SCOUR EVALUATION ON THE LITTLE BEAVER CREEK CROSSING ON GOSHEN ROAD MAHONING COUNTY, OHIO.

by

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ABSTRACT

Design of bridges over major waterways takes into consideration the effects of scour, or long term lowering of the river channel from an assumed datum. The effects of scour on a structure can be devastating, and many bridge failures have been attributed to this cause. A bridge over a small Northeastern Ohio stream, the North Fork of Little Beaver Creek, was studied to analyze and predict scour effects during 100 and 500 year flood events. The Federal Highway Administration (FHWA) methodology was used for this analysis. Scour estimates reveal that the bridge will most likely fail under both 100 year and 500 year events. This is primarily due to the undermining of the south abutment due to erosion. Had the structure not been subject to this cumulative damage, these flood events may not cause failure. Another conclusion found by this study is that a thorough understanding of the accepted scour analysis equations is required to produce accurate results.

It is suggested that scour countermeasures be immediately incorporated to ensure the safety of motorists using this bridge. Potential countermeasures include abutment strengthening through backfill and foundation construction, channel restoration to the original design configuration, tributary relocation and installation of riprap. In the absence of the onset of a major flood event, the bridge requires some of these improvements to offset the effect of the creek flowing partially under the south abutment.

DEDICATION

To Joe Ginocci and Mike Simonsic for encouraging me to be an engineer.

To Francine for her patience, understanding and encouragement throughout my academic pursuits.

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Chapter I: INTRODUCTION

1.1 Introduction

Engineering of bridges is an important factor in the development and maintenance of transportation systems. In order to provide the quickest, safest or most cost efficient route for transportation, it is often necessary to span natural valleys and crevasses with bridges. Many bridges span natural waterways having constant or intermittent hydrology, thereby subjecting the bridge supports to fluctuating hydraulic forces. Support structures which are placed on each end of the bridge are called abutments, and structures placed between spans are called piers. Each design varies according to site conditions, ideally subjecting the abutments and piers to minimum contact with the current. Abutments are often placed as far as economically possible from the main channel and skewed to accomplish this. In many instances, however, abutments are subject to hydraulic forces under normal flow and flood conditions. Piers are generally more susceptible to hydraulic forces, as they are often placed in, or near mid-channel. In general, structures placed on spread footings are most susceptible to the effects of scour. For this reason, piers and abutments are often placed on deep pilings.

When support structures are subject to hydraulic forces, erosion around and under these features is a primary concern, as excessive erosion can cause a bridge to fail. This erosion, or scour, has been responsible for the collapse of many bridges during high water conditions. One notable example is the five-span, multilane, New York Thruway bridge spanning Schoharie Creek which collapsed as a result of pier failure due to local scour. This 1987 accident caused the deaths of 10 people. For this reason, a technical advisory on the subject of Scour at Bridges was issued by the Federal Highway Administration of the U.S. Department of Transportation on September 16, 1988. One of the purposes of this advisory was to provide guidance on developing and implementing a scour evaluation program for existing bridges and using remedial measures on scour critical bridges. Scour critical is defined as being unable to withstand the scouring caused by a superflood (1.7 times 100 year flood) or less. It was recommended in this advisory that every existing bridge over a scourable stream should be evaluated concerning its vulnerability to floods in order to determine the prudent measures required for its protection. It is estimated that the added cost of making a bridge less vulnerable to scour is small when compared to the total cost of a failure, which can easily be two or three times the original cost of the bridge itself (FHWA, 1988).

The advisory made the following recommendations regarding scour evaluation on the existing bridges:

(1) An initial screening process should be developed to identify bridges most likely to be susceptible to scour damage and to establish a priority list for evaluation.

(2) Bridge scour evaluations should be conducted for each bridge to determine whether it is scour critical. A scour critical bridge is one with abutment or pier foundations which are rated as unstable due to:
a) observed scour at the bridge site or
b) a scour potential as determined from a scour evaluation study.

(3) The interim procedures in Chapter 5 of the publication titled Interim Procedures for Evaluating Scour at Bridges should be followed in conducting and documenting the results of scour evaluation studies.

(4) A plan of action should be developed for each existing bridge determined to be scour critical. The plan of action should include (FHWA, 1988):
a) instructions regarding the type and frequency of inspections to be

made at the bridge, particularly in regard to monitoring the performance and closing of the bridge, if necessary, during and after flood events.

b) a schedule for timely design and construction of scour countermeasures determined to be needed for the protection of the bridge.

Scour is the lowering of the streambed below a natural level or below an assumed datum. Scour depth is the depth of the bed material removed below this level. Total scour is that which results from the sum of three types: degradation; contraction; and local scour. Degradation is the long-term lowering (scouring) of the stream bed due to cumulative erosion. Contraction scour, or general scour is the erosion of material from the bed and banks across all or most of the width of a channel associated with a single event. Local Scour is restricted to a minor part of the width of a channel, and is caused by the acceleration of flow and the development of vortex systems induced by the obstructions to the flow (FHWA, 1988).

Although many studies have been conducted on these processes, equations for predicting the outcome in any situation can give highly variable results. This is due to the large number of variables found between structures, and the lack of physical data to support the equations. Many of the relationships in the equations used for the following analysis were developed in a laboratory setting. Because of this, the engineer must use good judgment in making decisions and measurements to be input into the scour prediction equations.

1.2 Project Objectives

The primary objectives of this project are outlined as follows:

1. Develop an acceptable methodology for scour evaluation using established procedures as guidelines.

2. Evaluate the potential bridge scour for a bridge in Northeastern Ohio designated by the County Engineer as a priority bridge.

3. Provide recommendations as to the action that should be taken to remedy any scour critical potential that may exist on that bridge.

4. Provide preventative strategies for pier and abutment design to avoid future scour critical situations.

1.3 Methodology

The methodology used in this analysis was formulated according to the guidelines established in the documents, Interim Procedures for Evaluating Scour at Bridges (FHWA, 1988) and Bridge Scour Evaluation Procedure for Minnesota Bridges (Minnesota Department of Transportation, 1995). Throughout the data gathering stage, the original methodology was adapted to meet the needs of the project with the available information. Organizational forms were obtained from the Minnesota Department of Transportation. The procedure used to accomplish each of the following steps is described in detail within this thesis. The general methodology was as follows:

1. Bridge selection.
2. Determination of analysis variables for runoff, elevation and scour equations.
3. Analysis of long term bed elevation change.
4. Scour analysis method selection.
5. Calculation of contraction scour.
6. Calculation of local scour at abutments.
7. Calculation of local scour at piers.
8. Plotting the total scour depths.
9. Bridge evaluation and recommendations.
10. Recommendations for future analysis methodology.

1.4 Bridge Selection

The first step of bridge selection is obtaining a list of priority bridges from the County Engineer. Bridge selection is based on identifying those with actual or potential problems including:

(1) bridges currently experiencing scour or that have experienced scour during past floods.

(2) bridges over streams with erodible beds with piers and abutments designed with spread footings or short pile foundations, superstructures with simple spans or non-redundant support systems that render them vulnerable to collapse in the event of foundation movement, and bridges with inadequate waterway openings or with designs that collect ice and debris.

(3) bridges on aggressive streams and waterways, including those with active degradation or aggradation of the stream bed, significant lateral movement or erosion of stream banks, steep slopes or high velocities, gravel or mining operations in the vicinity of the bridge, and histories of having damaged highways and bridges during past floods.

(4) bridges located on stream reaches with adverse flow characteristics such as crossings near stream confluences, crossings on sharp bends in a stream and location in alluvial fans.
(FHWA, 1988)

The Mahoning County Engineer's Bridge Department office was contacted January 28, 1997 about a scour susceptible bridge. The engineer noted that the Goshen Road Bridge over the Middle Fork of the Little Beaver Creek was currently experiencing scour and had experienced scour during past floods (Figure 1.1). This crossing was redesigned to address this problem in 1988, and has yet to receive allocation of funds. The engineer stated that some basic scour prevention methods, specifically the use of riprap, may be employed in the upcoming year to slow the scour process.

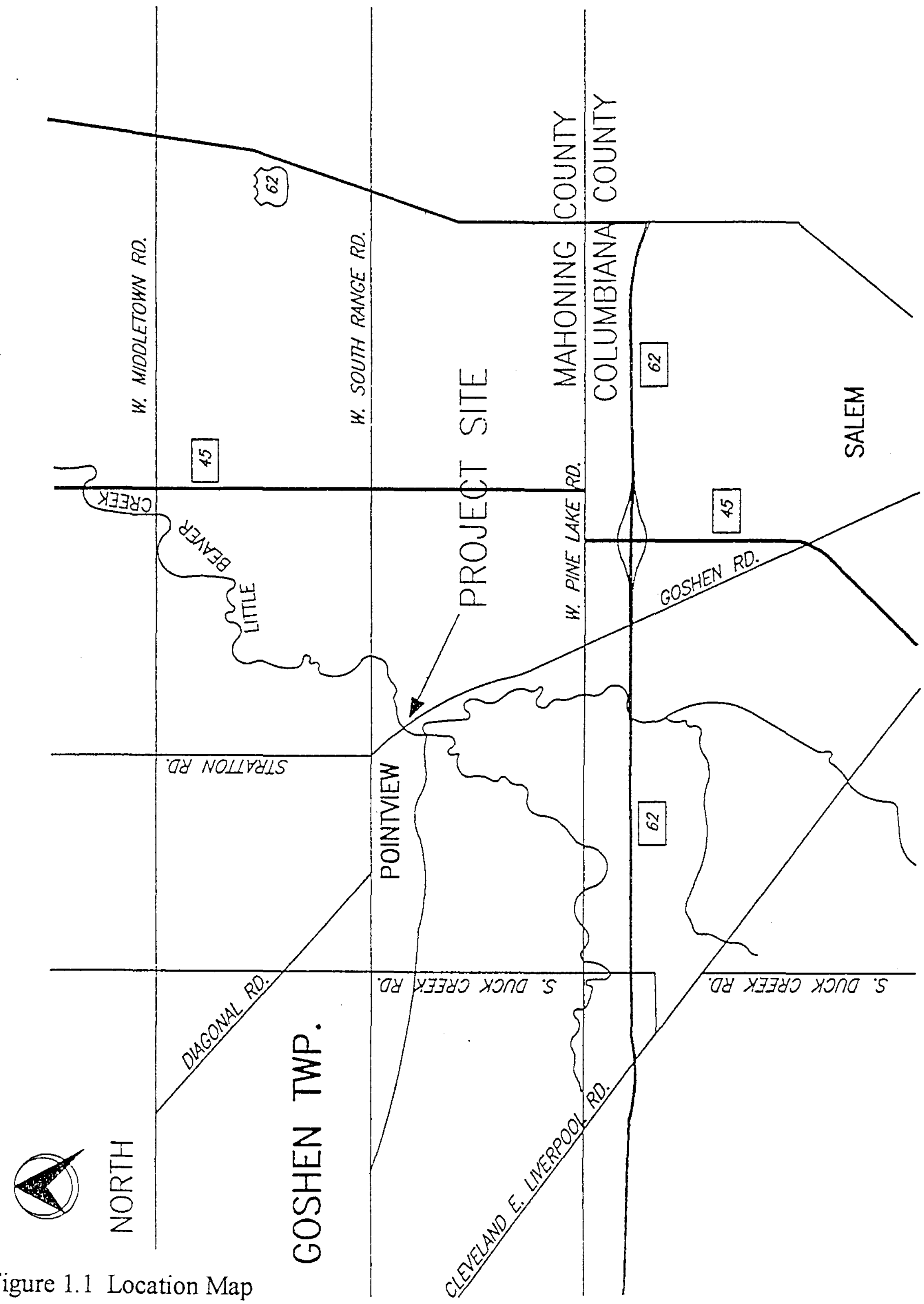


Figure 1.1 Location Map

A meeting was arranged for January 30, 1997, as a data collection step using the "Bridge Scour Analysis - Sources of Information" form provided by the Minnesota Department of Transportation (Appendix D, Figure D-1). This form organizes general bridge site information to provide a starting point for the analysis. Data gathered in this step included a bridge inspection report, bridge inventory and other sources of information (Appendix D, Figures D-5, D-6, D-7).

In the event that a large scale evaluation procedure is employed, it is recommended in the MnDOT document that the engineer follow a screening procedure to minimize engineering costs associated with full evaluations and address those with the most apparent problems first. The MnDOT document, "Bridge Scour Evaluation Procedure" specifies that bridges are screened in primary and secondary screening. This screening format provides a systematic approach to classifying bridges according to need. According to these criteria, primary screening rates a bridge as "low risk", "unknown foundation" or "scour susceptible" (Appendix D, Figure D-2). Using this evaluation tool, the bridge is rated as "scour susceptible" due to existing scour present at either abutment.

Secondary screening further reduces scour evaluation costs by reducing the number of bridges requiring detailed analysis. These criteria include seven parameters related to the performance of the bridge under scour conditions. These are: historical scour performance, scour resistant foundations, debris and blockage, geomorphic conditions affecting scour resistance, hydraulic conditions affecting scour resistance, structural conditions affecting scour resistance and monitored reduced risk bridges. Incorporating the secondary screening procedure (Appendix D, Figure D-4), the Goshen Road crossing requires evaluation of the monitored reduced risk bridges criteria only. Using this, the bridge is confirmed as "scour susceptible, rating J" requiring level 1 analysis. The level 1 scour evaluation, as defined by MnDOT involves performing the evaluation in accordance with the FHWA guidelines and specific Minnesota DOT conditions. The Goshen Road crossing was evaluated under the FHWA recommended procedures.

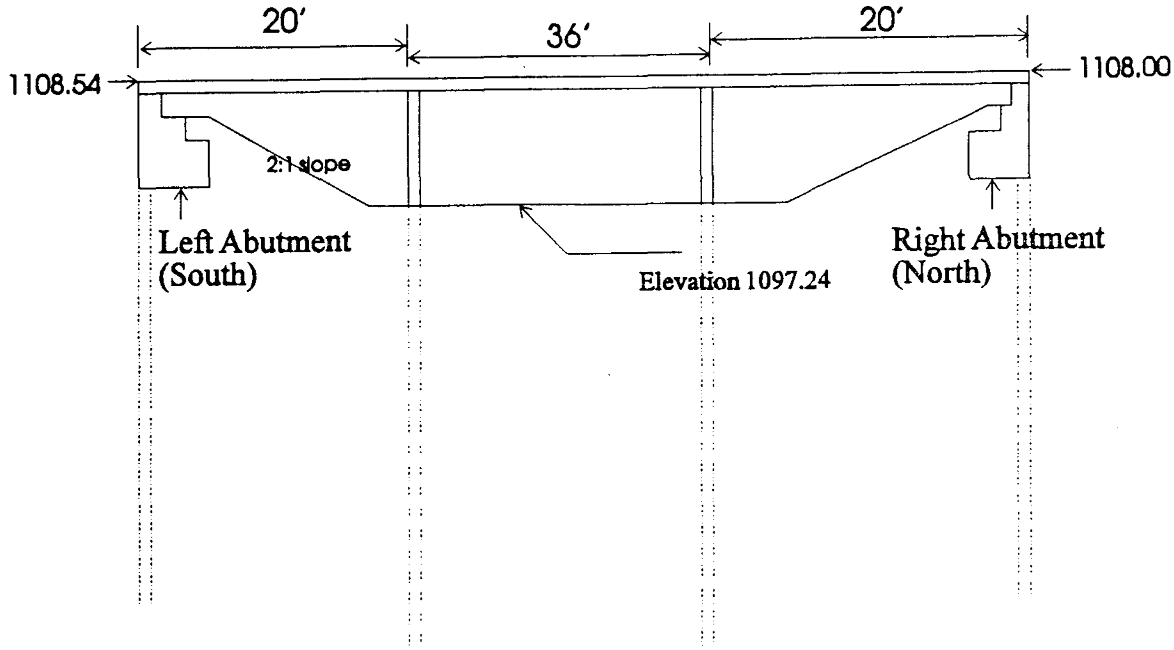


Figure 1.2 Original Bridge Profile

1.5 Summary Of Bridge Information

The Goshen Road bridge was constructed in 1958 consisting of 3 span concrete slab construction (20 ft, 36 ft, 20 ft). Span lengths are supported by abutments on both sides resting on underlying bedrock and 40 ft pilings. For the purpose of this project, the abutments on the southern and the northern sides of the channel are referred to as the south and north abutments, respectively (Figure 1.2). Piers are located between the spans, supported by 42 ft pilings. The deep piling design is an indication of the river's unpredictable meandering and potential for flooding.

The Middle Fork of the Little Beaver Creek watershed in southern Mahoning County is characterized by gentle rolling hills covered by forest, meadows and agricultural land. The creek flows in a northeast direction prior to the Goshen Road crossing with the southern portion of the watershed originating in the urban district of Salem. The remaining area is typical rural eastern Ohio geology, characterized by glacial till and outwash landscape providing soil types that are generally well drained. Interspersed throughout the watershed are ponds and swampy areas, but the rolling nature of the landscape dictates the characteristics of the stream that drains it.

The higher elevations of the watershed (Appendix C, Figure C-1) provide some buffering against flooding due to the scattered depressions. However, when the ground reaches saturation associated with heavy or prolonged precipitation events, the porous soil and rolling nature of the watershed can result in flooding. During dry periods, the stream assumes a meandering character through the agricultural properties. This variety is best demonstrated upstream from the Goshen Road crossing where the two tributaries join the main stem of the Middle Fork. These tributaries are characterized by a relatively high gradient while the main stem generally retains a low velocity character. This confluence is located approximately 1/8 mile upstream from the crossing (Figure 1.3).

A small tributary enters the Middle Fork immediately upstream of the Goshen Road crossing. This manmade channel drains some residences, forests and a small field on the north end of the watershed. Its location relative to the bridge plays a significant role in influencing the flow path of the channel under the bridge. A small storm sewer outlet is located on the south face of the channel on the downstream side of the bridge. These two factors have contributed greatly to the erosion around and under the south abutment during low and high flow conditions.

There is no known mining history in the area affecting the bridge, so any long term channel elevation fluctuations are not expected to occur. There is also no existing stream flow or scour data available on the creek, so on-site data accumulation is an important step in the analysis process.

During the original construction of the bridge, the stream channel was dredged to provide a perpendicular flow to the crossing. The original plans note a scour hole in the north side of the channel, which was backfilled. This and other features apparently indicated the variable nature of the stream to the engineer and that future meandering and potential high velocities were likely. This is evident through the use of deep pilings. It is apparent from the location of the upstream tributary and downstream storm outlet discharges that the original channel configuration would not retain its design (Figure 1.4).

During preliminary site inspection, it was noted that erosion from normal flow conditions and scour had undermined the south abutment. This erosion had been further advanced by erosion behind the downstream wingwall on the same side. The south front pier had accumulated a significant debris pile which had created a wide but shallow scour hole. The depths and locations of all scour areas were measured and drawn on upstream and downstream channel profiles (Figures 1.5, 1.6).

A form titled Bridge Scour Screening Data, developed by Minnesota DOT (AppendixD, D-3) was used to organize information. This is a useful tool for starting the evaluation and includes crossing data, structure data, hydraulic data and stream characteristics. Much of this can be obtained during the information gathering step from site visits and discussions with the engineer, while some information requires detailed calculations. The form also provides an outline for the final analysis.

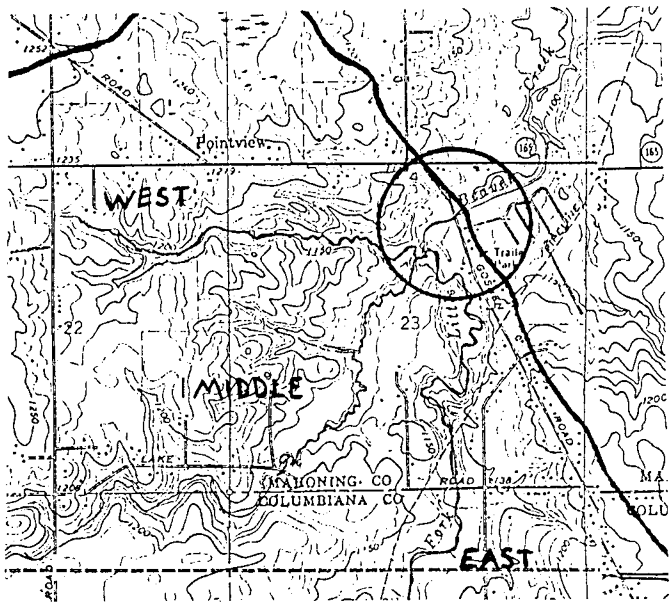


Figure 1.3 Stream Configuration

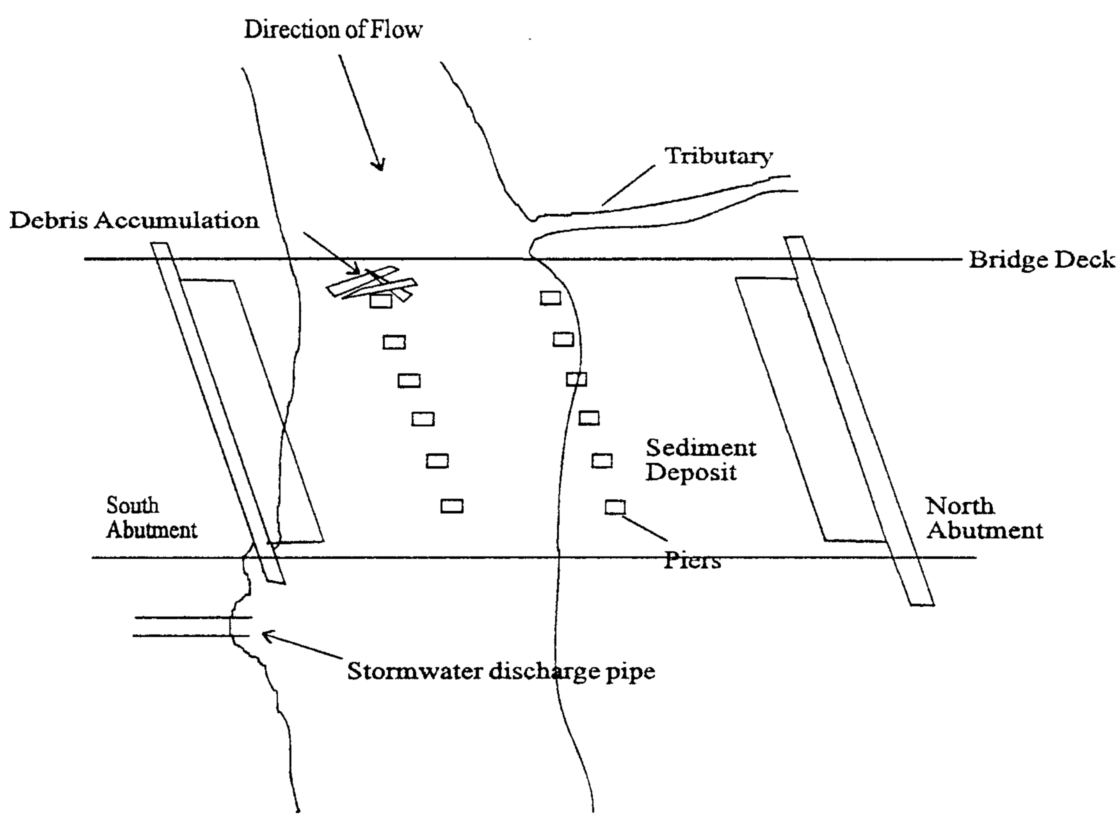


Figure 1.4 Crossing Configuration

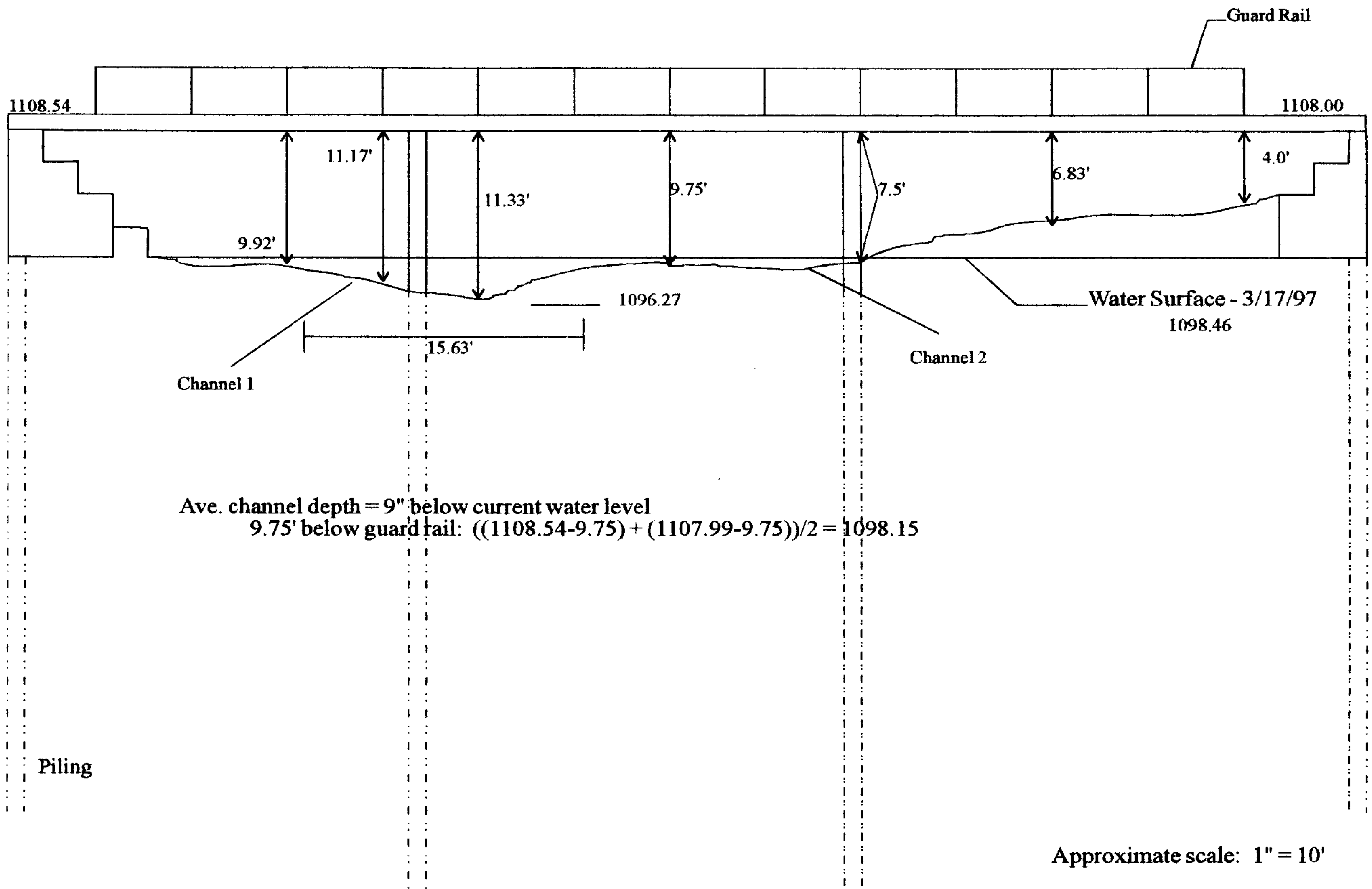


Figure 1.5 Channel Profile- Upstream face

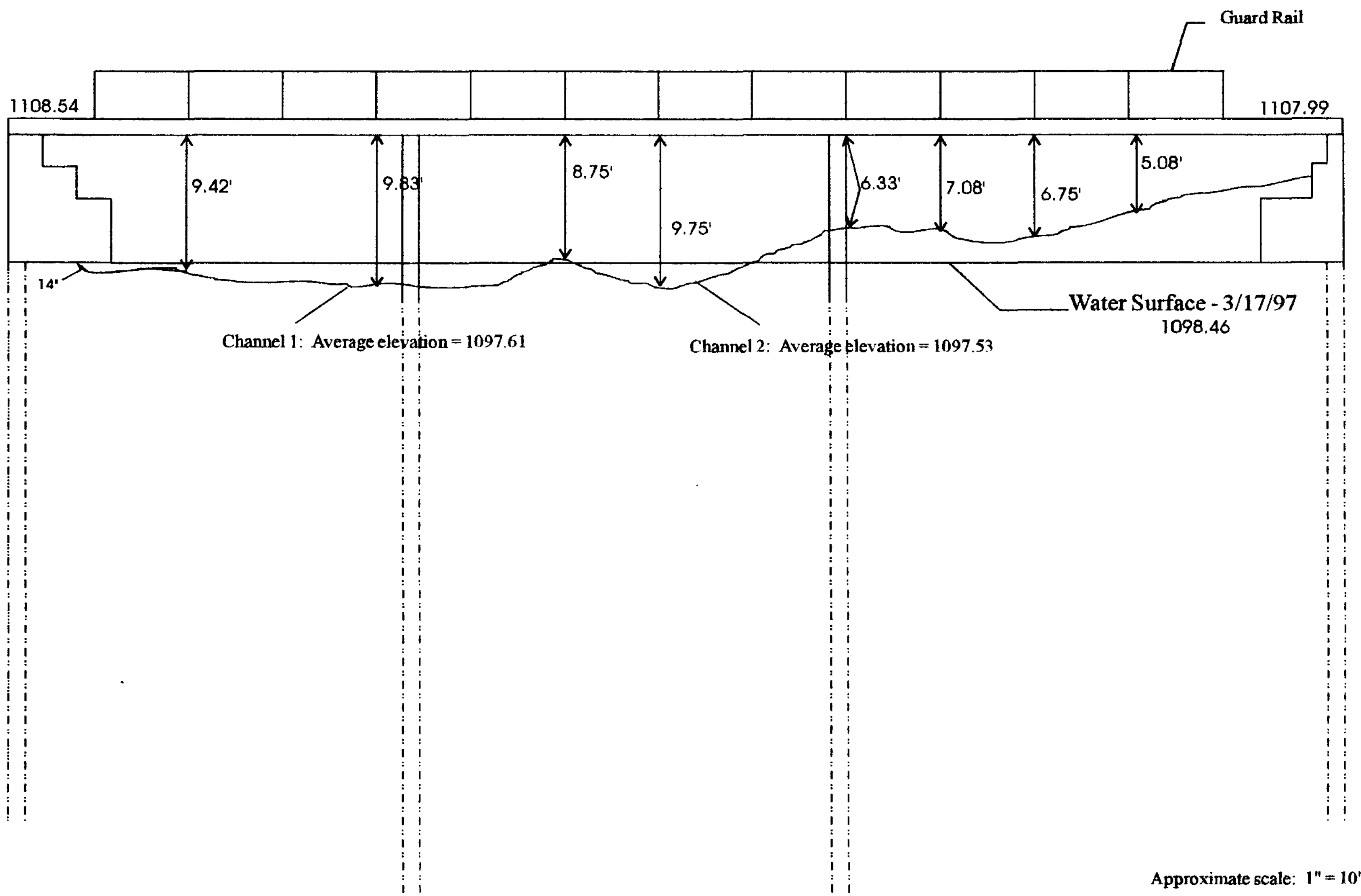


Figure 1.6 Channel Profile- Downstream face

Chapter II: SCOUR EVALUATION

2.1 Step 1. Determination of Analysis Variables

Analysis variables for the watershed characteristics and channel opening dimensions are required to calculate runoff and bridge hydraulics. Hydrologic variables include drainage area, longest distance of stream, average slope of watershed and runoff curve number. Hydraulic variables include bridge opening area, bridge wetted perimeter at various flow rates, slope of channel and predicted flow velocity under bridge.

In order to meet the specifications of the scour evaluation procedure, it is necessary to compute discharge magnitudes for a 100 and 500 year flood. These values were calculated using the SCS option in the HRQ computer program developed by Khan (1987). The channel's carrying capacity was then calculated for existing site conditions using the Manning equation.

2.1.1 The SCS Method - Background Data Acquisition

- Drainage area:

The soil conservation service method for predicting runoff for urban and small watersheds was chosen as the simplest approach for the study area . Data collection for this method included acquiring topographic maps, land use data and soil survey maps. USGS Topographic 7.5 minute quadrangle maps were acquired from the county engineers and the Ohio Department of Natural Resources. From these, the watershed was pieced together, overlapping the Hanover, Damascus and Salem quadrangles (Appendix C, Figure C-1). Site evaluation was required for some of the watershed delineation and used for sketching the boundaries on the topographic maps. The total drainage area was measured as 10,533 acres using a planimeter.

Table 2.1 Watershed Area Measurements

Trial	Area (acres)
1	10530
2	10539
3	10531
Average	10533

- Longest Distance:

The longest stretch of stream on the watershed is the main stem. The total length of the main stream was measured by placing a string on the map along the natural bends in the stream and measuring the length. This was measured to be 5.52 miles (29,145 ft). Length of the stream is needed to determine the time of concentration, which is the amount of time required by a drop of water to travel from the hydraulically farthest point in the watershed to the outlet of the stream. This is an important parameter required for predicting the peak flow rate during a precipitation event.

- Average Slope of Watershed:

The average slope of the watershed was computed using the Grid Method. A transparent 1 in x 1 in grid pattern was laid over the topographic map, and intersection points were marked. Horizontal and vertical intersection points with the 50 ft contour intervals were counted and slope was calculated using equations presented in Figure 2.1.

Figure 2.1 Calculation of Average Watershed Slope Using the Grid Method

$$S_h = \frac{n_h * h}{l} = \frac{87 * 50ft}{111,400ft} = .039\, ft/ft \qquad \text{(Equation 2.1)}$$

$$S_v = \frac{n_v * h}{l} = \frac{91 * 50ft}{101,000ft} = .046\, ft/ft \qquad \text{(Equation 2.2)}$$

$$S = (S_h + S_v)/2 = .043\, ft/ft$$

S_v = slope vertical
S_h = slope horizontal
n = total number of contour intersections by the horizontal and vertical grid lines
l = total length of grid line segments (horizontal and vertical) (ft)
h = contour interval (ft)
(Viessman, 1989)

- Curve Number:

The 7.5 minute quadrangle maps were also used to a certain extent for land use and cover calculations. The quadrangle maps provide rough locations and areas of wooded and residential areas. Areas for each land use from the topographic maps were evaluated in conjunction with site inspections. Curve number values are required with associated land uses to compute a composite curve number which in turn is computed using the HRQ computer program.

Table 2.2 Curve Number Inputs for Land Use

Land Use	Percentage of Watershed	Acreage	Condition	Curve Number *			
				A	B	C	D
Commercia l	2.2 %	230		89	92	94	95
Residential	9.9%	1042	1 acre ave.	51	68	79	84
Roads	2.4%	258	Paved, Open ditches	83	89	92	93
Woods	32.8%	3458	Good	25	55	70	77
Pasture	26.1%	2754	Fair	49	69	79	84
Agriculture	23.5%	2475	Straight Row, Good	67	78	85	89
Swamp	3.0%	316					

*(Veismann, 1989)
(Appendix B, Table B-1)

- Soil Types:

Soil surveys obtained from the Ohio Department of Natural Resources were used to estimate areas of various hydrologic soil types in for the watershed. The soil types located in the area were broken down by parcel and estimates were made on what percentage of each parcel contained each type. Each parcel has a 1 mi^2 area, allowing for the conversion of total acreage of each soil type to be made (Appendix B, Table B-2). The results of the analysis are shown in Table 2.3.

Table 2.3 Acreage of Various Soil Types in the Watershed

Soil type	Acreage
A	0
B	1756
C	8401
D	376
Total	10,533

(Appendix B, Table B-2)

2.1.2 The SCS Method - Computations

The HRQ computer program was used to compute the output of the model using the parameters calculated above. Peak flow rates for the watershed were computed using this program for 10, 25, 50 and 100 year floods. The HRQ computer program uses the SCS TR-55 graphical method (Viessman, 1989). This procedure estimates peak flow rates using a unit hydrograph computed for the watershed. USGS background data provided Ohio precipitation data for 24-hr floods for each of these precipitation events (Appendix C, Table C-1). The results of the HRQ modeling program provided peak flow rates for the precipitation data. Input parameters are as previously calculated, and represent the watershed's runoff capacity. The bridge hydraulics must be capable of handling the resultant flowrates presented in Table 2.4.

Table 2.4 Maximum Storm Events and Associated Runoff

Frequency	Inches of Precipitation over 24-hours for Mahoning County	Flowrate (cfs)
10 year	3.6	1793
25 year	4.1	2326
50 year	4.6	2893
100 year	4.8	3128
500 year	(1.7 * 100 year)	5318

(Appendix A, Table A-1)

2.1.3 Hydraulic Parameters

Hydraulic parameters for the bridge are required to compare with the bridge's carrying capacity for the predicted flood. This comparison dictates how the channel is modified by high flow rates The capacity of the bridge to carry flow is determined through the use of the Manning equation as shown in Figure 2.2.

Figure 2.2 The Manning Equation

$$V = \frac{Cm}{n}\left(\frac{A}{P}\right)^{2/3} S^{1/2}$$ *(Equation 2.3)*

V = Velocity (ft/s)
Cm = Chezy constant = 1.486
n = coefficient of friction
A = Area (ft^2)
P = Wetted perimeter (ft)
S = Slope of channel bed
(Veismann, 1989)

The Manning equation operates under the following assumptions:

- Open channel flow
- Depth does not vary in the channel under the bridge
- Constant velocity through the channel under the bridge

- <u>Wetted Perimeter</u>

The data accumulation step for use in the Manning equation requires the velocity of flow at various flow rates. It is necessary to calculate the depth of flow at these flowrates to provide the area and wetted perimeter variables. To accomplish this, on-site measurements of the channel openings were made using a surveyors tape. Both upstream and downstream channel profiles were measured and drawn (Figures 1.5, 1.6). The associated areas and wetted perimeter were calculated at one foot elevation increments using these measurements (Appendix C, Table C-2). From these values, velocity was calculated (Table 2.5).

- <u>Slope of Channel Bed</u>

Slope required for The Manning equation is different from that previously found for the entire watershed. The Manning equation slope refers to that of the channel bed under the structure. Slope measurements were taken from the quadrangle map, measuring the total distance between the contour lines immediately upstream and downstream of the bridge. This amounted to a drop of 30 ft. over a distance of 7800 ft. giving a slope of .00385, or .385% (Appendix C, Figure C-3).

- <u>Velocity</u>

The channel under the Goshen Road crossing varies slightly through the cross section of the bridge, but is assumed to be constant. For the purpose of the scour evaluation, the maximum velocity is used, and therefore, the downstream profile, which has the smallest cross-sectional area, was used in the Manning equation. The Manning equation n was assumed to be .035 for a natural channel material. Velocity calculations through the channel were then computed for each rainfall event category and are summarized in Table 2.5. The velocity results were then converted to flowrates using equation 2.4.

Table 2.5 Velocity and Flowrates Through Channel at Elevation Increments of 1 ft.

Flow Elevation (ft)	Velocity (ft/s)	Flowrate (cfs)
1098	1.78	37
1099	3.26	203
1100	5.06	654
1101	4.68	838
1102	5.68	1384
1103	6.25	1934
1104	6.78	2594
1105	7.50	3407
1106	8.16	4313
1107	8.86	5335

$Q = V*A$ *(Equation 2.4)*

Q = flowrate (cfs)

V = velocity (ft/s)

A = area (ft^2)

(Appendix B, Table B-3)

These velocity values were confirmed using a crude velocity measurement with a ping pong ball and a stopwatch. The procedure consisted of measuring a distance under the bridge and timing the travel of the ping pong ball over that distance. This method provides reliable physical evidence in absence of modern flow gauging equipment. The elevation of the water was measured to provide the flow rate calculation. The results of the experiment are as follows:

Flow elevation = 1098

Length of stream over which velocity was measured = 56.5 ft

Table 2.6 Ping Pong Ball Time Trials

Trial	Result (seconds)	Velocity (ft/s)
1	33.06	1.71
2	31.05	1.82
3	32.67	1.73
4	36.07	1.57
5	31.46	1.80
Average	32.86	1.72

- Other Parameters

Additional background data collection is required for scour analysis computations. This primarily consists of boring logs to provide subsurface structure bearing strength and bed material size distribution for predicting sediment transport. Subsurface information and soil types in the stream bed were obtained from the Mahoning County Soil Survey maps

(USDA, 1971) (Appendix C, Figure C-2). The soil survey maps provide good information about the grain size distribution with depth and depth to bedrock. These values for the soil type in the immediate area of the structure were assumed to be correct for the purpose of this analysis.

Soil types upstream of Goshen Road are consistent for approximately 1/8 mile. This is the location where the three branches of the creek, referred to as West, Middle and East, merge (Figure 1.3). The West and Middle branches consist of Wayland Silt Loam soil while the East consists of Lobdell Loam. The Lobdell Loam soil type makes up the entire area downstream of the confluence to one mile past the overpass. Floodplain soils upstream of the overpass consist of Orrville Silt Loam and Wayland Silt Loam soils, while floodplains immediately downstream consist of Lobdell and Wayland soils; the soil type directly influencing stream characteristics in the vicinity of the overpass is Lobdell Loam. (USDA, 1971). Typical engineering characteristics of Lobdell Loam are described in Figure 2.3.

Figure 2.3 Engineering Characteristics of Lodbell Loam

- Capability Unit Iiw-4: Well-drained to moderately well-drained
- Seasonal flood plain
- Depth to:
  - Seasonal high water table = 2-3 ft
  - Bedrock = < 6 ft
- Surface Profile:
  - 0-3 in Dark greyish-brown silt loam
  - 3-15 in Brown silt loam
  - 15-42 in Layered brown and dark yellowish-brown loam and sandy loam
- Percentage Passing Sieve:

	No. 4	No. 10	No. 200
Silt loam	98	95	72
Loam to Sandy loam	98	95	55

- AASHO: A-6 to A-4
- Permeability = 0.2 - 0.63 in./hr.
- Available Moisture Capacity

Silt loam	0.19 - 0.23
Loam to Sandy loam	0.12 - 0.17

- pH

Silt loam	5.1 - 6.0
Loam to Sandy loam	5.6 - 6.5

(USDA, 1971)

D_{50} = Grain size of 50% of grains
Assume: Approximately 50% of grains pass No. 200 Sieve
D_{50} = .08 mm (Tuma, 1973)

A summary of each of the analysis variables is presented in Table 2.7.

Table 2.7 Summary of Analysis Variables

Watershed Characteristics										
	Slope			4.2544	%					
	Length of Stream			29166	ft					
	Area			10,533	acres					
	Hydrolic Soil Type									
		B		1756	acres					
		C		8401	acres					
		D		376	acres					
Soil Characteristics										
	Grain Size									
	> .25 in diameter			2%						
	.005 in -.10 in diameter			32%						
	< .005 in diameter			66%						
							C	**N**		
Landuse Acreage			(%)			A	B	C	D	
	Commercial		2.18	230	acres	89	92	94	95	
	Residential		9.89	1042	acres	51	68	79	84	
	Roads		2.45	258	acres	83	89	92	93	
	Woods		32.83	3458	acres	25	55	70	77	
	Pasture		26.15	2754	acres	49	69	79	84	
	Agriculture		23.50	2475	acres	67	78	85	89	
	Hydrology		3.00	316	acres					74.18
			100	10533						
Hydraulic Variables										
	Peak Discharge (24 hr.)									
		10	1793	cfs						
		25	2326	cfs						
		50	2893	cfs						
		100	3128	cfs						
		500	5318	cfs						
	Current Area		602	ft^2						
	Channel Slope		0.00385							

2.2 Step 2. Analysis of Long Term Bed Elevation Change

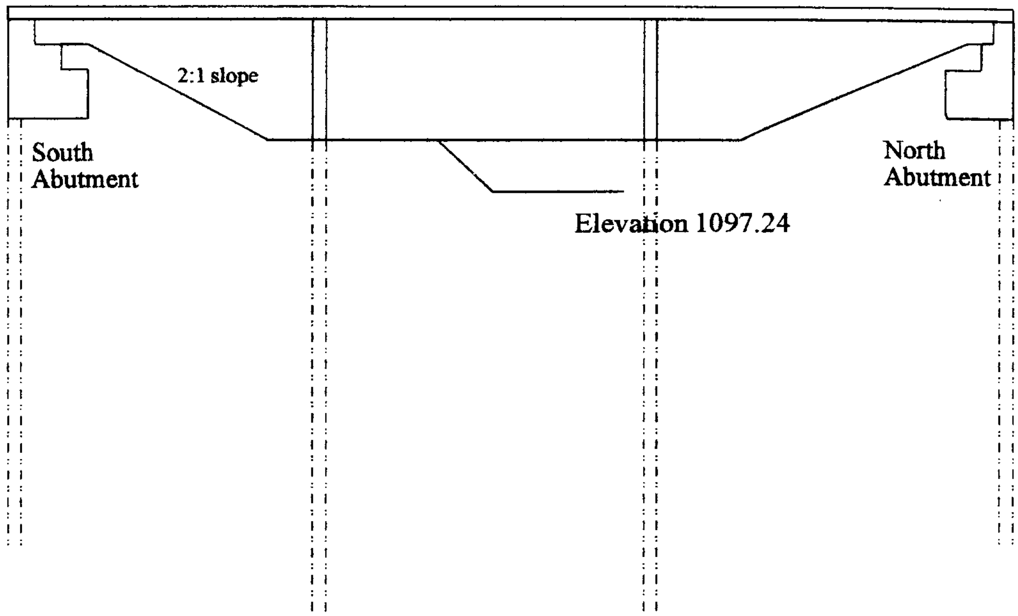
Aggradation and long term stream elevation changes are due to natural or man induced causes within the reach of the river. Aggradation is the deposition of eroded material from other sections of the river. The qualitative evaluation of long term bed elevation change can be made using a variety of techniques. Where significant aggradation or degradation is likely, 100 year estimates of elevation change can be made using any of the following:

- Corps of Engineers HEC 6
- Straight line extrapolation of present trends
- Engineering judgment
- Worse case scenarios

(FHWA, 1988)

In this project, long term stream bed elevation was found using extrapolation of present trends and engineering judgment. In the original plans, drafted in 1957, the channel elevation was graded to 1097.24 at the downstream section of the crossing. Current site measurements at this point found the average stream channel elevation to be 1097.57 (Figure 2.4). This shows a trend in of aggradation of approximately .00825 ft. per year, or 4.0 in. over 40 years. This rate of aggradation is not considered significant to warrant aggradation countermeasures. Since degradation is not a problem, the change in elevation will not affect the scour analysis. These results are based, however, on a mass balance approach, and don't account for sedimentation and erosion occurring at an equal rate in opposite sections of the channel. The section between channel 2 and the north abutment is experiencing sedimentation, and the channel has shifted away from this deposit. This process is the cause of the current scour occurring under the south abutment.

Original Channel Elevation



Current Channel Elevation - Downstream Profile

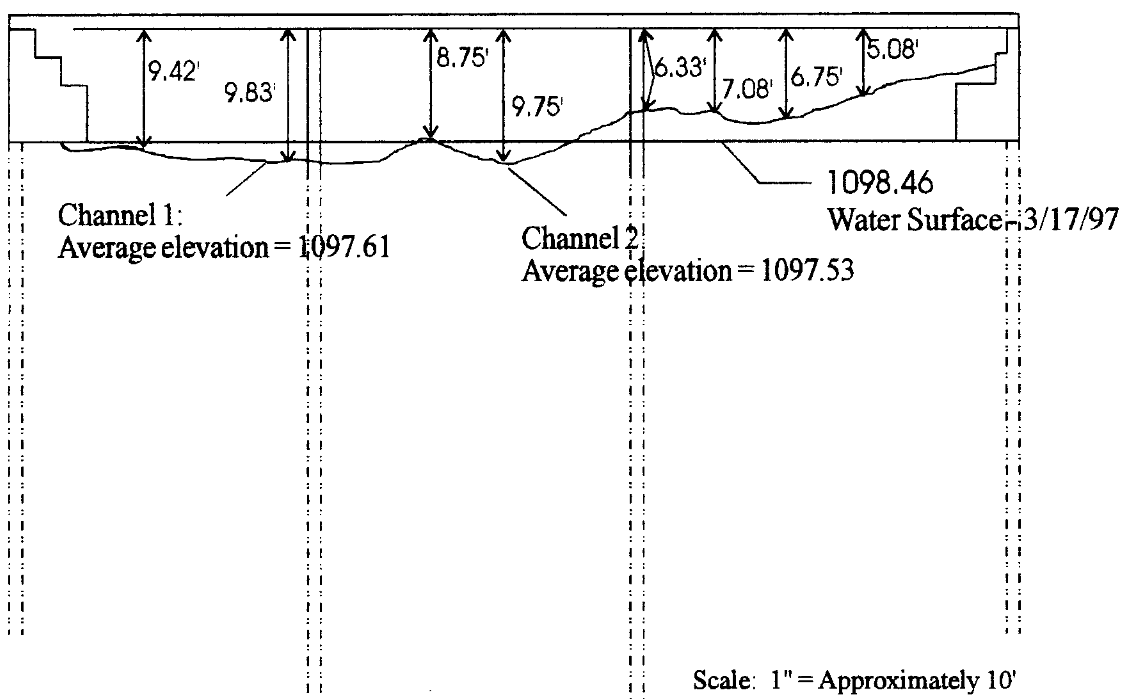


Figure 2.4 Analysis of Long-term Bed Elevation

2.3 Step 3. Scour Analysis Method Selection

The selection of the method of analysis of contraction and local scour is based on the inter-dependence of the contraction and local scour components. Either of two methods can be used depending on the situation. If contraction scour is occurring, local scour conditions are affected as a result in the changes in the hydraulic variables. In this situation, method one is used as follows:

- The natural channel's hydraulics for fixed bed condition based on existing site conditions are estimated.
- The expected profile and plan form changes are estimated.
- The natural channel hydraulics are adjusted based on the expected profile and plan form changes.
- A trial bridge opening is selected and the bridge hydraulics are computed.
- Contraction scour is estimated.
- The natural channel's geometry is iterated to reflect contraction scour and the channel hydraulics are revised until there is no significant change.
- Local scour is calculated using the revised hydraulics.
- The local scour depth extends below the predicted contraction scour depths.

(FHWA, 1988)

Method one uses an iteration process to arrive at acceptable estimates. However, from preliminary data gathering, it is apparent that if revised hydraulics are considered for local scour, the effects on the structure will not differ significantly. This is based on the lack of evidence of problematic local scour and the deep design of the pilings.

The alternative approach, method two, was selected for this analysis, and will provide adequate results since contraction scour is the primary area of concern. The approach is as follows:

- The natural channel's hydraulics for fixed bed condition based on existing site conditions is estimated.
- The expected profile and plan form changes is assessed.
- The fixed bed hydraulics are adjusted to reflect any expected long term profile or plan form changes.
- Contraction scour is estimated using an empirical contraction formula and the adjusted fixed bed hydraulics.
- Local scour is estimated using the adjusted fixed bed channel and bridge hydraulics.
- Add local and contraction scour to obtain the total scour.

(FHWA, 1988)

2.4 Step 4. Calculation of Contraction Scour

Contraction scour, or general scour, is the erosion of material from the bed and banks across all or most of the width of a channel. This can be caused by any of the following:

- the contraction of flow from natural or manmade obstructions
- change in downstream control of the water surface elevation
- location of bridge relative to a river bend

(FHWA, 1988)

For the Goshen Road crossing, contraction scour during floods is due primarily to the contraction of flow as the water drains from the flood plain through the channel. The water surface elevation was assumed to parallel the stream bed elevation change. This elevation was taken from the USGS quadrangle map, and decreases at an average slope of .035 ft/ ft The closest bend in the river is located approximately 250 ft. upstream from the crossing or about 5.7*w* (width of channel), thereby having no effect on the immediate river channel (FHWA, 1988).

The value of contraction scour is calculated accordingly for any of the following four cases:

> Case 1: Overbank flow on a flood plain being forced back to the main channel by the bridge approaches.
>
> Case 2: The normal river channel width becoming narrower either because of the bridge itself or the bridge being on a narrower stretch of river.
>
> Case 3: A relief bridge in the overbank area with little or no bed material transport in the overbank area.
>
> Case 4: A relief bridge over a secondary stream in the overbank area.

The Goshen Road bridge falls into case 1, that is, overbank flow at the approach of the bridge being forced through the reach of the channel with no overbank flow. This situation requires use of the methodology outlined in Figure 2.5.

Figure 2.5 Contraction Scour Computation Methodology - Laursen's Equation

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7}\left(\frac{W_1}{W_2}\right)^{K_1}\left(\frac{n_2}{n_1}\right)^{K_2}$$ *(equation 2.5)*

$y_s = y_2 - y_1$ = Average scour depth (ft)

y_1 = average depth of scour in the main channel (ft)
y_2 = average depth of scour in the contracted section (ft)
W_1 = width of the main channel (ft)
W_2 = width of the contracted section (ft)
Q_t = flow in the contracted section (cfs)
Q_c = flow in the main channel (cfs)
n_2 = Manning n for contracted section
n_1 = Manning n for main section
K_1 = Empirical constant
K_2 = Empirical constant
e = transport factor from the following:

Table 2.8 Estimation of Empirical Values for Laursen's Equation

V_{*c}/W	e	K_1	K_2	Mode of Bed Material Transport
<0.50	0.25	0.59	0.066	mostly contact bed material discharge
1.0	1.0	0.64	0.21	some suspended bed material discharged
>2.0	2.25	0.69	0.37	mostly suspended bed material discharged

$V_{*c} = (gy_1S_1)^{0.5}$, shear velocity
w = fall velocity of D_{50} bed material
g = gravity constant, 32.2 ft/s
S_1 = slope, energy grade line main channel

$$K_1 = \frac{6\,(2+e)}{7\,(3+e)}$$ *(equation 2.6)*

$$K_2 = \frac{6e}{7\,(3+e)}$$ *(equation 2.7)*

Notes:

1. The Manning n ratio can be significant for a condition of dune bed in the main channel and a corresponding plain bed, washed out dunes or anti-dunes in the contracted channel.

2. The average width of the bridge opening (W_2) is normally taken as the top width with the width of the piers subtracted.

3. Laursen's equation for a long contraction will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of the contraction or of the contraction is the result of the bridge abutments and piers.
(Laursen, 1960)

2.4.1 Calculation of Variables for Contraction Scour:

The calculations for Laursen's equation were performed using the bridge scour computer program, hy93, provided with the interim procedures from the FHWA (FHWA, 1988). All of the preceding equations are contained in the computer program.

Since the variables required for the program change with flowrate, it was necessary to determine the flow elevation corresponding to the desired flowrate. The SCS TR-55 method provided flowrates of 3128 cfs and 5318 cfs for 100 and 500 year floods, respectively. These flowrates were applied to the bridge areas to predict the water level during these events. This was done by interpolating elevations from SCS modeling flowrates and bridge hydraulic elevations as follows:

$$\frac{x - 1104}{1104 - 1103} = \frac{3128 - 2594}{3407 - 2594}$$

$$x = 1104.66$$

This elevation was checked using the Manning equation which provided a flowrate of 3128 cfs at elevation 1104.66 ft. Corresponding flow elevations are provided in Table 2.9.

Table 2.9 Surface Water Elevation Through the Channel at Different Flowrates

Storm Event (yr)	Flowrate (cfs)	Flow Elevation (ft)
100	3128	1104.66
500	5318	1107.00*

**Channel running full at 500 year flood.

These elevations were used to make field measurements for input variables to the hy93 computer program. Variables were entered into the program as described in Table 2.10.

Table 2.10 hy93 Computer Program Input Variables for Contraction Scour

Trial	Case	Approach Depth (ft)	Approach Width (ft)	Constricted Width (ft)	Contracted Flow (cfs)	Approach Flow (cfs)	V-ratio
100 yr.	I	6.51	355.5	73	3128	3128	39.05
500 yr.	I	9.46	613	76	5318	5318	43.32

The approach variables were measured at the nearest upstream point where the bridge does not affect the flow. The approach width and velocity parameters required measurements of the upstream flood plain that would be encompassed during each event. These areas were estimated using a surveyor's tape and a field walkover. Constricted parameters were calculated using the previously shown hydraulic analysis. The Manning n was assumed to be .035, and the Vratio was calculated as shown in Figure 2.6:

Figure 2.6 Vratio

$$\text{Vratio} = \frac{Shear\,V}{Fall\,V} = \frac{\left(gyS_1\right)^{.5}}{.023*} \qquad \textit{(Equation 2.8)}$$

* Fall Velocity = .007 m/s = .023 ft/s (From Figure 2.7)
g = acceleration due to gravity = 32.2 ft/s^2
y_1 = average depth of main channel
S_1 = slope of energy grade line
(Appendix, pg. 86)
(FHWA, 1988)

Results:

100 yr. contraction scour = -1.0 ft
500 yr. contraction scour = 2.0 ft

These results indicate that the contraction of the flow from the floodplain to the channel has a moderate effect on the depth of scour. In the event that the channel were subjected to a 100 year flood, no scouring effects due to contraction would be noticed.

In fact, the results indicate that sedimentation would occur, on average, across the channel. Due to the streams dune and antidune configuration, uniform sedimentation is unlikely as sediment will be "relocated" by the creation of scour holes and deposition of small grain dunes in the stream bed. A 500 year event would cause approximately 2 ft. of the streambed to be scoured.

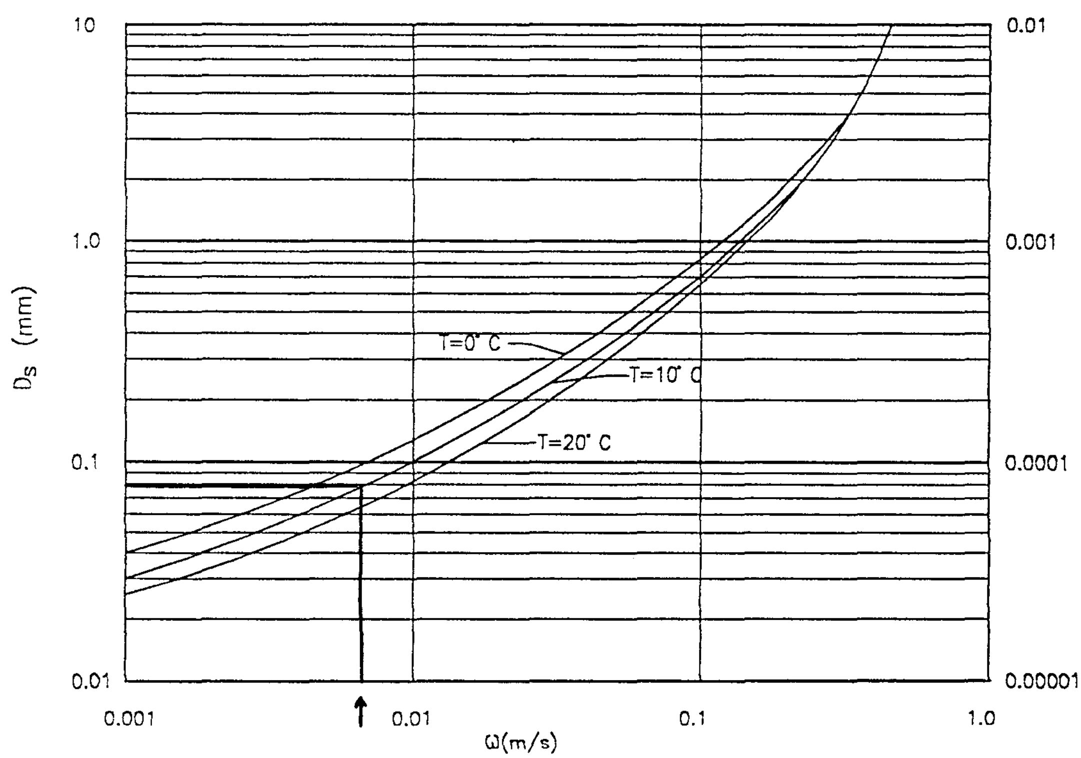


Figure 2.7 Fall Velocity of sand–sized particles. (Reproduced from FHWA, 1988)

2.5 Step 5. Calculation of Local Scour at Abutments

Local scour is erosion which is restricted to a minor part of the width of a channel and is caused by the acceleration of flow and the development of vortex systems induced by obstructions to the flow (FHWA, 1988). The two types of local scour are due to clear-water scour and live-bed scour. Clear-water scour is characterized by no movement of the bed material upstream of the obstruction, but the acceleration of the vortices causes the material around the base to move. Live-bed scour is dominant when bed material upstream is also moving. Both the north and south abutments are affected by clear water scour only, since no bed material is moving upstream. This is due to the heavy vegetative growth on the floodplain upstream from the bridge.

The equations used in the computer program are dependent on the following site condition scenarios:

Case 1: Abutments project into channel, no overbank flow upstream
Case 2: Abutments project into channel, overbank flow upstream
Case 3: Abutments set back from the channel more than 2.75* average scour depth
Case 4: Relief bridge
Case 5: Abutment set at edge of channel, overbank flow upstream
Case 6: Abutment length to flow depth ratio > 25
Case 7: Abutment set at an angle to the flow

Both of the abutments fall into case 5: Set at the edge of the channel, with overbank flow upstream (Figure 1.4).

2.5.1 Calculation of South Abutment Local Scour:

Equation 2.9 is used for calculating clear water scour (Figure 2.8).

Figure 2.8 Local Scour at Abutment Computations

$$\frac{Q_0}{q_{mc} y_0} = 2.75\left(\frac{y_s}{y_0}\right)\left[\left(\frac{y_s}{4.1 y_0}+1\right)^{7/6}-1\right] \qquad \textit{(equation 2.9)}$$

q_{mc} = the unit discharge in the main channel, Q/W (ft^2/s)
Q = discharge in main channel (cfs)
W = width of the main channel (ft)
Q_0 = overbank flow discharge (cfs)
y_0 = overbank flow depth (ft)
$y_s = y_2 - y_1$ = Average scour depth (ft)
(FHWA, 1988)

The skew angle of the river was measured using current site conditions for the angle of the flow to the placement of the abutments. For this abutment, the skew angle measures 15 degrees north. The approach depth is the depth of water in the main channel found using the bridge hydraulics for elevation of flow at associated flowrates. The following assumption is made to calculate the approach flow and the contracted flowrates:

$$Q_{approach} = Q_{contracted}$$

Computation input and outputs were determined as shown in Table 2.11.

Table 2.11 Input and Output Variables for Live Bed Local Scour for the South Abutment

Trial (yr)	Skew angle	Main Channel Flow at Approach (cfs)	Overbank Flow at Approach (cfs)	Overbank Depth at Approach (ft)	Main channel Depth (ft)	Main channel Width (ft)	Scour (ft)
100	15	3128	3128	3.51	3.0	73	5
500	15	5318	5318	6.46	3.0	76	8

2.5.2 Calculation of North Abutment Local Scour:

The north abutment has accumulated a significant amount of sediment and is subject to scour conditions during major flood events only. During low flow periods, the north abutment is protected from erosion by the accumulation of sediment. The north abutment is set at the edge of the channel and is subject to the same scour producing hydraulic conditions as the south abutment. The only parameter that differs for the analysis is the skew angle of the abutment to the channel. Computation input and output were determined as shown in Table 2.12.

Table 2.12 Input and Output Variables for Local Scour for the North Abutment

Trial (yr)	Skew angle	Main Channel Flow at Approach (cfs)	Overbank Flow at Approach (cfs)	Overbank Depth at Approach (ft)	Main channel Depth (ft)	Main channel Width (ft)	Scour (ft)
100	20	3128	3128	3.51	3.0	73	7
500	20	5318	5318	6.46	3.0	76	10

Results for local abutment scour:

South:
100 yr local abutment scour = 5 ft
500 yr local abutment scour = 8 ft

North:
100 yr local abutment scour = 7 ft
500 yr local abutment scour = 10 ft

Abutment scour equations frequently overestimate actual depths, and are often omitted from the investigation, basing conclusions primarily on contraction with pier scour (Barrett, 1997). This is not to suggest that abutment scour is not a factor in stability, rather that the results often overestimate a problem that can be avoided through the use of riprap. Due to the presence of abutment scour and the lack of riprap used on the Goshen Road abutments, the results obtained from the hy93 computer program are assumed to be correct for the purposes of this analysis.

2.6 Step 6. Calculation of Local Scour at Piers

Local scour at piers is a function of bed material size, flow characteristics, fluid properties and the geometry of the pier. There are many equations available for this calculation, based primarily on laboratory tests. The interim procedure manual recommends the use of the Colorado State University equation shown in Figure 2.9.

Figure 2.9 CSU Equation for Pier Scour

$$\frac{y_s}{y_1} = 2.0K_1K_2\left(\frac{a}{y_1}\right)^{0.65} Fr_1^{\,0.43} \qquad \textit{(equation 2.10)}$$

$$Fr_1 = \textit{Froude number} = \frac{V_1}{(gy_1)^{0.5}} \qquad \textit{(equation 2.11)}$$

y_s = scour depth (ft)
y_1 = flow depth just upstream of the pier (ft)
K_1 = correction for pier shape = 1.1 for square nose
K_2 = correction for angle of attack of flow = 1.0 for 1 ft. x 1 ft. pier.
a = pier width (ft)
(Richardson et al, 1987)

Input parameters for hy93 established by previous analysis and on-site measurements are summarized in Table 2.13.

Table 2.13 Input into hy93 for Pier Scour and Scour Predictions:

		100 year			500 year		
Pier #	**Angle of flow**	**Velocity at approach (ft/s)**	**Flow Depth (ft)**	**Scour (ft)**	**Velocity at approach (ft/s)**	**Depth (ft)**	**Scour (ft)**
1	10	5.74	6.51	4	2.70	9.46	3
2	10	7.27	6.51	4	8.84	9.46	5
3	5	7.27	6.51	4	8.84	9.46	4
4	10	7.27	6.51	4	8.84	9.46	4
5	5	7.27	6.51	4	8.84	9.46	4
6	20	5.74	6.51	5	2.70	9.46	4
7	20	7.27	6.51	5	8.84	9.46	6
8	15	7.27	6.51	5	8.84	9.46	5
9	10	7.27	6.51	4	8.84	9.46	5
10	5	7.27	6.51	4	8.84	9.46	4

All of the piers measure approximately 1 ft. X 1 ft. and have square shapes. Since the parameters vary for each of the piers, calculations were made for 10 piers. For example, pier number 1 was assumed to have an approach velocity equal to that of the approach flow, while pier number 2 is influenced by the velocity of flow in the contracted section. These approach piers had lower predicted scour values than would be expected due to the calculation at lower velocities.

It is noted in the methodology that the CSU equation gives results with varying accuracy depending on the stream configuration. For example, for a dune bed configuration the equation predicts equilibrium and maximum scour depths that are overestimated by 30%. Plane bed configuration or antidunes are predicted to maximum scour. An antidune configuration is characterized by depressions throughout the stream bed. Site observation dictates that the Little Beaver Creek is characterized by dune bed configuration from the presence of various sedimentary areas.

The results for the corresponding approach piers #1 and #5 may be underestimated since debris accumulation is not accounted for in the equation. The impact of an angle of flow larger than 15 degrees was obvious in these results, increasing scour depths by 1 to 2 feet. Despite the previously mentioned discrepancies in the validation of the pier scour results, engineering judgment indicates that the computer output is correct, from the presence of small scour holes around the piers and the lack of better data.

2.7 Step 7. Plotting the Total Scour Depths

The results of the hy93 scour analysis are summarized in Table 2.14 (Appendix A, Table A-2).

Table 2.14 Results of Scour Analysis

Scour Type	100 year Scour Depth from Datum	500 year Scour Depth from Datum
Contraction	-1 ft	2 ft
South Abutment (Clear water)	5 ft	8 ft
North Abutment (Clear water)	7 ft	10 ft
Pier	4 ft	4 ft

These values are plotted on the bridge profile for both 100 and 500 year flood events and are shown in Figures 2.10 - 2.13. Both profiles have the same shape as scour occurs around the same features.

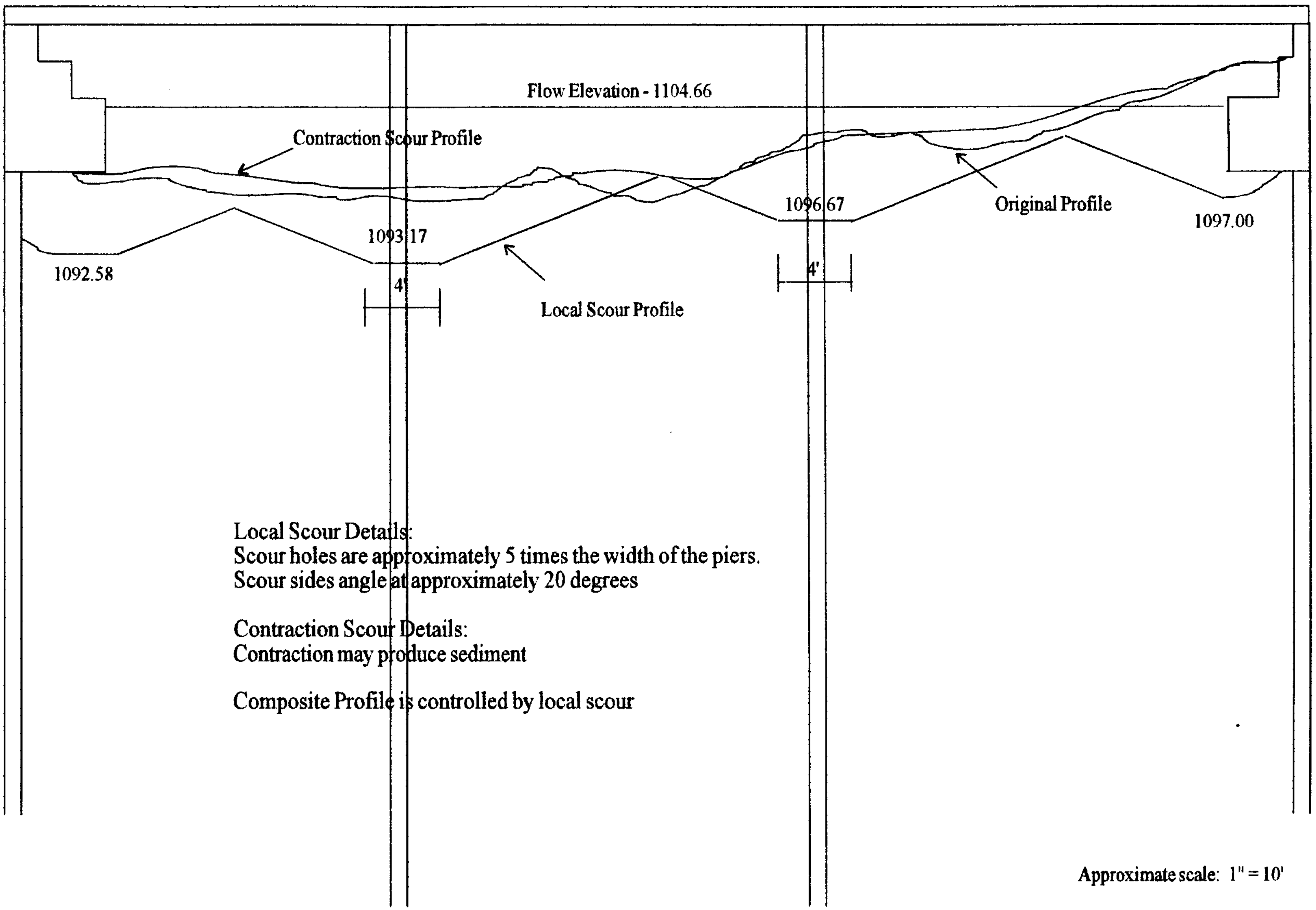


Figure 2.10 Channel Profile - 100 Year Scour Details

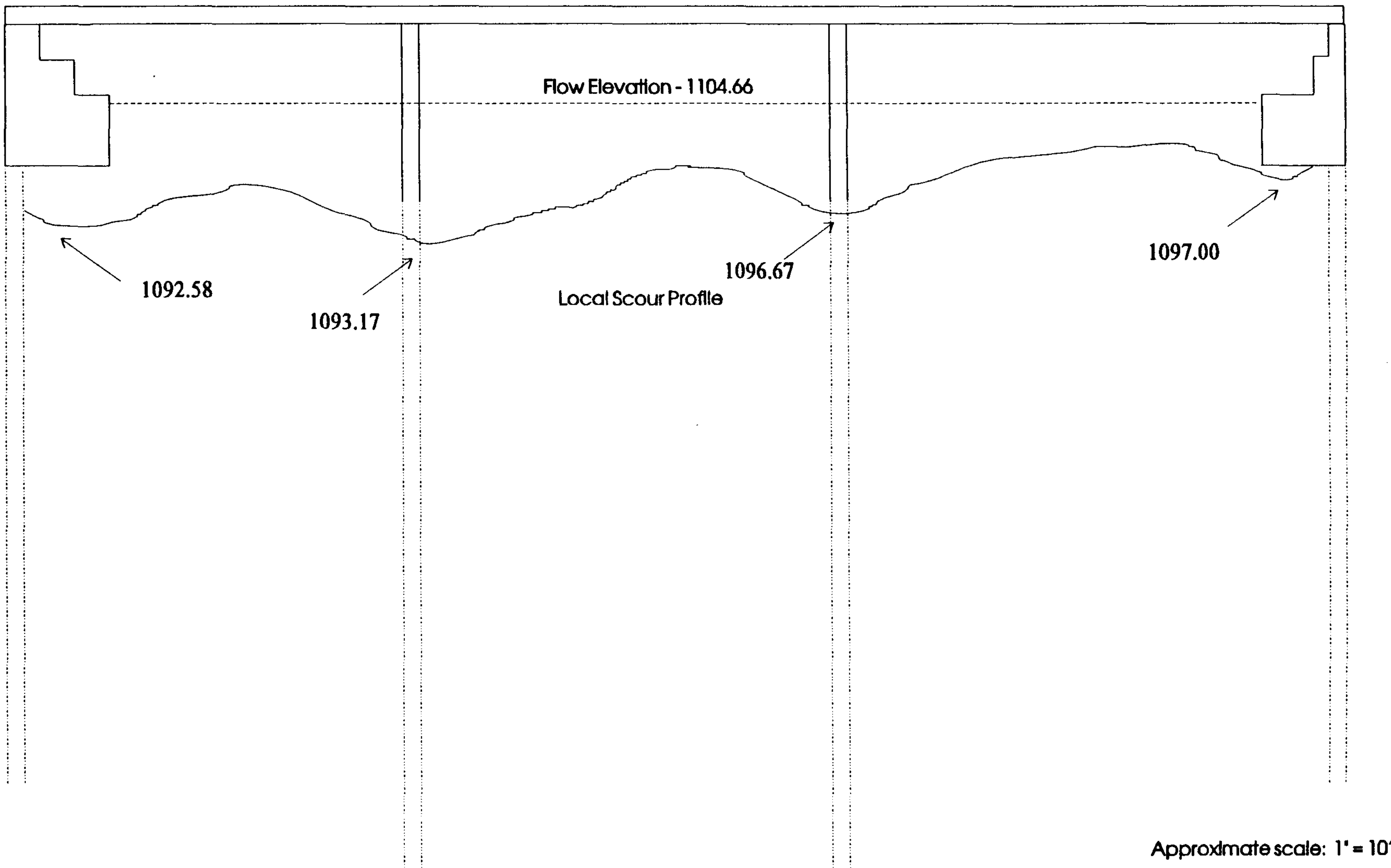


Figure 2.11 Channel Profile - 100 Year Scour Details (Composite)

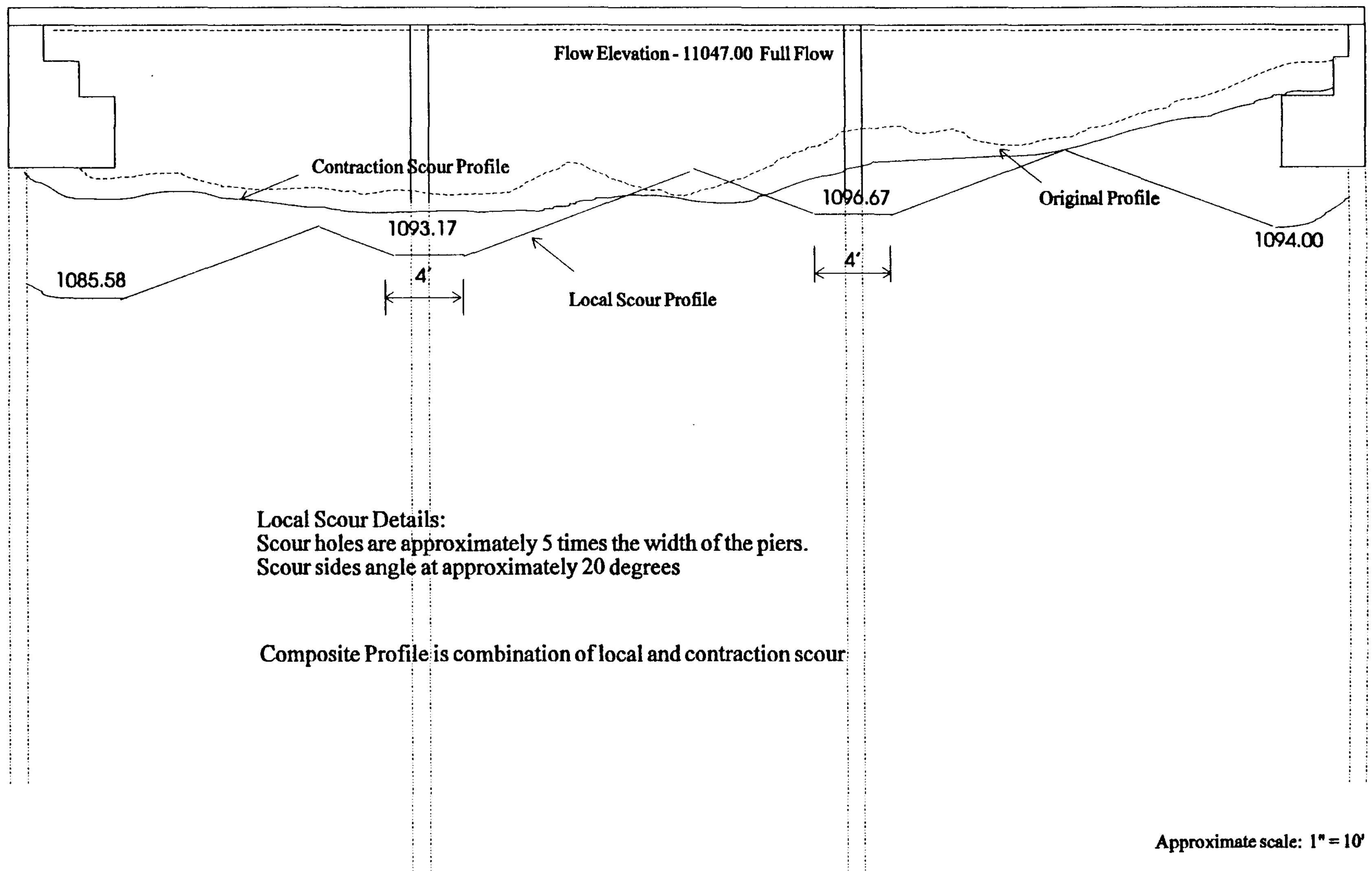


Figure 2.12 Channel Profile - 500 Year Scour Details

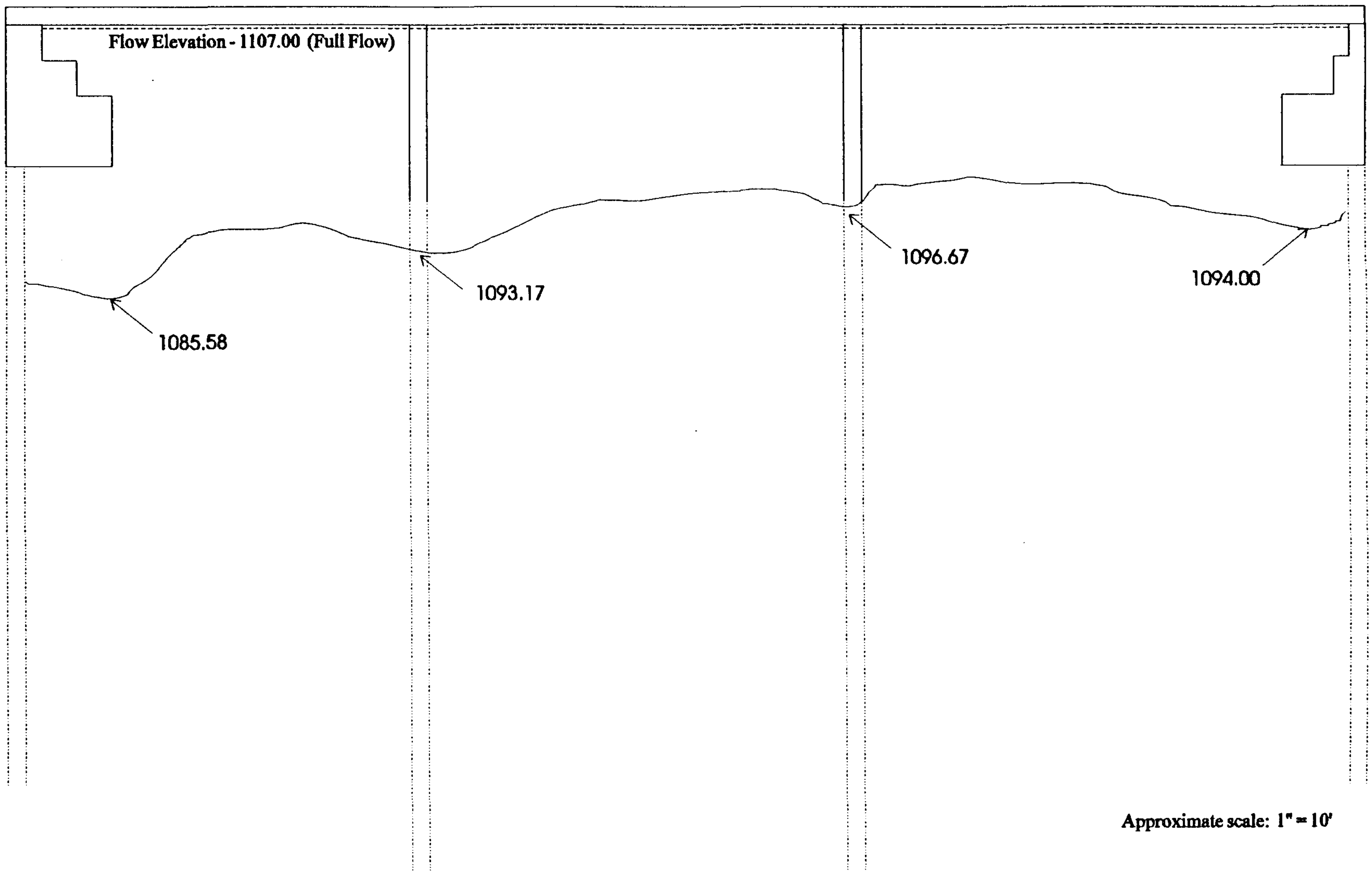


Figure 2.13 Channel Profile - 500 Year Flood Scour (Composite)

Chapter III: BRIDGE EVALUATION

3.1 Discussion

The bridge design is evaluated based on the results of the analysis. Considerations include waterway area, placement of piers and abutments in relation to each other, need for relief bridges, alignment of bridge abutments, location of crossing, competency of hydraulic study and overlapping of scour holes.

The area of the channel waterway is adequate for carrying flows greater than a 100 year flood. This event is considered highly unlikely (FHWA 1988) and the channel is capable of carrying this flow provided that the structure is not undermined.

The piers and abutments are adequately spaced to allow debris flows to pass through. Although debris accumulation is evident on the mid-channel piers, this is not a result of the placement of the structures. The use of relief bridges for this site is a possible consideration, and could be adopted to reduce the threat of failure during major floods. However, the installation of relief structures is not required because the current channel can adequately handle the hydraulics. Because of this, the costly option of relief bridges is not recommended for the Goshen Road crossing.

Abutment alignment was properly designed for the original channel geometry, but has failed due to the migration of the main channel. The creek approaches the south abutment at an of approximately 15 degrees and erosion is occurring under the abutment during normal flow conditions.

Placement of the bridge in the valley is acceptable due to the upstream and downstream location of bends in the river. The crossing is placed adequately downstream from the confluence of the three branches and upstream from a 90 degree change in flow direction. However, the bridges location in the vicinity of the tributary and storm outlet suggests that either the bridge or the tributaries should be relocated. The dominant site characteristics of the upstream confluence of the three main branches and the downstream river bend however, outweigh these factors for bridge relocation.

The hydraulic study was validated using a variety of techniques and confirmed the channel's adequate carrying capacity. The channel's ability to carry a 100 year flood at 70% capacity exhibits adequate design. The Manning equation provides accurate values for flow rates.

Local scour holes do not currently overlap. The widest scour hole is upstream and perpendicular to the front mid-channel pier. This is a result of the debris accumulation in this area, which develops vortices around the structure. The local scour holes from the piers and abutments will overlap during both 100 and 500 year floods. This is most noticeable for the 100 year flood scenario, in which contraction scour is not a problem and local scour dominates the predicted channel profile. For the 500 year event, the overlapping of scour holes does not significantly deepen the channel beyond contraction scour depths. In this scenario, the combination of local and contraction scour dictate the shape and depth of the composite channel profile.

The main erosion problem in the channel is under the south abutment. This erosion was caused by a combination of the two main site characteristics: the location of a small tributary immediately upstream from the north abutment, and an eight inch stormwater outfall located immediately downstream from the south abutment. The upstream tributary drains a small residential area and the adjacent crop field. This tributary carries heavy loads of sediment from the field and an additional amount from the residential area above. When the flow enters Beaver Creek, it quickly loses velocity and deposits the sediment at the front of the north abutment. Eventually this sediment has accumulated enough to shift the channel from the original grading to run flush against the abutment.

This problem is compounded by the accumulation of debris on the first pier which splits the flow into two distinct channels under the bridge (Figure 1.6). The channel to the north has some effect on eroding the sediment, but does not sufficiently scour the deposits to correct the flow of the channel. Under extreme conditions, this sediment will be subject to clear water scour and will be quickly eroded. This is due to the uncompacted nature of this material which is not supported by vegetation or large rocks. Because of this, the north abutment is most susceptible to local scour and should be monitored.

The stormwater outflow is located too close to the rear of the south abutment and has no sill to control erosion. Also, it is located at an approximate elevation of 1105, allowing the discharge to drop and cause backflow turbulence against the stream bank. The resultant erosion has encroached behind the abutment, leaving a gap between the abutment and the backfill.

The structure is adequately designed to withstand the hydraulic forces of Beaver Creek. However, the location of the discharges greatly affect the stability of the south abutment. In the absence of riprap to provide abutment protection, the original design is not resistant to scouring. This has led to an undermined abutment rating as scour critical, and requires the use of countermeasures.

3.2 Stability Analysis

The region and soil type in which Beaver Creek is located typically is supported by a bedrock layer at a depth of approximately 6 ft. However, the use of deep pilings in this design indicate that the Beaver Creek location either is devoid of this layer, or has a thin, penetrable layer of bedrock. The pilings are resting on bedrock at 40 and 42 ft, respectively (Department of Commerce, 1958). As a result, the channel bed lacks the typical present scour resistant layer. This has a significant effect on the stability of the structure, which is dependant on the scour resistance of the foundation material under the south abutment. It was noted during site visits that the material was similar to that of the stream bed, and is susceptible to fluid transport.

A stability analysis was performed for the abutments in the event that one is completely undermined. A simple approach was taken to estimate the ability of the piers to support the weight of the structure. Figures 3.1 and 3.2 describe the components of this analysis.

Figure 3.1 Abutment Stability Analysis

Vertical Forces

Abutment:

1. Location of Centroid
2. Weight of Abutment:
   - Cubic yards of concrete poured was taken from design plans (Mahoning County Engineer)
   - Weight of concrete was assumed to be equal to 2.5 times the weight of water. (Army Corp. of Engineers)
   - 190,382 lbs / 7 pilings* = 27, 197 lb/ piling

Superstructure:

- Cubic yards of concrete poured was taken from design plans (Mahoning County Engineer)
- Weight of concrete was assumed to be equal to 2.5 times the weight of water. (Army Corp. of Engineers)
- 502,031 lbs / 26 pilings supporting superstructure* = 27, 197 lb/ piling

* Pilings are assumed to equally distribute loads

Piling Rating:

Each piling is rated at 22 ton bearing capacity.

Summary: The weight of the structural components is greater than the bearing capacity of the pilings.

Horizontal Forces

Due to the location of the vertical forces, the primary moment resisting component is the approach slab key in the abutment. The required resistance of the key is estimated at 1012 lb/ft.

(Appendix B, Table B-7)

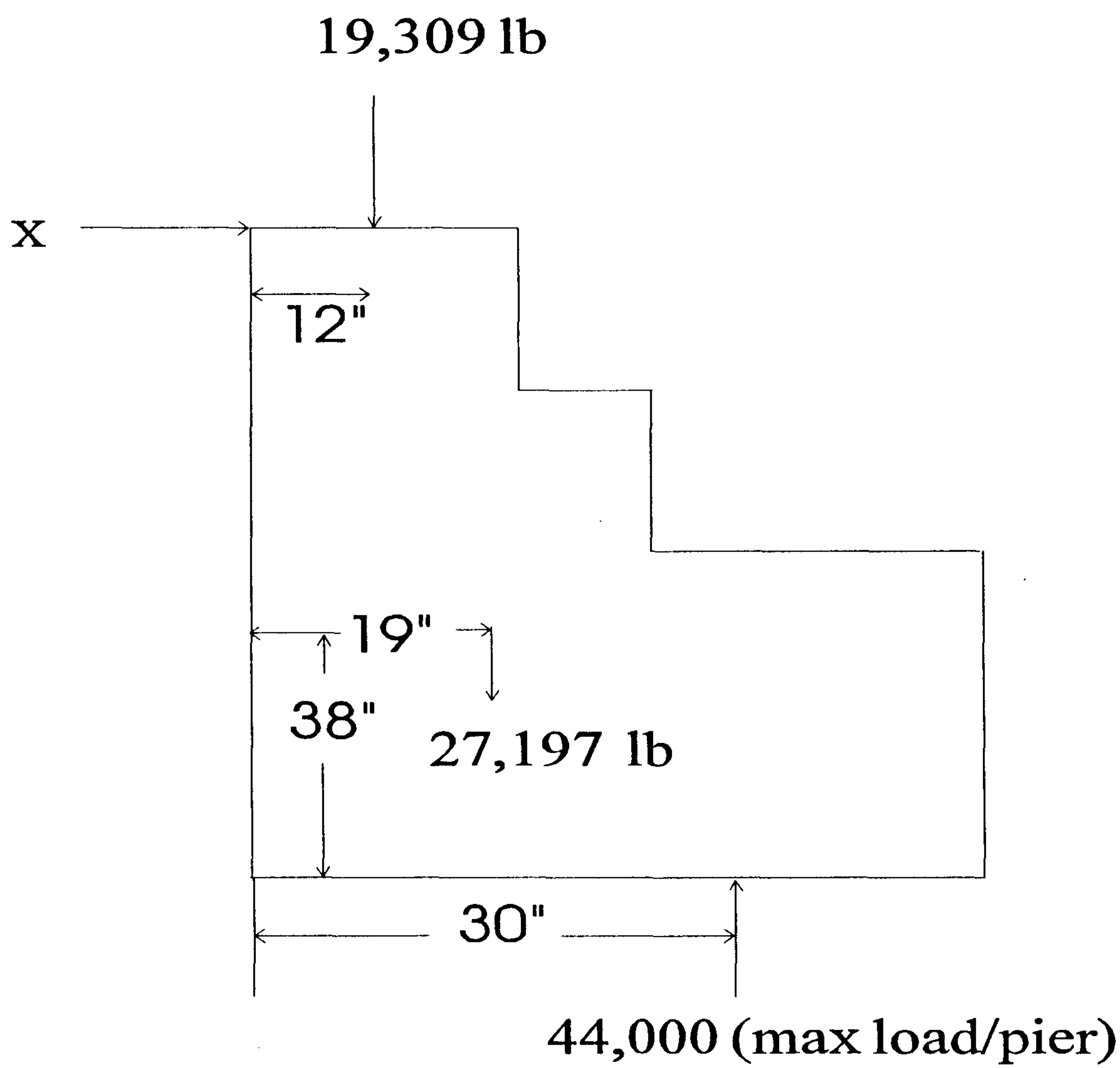


Figure 3.2 Abutment Stability Analysis

Chapter IV: BRIDGE RECOMMENDATIONS

4.1 Discussion

Recommendations are provided as to the action that should be taken to remedy any scour critical potential that may exist on the bridge. Any recommendation should address the relative extent of scour and the costs of constructed countermeasures. The Federal Highway Administration recommends that the following countermeasures be instituted:

(1) For the bridges classified as scour critical, a plan of action for installing scour countermeasures will be developed. The following measures will be examined for each bridge and the most suitable and economically efficient approach will be selected.

   a) Providing riprap at piers and abutments.
   b) Constructing guide banks (spur dikes).
   c) Constructing channel improvements.
   d) Strengthening the bridge foundation.
   e) Constructing sills or drop structures.
   f) Constructing relief bridges or lengthening existing bridges.

(2) The costs of constructing the scour countermeasures will be estimated for each scour critical bridge.

(3) Recommendations for monitoring, inspecting and closing of the bridge until the countermeasures are installed will be made for each scour critical bridge.

Many approaches could be taken to address the situation at the Little Beaver Creek Goshen Road crossing. These should be carefully examined by the County Engineer to decide the most cost effective approach. It is recommended that any countermeasures be instituted immediately as the bridge support structure is quickly degrading.

Since the south abutment is being affected by contraction imposed by external forces, it is recommended that a two phase approach be taken to the implementation of countermeasures.

Phase I: Infrastructure

The first step to alleviating the current problems is strengthening the bridge foundation under the south abutment. This should be done using a high strength material such as a concrete or grout foundation and be designed according to the AASHTO Manual for Bridge Maintenance, 1987. Although this is a costly option, some form of abutment reinforcement is necessary.

The back of the abutment adjacent to the approach slab should be backfilled in the areas where erosion has occurred. This will provide stability to the abutment during foundation strengthening and future events.

Riprap should be installed at the end of the abutment around the stormwater outfall. Riprap design can be accomplished by using the Ishbash equation for selecting stone diameter as shown in Figure 4.1. This procedure should also be used to select riprap for installation in front of both abutments.

Figure 4.1 Riprap Design Criteria

$$D_{50} = \left(\frac{.692V^2}{(s-1)2g} \right) \qquad \textit{(equation 4.1)}$$

D_{50} = average stone diameter (ft)
V = velocity against stone (ft/s) = V(contracted section) * 1.5
s = specific gravity of riprap material = 2.65
g = 32.2 ft/s
(FHWA, 1988)
For a 100 year flood, riprap diameter is sized as follows:

$$D_{50} = \left(\frac{.692(7.14)^2}{(1.65)64.4} \right) = 0.33ft = 4in$$

Short-term solutions can be used for the upstream tributary. The sediment which has shifted the channel flow can be excavated and upstream sediment dams can be placed in the tributary. Sediment dams will provide a cost-effective approach in the short-term, but will require long-term maintenance and may increase flooding in the adjacent field.

<u>Phase II - Runoff control</u>

In order to insure the long-term stability of the structure, the channel must be redirected to the middle of the opening. This can be accomplished through a variety of options. Most importantly, the upstream tributary should be relocated. This can be done by either redirecting the tributary upstream, or constructing a conduit under Goshen Road to discharge the runoff downstream. Redirecting the tributary to discharge upstream from the approach is possible but acts against the natural slope of the floodplain and may cause flooding of the adjacent field. Constructing a conduit under Goshen Road will provide the best prevention against future stream meandering through the reach of the channel. The downstream outfall of this tributary should be located where it has no effect on the structure. There are many potential locations for the outfall, and property easements must be considered. This option will provide long-term benefits by reducing sediment loading to the channel.

The stormwater outfall on the downstream end of the south abutment should be equipped with a sill. By providing a concrete sill, the discharge from this outfall will not contribute to additional erosion around the abutment.

4.2 Estimation of Construction Costs

Construction costs for the recommendations are estimated using the 1997 Construction Estimating Pricing Guide. Costs are estimated for on-site work only, excluding transportation and design costs, in Tables 4.1 and 4.2.

Table **4.1** Estimated Corrective Costs

1. Abutment Foundation
    Reinforced Foundation Mat
        < 10 cy = $233/cy
        > 10 cy = $157/cy
    Retaining wall during construction, 4 ft high, 5.5 ft deep = $21/s.f.
        46 ft X 9 ft X .5 ft = 77 cy = $12, 100

2. Riprap with steel mesh reinforcement:
        36in deep: $100/cy
        18in deep: $56.50/cy
        Required riprap (stormwater discharge) = 9 ft X 9 ft X 3 ft =9 cy = $900
        Required riprap (abutment protection discharge):
            = 18 ft X 9 ft X 3 ft X (2 Abutments) = 36 cy = $3600

3. Abutment Backfill
    Short term can consist of grout or slag fill. Long term will require excavation and bracing of the abutment, which cannot be estimated without detailed analysis.
    Slag Cost: $15/cy = $30 for 2 cy
    Labor Cost= $200 per day.
    Equipment Cost = $140/day
    Total = $200 + $140 + $30 = $370

4. Sediment Dams:
    Jute mesh: 100 square yard rolls: $1.09/square yard
    Hay bales: Place and remove: $375/ton
        Temporary sediment traps, 200 square yards, 2 ton hay = $968

5. Tributary routing
    Tunnel under roadway: 24-48 in outside diameter: $600/ ft
        For 40 feet = $24,000.
    Trench excavation: 4-6 ft deep: $3.72/cy
        To be estimated depending on location of culvert.

6. Excavation of channel
    With heavy equipment, average soil (12.5 cy/hr.)
    3 ft X 13 ft X 46 ft = 67 cy = 6 hrs.
    $210/day equipment cost
    $3.65/ cy labor cost = $250
    Grading = $477
    Disposal = $50/12 cy= $300
    Total = $1237

Table 4.2 Summary of Construction Costs

Option	Need	Estimated Cost
Foundation strengthening	high priority/ highly recommended	$12,100 (without excavation)
Riprap	high priority/ highly recommended	$5400
Abutment backfill	high priority/ highly recommended	$370
Sediment dams	possible/ highly recommended for short-term	$968
Tributary routing	possible/ highly recommended for long-term	$24,000 (without excavation)
Excavation of channel	possible	$1237

Chapter V: ON-GOING MONITORING

5.1 Discussion

The bridge has been rated as scour critical. The scour action plan recommends this bridge as a priority for installation of countermeasures. Until countermeasures are installed, the bridge should be monitored during high flows and closed if necessary. A program should be instituted to closely monitor the current progression of scour under the south abutment. After the required upgrades are made to the abutment, periodic inspections should be made to the site. Channel elevation profiles should be tracked to predict any future scour problems.

Chapter VI: FUTURE CONSIDERATIONS

6.1 Discussion

Development of a procedure for analyzing bridges for scour is an important aspect of this study, and procedures from the FHWA and Minnesota Department of Transportation (MnDOT) are used as guidelines throughout. It was apparent that the MnDOT screening procedure provides an efficient tool for large scale evaluations. In addition, HEC 18 (FHWA, 1993) provides a detailed summary of the procedure as a guidance document.

Scour equations should be used with discretion to assure important site conditions are addressed. Experience and sound engineering judgment play a vital role in this process. The equations have many shortcomings due to the fact that most were developed in the laboratory and that each bridge has highly variable characteristics. Because of this, the engineer must understand the applications of all the equations prior to use.

Strategies can be instituted to reduce the future risk of bridge failure from scour, including design, inspection and maintenance. Design considerations should focus on foundation and materials strengths. These include: abutments protected by properly designed riprap, piers or abutments on piles with pile tips more than 40 feet below the lowest channel bottom, or pile foundations located in stiff clay with high unconfined compressive strengths (MnDOT, 1995). Preventing the accumulation of debris and blockage can help reduce the possibility of increased channel velocities caused by head differentials. Debris prevention methods consist of constructing angled walls that force materials away from the piers and maintenance. Most importantly, during the life of the bridge, scheduled inspection should be performed to insure that significant scour conditions are not developing.

All hydraulic structures are susceptible to scour, especially bridges spanning rivers with the potential for high velocity flows. Many factors can increase the depth of scour, and should be considered during the siting, design and construction phases. The preceding analysis helped identify these primary factors as well as less apparent factors concerning placement of runoff outlets. In hindsight, the effect of the external factors is obvious. The challenge for civil engineers lies in having the ability to predict the eventual changes over time, and adequately design structures in a safe and cost effective manner to withstand the variations.

APPENDIXES

APPENDIX A Results

Table A-1 RESULTS OF HRQ COMPUTER PROGRAM

```
                H Y D R O - R O U T E
                     developed by
                Irfan A. Khan Ph.D., P.E.

Name of user ................ : Mike Rekstis
Dated ....................... : 05/03/97
Methodolgy used ............. : TR-55

   *****************************************

Little Beaver Creek
Mahoning/Columbiana Counties
Peak Discharge

   *****************************************
```

*** PHYSICAL DATA OF THE DRAINAGE BASIN ***

Subbasin No : 1

** Given Data **

Drainage area (acres) .... : 10533
Percentage of pond and swamp areas .... : 3
Longest distance to the outlet (ft) .... : 29166
Average slope of the basin (%) .... : 4.2544

*** Data for curve number ***

** Land Use Data **

Type of Landuse	Area (acres)	Curve Number A	B	C	D
Commercial	230	89	92	94	95
Residential	1042	51	68	79	84
Roads	258	83	89	92	93
Woods	3458	25	55	70	77
Pasture	2754	49	69	79	84
Agricultural	2475	67	78	85	89

** Computed Data **

Pond and swamp factor .... : .75
Computed time of concentration (hrs) .... : 4.534
Computed composite curve number .... : 74.18

***** SUBBASIN NO. 1 *****
***** TR55 PEAK DISCHARGE VALUES *****

Recurrence Interval (yrs)	24 - hour Rainfall (in)	Surface Runoff (in)	Pond/Swamp Factor --	Peak Discharge (cfs)
10	3.60	1.32	0.75	1793
25	4.10	1.68	0.75	2326
50	4.60	2.06	0.75	2893
100	4.80	2.22	0.75	3128

Table A-2 RESULTS OF hy93 COMPUTER PROGRAM FOR 100 YEAR EVENT

****

CONTRACTION SCOUR
CASE 1 Overbank flow on a flood plain being forced back to the main channel by the approaches to the bridge.
BRIDGE NUMBER

1 flow depth @ approach	y1	ft	=	6.51
2 width @ approach	w1	ft	=	355.5
3 width @ constriction	w2	ft	=	73
4 contracted flow	Qt	cfs	=	3128
5 main channel flow @ approach	Qc	cfs	=	3128
6 Vratio ShearV/FallV			=	39.05
7 Manning nRatio contracted /approach			=	.035

CONTRACTION SCOUR EQUATION 1 = -1 Ft

****

ABUTMENT SET AT THE EDGE OF CHANNEL Equation (10)
LEFT ABUTMENT
BRIDGE NUMBER

1 skew angle @ abutment	theta		deg	=	15
2 main channel flow @ approach		Qc	cfs	=	3128
3 overbank flow @ approach		Qo	cfs	=	3128
4 overbank depth @ approach		Yo	ft	=	3.51
5 main channel depth @ approach		y1	ft	=	3
6 width of main channel		W	ft	=	73

ABUTMENT SCOUR EQUATION 10 = 6 Ft
ABUTMENT SCOUR EQUATION 11 = 0 Ft

****

ABUTMENT SET AT THE EDGE OF CHANNEL Equation (10)
RIGHT ABUTMENT
BRIDGE NUMBER

1 skew angle @ abutment	theta		deg	=	20
2 main channel flow @ approach		Qc	cfs	=	3128
3 overbank flow @ approach		Qo	cfs	=	3128
4 overbank depth @ approach		Yo	ft	=	3.51
5 main channel depth @ approach		y1	ft	=	3
6 width of main channel		W	ft	=	73

ABUTMENT SCOUR EQUATION 10 = 7 Ft
ABUTMENT SCOUR EQUATION 11 = 0 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 1
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	10
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	5.74
5 depth of flow @ approach		y1	ft	=	6.51
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 2
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	10
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	7.27
5 depth of flow @ approach		y1	ft	=	6.51
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 3
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	5
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	7.27
5 depth of flow @ approach		y1	ft	=	6.51
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 4
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	10
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	7.27
5 depth of flow @ approach		y1	ft	=	6.51
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 5
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	5
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	7.27
5 depth of flow @ approach		y1	ft	=	6.51
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 6
BRIDGE NUMBER

1	attack angle of flow	theta		deg	=	20
2	length of pier		L	ft	=	1
3	width of pier		a	ft	=	1
4	velocity of flow @ approach		V	fps	=	5.74
5	depth of flow @ approach		y1	ft	=	6.51
6	pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 5 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 7
BRIDGE NUMBER

1	attack angle of flow	theta		deg	=	20
2	length of pier		L	ft	=	1
3	width of pier		a	ft	=	1
4	velocity of flow @ approach		V	fps	=	7.27
5	depth of flow @ approach		y1	ft	=	6.51
6	pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 5 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 8
BRIDGE NUMBER

1	attack angle of flow	theta		deg	=	15
2	length of pier		L	ft	=	1
3	width of pier		a	ft	=	1
4	velocity of flow @ approach		V	fps	=	7.27
5	depth of flow @ approach		y1	ft	=	6.51
6	pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 5 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 9
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	10
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	7.27
5 depth of flow @ approach		y1	ft	=	6.51
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 10
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	5
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	7.27
5 depth of flow @ approach		y1	ft	=	6.51
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

Table A-3 RESULTS OF hy93 COMPUTER PROGRAM FOR 500 YEAR EVENT

****

CONTRACTION SCOUR
CASE 1 Overbank flow on a flood plain being forced back to the main channel by the approaches to the bridge.
BRIDGE NUMBER

1 flow depth @ approach	y1	ft	=	9.46
2 width @ approach	w1	ft	=	613
3 width @ constriction	w2	ft	=	76
4 contracted flow	Qt	cfs	=	5318
5 main channel flow @ approach	Qc	cfs	=	5318
6 Vratio ShearV/FallV			=	47.08
7 Manning nRatio contracted /approach			=	.035

CONTRACTION SCOUR EQUATION 1 = 2 Ft

****

ABUTMENT SET AT THE EDGE OF CHANNEL Equation (10)
LEFT ABUTMENT
BRIDGE NUMBER

1 skew angle @ abutment	theta		deg	=	15
2 main channel flow @ approach		Qc	cfs	=	5318
3 overbank flow @ approach		Qo	cfs	=	5318
4 overbank depth @ approach		Yo	ft	=	6.46
5 main channel depth @ approach		y1	ft	=	3
6 width of main channel		W	ft	=	74

ABUTMENT SCOUR EQUATION 10 = 8 Ft
ABUTMENT SCOUR EQUATION 11 = 0 Ft

****

ABUTMENT SET AT THE EDGE OF CHANNEL Equation (10)
RIGHT ABUTMENT
BRIDGE NUMBER

1 skew angle @ abutment	theta		deg	=	20
2 main channel flow @ approach		Qc	cfs	=	5318
3 overbank flow @ approach		Qo	cfs	=	5318
4 overbank depth @ approach		Yo	ft	=	6.46
5 main channel depth @ approach		y1	ft	=	3
6 width of main channel		W	ft	=	74

ABUTMENT SCOUR EQUATION 10 = 10 Ft
ABUTMENT SCOUR EQUATION 11 = 0 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 1
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	10
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	2.7
5 depth of flow @ approach		y1	ft	=	9.46
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 3 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 2
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	10
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	8.84
5 depth of flow @ approach		y1	ft	=	9.46
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 5 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 3
BRIDGE NUMBER

1	attack angle of flow	theta		deg	=	5
2	length of pier		L	ft	=	1
3	width of pier		a	ft	=	1
4	velocity of flow @ approach		V	fps	=	8.84
5	depth of flow @ approach		y1	ft	=	9.46
6	pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 4
BRIDGE NUMBER

1	attack angle of flow	theta		deg	=	5
2	length of pier		L	ft	=	1
3	width of pier		a	ft	=	1
4	velocity of flow @ approach		V	fps	=	8.84
5	depth of flow @ approach		y1	ft	=	9.46
6	pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 5
BRIDGE NUMBER

1	attack angle of flow	theta		deg	=	5
2	length of pier		L	ft	=	1
3	width of pier		a	ft	=	1
4	velocity of flow @ approach		V	fps	=	8.84
5	depth of flow @ approach		y1	ft	=	9.46
6	pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 6
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	20
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	2.70
5 depth of flow @ approach		y1	ft	=	9.46
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 7
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	20
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	8.84
5 depth of flow @ approach		y1	ft	=	9.46
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 6 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 8
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	15
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	8.84
5 depth of flow @ approach		y1	ft	=	9.46
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 5 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 9
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	10
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	8.84
5 depth of flow @ approach		y1	ft	=	9.46
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 5 Ft

****

PIER SCOUR Equation (12)
PIER NUMBER 10
BRIDGE NUMBER

1 attack angle of flow	theta		deg	=	5
2 length of pier		L	ft	=	1
3 width of pier		a	ft	=	1
4 velocity of flow @ approach		V	fps	=	8.84
5 depth of flow @ approach		y1	ft	=	9.46
6 pier type code 1 - 5					1

PIER SCOUR EQUATION 12 = 4 Ft

APPENDIX B Spreadsheets

Table B - 1 AREAS OF VARIOUS LAND USE

Sectio	Area		Percentage of Landuse					Water/
		Commercial	Residential	Roads	Woods	Pasture	Agriculture	Swamp
1	12.32	0	5	2	30	25	37	1
2	7.88	0	10	2	27	17	27	17
3	3.72	0	10	2	60	13	5	10
4	1.33	0	5	2	55	30	2	6
5	31.63	0	5	2	23	30	30	10
5b	-0.76				20	80		
6	7.97	0	5	2	23	30	30	10
7	37.17	5	15	3	35	22	12	8
hanov	1.42	5	15	3	35	22	10	10
sale	10.72	5	15	3	35	22	10	10
	113.40							
% of Watershed		**2.17**	**9.89**	**2.45**	**32.80**	**26.10**	**23.50**	**3.00**
			Acres of Landuse					
1		0.00	0.62	0.25	3.70	3.08	4.56	0.12
2		0.00	0.79	0.16	2.13	1.34	2.13	1.34
3		0.00	0.37	0.07	2.23	0.48	0.19	0.37
4		0.00	0.07	0.03	0.73	0.40	0.03	0.08
5		0.00	1.58	0.63	7.27	9.49	9.49	3.16
5b		0.00	0.00	0.00	-0.15	-0.61	0.00	0.00
6		0.00	0.40	0.16	1.83	2.39	2.39	0.80
7		1.86	5.58	1.12	13.01	8.18	4.46	2.97
hanover		0.07	0.21	0.04	0.50	0.31	0.14	0.14
salem		0.54	1.61	0.32	3.75	2.36	1.07	1.07
		2.47	11.22	2.78	35.00	27.42	24.45	10.06
								113.40
Miles	0.36	1.63	0.40	5.08	3.98	3.55	1.46	16.46
Acres	230	1,042	258	3,458	2,754	2,475	316	10,533

Table B - 2 SUMMARY OF HYDROLOGIC SOIL TYPES

	Summary of Hydrologic Soil Types							
		(taken from SCS maps, pages and parcel #'s included)						
Soil type ratings:				Hydrologic				
	Abbr.	Name		rating	Composite rating			
	BCJ	Bogart-Chili-Jimtown		B-B-C	B		43560	
	CRW	Canfield-Ravenna-Wooste		C-C-C	C			
	SF	Sebring-Fitchville		D-C	D			
	WR	Wadsworth-Rittman		C-C	C			
	CW	Chili-Wayland		B-C/D	C			
	Can. W	Canfield-Wooster		C-C	C			
	CN	Chili-Negley		B-B	B			
	Mahoning County			Square	Square Miles of Each			
Page #	Parcel #	Soil Type	%	Footage	B	C	D	
58	27	B	80	34848	0.80			
		C	20	8712		0.20		
	34	B	100	43560	1.00			
57	28	B	40	17424	0.40			
		C	60	26136		0.60		
	29	C	100	43560		1.00		
	32	C	100	43560		1.00		
	33	B	25	10890	0.25			
		C	75	32670		0.75		
48	15	C	30	13068		0.30		
		D	70	30492			0.7	
	22	C	100	43560		1.00		
	23	D	5	2178			0.05	
		C	20	8712		0.20		
		B	75	32670	0.75			
47	16	C	100	43560		1.00		
	21	C	95	41382		0.95		
		B	5	2178	0.05			
					3.25	7.00	0.75	
			Acreage		2080	4480	480	7040
	Columbiana County				Square Miles of Each			
Page #	Parcel #	Soil Type	%		B	C	D	
1	25	C	90			0.90		
		C	10			0.10		
	26	C	80			0.80		
		C	20			0.20		
	35	C	70			0.70		
		C	30			0.30		
	36	C	35			0.35		
		C	55			0.55		
		C	10			0.10		

6	2	C		70			0.70		
		C		30			0.30		
	1	C		60			0.60		
		C		40			0.40		
	3	C		55			0.55		
		C		45			0.45		
	10	B		25		0.25			
		C		75			0.75		
	11	C		100			1.00		
	12	C		90			0.90		
		C		10		__	0.10		
						0.25	9.75		
						160	6240		6400
						2240	10720	480	13440
					Acres:	1756	8401	376	10533

Table B - 3 VELOCITY AND FLOWRATE CALCULATIONS USING THE MANNING EQUATION

	Mannings Equation, V =		Cm/n *R^(2/3) *S^(.5)			
		S =	0.00385			
		Cm =	1.486			
Flow		n =	0.035			
Elevation	**Area (ft^2)**	**Wetted P (ft)**	**R**	**V (ft/s)**	**Q (cfs)**	
1098	20.64	37.2	0.55	1.78	37	
1099	62	45.32	1.38	3.26	203	
1100	129	48.53	2.66	5.06	654	
1101	179	75.65	2.37	4.68	838	
1102	244	76.85	3.17	5.68	1384	
1103	309	84.62	3.66	6.25	1934	
1104	383	92.71	4.13	6.78	2594	
1104.66	**430**	**94**	**4.59**	**7.28**	**3129**	
1105	454	94.61	4.80	7.50	3407	
1106	529	97.07	5.45	8.16	4313	
1107	**601.81**	**97.5**	**6.17**	**8.86**	**5335**	
					Using Q=VA	
	Mannings Equation, V =		Cm/n *R^(2/3) *S^(.5)			
		S =	0.00385			
		Cm =	1.486			
		n =	0.035			
		Measurements:	Distance	56.6	ft	
			Time	32.05	s	
				=	1.77	ft/s

Table B - 4 AREA AND WETTED PERIMETER CALCULATIONS AT 1 FOOT INCREMENTS

	Little Beaver Creek Span Opening						
Upstream Profile							
Segment	Ht	Length	Triangle Factor	Area			
A1	2.08	25.25	1	52.60			
A2	2.83	24.75	1	70.13			
A3	2.83	22.83	1	64.69			
A4	1.23	20.33	1	25.01			
A5	2.19	20.33	0.5	22.27			
A6	9.14	13.00	1	118.82			
A7	2.19	13.00	0.5	14.24			
A8	9.90	13.00	1	128.70			
A9	6.83	13.00	1	88.79			
A10	0.67	13.00	0.5	4.36			
A11	5.42	13.00	1	70.46			
A12	2.83	1.00	1	2.83			
Pier supports	9.8	1	-2	-19.60			
				643.29		square feet	
Downstream Profile							
Segment	Ht	Length	Triangle Factor	Area			
A1	2.00	20.00	1	40.00			
A2	2.17	19.50	1	42.25			
A3	4.17	17.42	1	72.58			
A4	1.17	19.50	1	22.75			
A5	9.83	6.50	1	63.90			
A6	9.29	6.50	1	60.39			
A7	9.25	6.50	1	60.13			
A8	8.14	13.00	1	105.82			
A9	6.71	6.50	1	43.62			
A10	6.92	6.50	1	44.98			
A11	5.92	6.50	1	38.48			
A12	3.92	6.50	1	25.48			
A13	1.25	1.00	1	1.25			
Pier supports	9.8	1	-2	-19.60			
				602.01	square ft.		
Original Profile							
Segment	Ht.	Length	Triangle Factor	Area			
A1	2.08	17.75	1	36.98			
A2	8.58	15.00	0.5	64.35			
A3	10.20	43.00	1	438.60			
A4	8.03	15.00	0.5	60.23			
A5	2.17	17.75	1	38.46			
Pier supports	9.8	2.5	-2	-49.00			
				589.61		square feet	

Total Volume of Channel							
Average Square Footage		**Length of Span (ft)**			**Volume**		
Current:	622.65		36		22415	cubic ft	
Original	589.61		36		21226	cubic ft	
Average Channel Depth						**Average**	
	Length of Span		**Width of Span**		**Volume**	**Elevation**	
Current	36		79.5		22415	1099.338	
Original	36		79.5		21226	1099.753	
Downstream Profile		at elevation 1098					
Segment	**Ht**	**Length**	**Triangle Factor**	**Area**		**wpfactor**	
A1	0.00	20.00	1	0.00		0.00	
A2	0.00	19.50	1	0.00		0.00	
A3	0.00	17.42	1	0.00		0.68	
A4	0.17	19.50	1	3.32		14.80	
A5	0.85	6.50	1	5.53		6.50	
A6	0.62	6.50	1	4.03		6.53	
A7	0.62	6.50	1	4.03		6.53	
A8	0.41	6.50	1	2.67		2.17	
A9	0.00	6.50	1	0.00			
A10	0.00	6.50	1	0.00			
A11	0.00	6.50	1	0.00			
A12	0	6.5	1	0.00			
A13	0	1	1	0.00			
Pier supports	1.73	2.5	-1	-4.33			
				15.24	square f	37.20	ft
Downstream Profile		at elevation 1099					
Segment	**Ht**	**Length**	**Triangle Factor**	**Area**		**wpfactor**	
A1	0.00	20.00	1	0.00		0.00	
A2	0.00	19.50	1	0.00		0.00	
A3	0.68	17.42	1	11.84		0.68	
A4	1.17	19.50	1	22.75		20.67	
A5	1.85	6.50	1	12.03		6.50	
A6	0.93	6.50	1	6.01		6.57	
A7	0.93	6.50	1	6.01		6.57	
A8	0.62	13.00	1	8.06		4.34	
A9	0.00	6.50	1	0.00			
A10	0.00	6.50	1	0.00			
A11	0.00	6.50	1	0.00			
A12	0	6.5	1	0.00			
A13	0	1	1	0.00			
Pier supports	1.73	2.5	-1	-4.33			
				62.38	square f	45.32	ft

Downstream Profile		elevation	1100	Differential		2.85	
Segment	**Ht**	**Length**	**Triangle Factor**	**Area**		**Wet P.**	
A1	0.00	20.00	1	0.00		0.00	
A2	0.00	19.50	1	0.00		0.00	
A3	1.68	17.42	1	29.32		1.68	
A4	1.17	19.50	1	22.75		20.67	
A5	2.85	6.50	1	18.52		6.50	
A6	3.39	6.50	1	22.03		6.56	
A7	3.35	6.50	1	21.77		6.56	
A8	1.67	13.00	1	21.68		6.55	
A9	0.00	6.50	1	0.00			
A10	0.00	6.50	1	0.00			
A11	0.00	6.50	1	0.00			
A12	0	6.5	1	0.00			
A13	0	1	1	0.00			
Pier supports	2.73	2.5	-1	-6.83			
				129.26		48.53	ft
Downstream Profile		elevation	1101	Differential		3.85	
Segment	**Ht**	**Length**	**Triangle Factor**	**Area**		**Wet P.**	
A1	0.00	20.00	1	0.00		0.00	
A2	0.00	19.50	1	0.00		0.00	
A3	2.68	17.42	1	46.73		2.68	
A4	1.17	19.50	1	22.75		20.67	
A5	3.85	6.50	1	25.02		6.50	
A6	4.39	6.50	1	28.53		6.56	
A7	4.35	6.50	1	28.27		6.56	
A8	2.16	13.00	1	28.14		13.18	
A9	0.81	6.50	1	5.26		6.50	
A10	0.98	6.50	1	6.37		6.50	
A11	0.02	6.50	1	0.13		6.50	
A12	-1.94	6.5	0	0.00			
A13	-2.75	1	0	0.00			
Pier supports	3.85	2.5	-1	-12.12			
				179.10		75.65	ft

Downstream Profile		elevation	1102	Differential		4.85	
Segment	**Ht**	**Length**	**Triangle Factor**	**Area**		**Wet P.**	
A1	0.00	20.00	1	0.00		0.00	
A2	0.00	19.50	1	0.00		0.00	
A3	3.68	17.42	1	64.15		3.68	
A4	1.17	19.50	1	22.75		20.67	
A5	4.85	6.50	1	31.52		6.50	
A6	5.39	6.50	1	35.03		6.56	
A7	5.35	6.50	1	34.77		6.56	
A8	3.16	13.00	1	41.14		13.38	
A9	1.81	6.50	1	11.76		6.50	
A10	1.98	6.50	1	12.87		6.50	
A11	1.02	6.50	1	6.63		6.50	
A12	-0.94	6.5	0	0.00			
A13	-1.75	1	0	0.00			
Pier supports	4.85	2.5	-1	-17.12			
				243.52		76.85	ft

Downstream Profile		elevation	1103	Differential		5.85	
Segment	**Ht**	**Length**	**Triangle Factor**	**Area**		**Wet P.**	
A1	0.00	20.00	1	0.00		0.00	
A2	0.52	19.50	1	10.07		0.52	
A3	4.17	17.42	1	72.57		4.17	
A4	1.17	19.50	1	22.75		20.67	
A5	5.85	6.50	1	38.02		6.50	
A6	6.39	6.50	1	41.53		6.56	
A7	6.35	6.50	1	41.27		6.56	
A8	4.16	13.00	1	54.14		13.65	
A9	2.81	6.50	1	18.26		6.50	
A10	2.98	6.50	1	19.37		6.50	
A11	2.02	6.50	1	13.13		6.50	
A12	0.06	6.5	1	0.39		6.50	
A13	-0.75	1	0	0.00			
Pier supports	5.85	2.5	-1	-22.12			
				309.40		84.62	ft

Downstream Profile		elevation	1104	Differential		6.85	
Segment	**Ht**	**Length**	**Triangle Factor**	**Area**		**Wet P.**	
A1	0.00	20.00	1	0.00		0.00	
A2	1.52	19.50	1	29.57		1.52	
A3	4.17	17.42	1	72.57		4.17	
A4	1.17	19.50	1	22.75		20.67	
A5	6.85	6.50	1	44.52		6.50	
A6	7.39	6.50	1	48.03		6.56	
A7	7.35	6.50	1	47.77		6.56	
A8	5.16	13.00	1	67.14		13.99	
A9	3.81	6.50	1	24.76		6.50	
A10	3.98	6.50	1	25.87		6.50	
A11	3.02	6.50	1	19.63		6.50	
A12	1.06	6.5	1	6.89		6.50	
A13	0.25	1	1	0.25		6.75	
Pier supports	6.85	2.5	-1	-27.12			
				382.65		92.71	ft
Downstream Profile		elevation	1105	Differential		7.85	
Segment	**Ht**	**Length**	**Triangle Factor**	**Area**		**Wet P.**	
A1	0.35	20.00	1	7.00		0.35	
A2	2.17	19.50	1	42.25		2.17	
A3	4.17	17.42	1	72.57		4.17	
A4	1.17	19.50	1	22.75		20.67	
A5	7.85	6.50	1	51.02		6.50	
A6	8.39	6.50	1	54.53		6.56	
A7	8.35	6.50	1	54.27		6.56	
A8	6.16	13.00	1	80.14		14.39	
A9	4.81	6.50	1	31.26		6.50	
A10	4.98	6.50	1	32.37		6.50	
A11	4.02	6.50	1	26.13		6.50	
A12	1.75	6.5	1	11.38		6.50	
A13	0.75	1	1	0.75		7.25	
Pier supports	7.85	2.5	-1	-32.12			
				454.31		94.61	ft

Downstream Profile		elevation	1106	Differential		8.85	
Segment	**Ht**	**Length**	**Triangle Factor**	**Area**		**Wet P.**	
A1	1.35	20.00	1	27.00		1.35	
A2	2.17	19.50	1	42.25		2.17	
A3	4.17	17.42	1	72.57		4.17	
A4	1.17	19.50	1	22.75		20.67	
A5	8.85	6.50	1	57.52		6.50	
A6	9.39	6.50	1	61.03		6.56	
A7	9.35	6.50	1	60.77		6.56	
A8	7.16	13.00	1	93.14		14.84	
A9	5.81	6.50	1	37.76		6.50	
A10	5.98	6.50	1	38.87		6.50	
A11	5.02	6.50	1	32.63		6.50	
A12	2.75	6.5	1	17.88		6.50	
A13	1.75	1	1	1.75		8.25	
Pier supports	8.85	2.5	-1	-37.12			
				528.81		97.07	ft
Downstream Profile		elevation	1107	Differential		9.85	
Segment	**Ht**	**Length**	**Triangle Factor**	**Area**		**Wet P.**	
A1	2.35	20.00	1	47.00		2.35	
A2	2.17	19.50	1	42.25		2.17	
A3	4.17	17.42	1	72.57		4.17	
A4	1.17	19.50	1	22.75		20.67	
A5	9.85	6.50	1	64.02		6.50	
A6	10.39	6.50	1	67.53		6.56	
A7	10.35	6.50	1	67.27		6.56	
A8	8.16	13.00	1	106.14		15.35	
A9	6.81	6.50	1	44.26		6.50	
A10	6.98	6.50	1	45.37		6.50	
A11	6.02	6.50	1	39.13		6.50	
A12	3.75	6.5	1	24.38		6.50	
A13	1.25	1	1	1.25		7.75	
Pier supports	9.85	2.5	-1	-42.12		75.50	
				601.81		173.57	ft
						*when flowing full	
Approach Flow	at 100 year flood - elev. = 1104.66						
Area	**Length**	**Depth**	**Triangle factor**	**Area**	**P**		
Left Bank	99	3	0.5	148.5	99.05		
Stream	15	6.51	1	97.65	15.00		
Right Bank	94.5	3	0.5	204.75	94.55		
Field	147	0.64	1	94.08	98		
	355.5			544.98	306.59		
Approach Flow	at 500 year flood - elev. = 1107						
Area	**Length**	**Depth**	**Triangle factor**	**Area**	**P**		
Left Bank	127.8	6.11	0.5	390.43	127.95		
Stream	15	9.46	1	141.9	15.00		
Right Bank	469.8	6.11	0.5	1435.2	469.84		
	612.6			1967.6	612.79		

Table B - 5 vRATIO AT FLOW ELEVATIONS FOR 100 AND 500 YEAR FLOODS

Step 4: Conraction Scour				
Elevation	**Approach**	**Approach**	**Contracted**	**Contracted**
	Depth, ft (y1)	**Width , ft (w1)**	**Width, ft (w2)**	**Flow, cfs (Qt)**
1104.66	6.51	355.50	73.00	3128.00
1107	9.46	613.00	76.00	5318.00
	Approach	**Shear**	**Fall**	**Vratio**
	Flow,cfs (Qc	**Velocity (ft/s)**	**Velocity (ft/s)**	
	3128.00	0.898356666	0.023	39.05898549
	5318.00	1.082938687	0.023	47.08429074
	(width contracted/width approach)			
	* (contracted flow)			

Table B - 6 VELOCITY CALCULATIONS AT CONTRACTED VARIABLES

	Flood	Approach Variables				
	Frequency	Depth	Width	Area	Flow	V
	100	6.51	355.5	544.98	3128	5.74
	500 *	9.46	613	1967.6	5318	2.703
			Contracted Section Variables			
	Depth	Width	Area	Flow	V	
	6.51	75	430	3128	7.274418605	
	9.46	78	601.81	5318	8.836676027	

Table B - 7 LEFT ABUTMENT STABILITY ANALYSIS

Left Abutment Stability Analysis							
	Abutment						
		Area		x		y	
		1	360	7.5	2700	88	31680
		2	504	10.5	5292	63	31752
		3	2300	23	52900	25	57500
			3164		60892		120932
			21.97222	ft^2			
			1220.4	ft^3	location of x coor =		19.24526
			45.2	yd^3	location of y coor. =		38.22124
		Weight =	190382.4	lb =	27197.49		
			* using weight of concrete = 2.5 weight of water				
	Superstructure						
		Volume =	119 cu.yd				
		Weight =	502031.3	lbs			
		Wt. per piling =		19308.89	lbs		
			(26 pilings)				
	Horizontal Forces:						
		19,309 + 27,197 =		46506	>44000		
				Therefore pier fails if undermined			
	Required Resistance for approach slab key to prevent overturning:						
		M(A) = 0					
		x(100) + 27,197(19) - 44,000(30) = 0					
		x =	8032.57	lb/piling			
		=	56227.99	lb total			
		=	1012.387	lb/ft			
			Construction joint consists of 2" concrete key				
			and is likely to fail if backfill is eroded.				

APPENDIX C Maps & Data

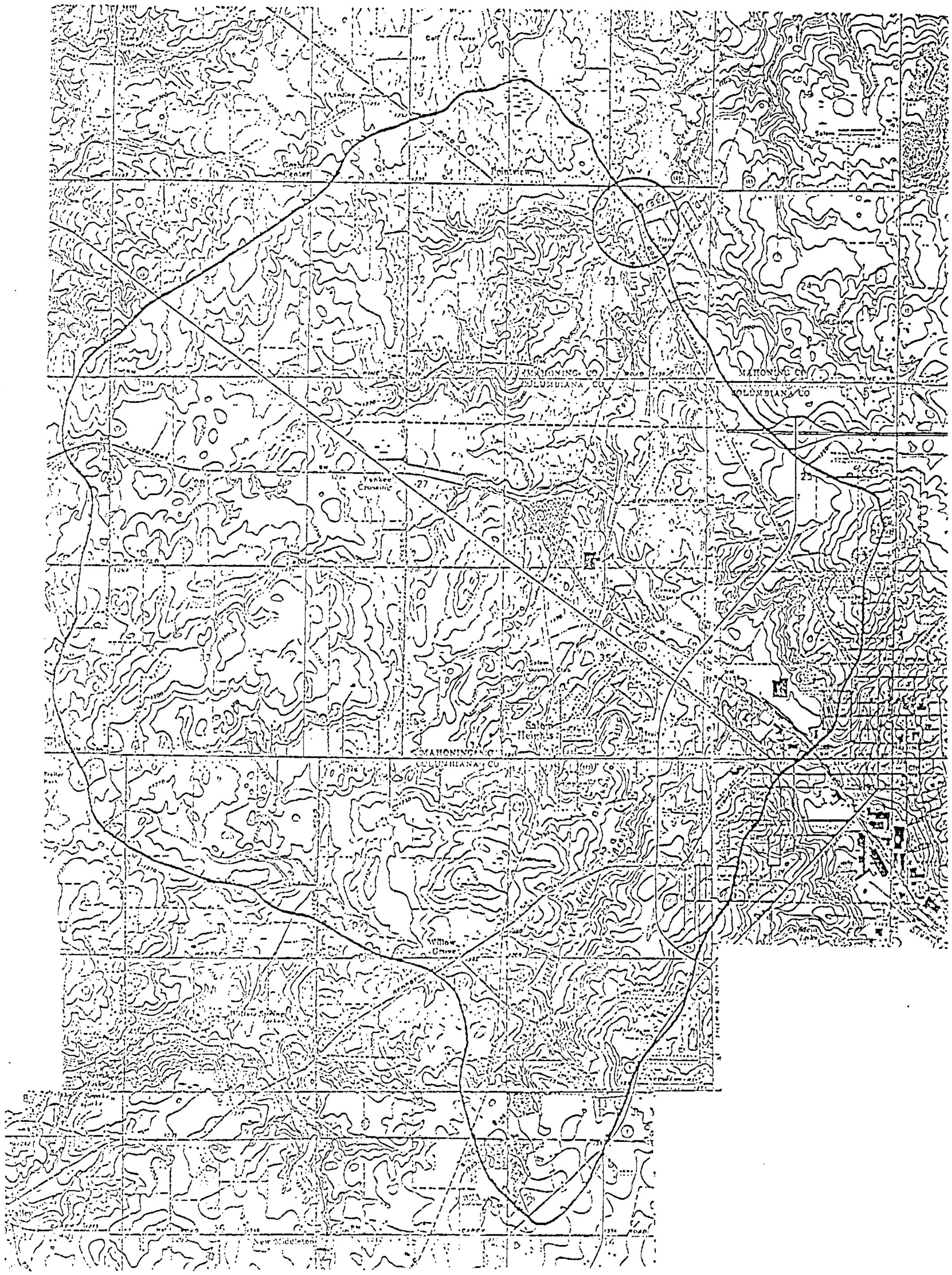


Figure C - 1 Topographic Map of the Little Beaver Creek North Fork Watershed

Figure C - 2 Grid Intersections for Watershed Slope Computation

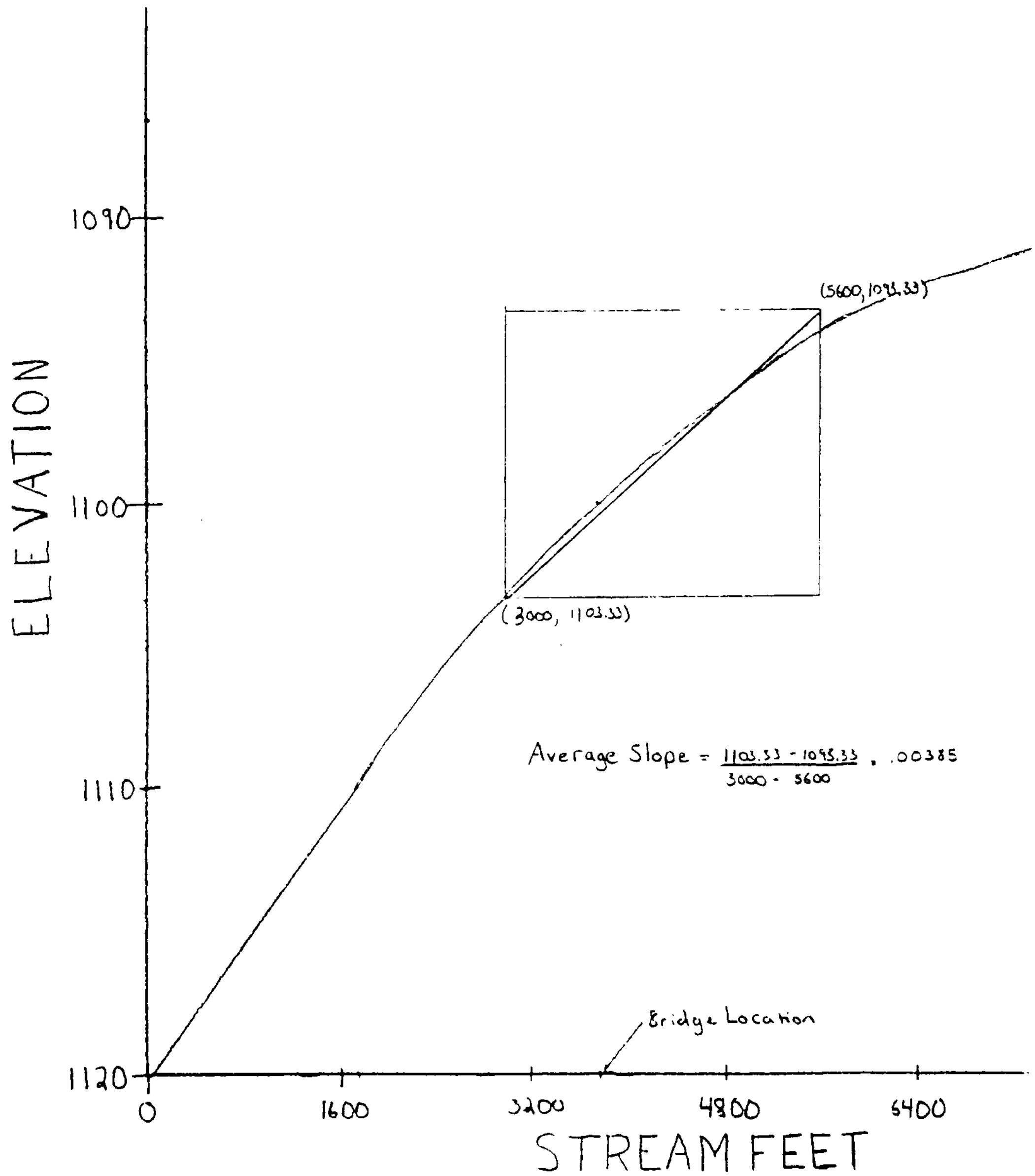


Figure C - 3 Average Slope of Stream Bed Computation

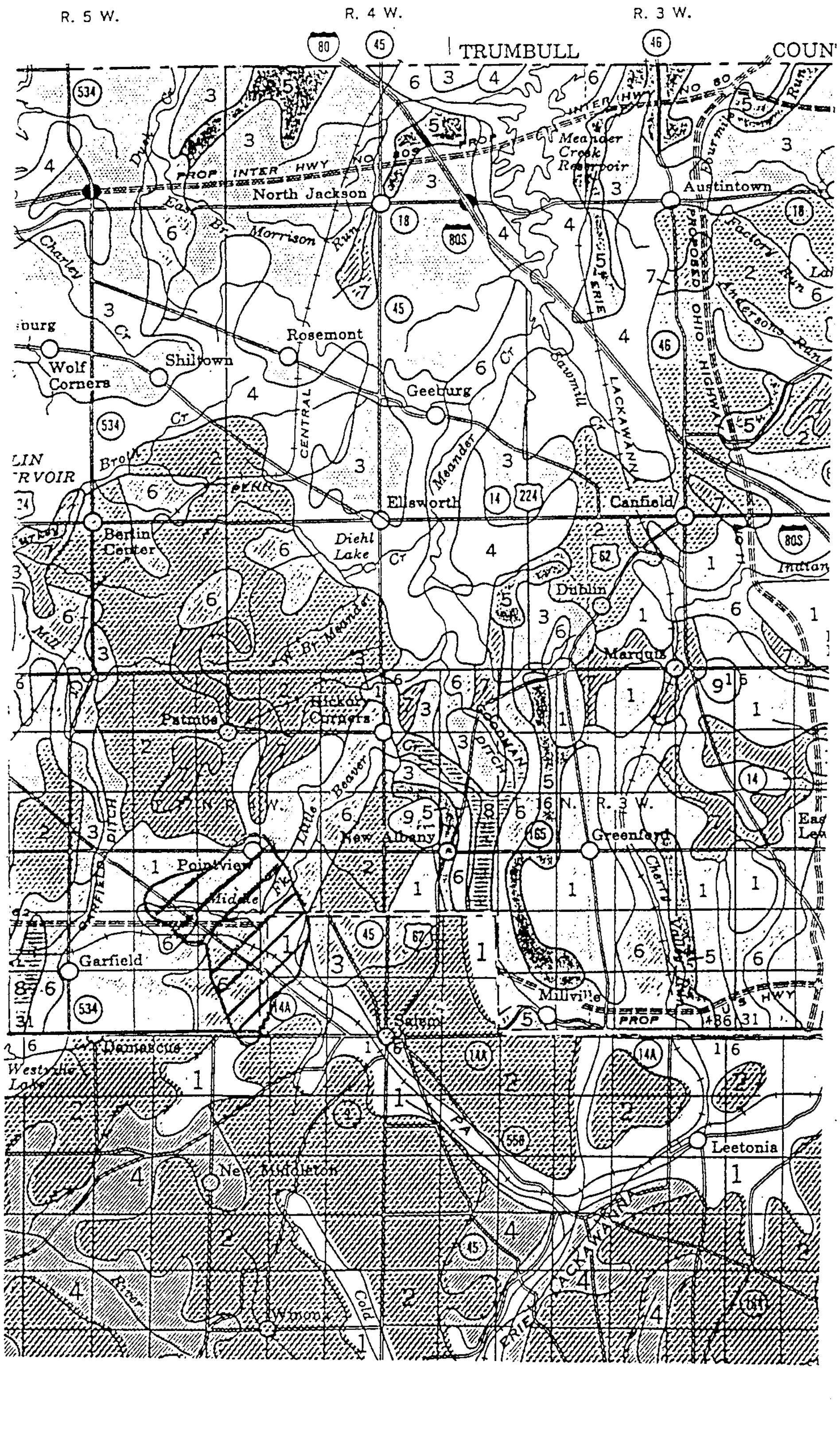


Figure C - 4 Soil Types

SOIL ASSOCIATIONS

1 Canfield-Ravenna-Wooster association: Mainly gently sloping, somewhat poorly drained to well-drained soils that have a fragipan in the subsoil; on uplands

2 Rittman-Wadsworth-Frenchtown association: Mainly gently sloping, moderately well drained to poorly drained soils that have a fragipan in the subsoil; on uplands

3 Mahoning-Ellsworth-Trumbull association: Nearly level to gently sloping, moderately well drained to poorly drained soils that have a moderately fine or fine textured subsoil; on uplands

4 Geeburg-Remsen-Trumbull association: Nearly level to gently sloping, moderately well drained to poorly drained soils that have a fine-textured subsoil; on uplands

5 Loudonville-Muskingum-Dekalb association: Gently sloping to steep, well-drained soils that are mostly moderately deep over sandstone or siltstone; on uplands

6 Bogart-Chili-Jimtown association: Gently sloping and sloping, well-drained to somewhat poorly drained soils that have a gravelly subsoil; on stream terraces and uplands

7 Sebring-Fitchville association: Nearly level to gently sloping, poorly drained and somewhat poorly drained soils that have a moderately fine textured subsoil; on former glacial lakebeds

8 Wayland-Orrville association: Nearly level, poorly drained and somewhat poorly drained soils on flood plains

9 Strip mine spoils association: Spoil piles of rock and glacial till

October 1969

Table C - 1 Ohio Precipitation Data

COUNTY/FLOOD FREQ.	1	2	5	10	25	50	100
ADAMS	2.5	2.8	3.5	4.0	4.6	5.0	5.5
ALLEN	2.3	2.7	3.3	3.8	4.3	4.8	5.2
ASHLAND	2.2	2.4	3.1	3.5	4.0	4.5	4.7
ASHTABULA	2.1	2.3	3.0	3.5	4.0	4.5	4.7
ATHENS	2.3	2.5	3.3	3.8	4.3	4.7	4.9
AUGLAIZE	2.4	2.7	3.4	3.9	4.4	4.9	5.2
BELMONT	2.2	2.5	3.2	3.7	4.3	4.7	4.9
BROWN	2.5	2.9	3.6	4.1	4.7	5.1	5.6
BUTLER	2.5	2.9	3.6	4.1	4.7	5.2	5.6
CARROLL	2.2	2.4	3.1	3.6	4.1	4.6	4.8
CHAMPAIGN	2.4	2.7	3.4	3.9	4.4	4.9	5.2
CLARK	2.4	2.7	3.4	3.9	4.5	4.9	5.3
CLERMONT	2.5	2.9	3.6	4.1	4.7	5.1	5.6
CLINTON	2.5	2.8	3.5	4.0	4.6	5.0	5.4
COLUMBIANA	2.2	2.4	3.2	3.7	4.2	4.6	4.8
COSHOCTON	2.2	2.4	3.2	3.6	4.1	4.6	4.8
CRAWFORD	2.2	2.5	3.2	3.6	4.1	4.6	4.8
CUYAHOGA	2.1	2.2	3.0	3.4	3.9	4.4	4.6
DRAKE	2.4	2.8	3.5	4.0	4.5	5.0	5.5
DEFIANCE	2.3	2.6	3.3	3.8	4.3	4.8	5.1
DELAWARE	2.3	2.5	3.3	3.7	4.2	4.7	4.9
ERIE	2.2	2.4	3.1	3.5	3.9	4.4	4.7
FAIRFIELD	2.3	2.5	3.3	3.7	4.3	4.7	4.9
FAYETTE	2.4	2.7	3.4	3.9	4.5	4.9	5.2
FRANKLIN	2.3	2.6	3.3	3.8	4.3	4.7	5.0
FULTON	2.2	2.6	3.2	3.7	4.2	4.6	5.0
GALLIA	2.4	2.6	3.4	3.8	4.4	4.9	5.1
GEAUGA	2.1	2.3	3.0	3.5	3.9	4.4	4.6
GREENE	2.4	2.8	3.5	4.0	4.6	5.0	5.4
GUERNSEY	2.2	2.4	3.2	3.7	4.2	4.6	4.0
HAMILTON	2.5	3.0	3.6	4.1	4.8	5.2	5.7
HANCOCK	2.3	2.6	3.3	3.7	4.2	4.7	5.0
HARDIN	2.3	2.7	3.3	3.8	4.3	4.8	5.1
HARRISON	2.2	2.4	3.0	3.7	4.2	4.7	4.9
HENRY	2.3	2.6	3.3	3.7	4.2	4.7	5.0
HIGHLAND	2.5	2.8	3.5	4.0	4.6	5.0	5.4
HOCKING	2.3	2.5	3.3	3.8	4.3	4.8	4.9
HOLMES	2.2	2.4	3.1	3.5	4.0	4.5	4.7
HURON	2.2	2.4	3.1	3.5	4.0	4.5	4.7
JACKSON	2.4	2.6	3.4	3.8	4.4	4.9	5.1
JEFFERSON	2.2	2.5	3.2	3.7	4.2	4.7	4.9
KNOX	2.2	2.4	3.2	3.6	4.1	4.6	4.8
LAKE	2.1	2.2	3.0	3.4	3.9	4.4	4.6
LAWRENCE	2.4	2.7	3.4	3.9	4.5	4.9	5.2
LICKING	2.3	2.5	3.2	3.7	4.2	4.6	4.8
LOGAN	2.3	2.7	3.4	3.8	4.4	4.8	5.2
LORAIN	2.1	2.3	3.0	3.4	3.9	4.4	4.6
LUCAS	2.2	2.5	3.2	3.6	4.1	4.6	4.9
MADISON	2.4	2.7	3.4	3.9	4.4	4.8	5.1
MAHONING	2.2	2.4	3.1	3.6	4.1	4.6	4.8
MARION	2.3	2.6	3.2	3.7	4.4	4.7	4.9
MEDINA	2.1	2.3	3.0	3.5	3.9	4.4	4.6
MEIGS	2.3	2.6	3.3	3.8	4.4	4.8	5.0
MERCER	2.4	2.8	3.4	3.9	4.5	5.0	5.4
MIAMI	2.4	2.8	3.5	4.0	4.5	5.0	5.4
MONROE	2.3	2.5	3.3	3.8	4.3	4.8	5.0

COUNTY/FLOOD FREQ.	1	2	5	10	25	50	100
MORROW	2.3	2.5	3.2	3.6	4.1	4.6	4.8
MUSKINGUM	2.2	2.4	3.2	3.6	4.2	4.6	4.8
NOBLE	2.3	2.5	3.2	3.7	4.2	4.7	4.9
OTTAWA	2.2	2.5	3.1	3.6	4.0	4.5	4.8
PAULDING	2.3	2.7	3.4	3.8	4.4	4.8	5.2
PERRY	2.3	2.5	3.2	3.7	4.2	4.7	4.9
PICKAWAY	2.4	2.6	3.4	3.8	4.4	4.8	5.0
PIKE	2.4	2.7	3.4	3.9	4.5	4.9	5.2
PORTAGE	2.1	2.3	3.1	3.5	4.0	4.5	4.7
PREBLE	2.5	2.9	3.6	4.1	4.6	5.1	5.6
PUTNAM	2.3	2.7	3.3	3.8	4.3	4.8	5.1
RICHLAND	2.2	2.4	3.1	3.6	4.0	4.5	4.7
ROSS	2.4	2.7	3.4	3.9	4.4	4.9	5.1
SANDUSKY	2.2	2.5	3.1	3.6	4.1	4.5	4.8
SCIOTO	2.4	2.7	3.5	3.9	4.5	5.0	5.3
SENECA	2.2	2.5	3.2	3.6	4.1	4.6	4.8
SHELBY	2.4	2.8	3.4	3.9	4.5	4.9	5.3
STARK	2.2	2.3	3.2	3.6	4.0	4.5	4.7
SUMMIT	2.1	2.3	3.0	3.5	3.9	4.4	4.6
TRUMBULL	2.1	2.4	3.1	3.6	4.1	4.6	4.7
TUSCARAWAS	2.2	2.4	3.1	3.6	4.1	4.6	4.8
UNION	2.3	2.6	3.3	3.8	4.3	4.8	5.0
VAN WERT	2.3	2.7	3.4	3.9	4.4	4.9	5.3
VINTON	2.3	2.6	3.3	3.8	4.4	4.8	5.0
WARREN	2.5	2.9	3.5	4.0	4.7	5.1	5.5
WASHINGTON	2.3	2.5	3.3	3.8	4.3	4.7	5.0
WAYNE	2.2	2.3	3.2	3.5	4.0	4.5	4.7
WILLIAMS	2.3	2.6	3.3	3.8	4.3	4.7	5.1
WOOD	2.2	2.6	3.2	3.7	4.2	4.6	4.9
WYANDOT	2.3	2.6	3.2	3.7	4.2	4.7	4.9

APPENDIX D Forms

Bridge No.: __________ Name: Goshen Road

County: Mahoning Stream: Little Beaver Creek

Location: Goshen Road South of Route 165

BRIDGE SCOUR ANALYSIS - Sources of Information

Bridge Office:

- _x_ Bridge Plan
- _X_ Bridge Inspection and Inventory Record

Hydraulics Office:

- ____ Correspondence File
- _X_ Quadrangle Map: __________
- ____ County Map (Nearby bridges with hydraulic data?)
- ____ Hydraulic Inventory (Computer)
- ____ DNR Watershed Maps
- ____ Flood Insurance Studies
- ____ Corps of Engineers Profiles
- ____ U.S.G.S. gage records
- ____ Photograph File
- ____ Bridge Survey File
- ____ Pile Report
- ____ Screening Worksheet

Other Offices:

- ____ Bridge Construction Folder (Record Center/Regional Eng)
- ____ Road Plans
- ____ Aerial Photos (Surveying and Mapping)
- ____ Boring logs (Foundations or Record Center Folder)

District:

- _X_ Bridge Engineer/Inspector
- ____ Highwater records
- _x_ Field check

Figure D - 1 Bridge Scour Analysis - Sources of Information

Date ____________
Name ____________
Hours: Off ____ Field ____

BRIDGE SCOUR SCREENING DATA

CROSSING DATA

Bridge # ________ C.S. _______ Route ________ County ____________

Stream ________________________ Location ______________ M.P. __________
S, T, R

ADT _________ Descriptive Location ______________________________

Quad Map _____________________ Main Channel? _____ Relief Bridge R or L

Bridge Plan? ______ Bridge Survey? _____ Date ____________

STRUCTURE DATA

Yr Built ____ Type __________________ Size __________ Low Member _______

Abutment Type ________________________ Projection into Channel (a) _______

Piers: Number ____ Length (L) _______ Width (a) _______ L/a ________

Type _________________ K1 _____ Angle of Attack _______ K2 _____

Footing: Spread _________ Length _____ Width _____ Top Elev ____________
Bottom Elev _________

Pile Cap________ Length _____ Width _____ Top Elev ____________
Bottom Elev _________

HYDRAULIC DATA

Drainage Area (sq mi) _______ Stream Slope _____ Flood of Record _______

Max Obs Highwater _______ Approx Flowline Elev _______ Road Sag Elev ______

Date Computed ____________

	Q	QOT or Q500	Q100
Flood Frequency	______	______	______
Discharge	______	______	______
Headwater Elev	______	______	______
Total Stage Increase	______	______	______
Min Waterway Below Elev. _______	______	______	______
Mean Velocity thry Structure Q/A	______	______	______
Main Channel Depth, y1	______	______	______
Main Channel Velocity, V1	______	______	______
$Fr1 = V1/\sqrt{gy1}$	______	______	______
Overbank Depth, L or R, y0	______	______	______
Overbank Velocity, L or R, V0	______	______	______
$Fr0 = V0/\sqrt{gy0}$	______	______	______

Figure D - 2 Bridge Scour Screening Data

STREAM CHARACTERISTICS

Bed Material Size	Main Channel	Left Overbank	Right Overbank
D50	_______	_______	_______
D16	_______	_______	_______
D84	_______	_______	_______
Boring Logs Available	_______	_______	_______

Remarks:

Geomorphology:
Straight _____; Meandering _____; Braided _____; Alluvial Fan _____

Aggradation _____; Degradation _____;

Flow Conditions:
Flashy _____; Perennial _____; Low Flow Discharge _____;

Remarks:

Nearest Tributaries - Location from Bridge Site:
Upstream _________; Downstream _________ (miles or feet)

Size of Tributary:
Upstream _________; Downstream _________ (cfs or % of flow)

Remarks on Potential Affect:

Distance to confluence with next stream:_______ miles

Remarks:

Location of bridge with respect to stream planform:
On Bend _____; Upstream of Bend _____; Downstream of Bend _____;

Island _____; Sketch Available _____

Bank Conditions	Left	Right
Stable	_______	_______
Erodible	_______	_______
Vegetated	_______	_______

Remarks:

Figure D - 3 Bridge Scour Screening - Coding Worksheet

BRIDGE SCOUR SCREENING - CODING WORKSHEET

Bridge #: ____________ Name: ______________________ Date: ___________

This worksheet is an aid to complete scour screening. Prior to starting the worksheet, you need information on foundations, historical scour problems, and existing scour protection. Circle Yes or No for each question and follow the directions.

1. Are there any existing or historical scour problems? o Scour at any pier. Movement, scour, or erosion at either abutment. o Channel lowering or lateral movement.	YES: Scour Susceptible, Code _J_ Worksheet Complete. NO: Go to Question 2.
2. Are any of the bridge foundations unknown?	YES: Unknown Foundations, Code _G_, Worksheet Complete. NO: Go to Question 3.
3. Do both abutments meet any of the following criteria? o Piling depth greater than 40 ft. o Adequate scour protection: Riprap (class III or larger), grouted riprap, or gabions, in good condition. o Spread on erosion resistant bedrock: Granite, basalt, gabbro, quartzite, or gneiss (not highly broken or fractured).	YES: Go to Question 4. NO: Scour Susceptible, Code _J_ Worksheet Complete.
4. Do all piers meet any one of the following criteria: o Piling depth greater than 40 ft. o Spread on erosion resistant bedrock: Granite, basalt, gabbro, quartzite, or gneiss (not highly broken or fractured). o No Piers.	YES: Go to Question 5. NO: Scour Susceptible, Code _J_ Worksheet Complete.
5. Are Questions 1 and 2: NO <u>and</u> Questions 3 and 4: YES	YES: Low Risk, Code _I_. NO: Scour code should already be assigned.

SECONDARY SCREENING OF MINNESOTA BRIDGES

Date: ______________________________

Signature of Professional Engineer performing Screening: ____________________
Registration Number: ____________________

Bridge Location:
Bridge Number: ____________________
County: ____________________
Township: ____________________
Roadway: ____________________
Stream: ____________________

Complete the following questionnaire consisting of 7 sections, in consecutive order, and place an **X** by the appropriate **scour screening rating code** listed below. Responses to questions in the various sections may result in rating the bridge without completing the questionnaire in total.

_____ Low risk for failure due to scour, Scour Code = I

_____ Scour susceptible, analysis required, Scour Code = J

_____ Limited risk to public, monitor in lieu of evaluation, Scour Code = K

_____ Scour safe, but action required, Scour Code = O

_____ Scour Critical, Monitoring required, Scour Code = R

1. HISTORICAL SCOUR PERFORMANCE:

   a. What is the Primary Screening Code: _______

   b. Has the bridge ever experienced scour which caused foundation undermining that has not been adequately corrected? __________

   If the answer to (b) is "yes", go to 7. If "no" or "unknown", go to 2.

2. SCOUR RESISTANT FOUNDATIONS:

   Answer the following questions for each substructure unit. Place the answer in the table on the next page.

   a. Are the foundations embedded in scour resistant rock such as basalt, gabbro, granite, gneiss, or quartzite, if not highly weathered, broken or fractured, based upon record drawings or construction records? Rock type is ________.

Figure D - 4 Example of Secondary Screening Worksheet

(b), (c), and (d) are only for bridges with drainage areas less than 400 mi^2:

b. For the foundations with piling, are the piling embedded in stiff clay (a clay with a shear strength greater than 2000 psf)?

c. Abutments only: are there adequately designed and functioning scour countermeasures in good stable condition protecting the abutments? (typical scour countermeasures include riprap, gabions, concrete paving)

d. Piers only: Is the average bottom of the pile tips more than 40 feet below the lowest river bottom elevation at the bridge site?

	Left Abutment	Pier No.___	Pier No.___	Pier No.___	Pier No.___	Right Abutment
a.						
b.						
c.		N. A.	N. A.	N. A.	N. A.	
d.	N. A.					N. A.

If there is at least one "yes" in each column in the above table, rate the bridge as "I" and proceed no further. If "no" or "unknown", go to 3.

3. DEBRIS AND BLOCKAGE:

a. Does debris collect or build up at the bridge and block at least 10% of the flow cross section? ________

b. Does ice in the form of jams or frazil collect or build up at the bridge and block at least 10% of the flow cross section? ________

If the answer to **either** of the above 2 questions is "yes" or "unknown", go to 7. If the answer to both questions is "no", go to 4.

4. GEOMORPHIC CONDITIONS AFFECTING SCOUR RESISTANCE:

a. Is the stream bed degrading? ________

b. For natural streams, are there channel bends of greater than 30 degrees within a distance of 4 times the channel width upstream of the bridge? ________

c. Are the stream banks unstable? ________

d. Are the bridge abutments or piers skewed to the direction of flow? ______

e. Is the effective flow width (width of flow during the 100 year flood) greater than 5 times the total bridge span or 5 times the bank full channel width? __________

If the answer to **any** of the above 5 questions is "yes" or "unknown", go to 7. If the answer to all the above questions is no, go to 5.

5. HYDRAULIC CONDITIONS AFFECTING SCOUR RESISTANCE:

Based upon known topographic information and water surface profile calculations or historical records or professional judgement, answer the following questions:

a. Is flood depth less than 3 feet and stream slope, within a mile of the bridge, less than 5 feet per mile? _______

b. Is flood depth less than 10 feet and stream slope, within a mile of the bridge, less than 1 foot per mile? __________

c. Is flood depth less than 20 feet and stream slope, within a mile of the bridge, less than 0.5 feet per mile? _________

d. For floods of magnitude greater than 50 years, is the average velocity through the bridge less than 3 fps in sand bed water courses or less than 5 fps in clay bed water courses? ________

If the answer to any of the above 4 questions is "yes", rate the bridge as "I" and proceed no further. If the answer to all of the above questions is "no" or "unknown", go to 6.

6. STRUCTURAL CONDITIONS AFFECTING SCOUR RESISTANCE:

If the bridge is multiple span, go to 7. If the bridge is a single span and the effective flood plain width is less than 5 times the span length, answer the following 3 questions. Otherwise, go to 7.

a. Is the bridge supported by concrete abutments on piles? __________

b. Is the bridge supported by timber abutments less than 6 feet high on piles?________

c. Is the bridge a single span with concrete abutments over a man made ditch with slope of less than 5 feet per mile or average ditch velocity less than 3 fps for a flood of magnitude 50 years or greater? __________

If the answer to any of the above 3 questions is "yes", rate the bridge as "I" and proceed no further. If the answer to **all** 3 questions is "no" or "unknown", go to 7.

7. MONITORED REDUCED RISK BRIDGES:

a. Is the bridge scheduled for replacement or installation of constructed scour countermeasures within 5 years? ________

b. Is the road classified as a Local Road or is the estimated average daily traffic (ADT) over the bridge less than 25? ________

c. Does the bridge or adjacent roadway overtop more often than on average every 5 years, requiring closure and therefore inspection before reopening? ________

d. Is the bridge supported by spread footings on rock and the can the rock condition be adequately examined during a routine inspection? ________

If the answer to either a, b, or c is "yes", and the local professional engineer having jurisdiction over the bridge inspection directs a monitoring program for the bridge, rate the bridge as "K". If the answer to d is yes, rate the bridge as "O", scour safe but action required in accordance with the instructions. If the answer to all 4 questions is "no" or "unknown", rate the bridge as "J" and perform a level 1 or level 2 scour evaluation or rate the bridge as "R" and monitor.

STATE OF OHIO DEPARTMENT OF TRANSPORTATION

(262)

BRIDGE INSPECTION REPORT

BR-86 REV 02-75

STRUCTURE FILE NUMBER	BRIDGE NUMBER (CO / ROUTE / UNIT)	YEAR BUILT
5 0 4 4 2 1 5	GOS / 61 / 93	9

DIST 04 BRIDGE TYPE CONCR/SLAB/CONT TYPE SERVICE 1 15 HOLE BROOK LITTLE BEAVER MAN

Item		Rating	Item		Rating
DECK					
1. FLOOR	1-CONC	1	2. WEARING SURFACE	1-CONC	2
3. CURBS, SIDEWALKS & WALKWAYS	0=NONE / 0=NONE		4. MEDIAN		
5. RAILING	4-STL	2	6. DRAINAGE	1-DVR SIDE W/O DRIP	1
7. EXPANSION JOINTS	2-COMP	1	8. SUMMARY		7
SUPERSTRUCTURE					
9. ALIGNMENT	MAX. SPAN=0	1	10. BEAMS/GIRDERS/SLAB	C-SLAB	1
11. DIAPHRAGMS or CROSSFRAMES	DIT. L THICK		12. JOISTS/STRINGERS		
13. FLOOR BEAMS			14. FLOOR BEAM CONNECTIONS		
15. VERTICALS			16. DIAGONALS		
17. END POSTS			18. TOP CHORD		
19. LOWER CHORD			20. LOWER LATERAL BRACING		
21. TOP LATERAL BRACING			22. SWAY BRACING		
23. PORTALS			24. BEARING DEVICES	0-OTHER	
25. ARCH			26. ARCH COLUMNS or HANGERS		
27. SPANDREL WALLS			28. PAINT	TYPE: 0 YEAR=	
29. PINS/HANGERS/HINGES			30. FATIGUE PRONE CONNECTIONS		
31. LIVE LOAD RESPONSE		5	32. SUMMARY		7
SUBSTRUCTURE FOOTER UNDERMINED					
33. ABUTMENTS	2-CONC	4	34. ABUTMENT SEATS		
35. PIERS	2-CONC	2	36. PIER SEATS		
37. BACKWALLS			38. WINGWALLS		1
39. FENDERS and DOLPHINS	SPANS=3		40. SCOUR (SOUTH WALL UNDERMINED)	4-SCOUR POSS.	4
41. SLOPE PROTECTION (PIERS:UNKNOWN FOUND)	PIERS=2		42. SUMMARY (ABUTMENT:UNKNOWN FOUND)		4
CULVERTS					
43. GENERAL			44. ALIGNMENT		
45. SHAPE			46. SEAMS		
47. HEADWALLS or ENDWALLS			48. SCOUR		
49.			50. SUMMARY		
CHANNEL					
51. ALIGNMENT		3	52. PROTECTION	0-OTHER	2
53. WATERWAY ADEQUACY		2	54. SUMMARY		4
APPROACHES					
55. PAVEMENT	2-ASPH	2	56. APPROACH SLABS		2
57. GUARDRAIL (POSTS ROTTING)		3	58. RELIEF JOINTS		
59. EMBANKMENT (ROLLED BENEATH APPROACH SLABS ON SOUTH END) BRDG.WIDTH=36		3	60. SUMMARY PCT.LEGAL=1?5		5
GENERAL					
61. NAVIGATION LIGHTS			62. WARNING SIGNS	MAINT. RESP: 3-COUNTY	
63. SIGN SUPPORTS (HVC ON=9999 UND=9999)			64. UTILITIES		
65. VERTICAL CLEARANCE			66. GENERAL APPRAISAL & OPERATIONAL STATUS	COND 4	STAT A

67. INSPECTED BY: [signature] — SIGNED — 76 PE — 78 INITIALS: KP

68. REVIEWED BY: Marilyn [illegible] — SIGNED — 81 PE: PE — 83 INITIALS: MSK

Figure D - 5 Goshen Rd. Bridge Inspection Report

Go[illegible] Road — CONCRETE AND STONE MASONRY BRIDGES — Sheet 1 of 1 Sheet — Date 16 March 1970

Type of bridge: Continuous Concrete Slab — No. of spans: 3 — Date built: 1958

Approx. safe load capacity: 24,000# Axle Load

Clear span	Length out to out of floor	Width between curbs	Width out to out	Width of sidewalks	Height of floor above bridge seat	Roadway clearance height	Width	Min. height	Provision for expansion
24'-30'-24'	79'-6"	36'-0" No Curbs	37'-6"	None	1'-1¼"	-	36'-0"	-	None

Type of railing: Steel H Posts Angle Top Rail, Deep Beam Guard Rail, 3'-0" High — Type and size of curb: No Curbs

Type and thickness of wearing surface: Concrete, Monolithic ¾" Thick

Description of floor drainage: Drains off Edge of Slab

Alignment and skew of structure: Straight 20° Skew RF — Approach slab: Reinf. Conc. 24'-0" Wide — Length 15'-0"

Lighting system: None

STREAM: Middle Fork Little Beaver Creek — Height from grade to stream bed: 11.7'± — Height from grade to high water: 3.2'±

Channel characteristics: Approx. width between banks: 20'± to 80'± — Channel depth: 5.0'± to 8.0'± — Nature of bottom: Clay Stone

Condition of banks: Satisfactory, Vegetation — Alignment of stream above and below structure: Irregular

Skew of normal flow: Parallel with Piers — Skew of flood flow: Parallel with Piers

CONCRETE SLAB

STD. DRAWING NO.

Thickness of slab: 13¼", 1" ♯ Bars at 6¾" at Center Spans 1, 2 and 3 at Bottom
1⅛" ♯ Bars at 6¾" at Piers at Top

CONCRETE BEAM

STD. DRAWING NO.

No. of beams	Spacing of beams	Width of beams	Height of beam including slab	Thickness of slab

CONCRETE GIRDER

STD. DRAWING NO.

Width of girder	Depth of girder at center	Depth of girder at ends	Floor beams size	Floor beams spacing	Thickness of floor slab

ARCHES

STD. DRAWING NO.

Material — Type — Shape

Type of rail or parapet

Clear span	Rise	Crown thickness	Spring thickness	Filling material	Depth of fill at crown	Height of parapet above grade

SUB-STRUCTURE

STD. DRAWING NO.

Abutments and piers	Material	Type	Height footing to bridge seat	Width of bridge seat	Length of bridge seat	Foundations (piling)	Wings (lengths, angles, etc.)
Rear	Concrete	Spill Thru	2'-2⅛"±	1'-3"	37'-3"	7-12BP53 Piles	3'-3"± Rt., 3'-1"± Lt.
Forward	Concrete	Spill Thru	2'-2⅛"±	1'-3"	37'-3"	7-12BP53 Piles	3'-1"± Rt., 3'-3"± Lt.
Pier #1	Concrete & Steel	Steel Pile & Cap	2'-2⅛"±	2'-6"	37'-3"	6-12BP53 Piles	
Pier #2	Concrete & Steel	Steel Pile & Cap	2'-2⅛"±	2'-6"	37'-3"	6-12BP53 Piles	

Bridge No.	County	Route No.	C.H. No.	Section	Strength	Roadway	Clearance	Type
3	Mahoning	-	61	Goshen A				

Figure D - 6 Goshen Rd. Bridge Inventory Record

BRIDGE INVENTORY INFORMATION
STRUCTURE FILE NUMBER 5044235 | INVENTORY BRIDGE NO. GOS 61 0093 | BI-191 PAGE 418 | 04/19/91
SUFFICIENCY RATING 78.4 | OVER: MOLE FORK LITTLE BEAVER | BR TYPE CONCRETE SLAB CONTIN.
DATE OF LAST INVENTORY UPDATE 02/09/87

(1) DISTRICT: 04 COUNTY: MAHONING (94) LOCATION: (95) FACILITY CARRIED
(2) FIPS CODE (96) ROUTE ON BRIDGE: COUNTY (97) ROUTE UNDER BRIDGE: NON-HIGHWAY
(9) DIRECTION OF TRAFFIC: 2-WAY TRAFFIC (10) TEMPORARY: NO (11) TRUCK NETWORK NO (12) PARALLEL: N
(88) INSP. COUNTY AGENCY (89) MAINT. COUNTY AGENCY (93) TYPE SERV. (ON): HIGHWAY (UNDER) WATERWAY

****** INVENTORY ROUTE DATA ******
(3) ROUTE ON/UNDER OVER: HIGHWAY SYSTEM: COUNTY HWY
ROUTE NO.: 61 DIR: DES: MAINLINE PREF.
(4) FEATURE INTERSECTED: MOLE FORK LITTLE BEAVER
(5) COUNTY: GOS MILEAGE: 0093 SPECIAL DESIG:
(6) AVG. DAILY TRAFFIC (ADT): 700 (7) ADT YEAR 1976
(8) TRUCK TRAF. 0 (14) FEDERAL-AID: NON-FEDERAL AID
(15) FUNCTIONAL CLASS: MINOR COLLECTOR-RURAL
(16) TOTAL MIN. HORIZONTAL CLEARANCE:
NON-CARD.: 0.0 FT CARD.: 36.0 FT
(17) PRACT. MAX VERT. CL: 99 FT. 99 IN. (18) DEFENSE: NO

****** INTERSECTED ROUTE DATA ******
(21) ROUTE ON/UNDER HIGHWAY SYSTEM
ROUTE NO.: DIR: DES: PREF
(22) FEATURE INTERSECTED:
(23) COUNTY: MILEAGE: SPECIAL DESIG:
(24) AVG. DAILY TRAFFIC (ADT): (21) ADT YEAR:
(26) TRUCK TRAF: (27) FEDERAL AID:
(28) FUNCTIONAL CLASS:
(29) TOTAL MIN. HORIZONTAL CLEARANCE:
NON-CARD.: FT CARD.: FT
(30) PRACT. MAX VERT. CL: FT. IN. (31) DEFENSE:

***** STRUCTURE INFORMATION *****
(33) BYPASS LENGTH 02 MILES
(34) LATITUDE: DEG MIN LONGITUDE: DEG MIN
(35) TOLL: ON FREE ROAD
(36) YEAR BUILT 1958 (37) MAJOR REHABILITATION 0000
(38) NO. LANES (ON) 02 NO. LANES (UNDER). 00
(39) HORIZ. CURVE DEG MIN (40) SKEW: 20 DEG.
(43) APP. RDWY. WIDTH. 36 FT
(44) BR. RDWY WIDTH 36.0 FT (45) DECK WIDTH: 36.0 FT
(46) MEDIAN TYPE NONE / NO BARRIER / NO JOINT
(47) BRIDGE MEDIAN NO MEDIAN
(48) SIDEWALKS: (LEFT) 0.0 FT (RIGHT) 0.0 FT
(49) TYPE CURB OR SIDEWALKS
(LEFT) MATL NONE TYPE: NONE
(RIGHT) MATL NONE TYPE NONE
(50) FLARED. NO (51) COMPOSITE:
(52) RAILING. STEEL POST & PANEL
(53) DECK DRAINAGE OVER THE SIDE (W/O DRIP STRIP)
(54) DECK TYPE. REINFORCED CONCRETE
(55) DECK PROTECTION EXTERNAL: NOT APPLICABLE
INTERNAL: NOT APPLICABLE
(56) WEARING SURFACE: CONCRETE (MONO) THICKNESS 01
(112A) DATE OF WEARING SURFACE:

(59) MAIN SPANS NUMBER: 3 TYPE: CONCRETE / SLAB / CONTIN
APP. SPANS NUMBER. 0 TYPE / /
(58) TOT SPANS. 3 (59) MAX SPAN: 30 FT (60) OVERALL LENG 82 FT

***** (63) SUBSTRUCTURE, (64) FOUNDATION AND SCOUR INFORMATION *****
ABUT-REAR MAT: CONCRETE TYPE: STUB-CAP PILE FND:
ABUT-FWD MAT: CONCRETE TYPE: STUB-CAP PILE FND:
PIER-PRED MAT: CONCRETE TYPE: CAPPED PILE FND:
PIER-OTHER MAT: TYPE: FND:
PIER-OTHER MAT: TYPE: FND:
NUMBER OF PIERS PREDOMINATE 02 OTHER: 00 OTHER 00
(64A) STREAM VELOCITY (65) SCOUR 5 SCOUR WITHIN LIMITS
(65A) UNDERWATER INSP: PROBE-ANNUAL (65B) CHANNEL PROT OTHER
(122) DATE OF LAST DIVING INSP. (140) DRAINAGE AREA. SQ MI

***** CLEARANCE INFORMATION *****
(67) MIN VERT. UNDERCLEAR: NON-CARD 00 FT 00 IN CARD 00 FT 00 IN
(68) MIN LAT. UNDER (FT): NON-CARD 99.9 / 99.9 CARD 99.9 / 99.9
(69) MIN VERT CLEAR ON BR: 99 FT 99 IN (71) VERT CLEAR-LIFT FT
(70) MIN LATERAL CLEAR: NON-CARD CARD

**** LOAD RATING INFORMATION ****
(42) DESIGN LOADING: HS-15
(73) OPERATING. 41 TON
INVENTORY 27 TON
OHIO PCT OF LEGAL LOAD 105
YEAR OF RATING 1973
(74) ANALYSIS: ENGINEER JUDGEMENT
(76) HS RATING: TON
ANALYSIS ON BARS: NO

***** (77) APPRAISAL *****
(INCLUDING CALCULATED ITEMS)
DECK GEOMETRY 8
UNDERCLEARANCE N
WATERWAY ADEQUACY 8
APPROACH ALIGNMENT 8
CALC GEN APPRAISAL 5
CALC DECK GEOMETRY 7
CALC. UNDERCLEARANCES N

***** APPROACH INFORMATION *****
(102) APPROACH GUARDRAIL STEEL BEAM
(103) APPROACH PAVEMENT BITUMINOUS (104) GRADE GOOD

***** CULVERT INFORMATION *****
(106) CULVERT TYPE NOT APPLICABLE (107) LENGTH FT
(108) DEPTH OF FILL FT (109) HEADWALLS

***** GENERAL INFORMATION *****
(114) MAIN MEMBER. SLAB (115) MOMENT PLATE
(116) EXPANSION JOINT: COMPRESSION SEAL
(117) BEARING DEVICES: OTHER /
(119) NAVIGATION CONTROL-NO VERT CLEAR 000 FT HORIZ CLEAR 0000 FT
(123) CRITICAL FEATURE INSP N MONTHS (125) HINGES NOT APPLICABLE
(126) FRACTURE CRITICAL INSP.
(128) LONG MEMBER NOT APPLICABLE (129) FRAMING. NOT APPLICABLE
(131) STRUCTURAL STEEL TYPE RAILING.
PAY WT: PRIME COAT PROTECT

Figure D - 7 Goshen Rd. Bridge Inventory Information

BRIDGE INVENTORY INFORMATION 04/19/91

STRUCTURE FILE NUMBER: 5044235 INVENTORY BRIDGE NO.: GOS 61 0093 BR TYPE: CONCRETE SLAB CONTIN

SUFFICIENCY RATING: 78.4 OVER: MOLE FORK LITTLE BEAVER

***** GENERAL INFORMATION (CONT.) *****

(61) HISTORIC SIGNIF: NOT HISTORIC (62) NBIS: YES

(91) SPECIAL FEATURES: /

(98-99) BORDER BRIDGE: STATE- RESP- SFN:

**** PROPOSED IMPROVEMENTS ****

(78) TYPE WORK: OTHER STRUCTURAL WORK BY AGENCY FORCES

(79) LENGTH: 200

(80) BRIDGE COST ($1000S): $12

(81) ROADWAY COST ($1000S): $1

(82) TOTAL PROJECT COST ($1000S): $20 (83) YEAR: 1986

(84) FUTURE ADT (ON BRIDGE): (85) YEAR OF FUTURE ADT:

**** PROGRAMMING INFO ****

PID NUMBER:

PID STATUS:

PID DATE:

**** INSPECTION SUMMARY ****

(I-8) DECK: 7

(I-32) SUPERSTRUCTURE: 8

(I-42) SUBSTRUCTURE: 5

(I-50) CULVERT:

(I-54) CHANNEL: 5

(I-60) APPROACHES: 5

(I-64) GENERAL APPRAISAL: 5

(I-64) OPERATIONAL STATUS: A

INSPECTION DATE: 07/17/90

(87) DESIG. INSP. FREQ: 12 MONTHS

**** (I-67) SURVEY ITEMS ****

RAILINGS: UNACCEPTABLE

TRANSITIONS: UNACCEPTABLE

GUARDRAIL: UNACCEPTABLE

RAIL ENDS: UNACCEPTABLE

PAVEMENT MARK: ACCEPTABLE

RESTRICT SIGN: NOT APPLICABLE

WARNING SIGN: NOT APPLICABLE

END MARKERS: NOT APPLICABLE

INSP. UPDATE DATE: 02/25/91

***** ORIGINAL PLANS INFORMATION *****

(133) CONTRACTOR:

(134) OHIO ORIGINAL CONSTRUCTION PROJECT NO.

(135) MICROFILM REEL:

(136) ORIG. CONSTRUCT. FED AID PROJ. NO:

(138) STANDARD DRAWING:

APERTURE CARDS: NO

***** REPAIR PROJECTS *****

(141)	(142)	(143)
(144)	(145)	(146)
(147)	(148)	(149)
(150)		

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