S =$	0.0023	
$T =$	62	°F

Notes: Setup time was 21 minutes. There was a layer of water on top of cover: put in piece of Styrofoam to evaluate if there was flow. Flow was not existent.

Test 2 - 100% cover, high Q, high y1

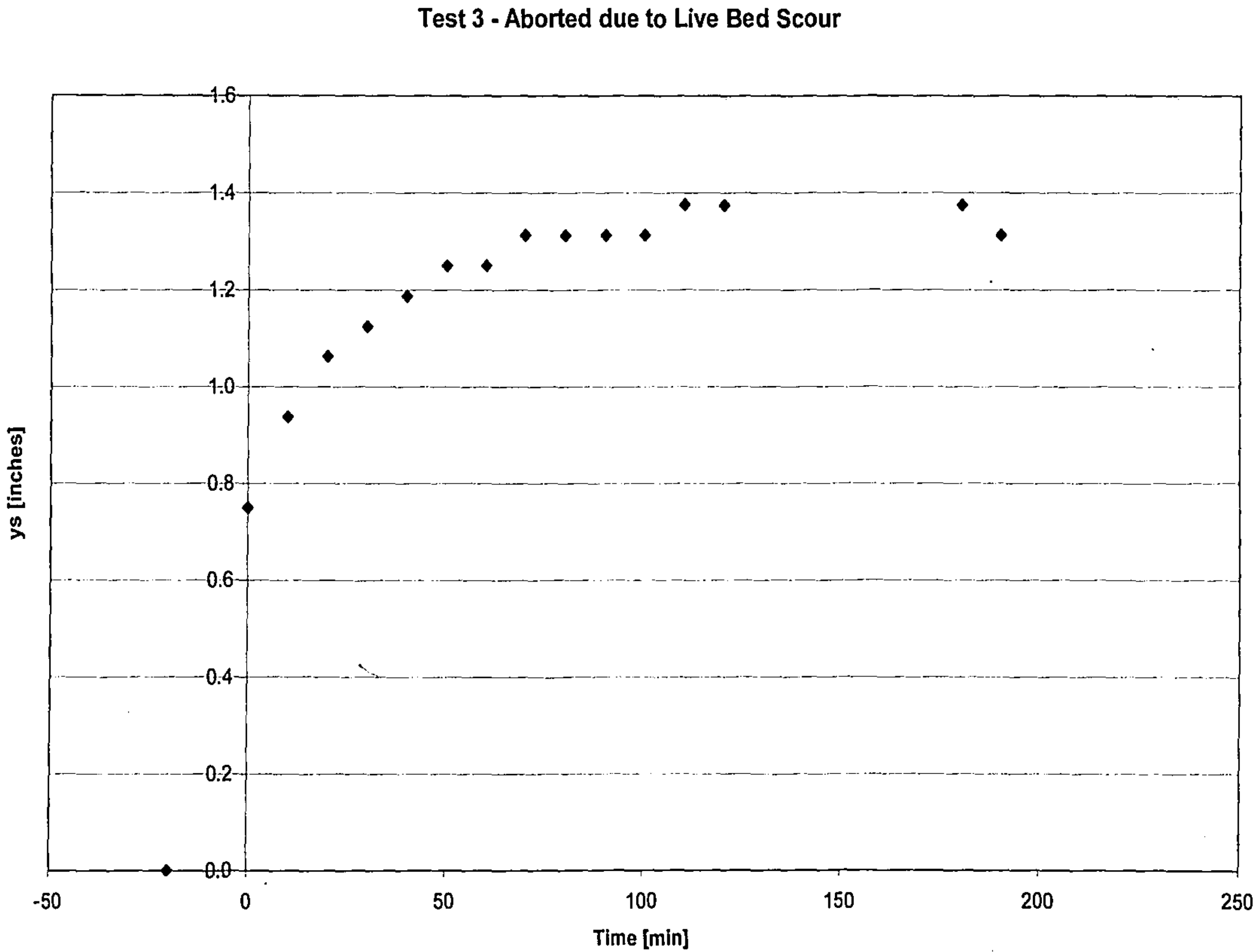


Test No.: 3
Date: 1/13/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
8:20	5 1/4	-0:20	-20	0.0000
8:40	4 1/2	0:00	0	0.7500
8:50	4 5/16	0:10	10	0.9375
9:00	4 3/16	0:20	20	1.0625
9:10	4 1/8	0:30	30	1.1250
9:20	4 1/16	0:40	40	1.1875
9:30	4	0:50	50	1.2500
9:40	4	1:00	60	1.2500
9:50	3 15/16	1:10	70	1.3125
10:00	3 15/16	1:20	80	1.3125
10:10	3 15/16	1:30	90	1.3125
10:20	3 15/16	1:40	100	1.3125
10:30	3 7/8	1:50	110	1.3750
10:40	3 7/8	2:00	120	1.3750
11:40	3 7/8	3:00	180	1.3750
11:50	3 15/16	3:10	190	1.3125
12:00	4 1/16	3:20	200	1.1875
12:10	4 1/16	3:30	210	1.1875

$L =$	11	ft
$L_f =$	11	ft
$L/y_p =$	26.4	
$B =$	1.5	ft
$Q =$	0.55	cfs
$y_1 =$	7	inch
$y_p =$	5	inch
$V_{avep} =$	0.88	ft/s
$V_1 =$	0.623	ft/s
$V_2 =$	0.656	ft/s
$V_{ave} =$	0.640	ft/s
$Q_v =$	0.560	cfs
$S =$	0.0023	
$T =$	66	°F

Notes: Setup time = 20 minutes, but there appeared to be a significant amount of scour in that time. y_1 was initially too high when I put the cover on and had to be lowered to adjust to the appropriate depth. This may have caused the large amount of initial scour. At the 50 minute mark, dunes were noticed upstream of the pier in the flume. These dunes end approximately 3 feet upstream of the pier. Possible causes: Entrance effects, sidewall effects (small contraction), effects due to the openings made for velocity probe, and effects due to having the velocity probe upstream. One can see bed movement along these dunes, but this movement does not seem to extend past the dunes. At the 2 hour mark, it should be noted that the dune formation does appear to be moving downstream. However, it is not yet affecting the pier. This bed movement shall be monitored until it is noticed that the scour hole is being affected, which at that point the test shall be aborted.
Test aborted due to live bed scour at 12:10 pm. It is noticed that the velocity calculated using the above flow rate and flume parameters is 0.88 ft/s, which is well above the velocity determined by the Shields Diagram analysis for incipient motion. A mistake in initial calculations yielded that a flow rate of up to 0.65 cfs could be used. This is (obviously) not the case, and subsequent tests will be performed at a lower flow rate.

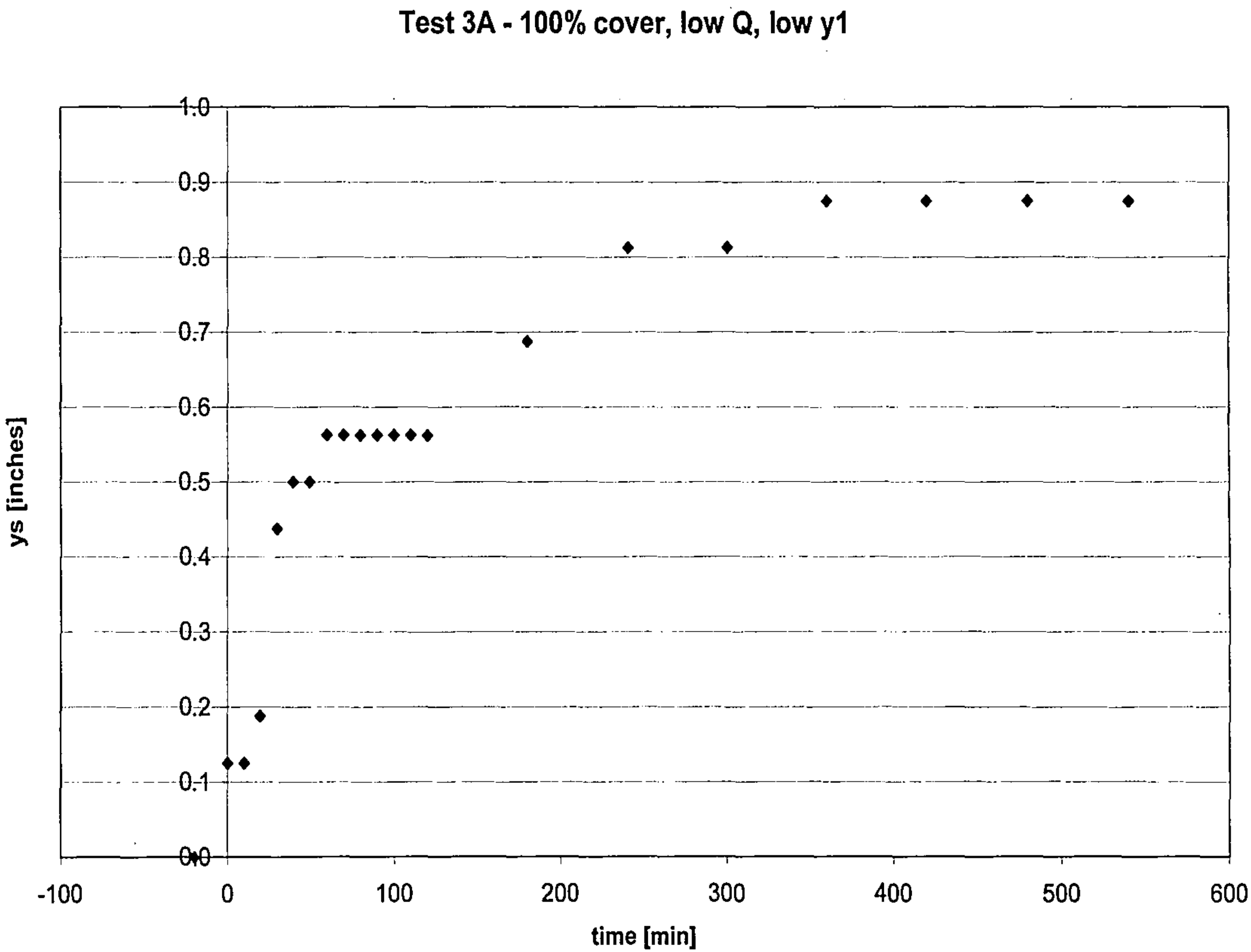

Test 3 - Aborted due to Live Bed Scour
ys [inches]
Time [min]
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
-50
0
50
100
150
200
250

Test No.: 3A
Date: 1/14/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
8:55	5 3/8	-0:20	-20	0.0000
9:15	5 1/4	0:00	0	0.1250
9:25	5 1/4	0:10	10	0.1250
9:35	5 3/16	0:20	20	0.1875
9:45	4 15/16	0:30	30	0.4375
9:55	4 7/8	0:40	40	0.5000
10:05	4 7/8	0:50	50	0.5000
10:15	4 13/16	1:00	60	0.5625
10:25	4 13/16	1:10	70	0.5625
10:35	4 13/16	1:20	80	0.5625
10:45	4 13/16	1:30	90	0.5625
10:55	4 13/16	1:40	100	0.5625
11:05	4 13/16	1:50	110	0.5625
11:15	4 13/16	2:00	120	0.5625
12:15	4 11/16	3:00	180	0.6875
13:15	4 9/16	4:00	240	0.8125
14:15	4 9/16	5:00	300	0.8125
15:15	4 1/2	6:00	360	0.8750
16:15	4 1/2	7:00	420	0.8750
17:15	4 1/2	8:00	480	0.8750
18:15	4 1/2	9:00	540	0.8750

$L =$	11	ft
$L_f =$	11	ft
$L/y_p =$	26.4	
$B =$	1.5	ft
$Q =$	0.38	cfs
$y_1 =$	7	inch
$y_p =$	5	inch
$V_{avep} =$	0.608	ft/s
$V_1 =$	0.558	ft/s
$V_2 =$	0.492	ft/s
$V_{ave} =$	0.525	ft/s
$Q_v =$	0.459	cfs
$S =$	0.0023	
$T =$	66	°F

Notes: Setup time was 20 minutes. Started velocity probe measurements ~1:30 pm. Took second round of velocity readings ~2:30 pm.

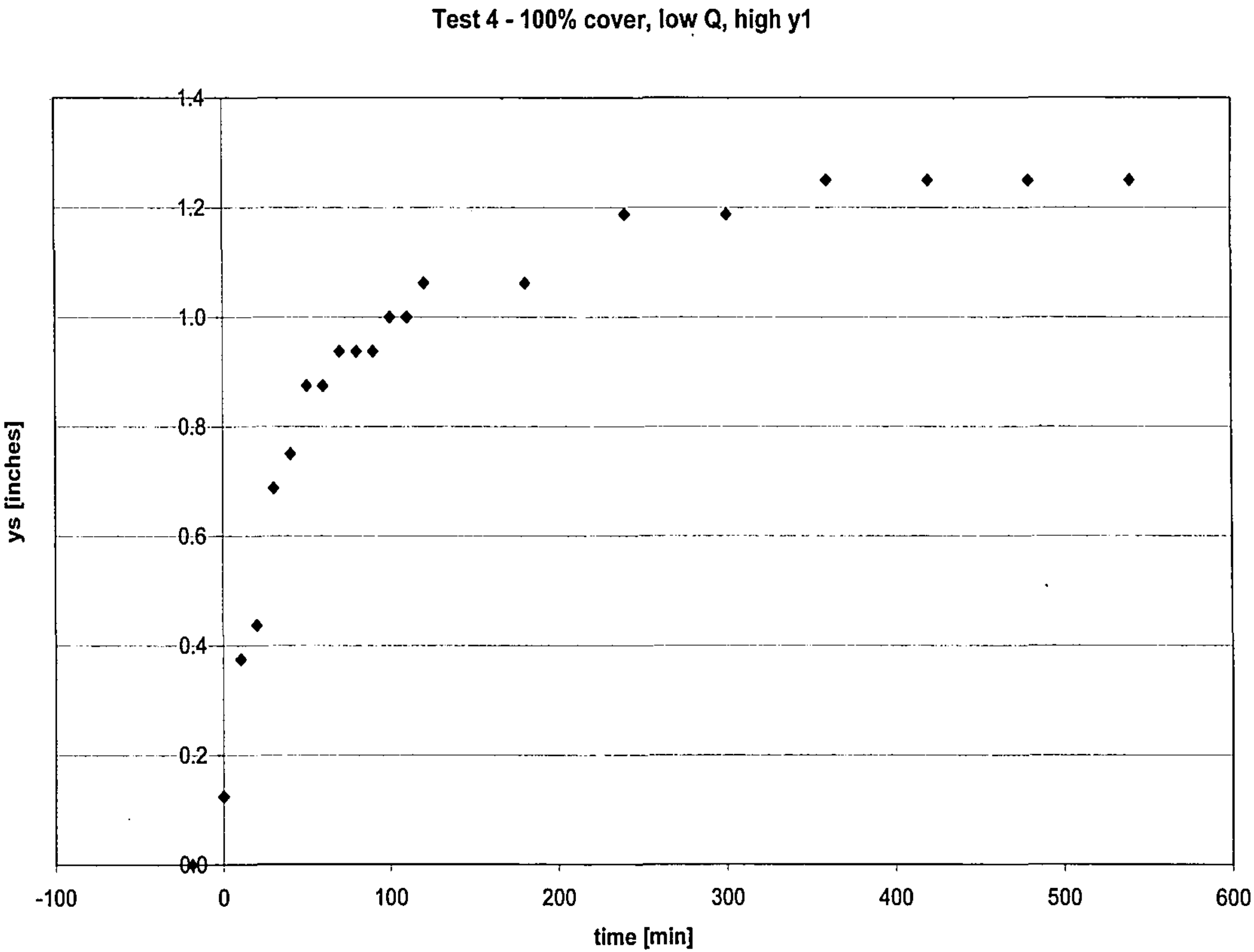

Test 3A - 100% cover, low Q, low y1
ys [inches]
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
-100
0
100
200
300
400
500
600
time [min]

Test No.: 4
Date: 1/15/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
8:57	5 3/4	-0:18	-18	0.0000
9:15	5 5/8	0:00	0	0.1250
9:25	5 3/8	0:10	10	0.3750
9:35	5 5/16	0:20	20	0.4375
9:45	5 1/16	0:30	30	0.6875
9:55	5	0:40	40	0.7500
10:05	4 7/8	0:50	50	0.8750
10:15	4 7/8	1:00	60	0.8750
10:25	4 13/16	1:10	70	0.9375
10:35	4 13/16	1:20	80	0.9375
10:45	4 13/16	1:30	90	0.9375
10:55	4 3/4	1:40	100	1.0000
11:05	4 3/4	1:50	110	1.0000
11:15	4 11/16	2:00	120	1.0625
12:15	4 11/16	3:00	180	1.0625
13:15	4 9/16	4:00	240	1.1875
14:15	4 9/16	5:00	300	1.1875
15:15	4 1/2	6:00	360	1.2500
16:15	4 1/2	7:00	420	1.2500
17:15	4 1/2	8:00	480	1.2500
18:15	4 1/2	9:00	540	1.2500

L =	11	ft
L_f =	11	ft
L/y_p =	26.4	
B =	1.5	ft
Q =	0.38	cfs
y_1 =	9	inch
y_p =	5	inch
V_{avep} =	0.608	ft/s
V_1 =	0.427	ft/s
V_2 =	0.459	ft/s
V_{ave} =	0.443	ft/s
Q_v =	0.498	cfs
S =	0.0023	
T =	64	°F

Notes: Setup time was 18 minutes. There was 2 inches of water on top of the cover.



Test 4 - 100% cover, low Q, high y1
ys [inches]
time [min]
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
-100
0
100
200
300
400
500
600

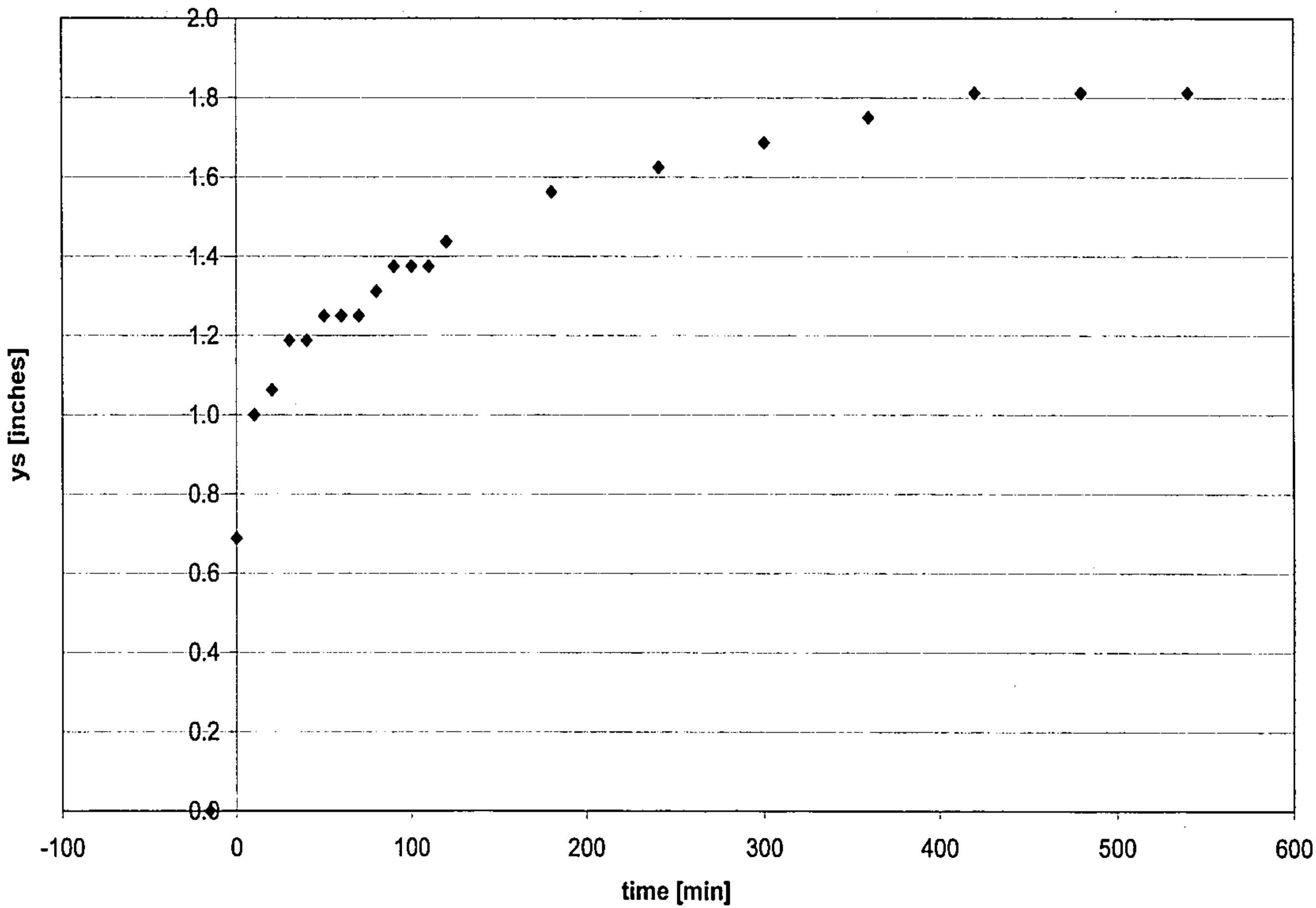
Test No.: 5
Date: 1/16/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
9:00	5 5/8	-0:15	-15	0.0000
9:15	4 15/16	0:00	0	0.6875
9:25	4 5/8	0:10	10	1.0000
9:35	4 9/16	0:20	20	1.0625
9:45	4 7/16	0:30	30	1.1875
9:55	4 7/16	0:40	40	1.1875
10:05	4 3/8	0:50	50	1.2500
10:15	4 3/8	1:00	60	1.2500
10:25	4 3/8	1:10	70	1.2500
10:35	4 5/16	1:20	80	1.3125
10:45	4 1/4	1:30	90	1.3750
10:55	4 1/4	1:40	100	1.3750
11:05	4 1/4	1:50	110	1.3750
11:15	4 3/16	2:00	120	1.4375
12:15	4 1/16	3:00	180	1.5625
13:15	4	4:00	240	1.6250
14:15	3 15/16	5:00	300	1.6875
15:15	3 7/8	6:00	360	1.7500
16:15	3 13/16	7:00	420	1.8125
17:15	3 13/16	8:00	480	1.8125
18:15	3 13/16	9:00	540	1.8125

L =	5.5	ft
L_f =	11	ft
L/y_p =	13.2	
B =	1.5	ft
Q =	0.46	cfs
y_1 =	5	inch
y_p =	7	inch
V_{avep} =	0.736	ft/s
V_1 =		ft/s
V_2 =		ft/s
V_{ave} =		ft/s
Q_v =		cfs
S =	0.0023	
T =	64	°F

Notes: Setup time was 15 minutes. Flow rate was >0.46 cfs for a short time while attempting to set up test. Velocity profile only taken at one point between 2:45pm and 3:15pm. Water is very clear today. No Marsh-McBirney readings taken today. See velocity probe data.

Test 5 - 50% cover, high Q, low y1



08

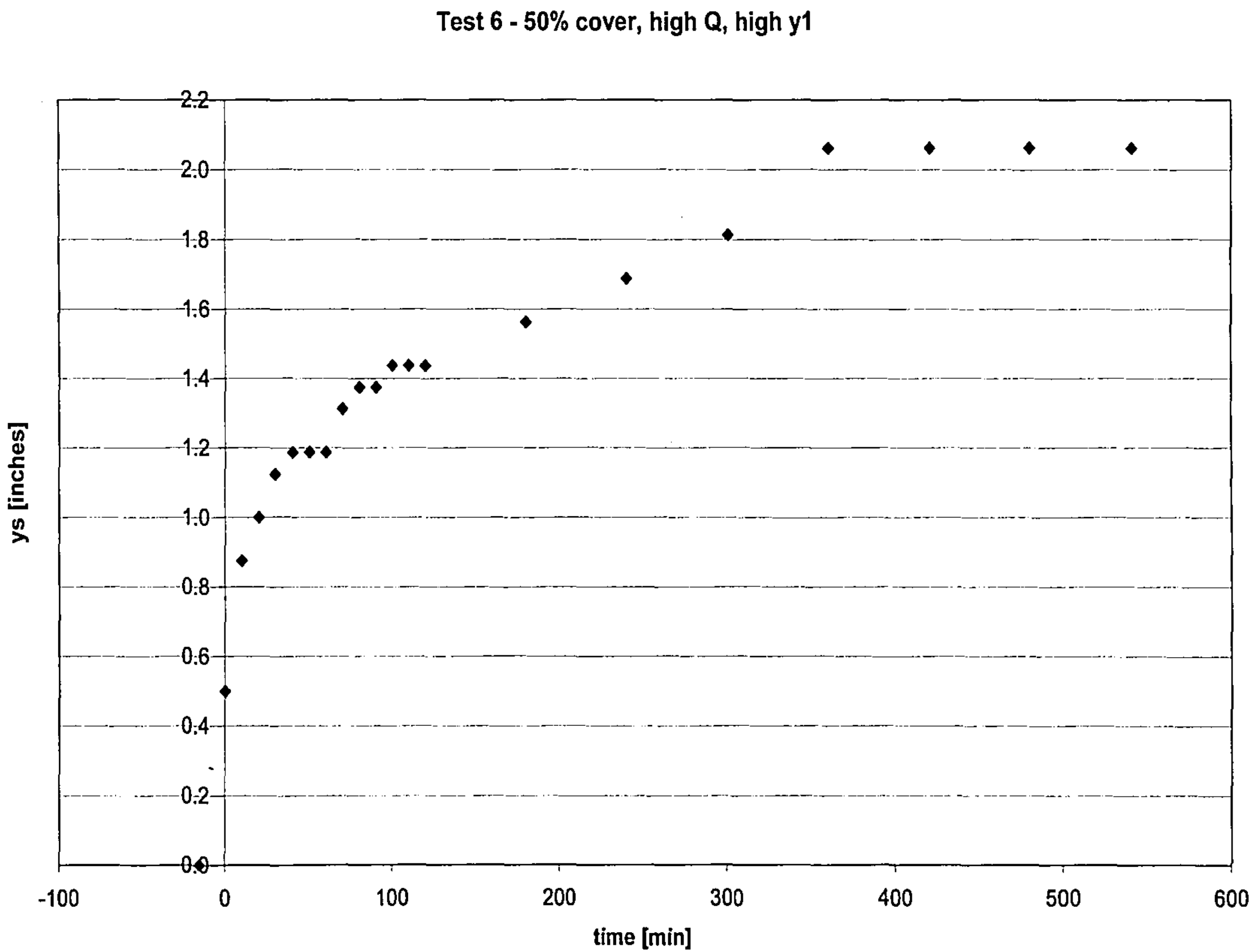
Test No.: 6
Date: 1/19/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
9:00	5 5/16	-0:15	-15	0.0000
9:15	4 13/16	0:00	0	0.5000
9:25	4 7/16	0:10	10	0.8750
9:35	4 5/16	0:20	20	1.0000
9:45	4 3/16	0:30	30	1.1250
9:55	4 1/8	0:40	40	1.1875
10:05	4 1/8	0:50	50	1.1875
10:15	4 1/8	1:00	60	1.1875
10:25	4	1:10	70	1.3125
10:35	3 15/16	1:20	80	1.3750
10:45	3 15/16	1:30	90	1.3750
10:55	3 7/8	1:40	100	1.4375
11:05	3 7/8	1:50	110	1.4375
11:15	3 7/8	2:00	120	1.4375
12:15	3 3/4	3:00	180	1.5625
13:15	3 5/8	4:00	240	1.6875
14:15	3 1/2	5:00	300	1.8125
15:15	3 1/4	6:00	360	2.0625
16:15	3 1/4	7:00	420	2.0625
17:15	3 1/4	8:00	480	2.0625
18:15	3 1/4	9:00	540	2.0625

$L =$	5.5	ft
$L_f =$	11	ft
$L/y_p =$	13.2	
$B =$	1.5	ft
$Q =$	0.46	cfs
$y_1 =$	9	inch
$y_p =$	5	inch
$V_{avep} =$	0.736	ft/s
$V_1 =$	0.492	ft/s
$V_2 =$	0.492	ft/s
$V_{ave} =$	0.492	ft/s
$Q_v =$	0.554	cfs
$S =$	0.0023	
$T =$	64	°F

Notes: Set up time = 15 minutes. There is water on top of the cover, does not appear to have a significant flow rate. Joint became loose and bent down a little.

Plan to fix this is to weight down dam in the front so that this slippage does not occur again. It is not known what effect this slippage will have on my scour.

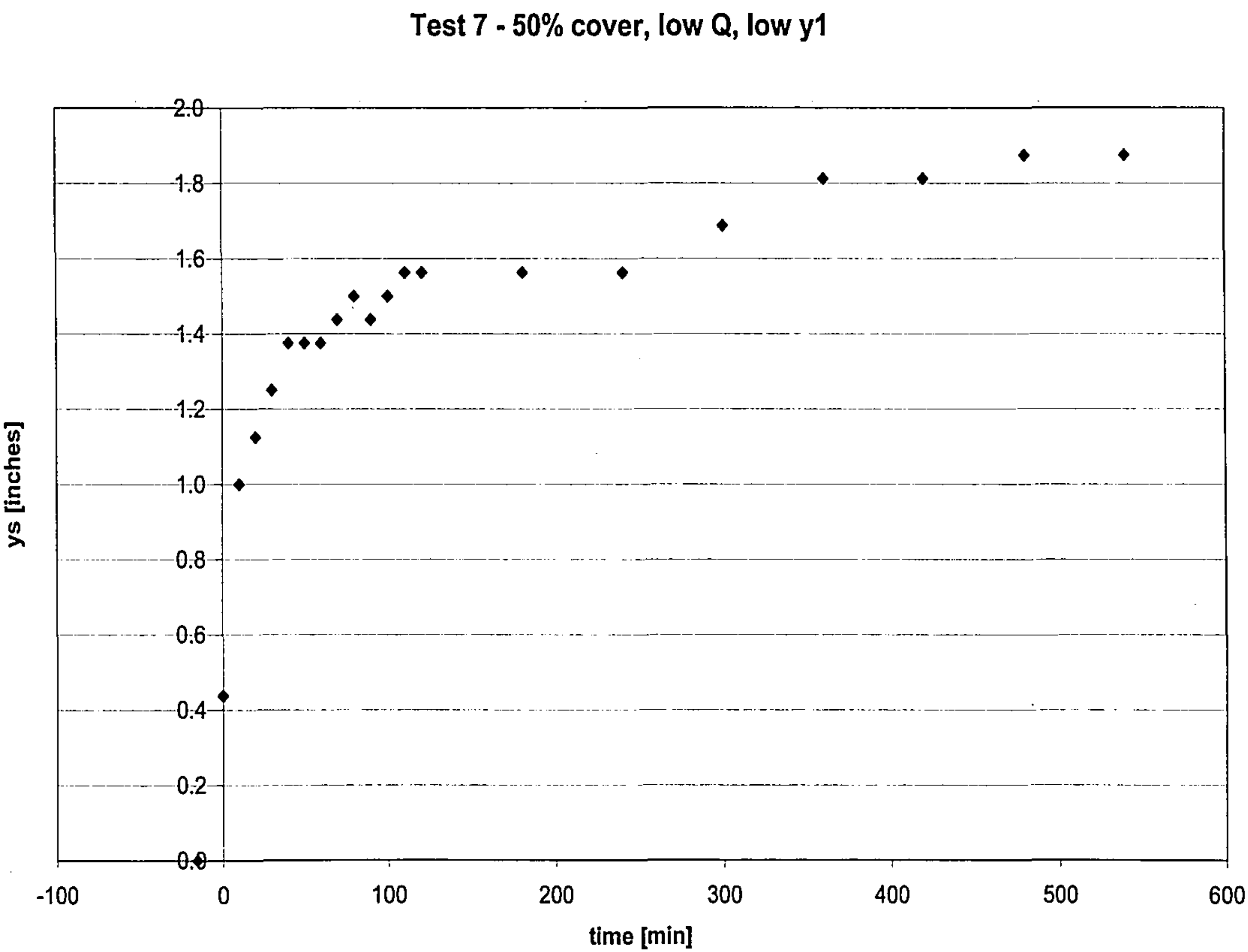

Test 6 - 50% cover, high Q, high y1
ys [inches]
time [min]
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-100
0
100
200
300
400
500
600

Test No.: 7
Date: 1/21/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
9:15	5 5/8	0:15	-15	0.0000
9:30	5 3/16	0:00	0	0.4375
9:40	4 5/8	0:10	10	1.0000
9:50	4 1/2	0:20	20	1.1250
10:00	4 3/8	0:30	30	1.2500
10:10	4 1/4	0:40	40	1.3750
10:20	4 1/4	0:50	50	1.3750
10:30	4 1/4	1:00	60	1.3750
10:40	4 3/16	1:10	70	1.4375
10:50	4 1/8	1:20	80	1.5000
11:00	4 3/16	1:30	90	1.4375
11:10	4 1/8	1:40	100	1.5000
11:20	4 1/16	1:50	110	1.5625
11:30	4 1/16	2:00	120	1.5625
12:30	4 1/16	3:00	180	1.5625
13:30	4 1/16	4:00	240	1.5625
14:30	3 15/16	5:00	300	1.6875
15:30	3 13/16	6:00	360	1.8125
16:30	3 13/16	7:00	420	1.8125
17:30	3 3/4	8:00	480	1.8750
18:30	3 3/4	9:00	540	1.8750

$L =$	5.5	ft
$L_f =$	11	ft
$L/y_p =$	13.2	
$B =$	1.5	ft
$Q =$	0.38	cfs
$y_1 =$	7	inch
$y_p =$	5	inch
$V_{avep} =$	0.608	ft/s
$V_1 =$	0.492	ft/s
$V_2 =$	0.525	ft/s
$V_{ave} =$	0.509	ft/s
$Q_v =$	0.445	cfs
$S =$	0.0023	
$T =$	64	°F

Notes: Setup time = 15 minutes.

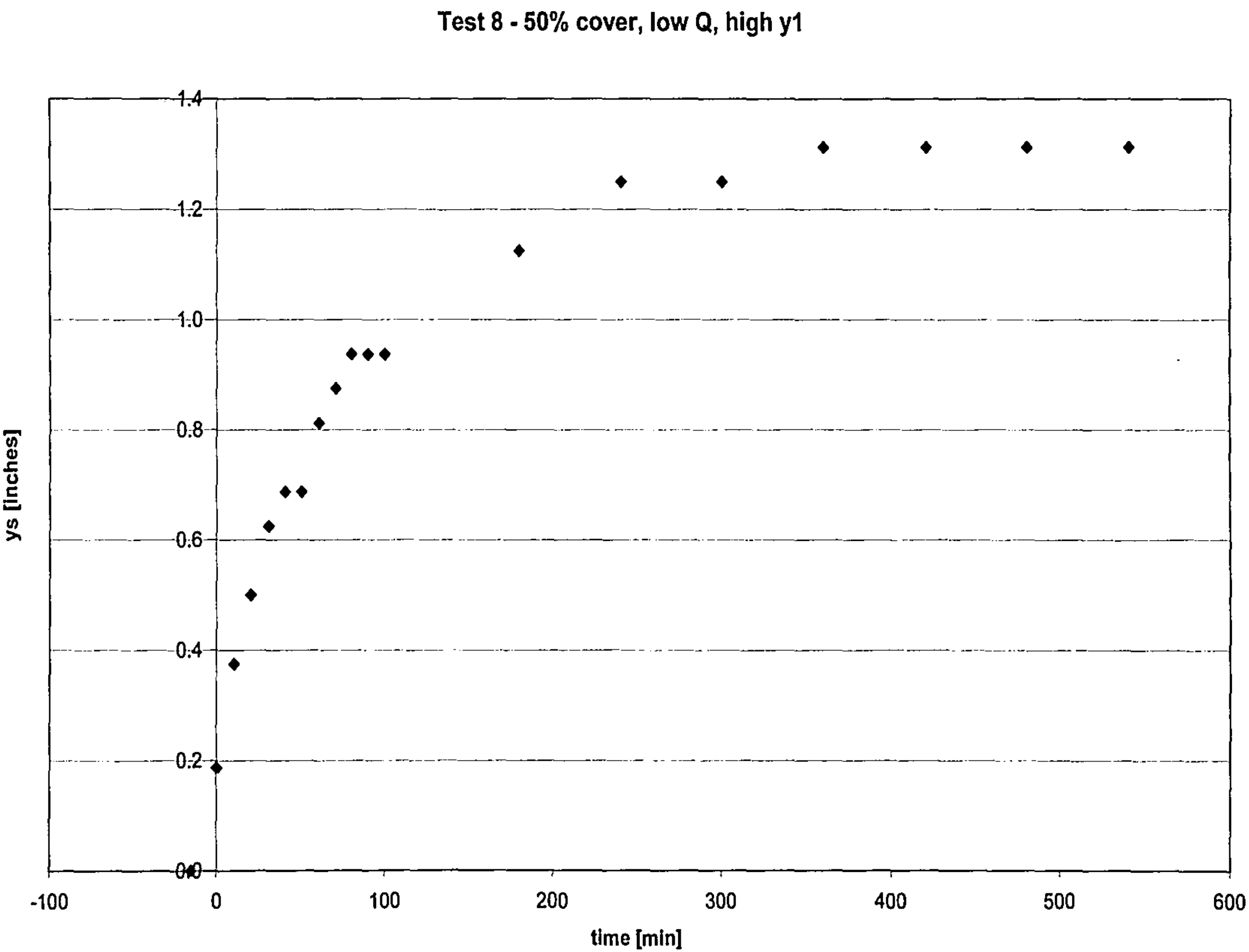

Test 7 - 50% cover, low Q, low y1
ys [inches]
time [min]
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-100
0
100
200
300
400
500
600

Test No.: 8
Date: 1/22/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
8:45	5 9/16	-0:15	-15	0.0000
9:00	5 3/8	0:00	0	0.1875
9:10	5 3/16	0:10	10	0.3750
9:20	5 1/16	0:20	20	0.5000
9:30	4 15/16	0:30	30	0.6250
9:40	4 7/8	0:40	40	0.6875
9:50	4 7/8	0:50	50	0.6875
10:00	4 3/4	1:00	60	0.8125
10:10	4 11/16	1:10	70	0.8750
10:20	4 5/8	1:20	80	0.9375
10:30	4 5/8	1:30	90	0.9375
10:40	4 5/8	1:40	100	0.9375
12:00	4 7/16	3:00	180	1.1250
13:00	4 5/16	4:00	240	1.2500
14:00	4 5/16	5:00	300	1.2500
15:00	4 1/4	6:00	360	1.3125
16:00	4 1/4	7:00	420	1.3125
17:00	4 1/4	8:00	480	1.3125
18:00	4 1/4	9:00	540	1.3125

$L =$	5.5	ft
$L_f =$	11	ft
$L/y_p =$	13.2	
$B =$	1.5	ft
$Q =$	0.38	cfs
$y_1 =$	9	inch
$y_p =$	5	inch
$V_{avep} =$	0.608	ft/s
$V_1 =$	0.394	ft/s
$V_2 =$	0.459	ft/s
$V_{ave} =$	0.427	ft/s
$Q_v =$	0.480	cfs
$S =$	0.0023	
$T =$	64	°F

Notes: Setup time = 15 minutes. Readings were not taken between 10:45 am and 12:00 pm because I have class.


Test 8 - 50% cover, low Q, high y1
ys [inches]
time [min]
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-100
0
100
200
300
400
500
600

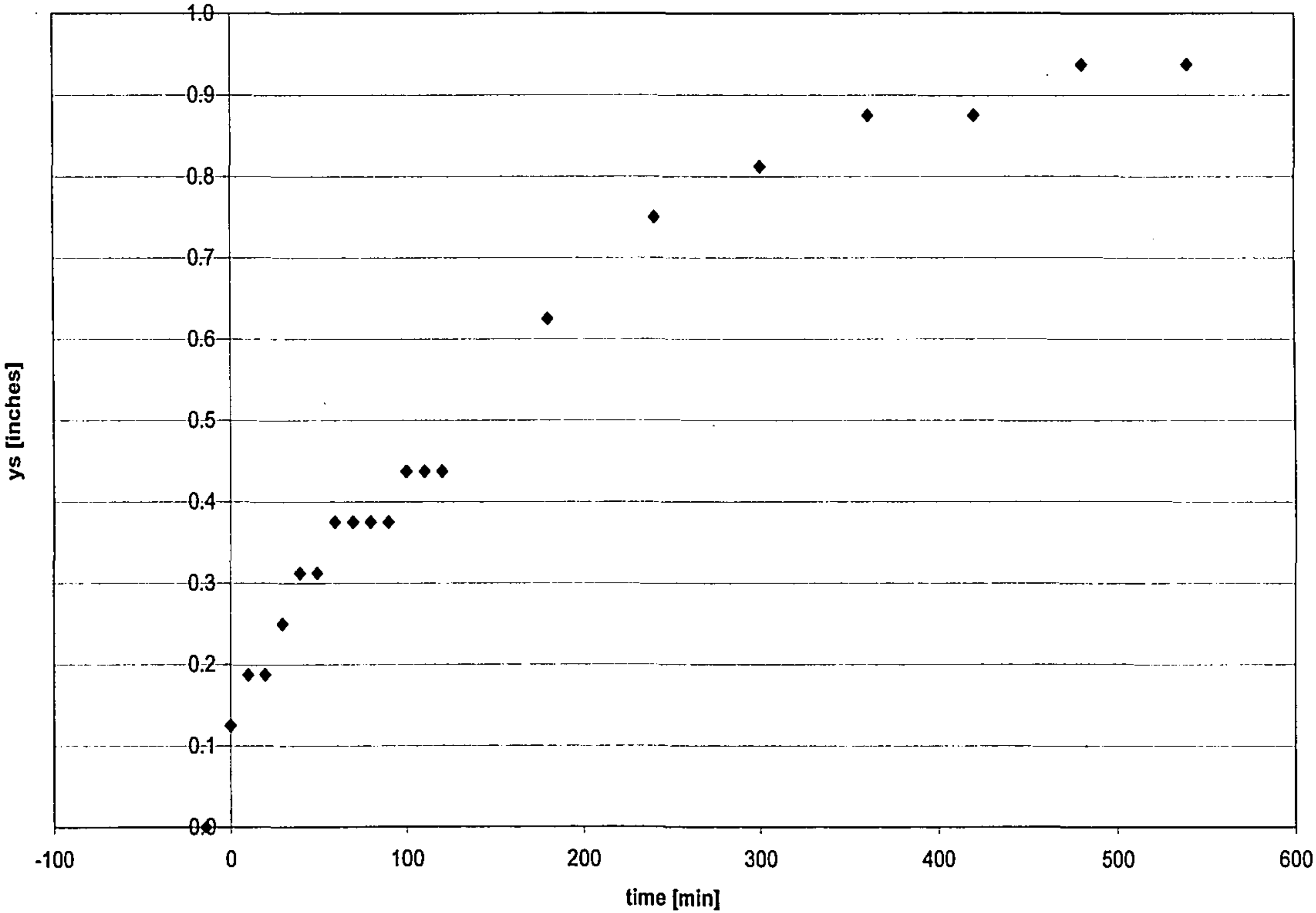
Test No.: 9
Date: 1/26/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
8:41	5 7/16	-0:14	-14	0.0000
8:55	5 5/16	0:00	0	0.1250
9:05	5 1/4	0:10	10	0.1875
9:15	5 1/4	0:20	20	0.1875
9:25	5 3/16	0:30	30	0.2500
9:35	5 1/8	0:40	40	0.3125
9:45	5 1/8	0:50	50	0.3125
9:55	5 1/16	1:00	60	0.3750
10:05	5 1/16	1:10	70	0.3750
10:15	5 1/16	1:20	80	0.3750
10:25	5 1/16	1:30	90	0.3750
10:35	5	1:40	100	0.4375
10:45	5	1:50	110	0.4375
10:55	5	2:00	120	0.4375
11:55	4 13/16	3:00	180	0.6250
12:55	4 11/16	4:00	240	0.7500
13:55	4 5/8	5:00	300	0.8125
14:55	4 9/16	6:00	360	0.8750
15:55	4 9/16	7:00	420	0.8750
16:55	4 1/2	8:00	480	0.9375
17:55	4 1/2	9:00	540	0.9375

$L =$	2.75	ft
$L_f =$	11	ft
$L/y_p =$	6.6	
$B =$	1.5	ft
$Q =$	0.38	cfs
$y_1 =$	9	inch
$y_p =$	5	inch
$V_{avep} =$	0.608	ft/s
$V_1 =$	0.328	ft/s
$V_2 =$	0.427	ft/s
$V_{ave} =$	0.377	ft/s
$Q_v =$	0.424	cfs
$S =$	0.0023	
$T =$	64	°F

Notes: No 2D velocity probe readings taken for the 25% and lower cover cases, as not to interfere with the flow mechanism near the pier.

Test 9 - 25% cover, low Q, high y1

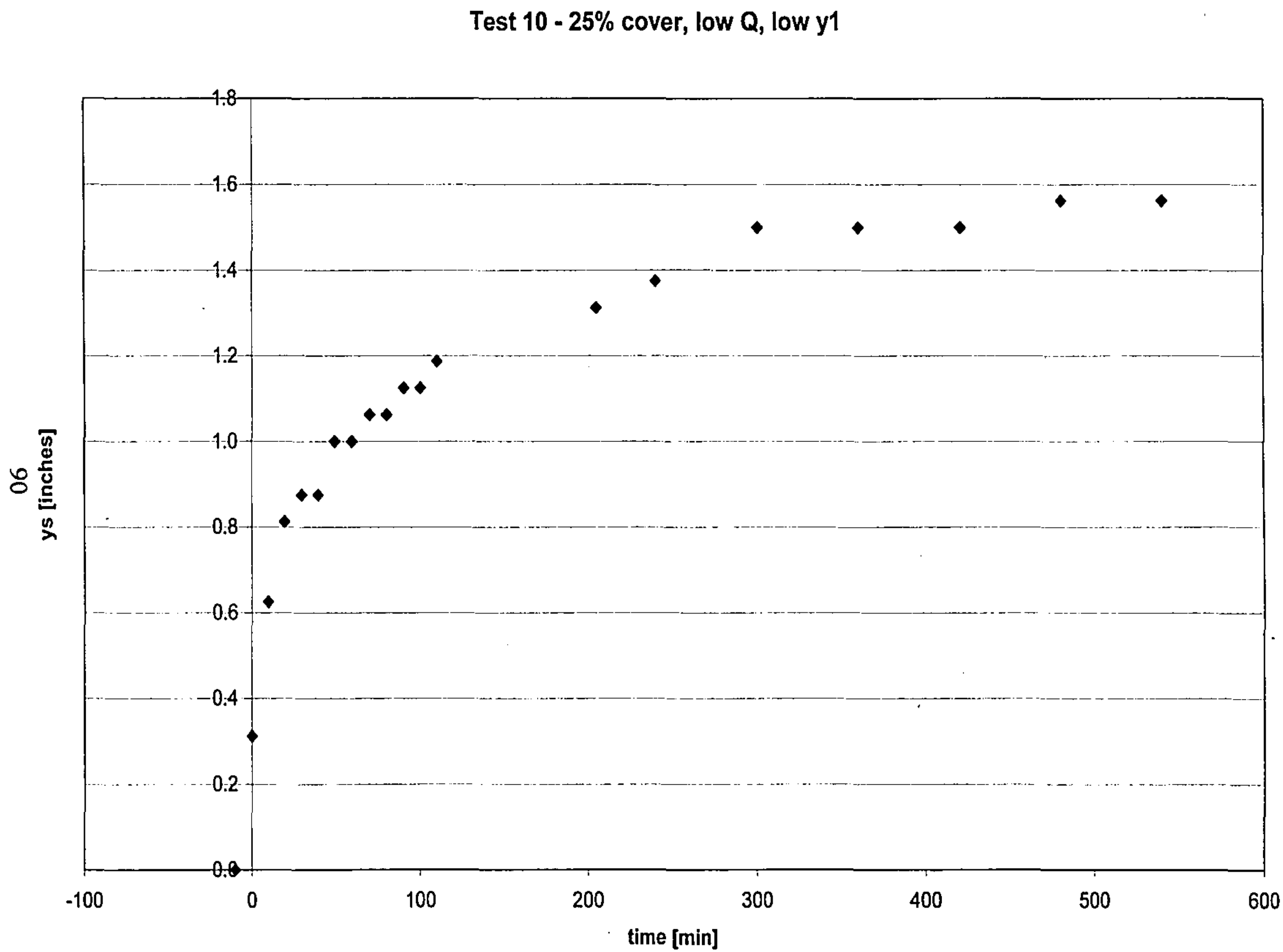


Test No.: 10
Date: 1/27/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
8:35	5 9/16	-0:10	-10	0.0000
8:45	5 1/4	0:00	0	0.3125
8:55	4 15/16	0:10	10	0.6250
9:05	4 3/4	0:20	20	0.8125
9:15	4 11/16	0:30	30	0.8750
9:25	4 11/16	0:40	40	0.8750
9:35	4 9/16	0:50	50	1.0000
9:45	4 9/16	1:00	60	1.0000
9:55	4 1/2	1:10	70	1.0625
10:05	4 1/2	1:20	80	1.0625
10:15	4 7/16	1:30	90	1.1250
10:25	4 7/16	1:40	100	1.1250
10:35	4 3/8	1:50	110	1.1875
12:10	4 1/4	3:25	205	1.3125
12:45	4 3/16	4:00	240	1.3750
13:45	4 1/16	5:00	300	1.5000
14:45	4 1/16	6:00	360	1.5000
15:45	4 1/16	7:00	420	1.5000
16:45	4	8:00	480	1.5625
17:45	4	9:00	540	1.5625

L =	2.75	ft
L_f =	11	ft
L/y_p =	6.6	
B =	1.5	ft
Q =	0.38	cfs
y_1 =	7	inch
y_p =	5	inch
V_{avep} =	0.608	ft/s
V_1 =	0.427	ft/s
V_2 =	0.492	ft/s
V_{ave} =	0.459	ft/s
Q_v =	0.402	cfs
S =	0.0023	
T =	63	°F

Notes: No readings taken between 10:45 am and 12:00 pm because I have class.



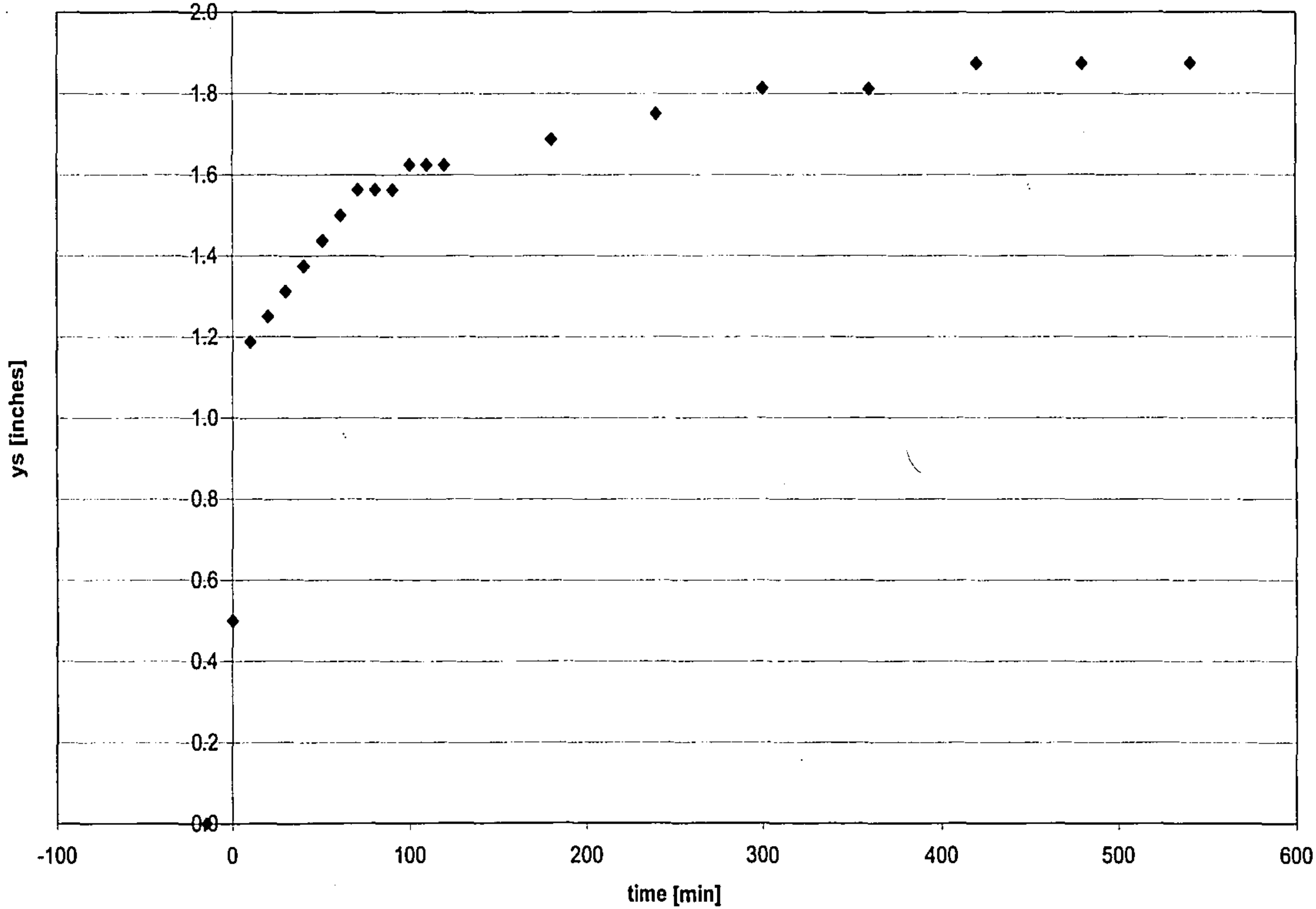
Test 10 - 25% cover, low Q, low y1
ys [inches]
time [min]
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-100
0
100
200
300
400
500
600

Test No.: 11
Date: 1/28/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
8:55	5 5/8	-0:15	-15	0.0000
9:10	5 1/8	0:00	0	0.5000
9:20	4 7/16	0:10	10	1.1875
9:30	4 3/8	0:20	20	1.2500
9:40	4 5/16	0:30	30	1.3125
9:50	4 1/4	0:40	40	1.3750
10:00	4 3/16	0:50	50	1.4375
10:10	4 1/8	1:00	60	1.5000
10:20	4 1/16	1:10	70	1.5625
10:30	4 1/16	1:20	80	1.5625
10:40	4 1/16	1:30	90	1.5625
10:50	4	1:40	100	1.6250
11:00	4	1:50	110	1.6250
11:10	4	2:00	120	1.6250
12:10	3 15/16	3:00	180	1.6875
13:10	3 7/8	4:00	240	1.7500
14:10	3 13/16	5:00	300	1.8125
15:10	3 13/16	6:00	360	1.8125
16:10	3 3/4	7:00	420	1.8750
17:10	3 3/4	8:00	480	1.8750
18:10	3 3/4	9:00	540	1.8750

$L =$	2.75	ft
$L_f =$	11	ft
$L/y_p =$	6.6	
$B =$	1.5	ft
$Q =$	0.46	cfs
$y_1 =$	7	inch
$y_p =$	5	inch
$V_{avep} =$	0.736	ft/s
$V_1 =$	0.427	ft/s
$V_2 =$	0.558	ft/s
$V_{ave} =$	0.492	ft/s
$Q_v =$	0.431	cfs
$S =$	0.0023	
$T =$	64	°F

Test 11 - 25% cover, high Q, low y1



Test No.: 12

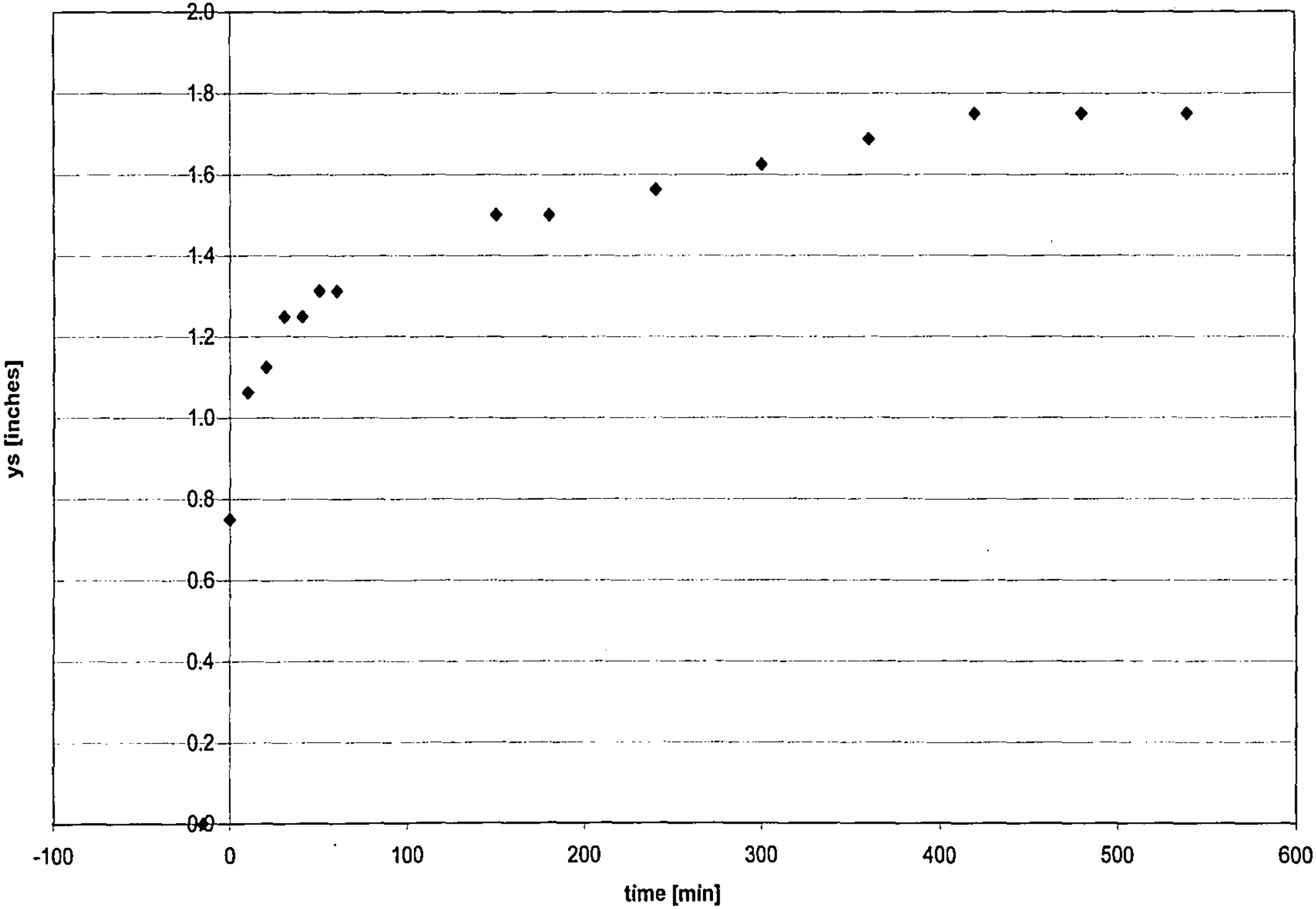
Date: 1/29/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
9:15	5 11/16	-0:15	-15	0.0000
9:30	4 15/16	0:00	0	0.7500
9:40	4 5/8	0:10	10	1.0625
9:50	4 9/16	0:20	20	1.1250
10:00	4 7/16	0:30	30	1.2500
10:10	4 7/16	0:40	40	1.2500
10:20	4 3/8	0:50	50	1.3125
10:30	4 3/8	1:00	60	1.3125
12:00	4 3/16	2:30	150	1.5000
12:30	4 3/16	3:00	180	1.5000
13:30	4 1/8	4:00	240	1.5625
14:30	4 1/16	5:00	300	1.6250
15:30	4	6:00	360	1.6875
16:30	3 15/16	7:00	420	1.7500
17:30	3 15/16	8:00	480	1.7500
18:30	3 15/16	9:00	540	1.7500

$L =$	2.75	ft
$L_f =$	11	ft
$L/y_p =$	6.6	
$B =$	1.5	ft
$Q =$	0.46	cfs
$y_1 =$	9	inch
$y_p =$	5	inch
$V_{avep} =$	0.736	ft/s
$V_1 =$	0.410	ft/s
$V_2 =$	0.427	ft/s
$V_{ave} =$	0.418	ft/s
$Q_v =$	0.471	cfs
$S =$	0.0023	
$T =$	64	°F

Notes: The well around the pier became unattached.

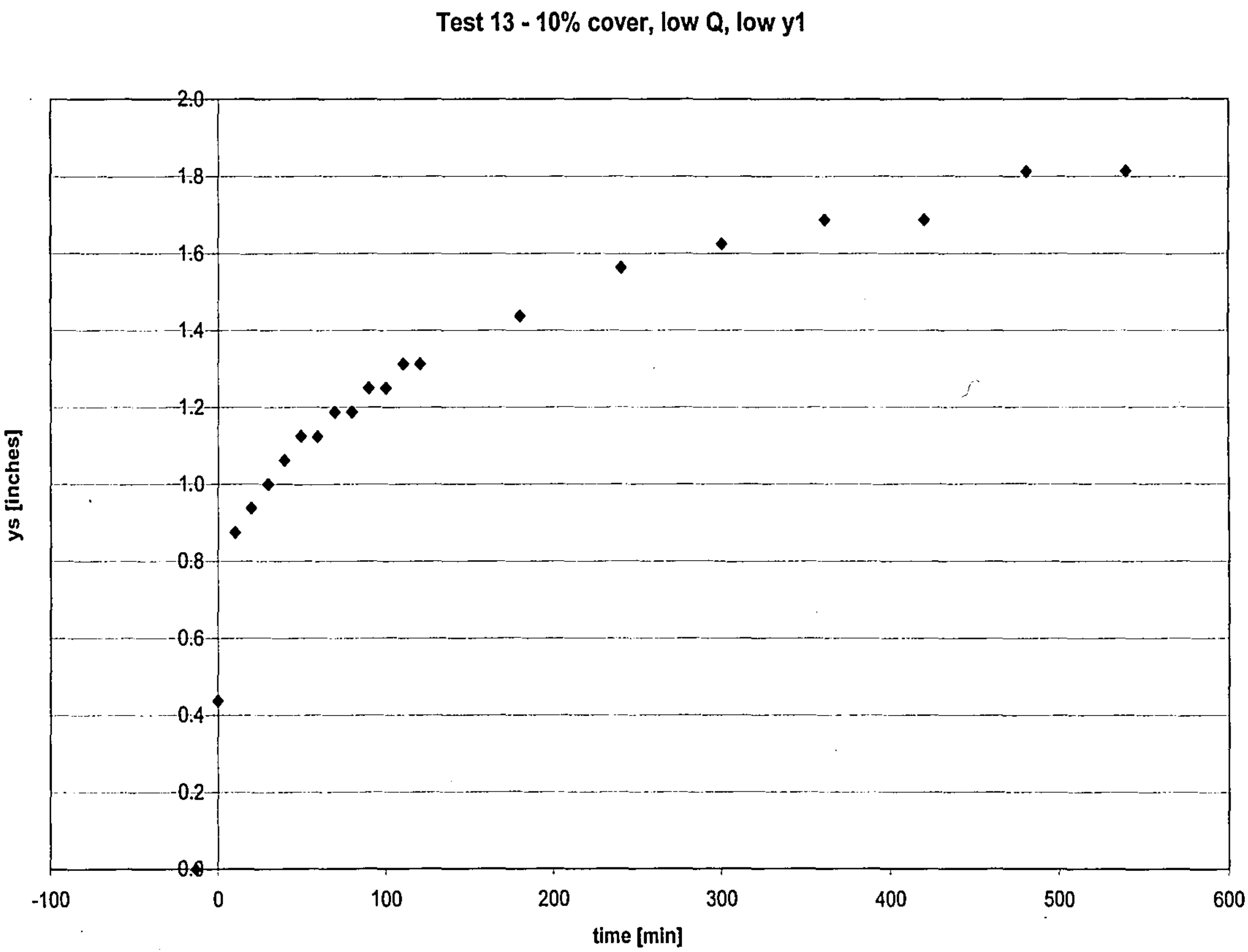
Test 12 - 25% cover, high Q, high y1



Test No.: 13
Date: 1/30/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
8:27	5 13/16	-0:13	-13	0.0000
8:40	5 3/8	0:00	0	0.4375
8:50	4 15/16	0:10	10	0.8750
9:00	4 7/8	0:20	20	0.9375
9:10	4 13/16	0:30	30	1.0000
9:20	4 3/4	0:40	40	1.0625
9:30	4 11/16	0:50	50	1.1250
9:40	4 11/16	1:00	60	1.1250
9:50	4 5/8	1:10	70	1.1875
10:00	4 5/8	1:20	80	1.1875
10:10	4 9/16	1:30	90	1.2500
10:20	4 9/16	1:40	100	1.2500
10:30	4 1/2	1:50	110	1.3125
10:40	4 1/2	2:00	120	1.3125
11:40	4 3/8	3:00	180	1.4375
12:40	4 1/4	4:00	240	1.5625
13:40	4 3/16	5:00	300	1.6250
14:40	4 1/8	6:00	360	1.6875
15:40	4 1/8	7:00	420	1.6875
16:40	4	8:00	480	1.8125
17:40	4	9:00	540	1.8125

L =	1.083	ft
L_f =	11	ft
L/y_p =	2.6	
B =	1.5	ft
Q =	0.38	cfs
y_1 =	5	inch
y_p =	7	inch
V_{avep} =	0.608	ft/s
V_1 =	0.394	ft/s
V_2 =	0.492	ft/s
V_{ave} =	0.443	ft/s
Q_v =	0.388	cfs
S =	0.0023	
T =	65	°F


Test 13 - 10% cover, low Q, low y1
ys [inches]
time [min]
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-100
0
100
200
300
400
500
600

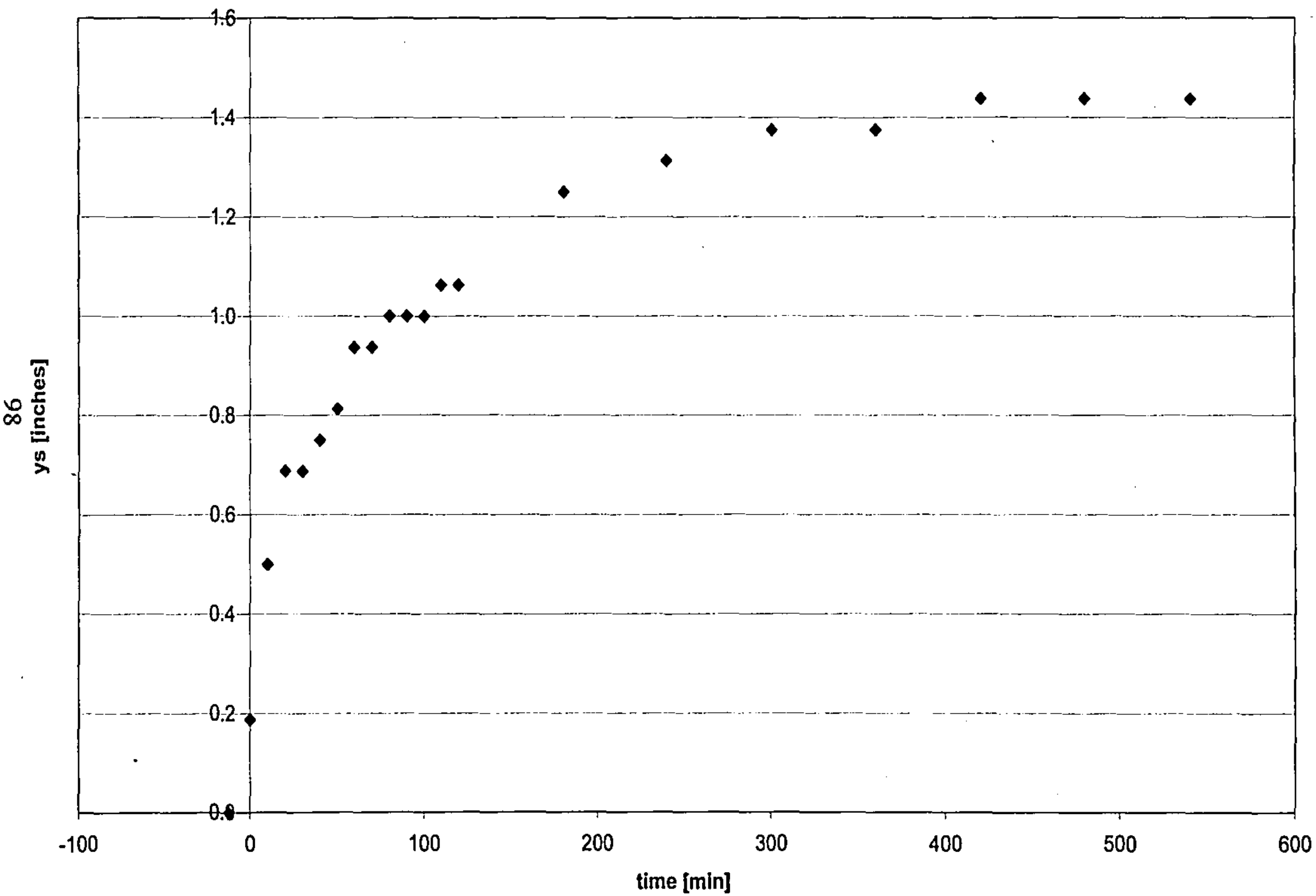
Test No.: 14
Date: 2/2/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
9:18	5 5/8	-0:12	-12	0.0000
9:30	5 7/16	0:00	0	0.1875
9:40	5 1/8	0:10	10	0.5000
9:50	4 15/16	0:20	20	0.6875
10:00	4 15/16	0:30	30	0.6875
10:10	4 7/8	0:40	40	0.7500
10:20	4 13/16	0:50	50	0.8125
10:30	4 11/16	1:00	60	0.9375
10:40	4 11/16	1:10	70	0.9375
10:50	4 5/8	1:20	80	1.0000
11:00	4 5/8	1:30	90	1.0000
11:10	4 5/8	1:40	100	1.0000
11:20	4 9/16	1:50	110	1.0625
11:30	4 9/16	2:00	120	1.0625
12:30	4 3/8	3:00	180	1.2500
13:30	4 5/16	4:00	240	1.3125
14:30	4 1/4	5:00	300	1.3750
15:30	4 1/4	6:00	360	1.3750
16:30	4 3/16	7:00	420	1.4375
17:30	4 3/16	8:00	480	1.4375
18:30	4 3/16	9:00	540	1.4375

$L =$	1.083	ft
$L_f =$	11	ft
$L/y_p =$	2.6	
$B =$	1.5	ft
$Q =$	0.38	cfs
$y_1 =$	9	inch
$y_p =$	5	inch
$V_{avep} =$	0.608	ft/s
$V_1 =$	0.328	ft/s
$V_2 =$	0.394	ft/s
$V_{ave} =$	0.361	ft/s
$Q_v =$	0.406	cfs
$S =$	0.0023	
$T =$	64	°F

Notes: The water became very dirty ~11 am. Had to turn the flow rate up a little at this point, it seemed to dip down a small bit when other systems were turned on for CE222 lab.

Test 14 - 10% cover, low Q, high y1

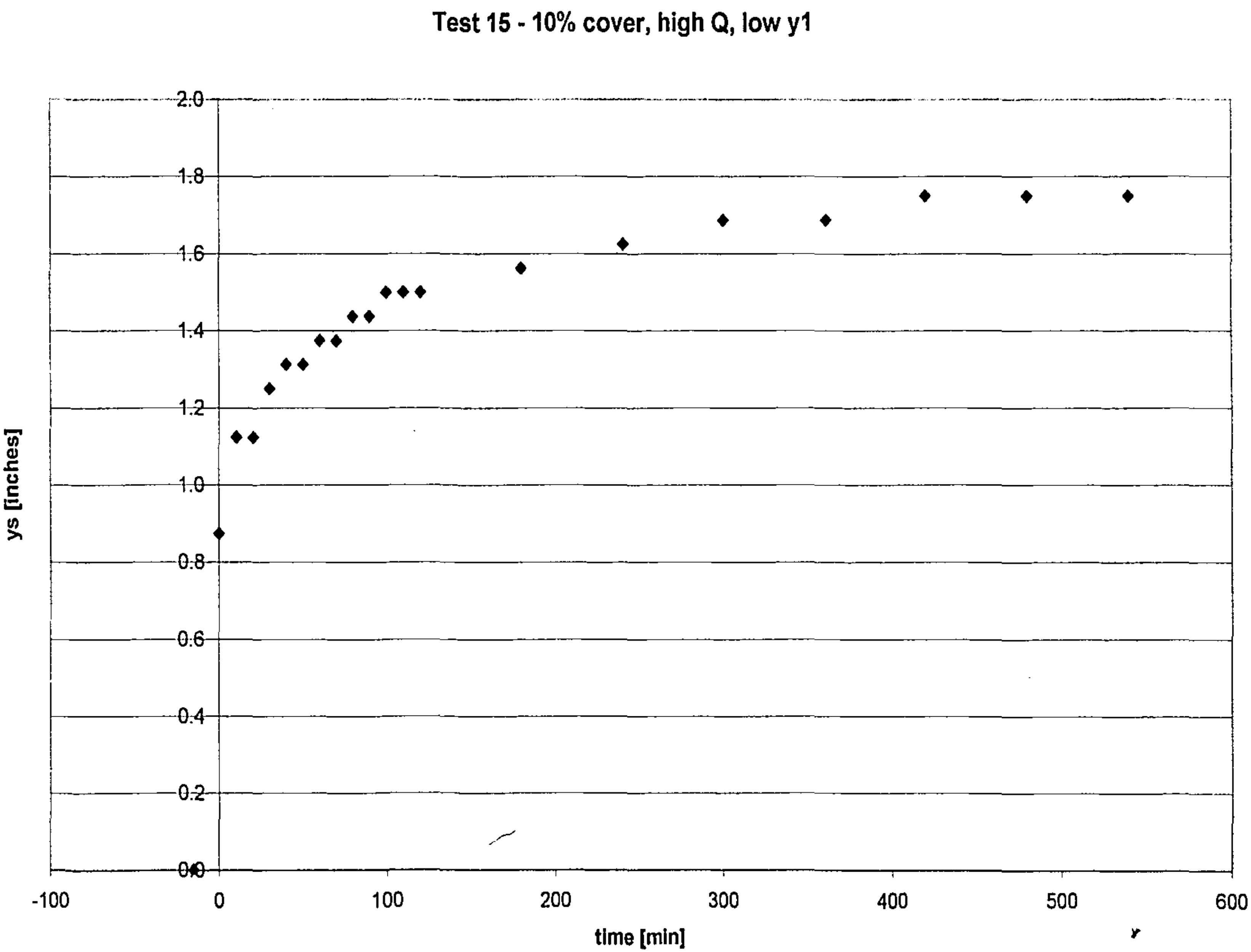


Test No.: 15
Date: 2/4/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
9:05	5 1/2	-0:15	-15	0.0000
9:20	4 5/8	0:00	0	0.8750
9:30	4 3/8	0:10	10	1.1250
9:40	4 3/8	0:20	20	1.1250
9:50	4 1/4	0:30	30	1.2500
10:00	4 3/16	0:40	40	1.3125
10:10	4 3/16	0:50	50	1.3125
10:20	4 1/8	1:00	60	1.3750
10:30	4 1/8	1:10	70	1.3750
10:40	4 1/16	1:20	80	1.4375
10:50	4 1/16	1:30	90	1.4375
11:00	4	1:40	100	1.5000
11:10	4	1:50	110	1.5000
11:20	4	2:00	120	1.5000
12:20	3 15/16	3:00	180	1.5625
13:20	3 7/8	4:00	240	1.6250
14:20	3 13/16	5:00	300	1.6875
15:20	3 13/16	6:00	360	1.6875
16:20	3 3/4	7:00	420	1.7500
17:20	3 3/4	8:00	480	1.7500
18:20	3 3/4	9:00	540	1.7500

L =	1.083	ft
L_f =	11	ft
L/y_p =	2.6	
B =	1.5	ft
Q =	0.46	cfs
y_1 =	7	inch
y_p =	5	inch
V_{avep} =	0.736	ft/s
V_1 =	0.492	ft/s
V_2 =	0.492	ft/s
V_{ave} =	0.492	ft/s
Q_v =	0.431	cfs
S =	0.0023	
T =	64	°F

Notes: Duct tape seems to be holding the Styrofoam a little low, not quite reaching the underside of the braces. In time, the Styrofoam did rise up. ~3pm, electrical circuit board for pump burned out. Dan switched to the auxiliary pump, there was no noticeable change in flow rate.


Test 15 - 10% cover, high Q, low y1
ys [inches]
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-100
0
100
200
300
400
500
600
time [min]

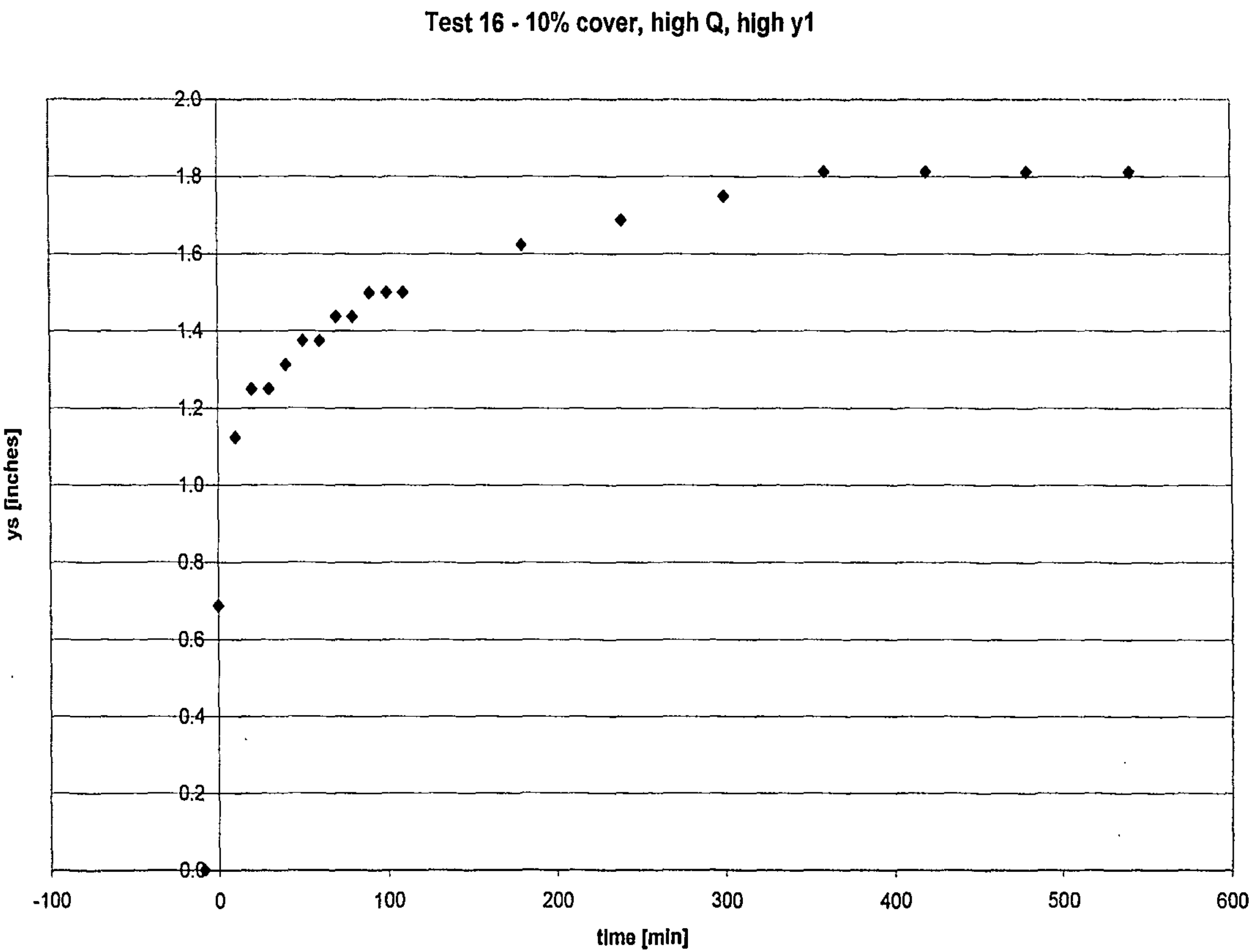
Test No.: 16

Date: 2/5/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
8:41	5 5/8	-0:09	-9	0.0000
8:50	4 15/16	0:00	0	0.6875
9:00	4 1/2	0:10	10	1.1250
9:10	4 3/8	0:20	20	1.2500
9:20	4 3/8	0:30	30	1.2500
9:30	4 5/16	0:40	40	1.3125
9:40	4 1/4	0:50	50	1.3750
9:50	4 1/4	1:00	60	1.3750
10:00	4 3/16	1:10	70	1.4375
10:10	4 3/16	1:20	80	1.4375
10:20	4 1/8	1:30	90	1.5000
10:30	4 1/8	1:40	100	1.5000
10:40	4 1/8	1:50	110	1.5000
11:50	4	3:00	180	1.6250
12:50	3 15/16	4:00	240	1.6875
13:50	3 7/8	5:00	300	1.7500
14:50	3 13/16	6:00	360	1.8125
15:50	3 13/16	7:00	420	1.8125
16:50	3 13/16	8:00	480	1.8125
17:50	3 13/16	9:00	540	1.8125

$L =$	1.083	ft
$L_f =$	11	ft
$L/y_p =$	2.6	
$B =$	1.5	ft
$Q =$	0.46	cfs
$y_1 =$	9	inch
$y_p =$	5	inch
$V_{avep} =$	0.736	ft/s
$V_1 =$	0.394	ft/s
$V_2 =$	0.361	ft/s
$V_{ave} =$	0.377	ft/s
$Q_v =$	0.424	cfs
$S =$	0.0023	
$T =$	64	°F

Notes: I am in class 10:45 am to 12:00 pm.


Test 16 - 10% cover, high Q, high y1
ys [inches]
time [min]
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-100
0
100
200
300
400
500
600

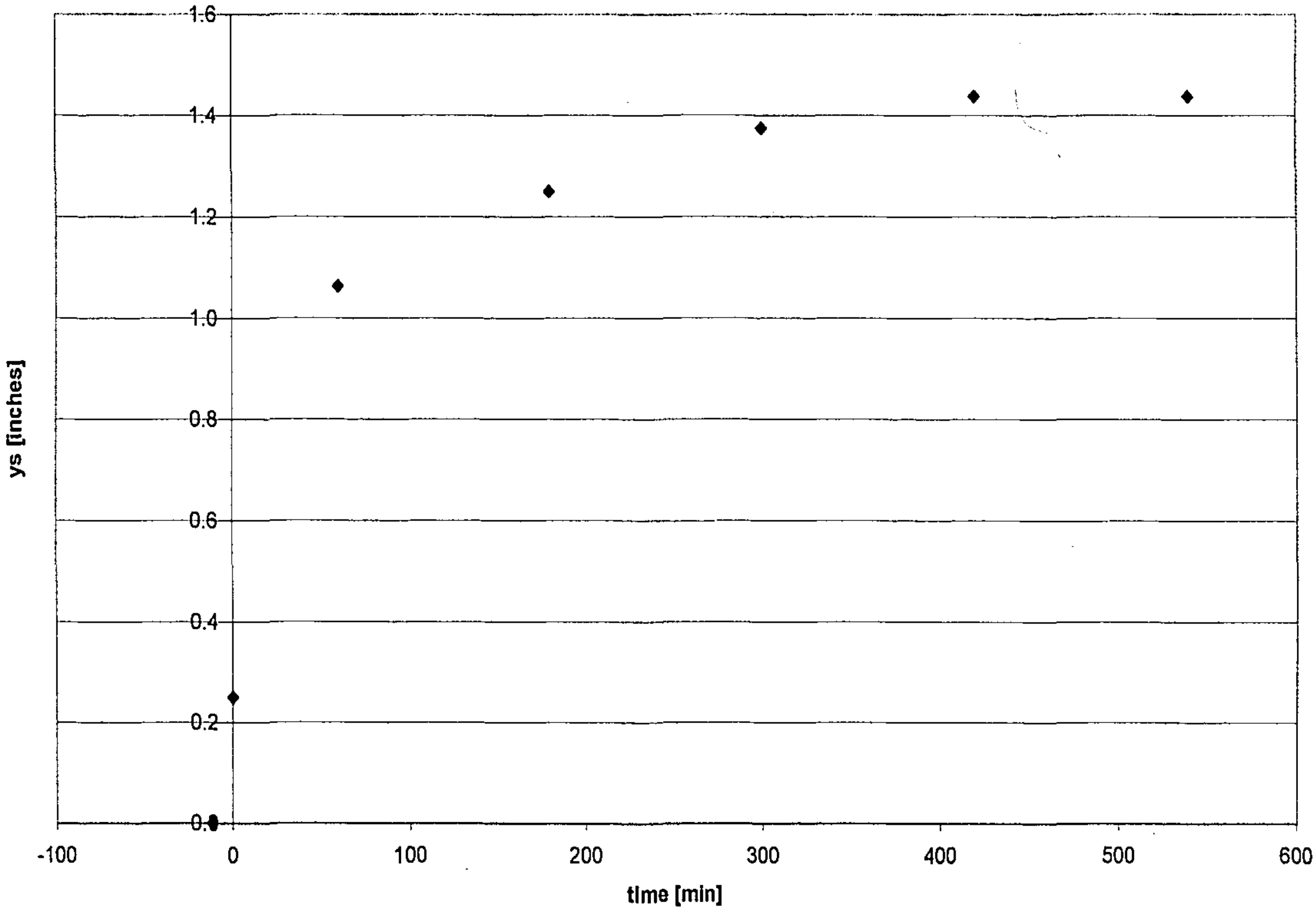
Test No.: 9A
Date: 2/11/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
9:05	5 5/8	-0:12	-12	0.0000
9:17	5 3/8	0:00	0	0.2500
10:17	4 9/16	1:00	60	1.0625
12:17	4 3/8	3:00	180	1.2500
14:17	4 1/4	5:00	300	1.3750
16:17	4 3/16	7:00	420	1.4375
18:17	4 3/16	9:00	540	1.4375

$L =$	2.75	ft
$L_f =$	11	ft
$L/y_p =$	6.6	
$B =$	1.5	ft
$Q =$	0.38	cfs
$y_1 =$	9	inch
$y_p =$	5	inch
$V_{avep} =$	0.608	ft/s
$V_1 =$	0.328	ft/s
$V_2 =$	0.394	ft/s
$V_{ave} =$	0.361	ft/s
$Q_v =$	0.406	cfs
$S =$	0.0023	
$T =$	64	°F

Notes: This is a repeat of the test performed on 1/26/04. It was noticed at the very beginning of the test, water was leaking by the dam that was set up and overtopping the ice. About 1 ½ hours into the test, a piece of Styrofoam was allowed to set on the water on top of the ice, and no forward velocity was observed. However, the initial overtopping may contribute to a smaller flow rate passing under the ice cover at the beginning of the test, where the highest rate of scour occurs. This may cause a smaller equilibrium scour value. Less readings were taken for this repeat test, since the equilibrium value is all that is desired.

Test 9A - 25% cover, low Q, high y1

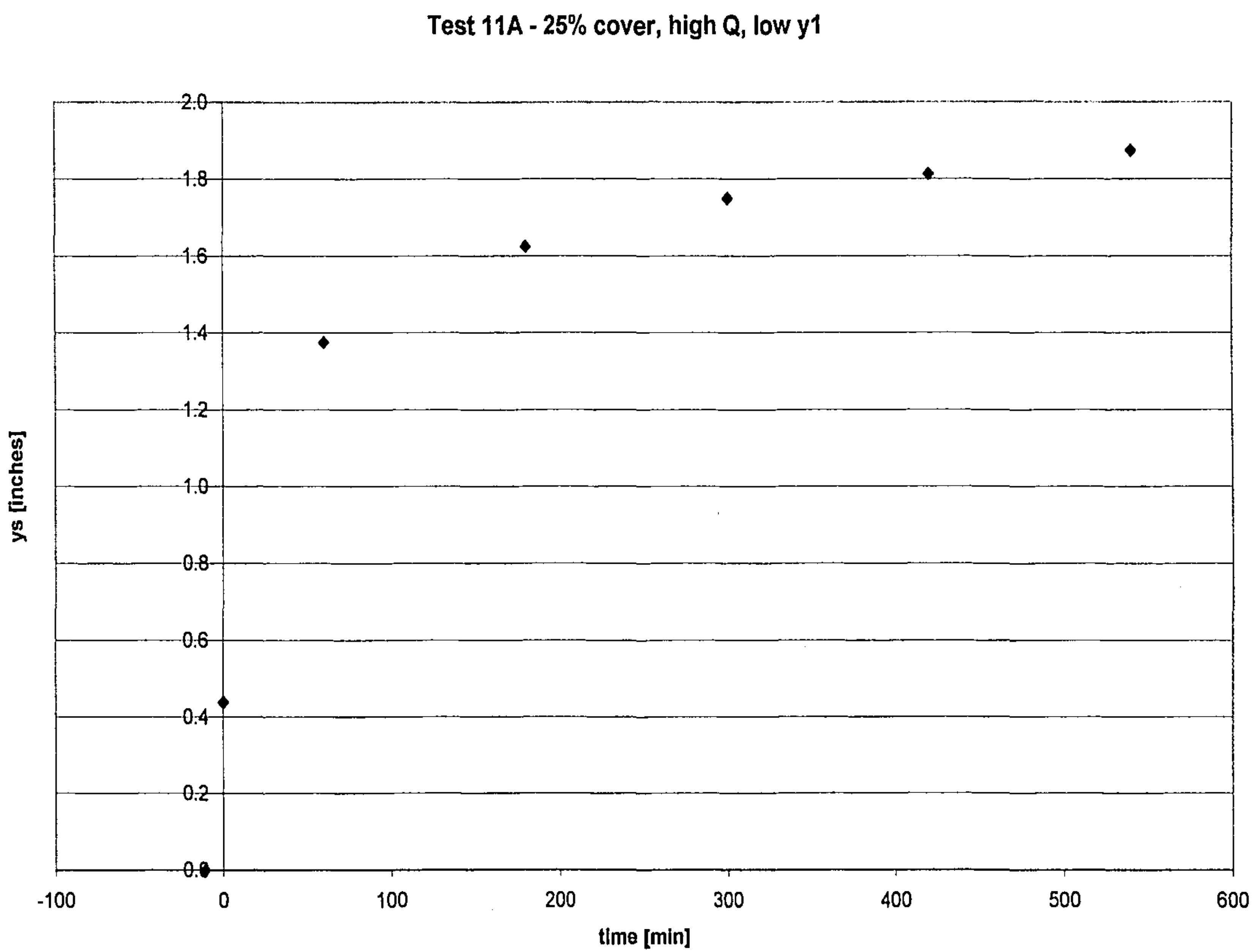


Test No.: 11A
Date: 2/11/2004

Time	Tape Reading [inch]	Elapsed Time [hr:min]	Elapsed Time [min]	y_s [inch]
9:06	5 9/16	-0:11	-11	0.0000
9:17	5 1/8	0:00	0	0.4375
10:17	4 3/16	1:00	60	1.3750
12:17	3 15/16	3:00	180	1.6250
14:17	3 13/16	5:00	300	1.7500
16:17	3 3/4	7:00	420	1.8125
18:17	3 11/16	9:00	540	1.8750

L =	2.75	ft
L_f =	11	ft
L/y_p =	6.6	
B =	1.5	ft
Q =	0.46	cfs
y_1 =	7	inch
y_p =	5	inch
V_{avep} =	0.736	ft/s
V_1 =	0.558	ft/s
V_2 =	0.558	ft/s
V_{ave} =	0.558	ft/s
Q_v =	0.488	cfs
S =	0.0023	
T =	64	°F

Notes: This is a repeat of the test performed on 1/28/04. Less readings were taken for this repeat test, since the equilibrium value is all that is desired.


Test 11A - 25% cover, high Q, low y1
ys [inches]
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-100
0
100
200
300
400
500
600
time [min]

APPENDIX B: VELOCITY PROFILE DATA

This appendix contains the data that was collected during the velocity profile portion of this experiment. The results are presented in two sections: the 100% cover case and the 50% cover case. The data presented for each test is as follows: a table of the time-averaged velocity readings and graphs of the velocity vs. height data. Two sets of data were taken for the 100% case, one 5 feet upstream of the pier and one 2.5 feet upstream of the pier. One set of data was taken for the 50% case at 2.5 feet upstream of the pier. The data for the second point taken on January 15, 2004, was lost in data processing and will not be retaken.

January 14 2004 - Pt 1				January 14 2004 - Pt 2				January 15 2004 - Pt 1			
Time [sec]	Marker #	Vx [cm/s]	Vx/Vc	Time [sec]	Marker #	Vx [cm/s]	Vx/Vc	Time [sec]	Marker #	Vx [cm/s]	Vx/Vc
Average:	1	16.051	0.560	Average:	1	14.153	0.494	Average:	1	15.610	0.545
Average:	2	16.802	0.586	Average:	2	16.65633	0.581	Average:	2	17.478	0.610
Average:	3	17.53567	0.612	Average:	3	17.52167	0.612	Average:	3	18.28067	0.638
Average:	4	17.86567	0.624	Average:	4	17.672	0.617	Average:	4	18.40467	0.642
Average:	5	17.763	0.620	Average:	5	18.18433	0.635	Average:	5	19.38833	0.677
Average:	6	17.67933	0.617	Average:	6	18.46433	0.644	Average:	6	18.85633	0.658
Average:	7	17.894	0.625	Average:	7	18.25833	0.637	Average:	7	19.12167	0.667
Average:	8	17.611	0.615	Average:	8	18.39367	0.642	Average:	8	19.164	0.669
Average:	9	17.27433	0.603	Average:	9	18.15867	0.634	Average:	9	19.267	0.672
Average:	10	17.17533	0.599	Average:	10	18.412	0.643	Average:	10	19.07767	0.666
Average:	11	12.71367	0.444	Average:	11	17.009	0.594	Average:	11	18.33867	0.640
								Average:	12	15.923	0.556
Max Value:	7	17.894	0.625	Max Value:	6	18.46433	0.644	Max Value:	5	19.38833	0.677

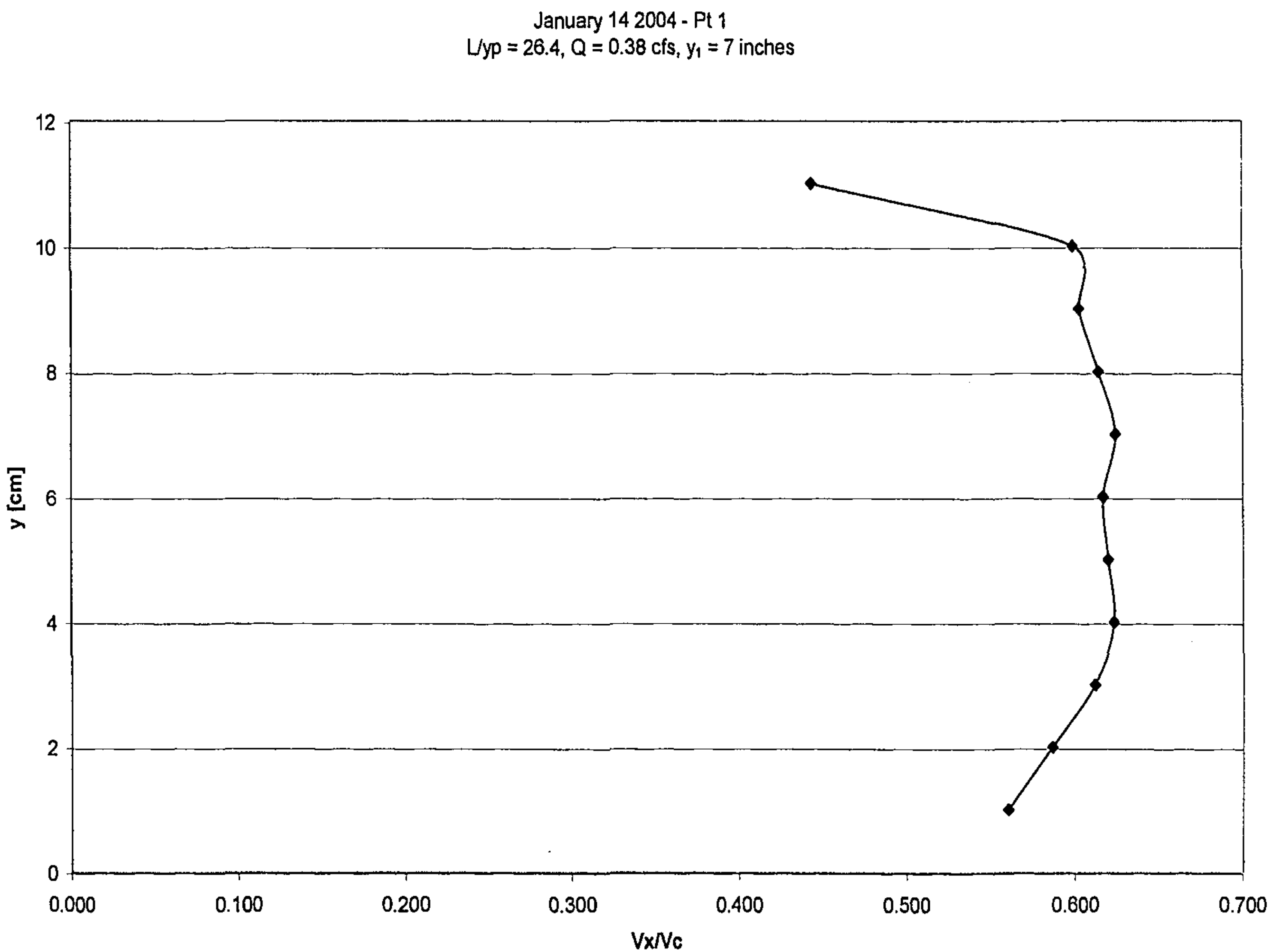
L/y_p =	26.4		L/y_p =	26.4		L/y_p =	26.4	
Q =	0.38	cfs	Q =	0.38	cfs	Q =	0.38	cfs
y1 =	7	inch	y1 =	7	inch	y1 =	9	inch
Vc =	0.94	ft/s	Vc =	0.94	ft/s	Vc =	0.94	ft/s
Vc =	28.6512	cm/s	Vc =	28.6512	cm/s	Vc =	28.6512	cm/s

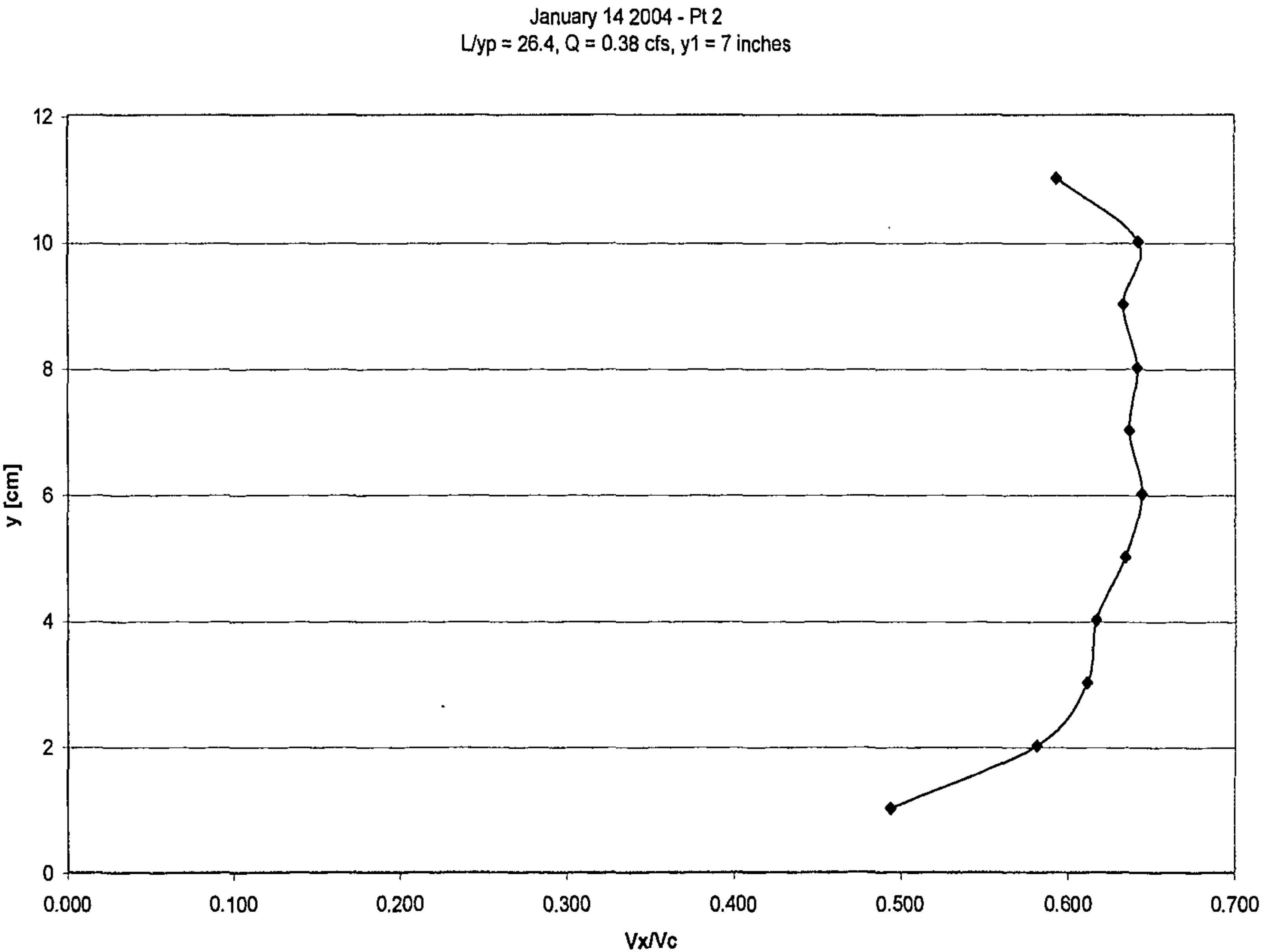
January 15 2004 - Pt 2				January 16 2004				January 19 2004			
Time [sec]	Marker #	Vx [cm/s]	Vx/Vc	Time [sec]	Marker #	Vx [cm/s]	Vx/Vc	Time [sec]	Marker #	Vx [cm/s]	Vx/Vc
Average:	1	15.198	0.530	Average:	1	21.124	0.737	Average:	1	23.516	0.821
Average:	#DIV/0!	#DIV/0!	#DIV/0!	Average:	2	22.74367	0.794	Average:	2	25.03733	0.874
Average:	#DIV/0!	#DIV/0!	#DIV/0!	Average:	3	23.20967	0.810	Average:	3	25.11633	0.877
Average:	#DIV/0!	#DIV/0!	#DIV/0!	Average:	4	23.206	0.810	Average:	4	24.96633	0.871
Average:	#DIV/0!	#DIV/0!	#DIV/0!	Average:	5	23.32767	0.814	Average:	5	24.785	0.865
Average:	#DIV/0!	#DIV/0!	#DIV/0!	Average:	6	23.28233	0.813	Average:	6	24.95667	0.871
Average:	#DIV/0!	#DIV/0!	#DIV/0!	Average:	7	23.452	0.819	Average:	7	24.42967	0.853
Average:	#DIV/0!	#DIV/0!	#DIV/0!	Average:	8	23.33333	0.814	Average:	8	24.224	0.845
Average:	#DIV/0!	#DIV/0!	#DIV/0!	Average:	9	23.14233	0.808	Average:	9	24.143	0.843
Average:	#DIV/0!	#DIV/0!	#DIV/0!	Average:	10	22.52533	0.786	Average:	10	23.065	0.805
Average:	#DIV/0!	#DIV/0!	#DIV/0!	Average:	11	22.458	0.784	Average:	11	21.19967	0.740
				Average:	12	21.708	0.758	Average:	12	19.242	0.672
				Average:	13	19.88233	0.694				
Max Value:			#DIV/0!	Max Value:	7	23.452	0.819	Max Value:	3	25.11633	0.877

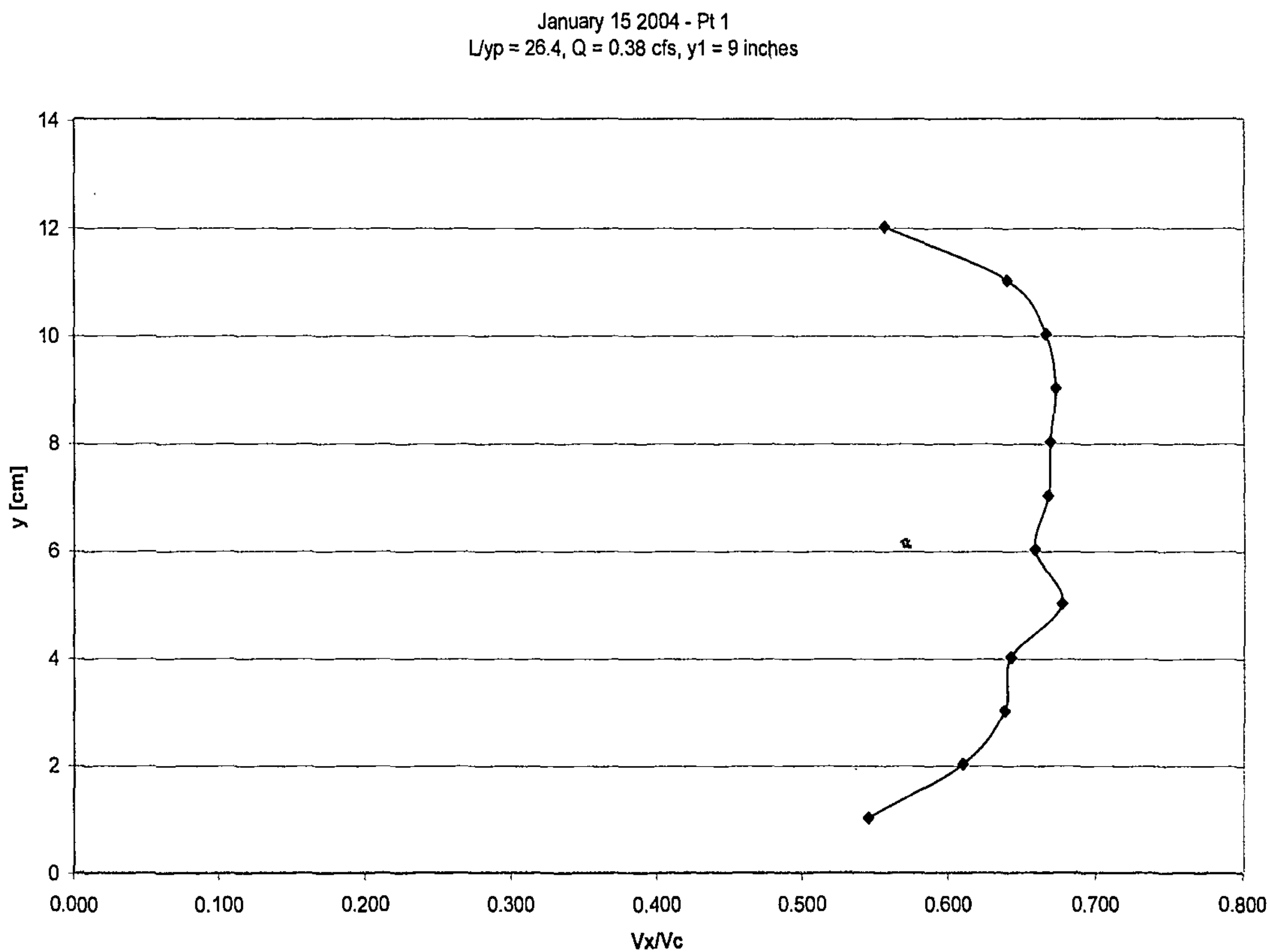
L/y_p =	26.4		L/y_p =	13.2		L/y_p =	13.2	
Q =	0.38	cfs	Q =	0.46	cfs	Q =	0.46	cfs
y1 =	9	inch	y1 =	7	inch	y1 =	9	inch
Vc =	0.94	ft/s	Vc =	0.94	ft/s	Vc =	0.94	ft/s
Vc =	28.6512	cm/s	Vc =	28.6512	cm/s	Vc =	28.6512	cm/s

January 21 2004				January 22 2004			
Time [sec]	Marker #	Vx [cm/s]	Vx/Vc	Time [sec]	Marker #	Vx [cm/s]	Vx/Vc
Average:	1	21.101	0.736	Average:	1	19.466	0.679
Average:	2	22.43333	0.783	Average:	2	19.80667	0.691
Average:	3	22.00533	0.768	Average:	3	19.898	0.694
Average:	4	22.26767	0.777	Average:	4	20.03433	0.699
Average:	5	22.78467	0.795	Average:	5	20.00233	0.698
Average:	6	22.68233	0.792	Average:	6	19.87567	0.694
Average:	7	22.992	0.802	Average:	7	19.78767	0.691
Average:	8	22.865	0.798	Average:	8	19.37467	0.676
Average:	9	21.33033	0.744	Average:	9	19.627	0.685
Average:	10	21.05833	0.735	Average:	10	18.669	0.652
Average:	11	20.01467	0.699	Average:	11	17.30967	0.604
Max Value:	7	22.992	0.802	Max Value:	4	20.03433	0.699

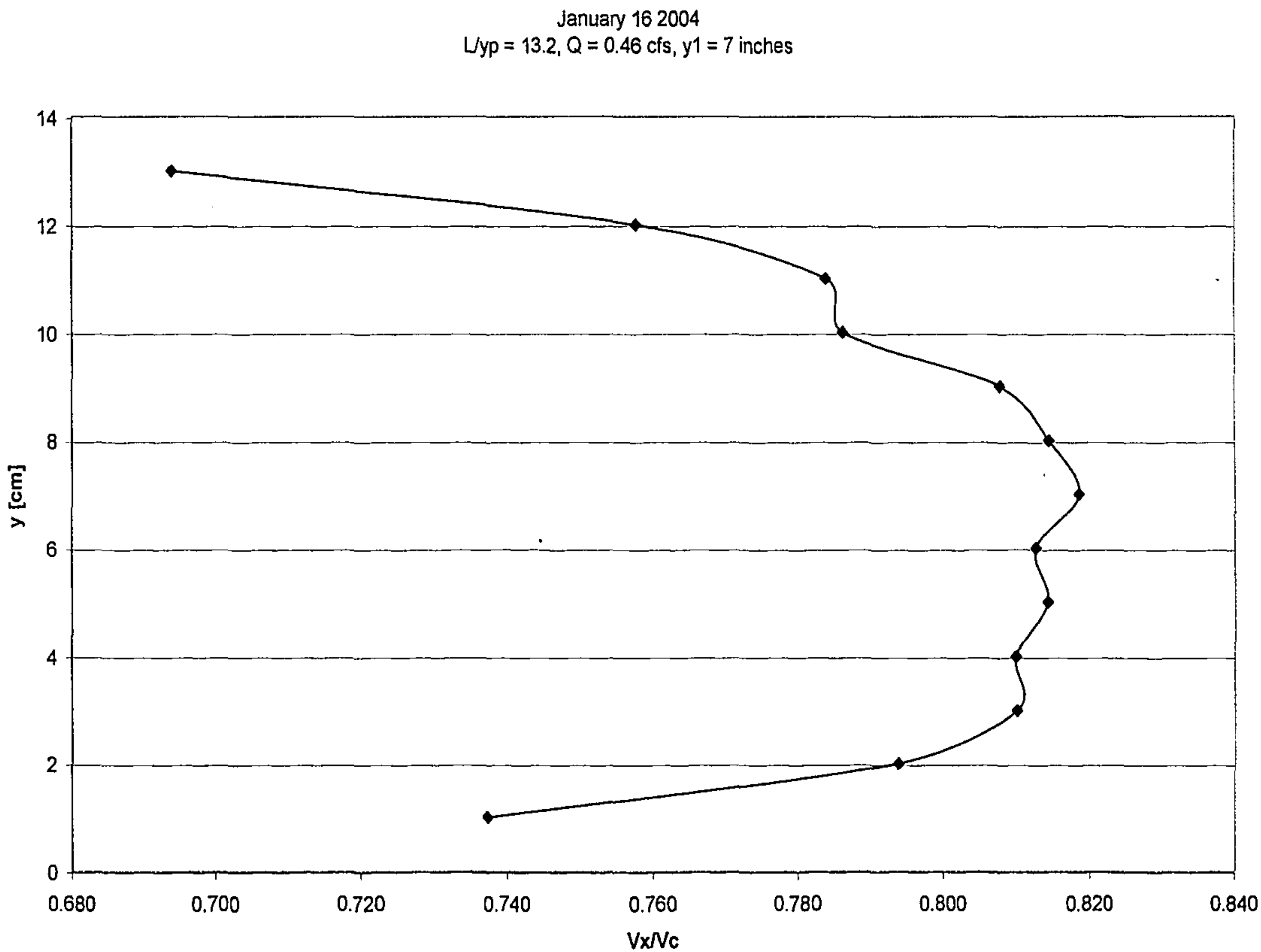
L/y_p =	13.2		L/y_p =	13.2	
Q =	0.38	cfs	Q =	0.38	cfs
y1 =	7	inch	y1 =	9	inch
Vc =	0.94	ft/s	Vc =	0.94	ft/s
Vc =	28.6512	cm/s	Vc =	28.6512	cm/s

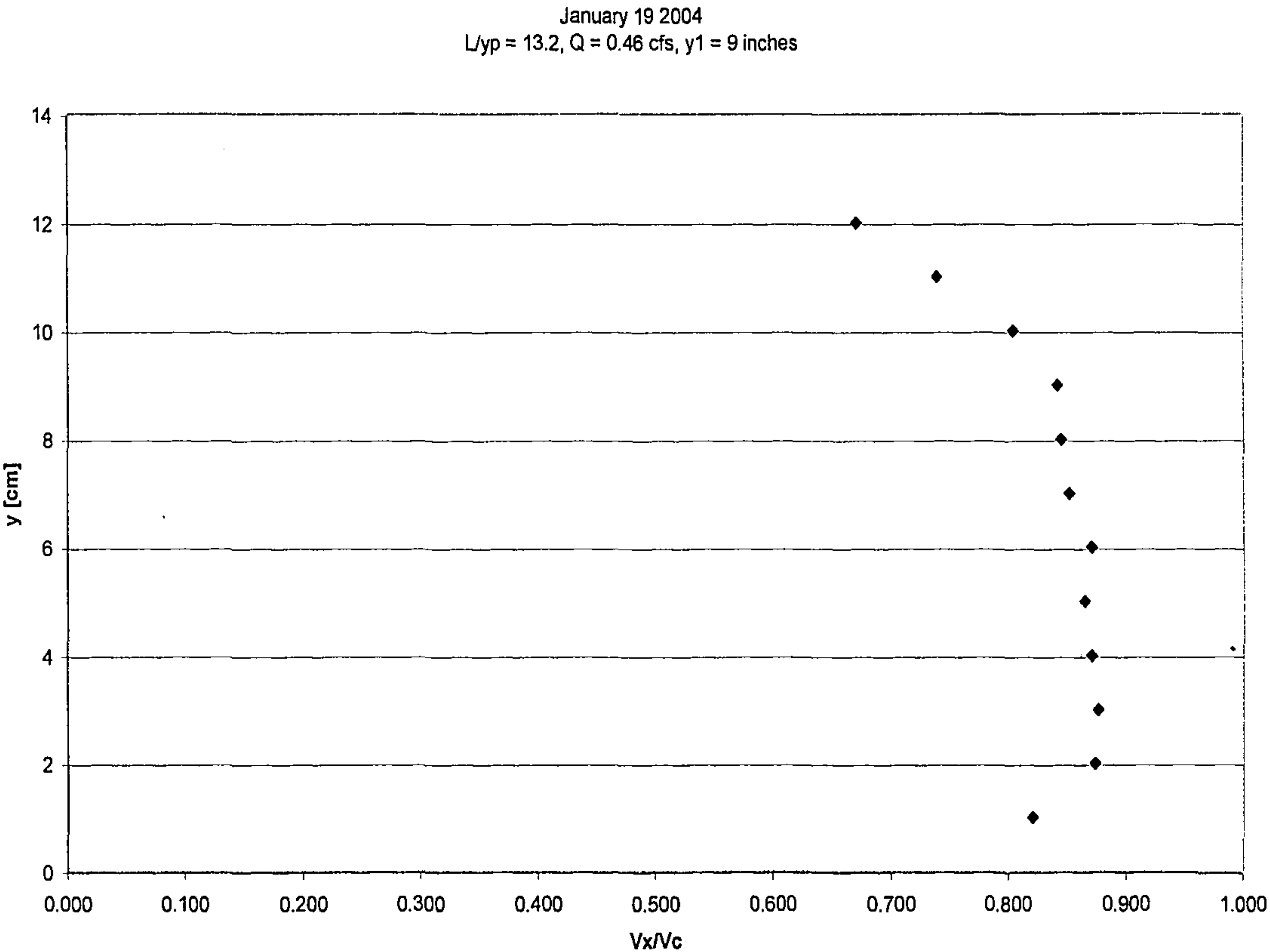

January 14 2004 - Pt 1
L/yp = 26.4, Q = 0.38 cfs, y_1 = 7 inches
y [cm]
0
2
4
6
8
10
12
Vx/Vc
0.000
0.100
0.200
0.300
0.400
0.500
0.600
0.700

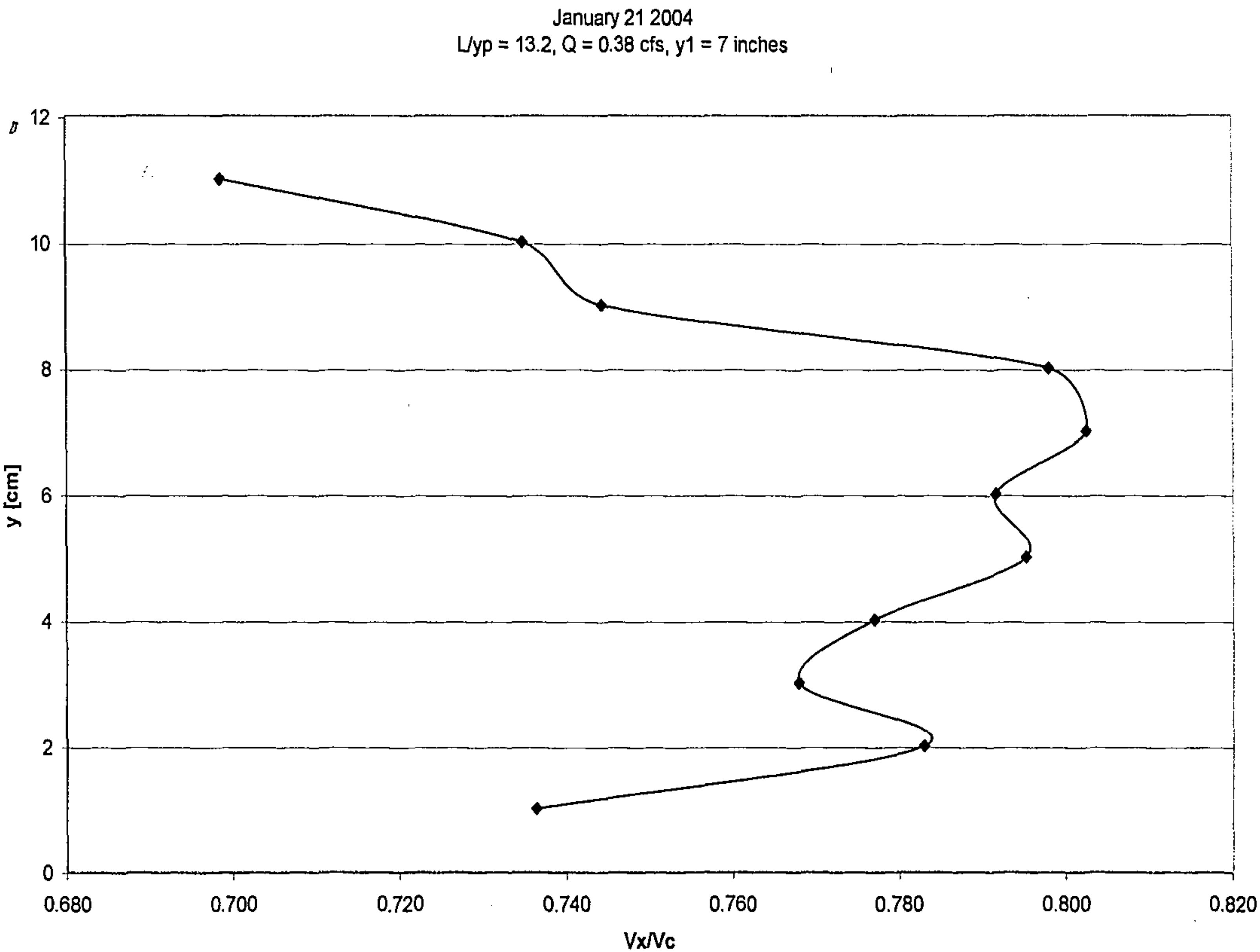

January 14 2004 - Pt 2
L/yp = 26.4, Q = 0.38 cfs, y1 = 7 inches
y [cm]
Vx/Vc
0
2
4
6
8
10
12
0.000
0.100
0.200
0.300
0.400
0.500
0.600
0.700

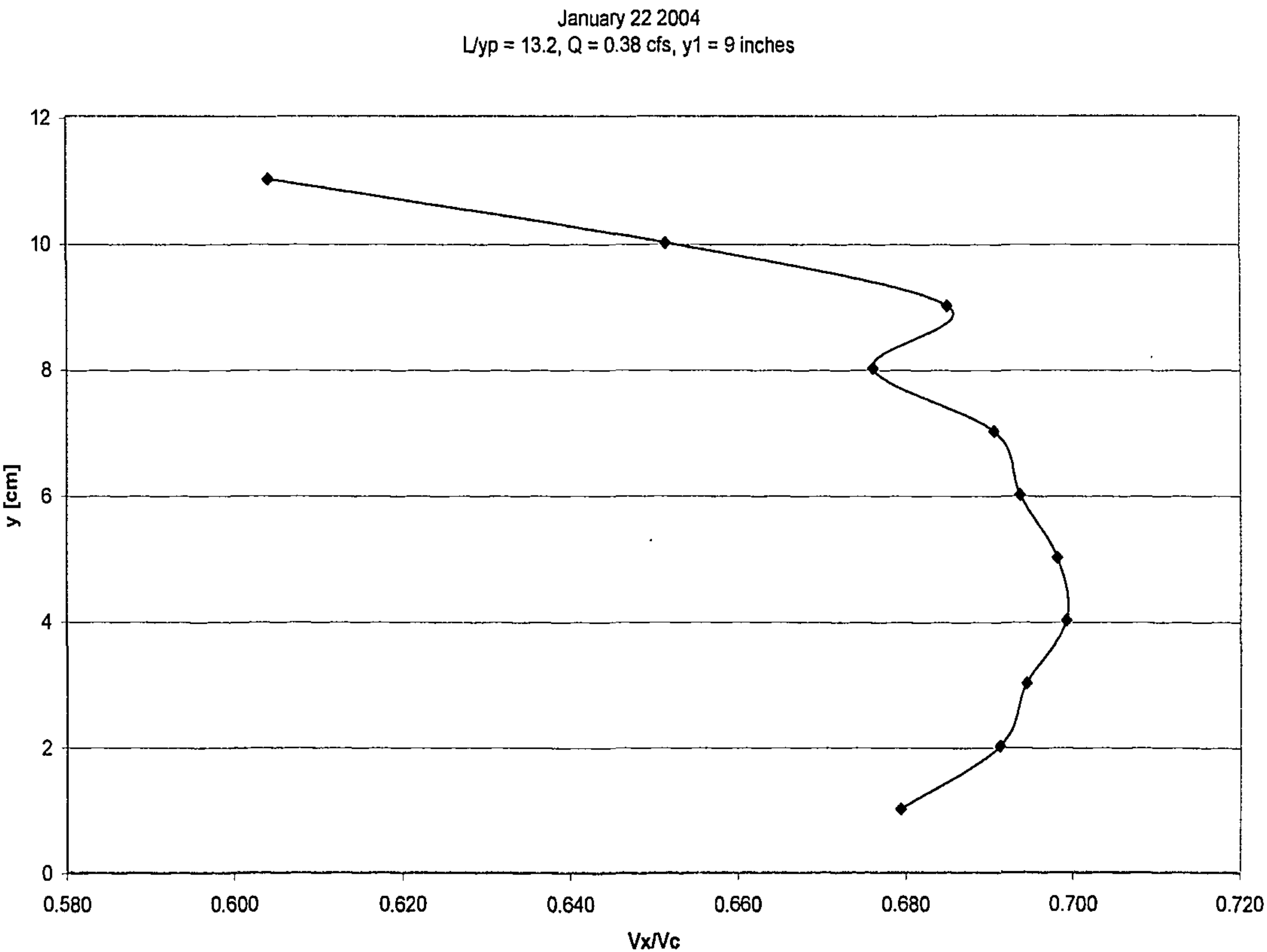


January 15 2004 - Pt 1
L/yp = 26.4, Q = 0.38 cfs, y1 = 9 inches
14
12
10
8
6
4
2
0
y [cm]
0.000
0.100
0.200
0.300
0.400
0.500
0.600
0.700
0.800
Vx/Vc


January 16 2004
L/yp = 13.2, Q = 0.46 cfs, y1 = 7 inches
y [cm]
Vx/Vc
0
2
4
6
8
10
12
14
0.680
0.700
0.720
0.740
0.760
0.780
0.800
0.820
0.840


January 19 2004
L/yp = 13.2, Q = 0.46 cfs, y1 = 9 inches
y [cm]
Vx/Vc
14
12
10
8
6
4
2
0
0.000
0.100
0.200
0.300
0.400
0.500
0.600
0.700
0.800
0.900
1.000


January 21 2004
L/yp = 13.2, Q = 0.38 cfs, y1 = 7 inches
y [cm]
0
2
4
6
8
10
12
Vx/Vc
0.680
0.700
0.720
0.740
0.760
0.780
0.800
0.820


January 22 2004
L/yp = 13.2, Q = 0.38 cfs, y1 = 9 inches
y [cm]
12
10
8
6
4
2
0
0.580
0.600
0.620
0.640
0.660
0.680
0.700
0.720
Vx/Vc

APPENDIX C: VITA

Karen Elizabeth Miranda was born on March 23, 1981 in Plainview, NY to Keith and Mary Ann Miranda. She graduated from Lehigh University with a Bachelor of Science degree in Civil and Environmental Engineering on May 19, 2003. As a recipient of the Lehigh University Presidential Scholarship, Karen continued on work towards her Master of Science degree in the summer of 2003.

While enrolled at Lehigh, Karen was a member of Tau Beta Pi, Chi Epsilon, Lehigh University Tour Guides, and several Music Department organizations. She served as the initiation coordinator for Tau Beta Pi from spring 2002 to spring 2003 as well as the publicity manager for the Lehigh University Marching 97 from spring 2001 to spring 2003.

Upon graduation from Lehigh with her Master of Science degree in Civil and Environmental Engineering, Karen will be employed at Parsons Energy & Chemical in Reading, Pennsylvania. She is a licensed Engineering-In-Training in Pennsylvania.

END OF
TITLE