HYDROLOGIC AND HYDRAULIC REPORT
FOR
SR 3001, SECTION 01B
OVER
SWABIA CREEK

BOROUGH OF ALBURTIS AND
LOWER MACUNGIE TOWNSHIP
LEHIGH COUNTY

Prepared for:

Pennsylvania Department of Transportation
Engineering District 5-0
Allentown, PA

CCJM

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HYDROLOGIC AND HYDRAULIC REPORT

SR 3001, SECTION 01B
SEGMENT 0020, OFFSET 0092

OVER
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LEHIGH COUNTY

"I, Dr. Charles Arnold, P.E. hereby certify, pursuant to the penalties of 18 Pa.C.S.A., Section 4904, that to the best of my knowledge, information and belief, the information contained in the accompanying plans, specifications and reports has been prepared in accordance with accepted engineering practice, is true and correct, and is in conformance with Chapter 105 of the rules and regulations of the Department of Environmental Protection."

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
CHARLES DAVID ARNOLD
ENGINEER
No. 057303-E

Signature

April 17, 2008
Date

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I. INTRODUCTION AND PROJECT DESCRIPTION

The intent of this report is to provide information to the Department in support of an application for a Water Obstruction Permit. The existing structure will be replaced with a slightly larger structure that provides a larger hydraulic opening. Specific dimensions are detailed below. The existing structure will be replaced at the same approximate location. The alignment for the proposed roadway and structure will remain the same as the existing, with no skew, and the channel work will be minimized. It is the intent of the hydraulic study to provide information demonstrating that the proposed minimum clear span will not cause intolerable backwater conditions or excessive mean velocities that result in scour.

Hydrologic and hydraulic analyses were performed for the existing and proposed structures. The hydraulic analysis was performed using the U.S. Army Corps of Engineer's, HEC-RAS (V3.1.3) computer software.

An Individual Section 404 permit will not be required for the proposed construction activities associated with this project. The Pennsylvania State Programmatic General Permit No. 3 (PASPGP-3) applies to the proposed activities.

1. Project Description

The project involves the replacement of the existing bridge carrying SR 3001 over Swabia Creek. The bridge is located south of the Borough of Alburtis in Lower Macungie Township, Lehigh County (as shown on the location map in Figure 1). The existing structure is a 20.5 ft Steel I-beam bridge having a clear opening of that varies from 16.4 to 17.1 ft with no hydraulic skew. It has a curb-to-curb width of 20.4 ft and an under-clearance ranging between 4.55 ft and 5.05 ft. The existing hydraulic opening is approximately 67 sq. ft. Figure 2 contains a sketch of the existing bridge and Figure 3 shows the proposed structure. SR 3001 is classified as an Urban Collector with an estimated 2001 ADT of 1732 and a predicted 2021 ADT of 2574.

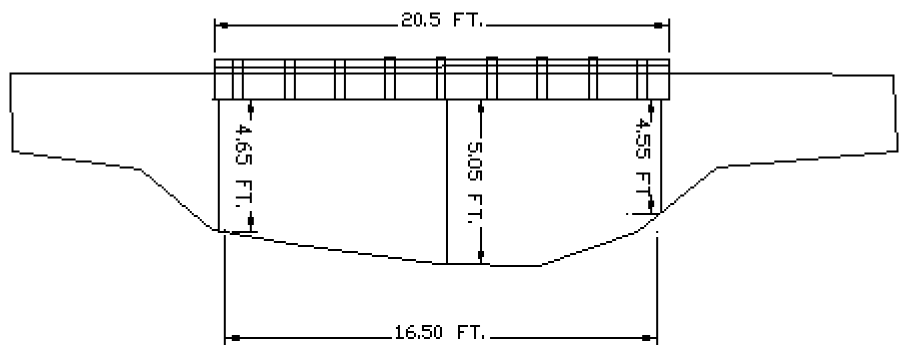
The existing roadway alignment runs from north to south approximately perpendicular to Swabia Creek. The proposed line and grade remain identical to the existing line and grade and were reviewed by PennDOT as part of the Approved Safety Review on June 8, 2005.

Several structure alternatives were examined as part of this study. Based on the following hydraulic analysis, a bridge hydraulic opening of at least 71.5 sq. ft is recommended. The recommended structure alternative was selected from three structure types after consideration of hydraulic performance, environmental impacts, length of construction period and cost effectiveness. The recommended structure is a Pre-Cast Reinforced Concrete Culvert that has a hydraulic opening of 81.8 sq. ft.

Figure 1: Location Map

See “Project Location Map” section of the JPA-ECMS submittal.

Figure 2 : Existing Structure Sketch



UPSTREAM FACE OF EXISTING STRUCTURE

SR 3001, SECTION 01B
LEHIGH COUNTY

FIGURE 2: EXISTING STRUCTURE

(NOT TO SCALE)

Figure 3: Proposed Plan View

See the "Other: Cross Sections" section of the JPA ECMS submittal.

II. SITE DATA

1. Location

The bridge is located South of the Borough of Alburtis on SR 3001 (Franklin Street) over Swabia Creek in Lower Macungie Township, Lehigh County. The exact location of the project is shown in Figure 1. The watershed boundary for the SR 3001 Bridge over Swabia Creek is delineated in Attachment A. Locations of the nearest upstream and downstream structures are shown on Figure 1 and in Attachment A.

2. Existing Structures

The existing single span bridge structure is a steel I-beam bridge having a clear opening that varies from 16.4 to 17.1 ft. and is supported by reinforced concrete structures that are perpendicular to the centerline of S.R. 3001. The streambed runs parallel to the abutments through a vertical opening ranging between 4.55 ft. and 5.05 ft. The S.R. 3001 roadway has been shown to be overtopped by a 25-year flood event.

The bridge width is 19.5 ft. wide from curb to curb and is posted for a weight limit of 15 Tons for a single vehicle and has a speed limit of 25 miles per hour on the roadway. The approach roadways are approximately 18 ft. wide with no shoulders.

There are two existing bridges crossing this branch of Swabia Creek; one upstream and another downstream. The downstream bridge is a twin cell concrete box culvert. Each box is 18 feet wide with an average clear opening 5.9 feet tall. The adjacent box walls form a 2.1 ft wide wall in the middle of the bridge span of 38.1 feet. The 21.8 feet, curb to curb, bridge is located on Church Street approximately 0.68 miles downstream. The upstream bridge is a single span 21.2 feet Steel I-Beam having an approximate clear opening of 15 ft and an under-clearance of 5.85 ft located on Gun Club Road, T468, approximately 0.44 miles upstream. Neither bridge is located close enough to affect the hydraulics within the study area.

Photographs of the project site and upstream and downstream structures are presented on pages 9 through 13. A photo location map is shown on Figure 4. Locations of the upstream and downstream bridges are shown on Figure 1 and on the drainage area map in Attachment A.

3. Flood History

Photos were provided by the family of Charles C. Knerr who live just off the North West Quadrant of the bridge site. One of the floods is shown in the photo which follows. Mr. Knerr commented that “there have been approximately five such floods in the last fifty years of approximately the same degree as seen in the photograph". He also indicated that "floods of lesser degree typically occur once every two to three years" and "whenever the National Weather Service issues a Small Stream Flooding Watch for Lehigh County or Berks County, the property is in jeopardy of flooding". The storm in this particular photo, was the 1972 flood associated with Hurricane Agnes. Notice, in the upper right corner of the photo, that Amity Machine Corporation's building is up and out of the floodplain. Residential homes located on the northern side of the stream are also beyond the floodplain.

A discussion of the Flood History is presented in the Hydrology Section, in light of the anticipated flows.

Photo 1. PHOTO OF BRIDGE OVERTOPPING, PROVIDED BY CHARLES C. KNERR

4. Stream Characteristics

Swabia Creek is a fairly narrow stream with continuous flow ranging from 10 to 16.5 feet in width at the bottom of banks. The streambed has stones of 6-9 inch diameter intermixed with smaller stones and silt. Small pools and sandy areas can be observed at various points at the base of the bank. Trees and shrubs grow to the edge of the stream and fallen branches can be observed over the width of the stream.

Swabia Creek is stocked with trout and is classified as a HQ-CWF, High Quality Cold Water Fishery, according to Chapter 93, Title 25, of the Pennsylvania Code. It is also on the Fish Commission wild trout waters list. Because of the HQ-CWF classification, construction activities which could affect the stream may be prohibited between March 1 and June 15 (for stocked trout streams) and between October 1 and December 31 (for wild trout streams) unless the PA Fish Commission grants a waiver. Although there are no wetlands within the construction area, significant wetlands are present just outside the construction work area. Impacts to these wetlands will be avoided through the use of the appropriate erosion and sediment control measures.

The upstream banks are covered with trees and shrubs out to a distance of 30-50 feet on both sides of the stream. Outside this area is mown grass on the right bank, which ends approximately 300 feet from the stream to the Amity Machine Corporation parking area. The left bank, outside the central tree and shrub covered banks, consists of mowed grass around a pond and residence which turns into a thickly vegetated car salvage lot approximately 290 feet upstream. The salvage lot extends some 250-300 feet to the left of the streambed and then changes to a mowed grass field.

The downstream banks are also covered with trees and shrubs out to a distance of 30-40 feet on both sides of the stream. Outside this central area the right bank consists of a cornfield to the right limits of the cross sections through all of the profile downstream. The left overbank consists of dense grass and a few low shrubs out from the central bank areas for a distance of 160 feet, which then becomes a mowed grass yard below a residence on the far left of the cross sections. This pattern of dense grass to mown grass continues throughout the profile downstream.

5. Watershed Description

The drainage area, delineated in Attachment A, is approximately 2.54 sq. miles. The land within the watershed is approximately 40% forest with a mixture of low density residential, farmland and pasture. The watershed ranges in elevation from 423 ft up to 1120 ft.

Figure 4: Photo Location Map

See the "Photos with Map" section of the JPA ECMS submittal.

The following photos are found in the "Photos with Map section of the JPA ECMS submittal.

- Photo 2. Downstream Face of Existing Structure
- Photo 3. Upstream Face of Existing Structure
- Photo 4. Near Approach of Existing Structure.
- Photo 5. Far Approach of Existing Structure.
- Photo 6. Looking Downstream from Bridge Deck.
- Photo 7. Looking Upstream from Bridge Deck.
- Photo 8. Upstream Face of Upstream Structure on T-468, Gun Club Road.
- Photo 9. Upstream Face of Downstream Structure on Church Street.

III. HYDROLOGIC ANALYSIS

1. Estimation and Comparison of Flow Rates

There are no gaging stations near the project area. A gaging station near East Texas, PA is located upstream from where the Swabia Creek enters the Little Lehigh Creek. Another gaging station is near the western edge of Allentown, located downstream from where Swabia Creek enters the Little Lehigh Creek.

PennDOT's, Design Manual 2, Section 10.6.C.2 indicates that "if a project lies within a Federal Emergency Management Agency (FEMA) study area, the flood discharges reported in the FEMA study should be reviewed and incorporated into the hydrological analysis".

A detailed FEMA study of Swabia Creek was done in 1974 for Lower Macungie Township which included the bridge on SR 3001. A copy of the detailed FEMA is provided in Attachment C, with section 25.90 being the bridge section on SR 3001. Although the detailed FEMA input file does not indicate the return period of the four flows used in its calculations, the results for the flows and their water surface elevation were plotted by FEMA and a legend delineating the flows was given. These flow rates are tabulated in Table 1.

The flow rates given in the FEMA study are based on the stream flow records obtained from the gaging station near the western edge of Allentown. Discharges for the various return periods were determined using a method in Beard's "Statistical Methods of Hydrology"[1]. The August 1976 Flood Insurance Study for the Township of Lower Macungie notes that "flows in each of the Little Lehigh Creek tributaries were determined assuming a direct relationship between discharges and the square root ratios of drainage areas".

[1] Beard, Leo R., "Statistical Methods in Hydrology", U.S. Army Corps of Engineers, Sacramento, California District, January 1962.

The drainage area was calculated to be 2.54 sq. miles (6.57 sq. km.) using the map in Attachment A. The map was constructed from the Allentown West and East Greenville, Pennsylvania U.S.G.S. 7.5 minute quadrangles. The drainage area for the project site constitutes only 3.14 per cent of the drainage area for the Allentown gaging station. This percentage is less than the 50 percent required for Section 3.4.1, Analysis of Stream Gage Records, in Penn DOT Design Manual 2 Section 10.6.C.4.a. The FEMA flows, as determined using a ratio of drainage areas, do not meet current design and modeling practices for sizing highway structures. A regression model was therefore used to model the flows at the bridge.

The USGS (National Flood Frequency, NFF, version 3.0) Program was used to determine the design flood discharges for the SR 3001 structure. This program requires estimates of the forest cover, percent urban land use and carbonate within the watershed area. Using the Allentown West and East Greenville USGS maps, estimates of 40% forest area and 1% urban were determined. A carbonate estimate of 63 percent was taken from the NFF: Water Resources Investigation Report 00-4189[2] for the nearest downstream stream gaging station on the Little Lehigh Creek, near Allentown. The results are summarized in Table 1.

The PSU-IV method, “Procedure for Estimating Design Flood Peaks on Ungaged Watersheds” (Pennsylvania State University, April 1981) was used to determine flow estimates for comparative purposes. The centroid of the drainage area above the structure is approximately located at latitude N 40°29’06”, longitude W 75°36’37”. According to Plate 1 of the PSU-IV method, the drainage area is divided by the line of delineation between Flood Regions 1 and 2. The bridge on SR 3001 lies in Region 2.

The PSU-IV flow rates for both Regions 1 and 2 were determined for illustrative purposes. The divide elevation for Region 1 was 990.6 feet (304.8 m). The forest cover percentage for Region 2 was measured as the green areas on the USGS map and was calculated to be approximately 40%. The standard deviation, S_y, and skew coefficient, G, for both regions were determined from Plates 2 and 3 respectively, as shown in Attachment C. Based on Plates 2 and 3, S_y was determined to be 0.272 and G was determined to be 0.348. No other adjustments were required for urbanization, carbonate rock, or watershed size. The above variables were then input into PSU-IV and flows for various storm events were computed. The results are summarized in Table 1 and the model output is presented in Attachment C.

The Borough of Alburtis has adopted the June 1999 Lehigh Valley Planning Commission ACT 167, Storm Water Management Plan for the Little Lehigh

2 Stuckey, Marla H., and Reed, Lloyd A., “Techniques for Estimating Magnitude and Frequency of Peak Flows for Pennsylvania Streams”, Water-Resources Investigations Report 00-4189, U.S. Department of the Interior, U.S. Geological Survey, Lemoyne, Pennsylvania, 2000.

Creek Watershed which includes the Swabia Creek watershed. The construction proposed in this bridge replacement involves widening the road so that an additional impervious surface of approximately 4,000 square feet is added to the roadway. According to Section IV, Article A of the ACT 167 Plan, the proposed improvements constitute a "New Development" of less than 10,000 square feet. Under the ACT 167 Plan a "New Development" is exempt from the runoff control plan and is expected to have an insignificant impact on the watershed-level runoff characteristics. Flow rates from this plan are given in Table 1 for illustrative purposes only.

SR 3001, Section 01B is classified as an Urban Collector. Penn DOT Design Manual 2 Section 10.6.E indicates that the minimum design storm for an Urban Collector is the 10-year storm event yet permits larger storms for more conservative designs. A 25-year storm will be used as the design storm for the bridge. As can be seen from Table 1, there is a great variation in the predicted 25-year flow values, ranging from between 90 and 200 ft^3/s for the FEMA study to 1544.2 ft^3/s for the ACT 167 study.

The bridge on SR 3001 lies in the PSU-IV Region 2. The USGS NFF Program values, which account for an additional carbonate deposits factor, compare most closely with the PSU-IV Region 2 values, while the ACT 167 values provide greater estimated values than the other methods since it accounts for future build-out of the watershed.

TABLE 1
Summary of Discharges in ft^3/s

Frequency	***2.33***	***5***	***10***	***25***	***50***	***100***	***500***
FEMA, Jan 1974			90		200	280	1100
PSU-IV, Region 1	280	468	637	899	1135	1407	2392
PSU-IV Region 2	139	233	317	447	564	699	1189
USGS NFF Program			214.9	350.6	489.8	670.1	1311.8
ACT 167 1999	379.8		1016.8	1544.2		2780.6	

In the following hydraulic analysis, the FEMA flows will be used to calibrate the HEC-RAS model. This will allow the model to better reflect the physical situation found at the time of the original FEMA study.

The USGS NFF flows will be used in the design of the bridge hydraulic opening and the analysis of the 100-year floodplain.

An analysis of the existing and proposed FEMA 100-year flows was performed to show consistency with the current FEMA floodplain. Results of the FEMA flow analysis are presented in Section VI: Floodplain Management.

Analyses of the ACT 167 100-year flows were done to illustrate consistency with the Lehigh County Stormwater Management Plan. Results of the ACT 167 flows analysis are presented in Section V: Stormwater Management Analysis.

2. Discussion of the Flooding History

The calibrated HEC-RAS model developed in the next section was run to help evaluate the flood levels in Photo 1, presented by Mr. Charles Kneer. Companion photos to the one presented showed several people walking near the low point on the bridge approach road near to the Kneer house. The water appears to be 9-12 inches deep at that point. By running the HEC-RAS Model, it was determined that flow rates in the range of 2000-2400 ft^3/s were needed to produce the depth shown. These flow rates correspond to a flooding frequency greater than a 500-year event.

The overbank areas of Swabia Creek, both upstream and downstream of SR 3001, are wooded right to the bottom edge of bank. A blockage could drastically reduce the flow rate needed to overtop the bridge. Mr. Kneer has reported that “it is very common in times of high water, and especially in times of flood conditions, that large branches and long logs which naturally fall into the creek’s current are swept downstream”. Mr. Kneer also notes that “with the present free span bridge this debris in times of high water simply travels under the bridge”. It would not be unreasonable, therefore, to assume that some level of blockage due to branches being washed downstream could occur 5 times in 50 years.

The HEC-RAS model of the existing conditions predicts both an overtopping of the near approach road in the overbank area on Amity Machine Corporation property at the 25 year storm level and some flow in the overbank areas near to the stream channel on Mr. Kneer’s property. If a level of blockage would occur with a smaller frequency storm, the level of flooding reported by Mr. Kneer would be reasonable.

IV. HYDRAULIC ANALYSIS

The U.S. Army Corps of Engineers HEC-RAS computer software was used to determine water surface profiles for the Swabia Creek. Topographic survey information for the project area was shot and compiled by C.C. Johnson & Malhotra, P.C. A total of 14 cross-sections were cut based on the topographic survey. The locations of the cross sections are presented in Attachments F, G and H. The survey information was checked by the consultant staff during a field reconnaissance. The normal span of the hydraulic opening for the existing structure was measured as 16.5 ft with a low chord elevation 437.73 ft.

Normal depths computed by HEC-RAS using streambed slopes input at the most upstream and downstream cross-sections of the reach under study were used as the boundary conditions for the existing and proposed hydraulic models. All other modeling parameters including the roadway surface elevations over the bridge and its approach roads were based on information from the topographic survey.

Two models (a high flow and low flow model) were developed for the existing and proposed conditions based on the flow over SR 3001. During low volume flow rates, the high right bank downstream limits conveyance in the first few cross sections downstream of the roadway. During high flow rates the creek will overtop the roadway, thereby bypassing the high downstream bank and providing conveyance in the overbank area in the first few cross sections downstream.

Water surface elevations were computed using the energy equation. At one cross-section (RS 946.44) the energy equation resulted in very shallow flows, and consequently very high velocities, for the 25-year storm only. This result was nonsensical and deemed inappropriate. The modeling parameters were changed to utilize both the energy and momentum equations and report the highest. All output elevations remained the same (as the energy equation) and the RS 946.44 output is more consistent with steadily varying flow.

1. Calibration of the HEC-RAS model

The project area is in a detailed FEMA study area, and a copy of the detailed HEC 2 printout is found in Attachment C. This FEMA study was used to calibrate the HEC-RAS model for the bridge. Fourteen cross sections were used in the HEC-RAS model, covering a width of approximately 350 feet to each side of the channel and approximately 450 feet up and down the stream from SR 3001. It was assumed that there has been no significant change in surface roughness due to new construction in the overbank and channel areas since the FEMA study was done in 1974.

The FEMA study gives the water surface elevation at the bridge over SR 3001 for four flow rates. Once the basic geometry of the bridge and stream cross sections was developed the model was run for the four flow rates given in the FEMA study. The Manning's roughness coefficient was inputted based on field observations made on October 3, 2000. The coefficients were then modified within reason so as to calibrate the model to the water surface elevations given in the FEMA study. Table 2 presents a summary of the results of the calibration for the four water surface elevations at the bridge. The October 2000 field notes and complete results of the calibration study are given in Attachment F.

The FEMA study and the HEC-RAS model both show the 100-year storm going through a hydraulic jump as it passes through the bridge. The 10 year storm never left the channel banks and was used to calibrate the surface roughness in the channel itself, while the 500 year storm was used to calibrate the surface roughness in the overbank areas. Table 2 shows that the HEC-RAS model was able to match the FEMA 10 year water surface elevation and was 0.12 feet below the FEMA 500 year storm elevations.

TABLE 2
Calibration to FEMA Elevations

Frequency	***Flow rate***	***FEMA- W.S.El.***[3]	***HEC-RAS W.S.El.***	***DIFFERENCE***
10 year storm	90	435.29	435.30	+ 0.01
50 year storm	200	436.12	435.95	- 0.17
100 year storm	280	436.57	436.50	- 0.07
500 year storm	1100	438.92	438.80	- 0.12

The Manning's roughness coefficient, n, varied across each section, from the heavy brush and tree areas next to the channel to mowed areas on the outer areas of the overbanks. The values used in the calibrated model are as follows.

TABLE 3
Manning's n Values used in the HEC-RAS Model

Area	Manning's n
➢ Trees w/ heavy brush and old cars	**0.12**
➢ Trees with heavy brush	**0.07**
➢ Trees w/ mowed grass/light brush	**0.065**
➢ Heavy Brush, no trees	**.05**
➢ Channel Bottom	**.04**
➢ Mowed Areas in the overbank	**.035**

[3] FEMA water surface elevations given in the Attachment C printout were based on the 1929 USGS benchmarks and were modified to the 1988 USGS benchmarks by subtracting 0.67 feet to match the field survey data used in the HEC-RAS model.

2. Existing Conditions with the Design Flows

The calibrated model was then used as the existing geometric model to be run with various flows indicated in Table 1. These flows included the FEMA 100 year, the USGS NFF flows and the ACT 167 current and proposed 100 year flows. The results for the existing conditions were used in a comparison to the results from the proposed model, with a modified bridge opening, run at the same flow rates.

Discharges were analyzed up to the ACT 167 proposed 100-year storm of 2780.6 ft^3/s. Results for the ACT 167 flows and their impact on the 100 year floodplain are given in Section V: Stormwater Management Analysis. Results for the FEMA 100-year storm are presented in Section VI: Floodplain Management. The storms of primary interest in this section are the design storm and the 100-year storm using the USGS NFF flows. The design flood for the structure is the 25-year storm with a peak discharge of 351 ft^3/s. The 100-year storm peak discharge for the project is 670 ft^3/s.

During the 25-year storm the headwater elevation (at River Station 946.44 ft) is 437.59 ft with a velocity of 2.51 ft/s. The existing near approach roadway will be overtopped under the 25-year flood conditions. The 100-year storm produces a headwater elevation of 438.39 ft with a velocity of 2.32 ft/s. The roadway will be overtopped under the 100-year flood conditions.

3. Proposed Bridge Configurations

The project alternatives examined various structural replacement solutions taking into consideration current design criteria, the bridge rating to carry all legal traffic conditions; and elimination of current safety issues. The bridge replacement focused on four common aspects of the project: hydraulic opening, stream impacts, wetland impacts and roadway impacts. One public safety issue is that Franklin Street is within a school bus route without an alternative routing. The bridge must therefore be constructed between June 15th and September 1st. The bridge will be replaced while a full detour of the project site is in effect.

A single span bridge, a single cell box culvert and a single span concrete arch were studied using the hydraulic opening as the common element between the structures. The minimum hydraulic opening was determined to be 20’-0” by 4’-1” to adequately pass the 25-year storm and prevent increases in the back-water elevation. A brief description of the structural alternatives with opinions of probable cost has been summarized in the following paragraphs.

Alternative #1. Single Span Bridge

A single span bridge with a 17” spread box beam was considered to span Swabia Creek. The clear span was set at the minimum opening of 20’-0” and with an average under-clearance of 4’-2 ½”. A scour analysis indicates that the abutment footers will require foundation piles to bedrock for adequate protection against settlement. The principle advantage of this structure is the clear opening under the structure and its constructability with the stream diverted through the center of the bridge opening. Cofferdams would be placed around each respective abutment during its construction and then removed prior to the placement of the pre-cast spread box beams. The disadvantages include the long construction period (greater than the 75 days needed to satisfy the school bus routing restriction) required for driven pile foundations, cast in place concrete, increased long term maintenance and the cost of the structure. The estimated Opinion of Probable Construction Cost for the structure is $252,131 with an estimated construction duration of 135 days.

Alternative #2: Single Span Box Culvert

A pre-cast reinforced concrete box culvert alternative was analyzed with a hydraulic opening of 20’-0” by 5’-6” that is expected to silt in around fish baffles to a depth of approximately 1’-5”, providing an under-clearance of 4’-1”. The primary advantages of the single cell culvert are the clear opening and a construction period of less than 75 days (thus completing the construction within the school bus routing restriction) and reduced long-tem maintenance costs. The disadvantages include a concrete bottom (although it is expected to silt in) and the need for a temporary stream by-pass during the construction period. Cofferdams would be placed across the stream upstream of the structure to divert the stream into an open channel and pipe system which by-passes the construction area. Diverted waters are returned to the main channel just downstream of the construction area. The estimated Opinion of Probable Construction Cost for the structure is $156,442 with an estimated construction duration of 60 days.

Alternative #3: Single Span Concrete Arch

A pre-cast reinforced concrete arch bridge was analyzed with a hydraulic opening of 20’-0” wide with a mid height of 5’-0”. A scour analysis indicates that the abutment footers will require foundation piles to bedrock for adequate protection against settlement. Footers on driven piles and abutments will be installed as base supports for the bottom legs of the arch. Fill of 12” (including the road pavement section and sub-base) will cover the mid-span with deeper fill over the vertical legs of the arch. The disadvantages include the long construction period (greater than the 75 days needed to satisfy the school bus routing restriction) required for driven pile foundations, and the need for a temporary stream by-pass during the construction period. Cofferdams would be placed across the stream upstream of the structure to divert the stream into an open channel and pipe system which by-passes the construction area. Diverted waters are returned to the main channel just downstream of the construction area. The estimated Opinion of Probable Construction Cost for the structure is $215,479 with an estimated construction duration of 135 days.

4. Analysis of Alternative Structures

4.A Environmental Concerns

The major environmental concern of the project is the Swabia Creek environment and habitat. The ideal alternative solution minimizes impacts to the channel form, habitat and environment. Temporary impacts to Swabia Creek will be attributed to actions related to the installation of the proposed structure and will be ameliorated by the use of approved erosion and sediment control measures. Wetlands were identified near to, but outside of the construction area. They will be clearly identified for avoidance during the construction period.

Several techniques are proposed to minimize the impacts: construction under a detour, erosion and sediment control measures, channel control during construction, and concrete fish baffles (for the box culvert alternative). All alternatives will re-route roadway traffic from S.R. 3001 to State-maintained roadways to provide clear work zones to minimize stream and vegetation impacts. The erosion and sediment control measures will be removed once construction is complete. The natural stream substrate will be replaced to its original width and general condition after construction. Vegetation will return to preconstruction conditions after the bridge replacement is complete.

Outlet protection can be maintained with rip-rap stabilization. Rip-rap will be placed along the wing-walls and abutments of the spread box beam bridge and the concrete arch for sour protection. For the box culvert alternative, concrete aprons and cut-off walls will be provided across the culvert inlet and outlet, and the culvert invert will be depressed 1’-5” below the existing streambed. Concrete baffles will be provided to allow the streambed to silt in to provide a natural streambed habitat.

- The spread box beam bridge and the concrete arch do not permanently impact the channel bottom, thereby providing natural stream habitat. There is a slight risk due to scour and because of the low clearance under the bridge, scour maintenance would be costly.
- The box culvert has a depressed bottom that will silt in to an average depth of 1’-5”, providing a natural habitat and stream armoring. The box culvert is designed to guard against long term risks associated with scour. This is accomplished through the use of a depressed bottom and a concrete apron and cut-off wall, across the inlet and outlet, to guard against undermining of the structure. Large storms may scour the silts out of the culvert, but it is anticipated that the stream will replenish the materials naturally.

4.B Minimizing Impacts

The project objective is to replace the existing structure with a new structure that provides a safe travel way for the residents of the Borough of Alburtis and Macungie Township while minimizing initial construction costs, environmental impacts and long term maintenance costs. Each proposed alternative achieves the structural requirements for a safe travel way. The primary variation between the alternatives is the environmental impacts to Swabia Creek and its floodplain. Secondary impacts include the construction period requirements and cost.

Investigation of the hydraulic performance for the alternatives indicates only minor differences. All three alternatives improve the hydraulic conditions of Swabia Creek with respect to the existing structure. All three alternatives pass the 25-year design storm without overtopping the roadway. Although all three alternatives provide a reduction in the 100-year water surface elevations, the concrete arch alternative provides the least improvement in the hydraulic performance. This reduced performance is a result of a slightly smaller flow area under the arch, in comparison to the square cornered openings of the bridge and culvert.

Investigation of the impacts to the streambed indicate no differences between the open bottom bridge and concrete arch versus the use of a depressed bottom and fish baffles with the box culvert. The single span bridge and concrete arch have minimal impacts to the natural streambed and its habitat. A scour analysis indicates contraction scour depths of 2 ft. for the 100-year event and up to 16 ft. of total scour for the 500-year event. As per DM-4 the top of footer would be set at the 100-year scour depth and driven piles would be required to guard against the 500-year event. The box culvert is designed with fish baffles which, as per DM-2 Section 10.11.B, “help to simulate natural conditions by promoting the deposition and retention of stream bed material inside the culvert”. The design is provided as per DM-2 Section 10.11.E.1.a “such that the 1’-0” depression below the natural stream bed elevation is filled in with sediment”. The depression depth of the box provides more than 1’-0” for the natural stream processes to silt in. In addition to providing habitat for Swabia Creek, the box culvert provides protection against long term risk associated with scour due to its concrete bottom.

The time of construction is critical in the selection of the structure type. The box culvert alternative is the only alternative that can be counted on to meet the 75 day time period needed to allow a full detour of the construction area. The single span bridge and concrete arch will require additional time for the concrete to cure and to drive piles for the footer foundation and will not be able to meet the desired construction time period.

A comparison of costs indicates that the box culvert is the most economical alternative. The single span bridge and concrete arch require driven piles foundations, thus driving their costs above that of the box culvert. Long term maintenance costs add to the costs of the single span bridge in comparison to the other two alternatives.

TABLE 4
Comparison of Alternatives

Alternative	Hydraulic Performance 25-year	Change in 100-year WSE From Existing	Requires Temp. Stream By-Pass	Total Estimated Construction Cost	Estimated Construction Duration (Days)
1) Single Span Bridge	Under Bridge	-0.25 ft.	No	$252,131	135
2) Box Culvert	Under Bridge	-0.19 ft.	Yes	$156,442	60
3) Concrete Arch	Under Bridge	-0.01 ft.	Yes	$215,479	135

4.C Recommended Structure

Both the single span bridge and the concrete arch bridge will not be able to meet the required construction period window.

Based on factors of constructability, initial cost, long term maintenance and impacts to the stream, the recommended alternative for the replacement of the existing bridge along S.R. 3001 over Swabia Creek is the pre-cast reinforced concrete box culvert with cast-in-place headwalls and wing-walls. In addition to being the most economical of the feasible alternatives, the alternative provides a fish baffle system which minimizes the impacts to Swabia Creek as mentioned previously.

5. Recommended Bridge Opening

Based on the hydraulic analysis, the recommended bridge opening is at least 71.5 sq. ft having at least a 20 ft hydraulic width (normal to the stream flow). All three alternatives discussed in Section 4 meet these criteria. The bridge and box culvert provide a slightly larger opening than recommended (81.9 sq ft). In the following discussion, results will be presented for the recommended alternative only.

The recommended structure is a single cell Pre-Cast Concrete Box with a clear opening of 20 ft and upstream low chord elevation of 437.08 ft. The Concrete Box structure will have a vertical opening of 5.50 feet which is expected to silt in to a depth of approximately 1.5 feet so that an under-clearance of 4.08 ft. remains. An opening 20 by 4.08 feet was used as the clear opening for the proposed structure. There is no hydraulic skew in the bridge.

The existing and proposed conditions were compared at a common cross-section just upstream of the proposed bridge, (cross-section #8 at River Station 946.44 ft). The USGS NFF flow analysis indicates that the headwater elevation during the 25-year design flood event for the proposed structure, 436.82 ft, is lower than that of the existing, 437.59 ft. The headwater elevation during the 100-year flood event for the proposed conditions, 438.20 ft, is 0.19 ft less than that of the existing conditions, 438.39 ft. There will be no overtopping of the bridge or its approach roads during the 25-year event, while the near approach road will be overtopped under the 100-year flood conditions. Figure 5 (presented in the map section of this submittal) shows the existing and proposed 100-year floodplain map for NFF Flows.

Overall, the hydraulic conditions will be improved due to increased hydraulic efficiency of the proposed structure. Results are summarized in Tables 5 through 8 and the HEC-RAS model output can be found in Attachments G and H.

TABLE 5
25-YEAR WATER SURFACE ELEVATIONS
EXISTING AND PROPOSED
USGS NFF FLOWS

X-Sn. Location				Water Surface Elevations (ft)		
X-Sn. #		River Station (ft)	Distance to Next X-Sn. (ft)	Existing Bridge	Proposed Bridge	Difference
Upstream	14	1382.67	150	439.30	439.30	0.00
	13	1232.67	75	438.94	438.93	-0.01
	12	1157.64	75	438.40	438.42	+0.02
	11	1082.67	50	437.82	437.63	-0.19
	10	1032.67	50	437.68	437.20	-0.48
	9	982.67	50	437.61	436.79	-0.82
	8	946.44	58.49	437.59	436.82	-0.77
	Existing and Proposed Bridge @ River Station 930					
	7	887.95	17.95	436.35	436.25	-0.10
	6	875	54.15	436.49	436.24	-0.25
	5	820.85	66.85	436.01	435.94	-0.07
	4	754	50	435.24	435.24	0.00
	3	704	100	434.58	434.58	0.00
	2	604	100	433.84	433.84	0.00
	1	504		433.72	433.72	0.00

NOTE: Cross section locations are shown in Attachments G and H.

TABLE 6
100-YEAR WATER SURFACE ELEVATIONS
EXISTING AND PROPOSED
USGS NFF FLOWS

X-Sn. Location				**Water Surface Elevations (ft)**		
X-Sn. #		**River Station (ft)**	**Distance to Next X-Sn. (ft)**	**Existing Bridge**	**Proposed Bridge**	**Difference**
Upstream	14	1382.67	150	439.84	439.83	-0.01
	13	1232.67	75	439.57	439.57	0.00
	12	1157.64	75	439.11	439.10	-0.01
	11	1082.67	50	438.60	438.50	-0.10
	10	1032.67	50	438.44	438.28	-0.16
	9	982.67	50	438.37	438.18	-0.19
	8	946.44	58.49	438.39	438.20	-0.19
	Existing and Proposed Bridge @ River Station 930					
	7	887.95	17.95	436.30	436.22	-0.08
	6	875	54.15	436.38	436.38	0.00
	5	820.85	66.85	436.23	436.22	-0.01
	4	754	50	435.59	435.59	0.00
	3	704	100	434.81	434.80	-0.01
	2	604	100	434.28	434.28	0.00
	1	504		434.16	434.16	0.00

NOTE: Cross section locations are shown in Attachments G and H.

TABLE 7
25 -YEAR CHANNEL VELOCITIES
EXISTING AND PROPOSED
USGS NFF FLOWS

X-Sn. Location				Channel Velocities (ft/s)		
X-Sn. #		River Station (ft)	Distance to Next X-Sn. (ft)	Existing Bridge	Proposed Bridge	Difference
Upstream	14	1382.67	150	3.40	3.40	0.00
	13	1232.67	75	3.29	3.27	-0.02
	12	1157.64	75	5.15	5.06	-0.09
	11	1082.67	50	4.63	5.38	+0.75
	10	1032.67	50	3.65	5.11	+1.46
	9	982.67	50	3.04	5.36	+2.32
	8	946.44	58.49	2.51	3.68	+1.17
	Existing and Proposed Bridge @ River Station 930					
	7	887.95	17.95	5.26	3.10	-2.36
	6	875	54.15	2.60	2.63	+0.03
	5	820.85	66.85	4.85	2.74	-2.11
	4	754	50	5.02	5.02	0.00
	3	704	100	4.72	4.72	0.00
	2	604	100	1.70	1.70	0.00
	1	504		1.54	1.54	0.00

NOTE: Cross section locations are shown in Attachments G and H.

TABLE 8
100-YEAR CHANNEL VELOCITIES
EXISTING AND PROPOSED
USGS NFF FLOWS

X-Sn. Location				Channel Velocities (ft/s)		
X-Sn. #		River Station (ft)	Distance to Next X-Sn. (ft)	Existing Bridge	Proposed Bridge	Difference
Upstream	14	1382.67	150	3.30	3.31	+0.01
	13	1232.67	75	3.25	3.25	0.00
	12	1157.64	75	5.53	5.55	+0.02
	11	1082.67	50	5.04	5.40	+0.36
	10	1032.67	50	4.29	4.71	+0.42
	9	982.67	50	3.64	4.07	+0.43
	8	946.44	58.49	2.41	2.85	+0.44
	Existing and Proposed Bridge @ River Station 930					
	7	887.95	17.95	5.45	6.42	+0.97
	6	875	54.15	2.52	2.35	-0.17
	5	820.85	66.85	2.01	1.97	-0.04
	4	754	50	5.82	5.82	0.00
	3	704	100	5.60	5.59	-0.01
	2	604	100	1.78	1.78	0.00
	1	504		1.85	1.85	0.00

NOTE: Cross section locations are shown in Attachments G and H.

6. Scour Investigation and Rip-Rap Sizing

PennDOT Design Manual 4, Section 7.2.1 “Scour Investigation” states that “ No scour analysis for pipe or box culvert is required. Refer to BD 632M for scour protection details for box culverts.” Details are given as per specification BD-632M as presented in Attachment I.

Rip-rap protection for local scour was sized for scour and erosion undercutting of the abutments and front and rear aprons. Calculations based on the 100-year event (scour design flood) and the 500-year (super-flood) events indicate that a R-7 rip-rap (with and average diameter of 18”) to a depth of three feet should be used. Details of the calculations are provided in Attachment I.

7. Temporary Crossing

Construction will be done under a full detour condition. No temporary crossing for vehicular traffic will be provided at the site.

The single cell box culvert will be installed as a single unit, requiring a stream by-pass while the abutments are removed and the box culvert is installed. The stream will be diverted through a by-pass during the construction. The by-pass details are provided in the Erosion and Sediment Pollution Control Plans, while a HEC-RAS model of the temporary by-pass is included in Attachment H.

V. STORMWATER MANAGEMENT ANALYSIS

The project is a bridge replacement on the same alignment of the existing bridge, with minor roadway work adding impervious cover to the drainage area. The project will have little or no effect on the overall hydrology of the Swabia Creek watershed. The project will improve the hydraulics within the project area, but overall the creek will not be significantly impacted by the project.

An ACT 167 Stormwater Management Plan exists for the Little Lehigh Creek Watershed. The flow rates from the plan were presented in Section IV: Hydrologic Analysis. The construction proposed in this bridge replacement involves widening the road so that an additional impervious surface of approximately 4,000 square feet is added to the roadway. This constitutes a new development of less than 10,000 square feet and is expected to have an insignificant impact on the watershed-level runoff characteristics. According to Section IV, article A of the ACT 167 Stormwater Management Plan, the proposed improvements constitute “New Development” which are exempt from the runoff control plan.

The HEC-RAS model for both the existing and proposed conditions was run using the flows from the ACT 167 Stormwater Management Plan. Table 9 shows the results of the analysis and indicates a maximum increase in the water surface elevation of 0.01 ft occurring within the road right-of way just upstream of the bridge. Overall the results indicate no significant increases (less than 0.1 feet increase) in the proposed 100-year flood level over that of the existing 100-year flood level. The proposed bridge is therefore consistent with the ACT 167 Stormwater Management Plan.

Figure 6 shows the 100-year floodplain maps for the 1999 ACT 167 Stormwater Management Plan. At the scale drawn, the existing and proposed 100-year flood limits give the same approximate line of demarcation (as anticipated under conditions of no significant difference). Figure 6: 100-year Floodplain Map – ACT 167 Flows is provided in the “Other 100-year Floodplain Map” section of the JPA ECMS submittal.

Copies of the above hydrology and hydraulic analysis were sent to the Township and County officials for their review and comment. Signed letters of concurrence are provided in Attachment N.

Township and County officials have been notified of the proposed bridge replacement in accordance with PA ACT 14, P.L. 834, and no objections have been raised concerning the proposed project. Copies of the Act 14 Notifications and Receipts have been provided in Attachment N.

TABLE 9
100-YEAR WATER SURFACE ELEVATIONS
EXISTING AND PROPOSED
ACT 167 FLOWS

X-Sn. Location				Water Surface Elevations (ft)		
X-Sn. #		River Station (ft)	Distance to Next X-Sn. (ft)	Existing Bridge	Proposed Bridge	Difference
Upstream	14	1382.67	150	441.32	441.31	-0.01
	13	1232.67	75	440.83	440.82	-0.01
	12	1157.64	75	440.61	440.60	-0.01
	11	1082.67	50	440.31	440.31	0.00
	10	1032.67	50	440.10	440.10	0.00
	9	982.67	50	439.41	439.41	0.00
	8	946.44	58.49	439.52	439.53	+0.01
	Existing and Proposed Bridge @ River Station 930					
	7	887.95	17.95	437.46	437.44	-0.02
	6	875	54.15	437.49	437.49	0.00
	5	820.85	66.85	437.21	437.21	0.00
	4	754	50	436.50	436.50	0.00
	3	704	100	435.79	435.78	-0.01
	2	604	100	435.69	435.69	0.00
	1	504		435.57	435.57	0.00

NOTE: Cross section locations are shown in Attachments G and H.

VI. FLOODPLAIN MANAGEMENT

A detailed HEC-2 FEMA study for the project area was done in 1974. In July 16, 2004 the FEMA map was revised but remains based on the same flow rates and hydraulic conditions. The detailed FEMA water flow rates and water surface elevations were used to calibrate a HEC-RAS model of the site. The HEC-RAS model was then run using the FEMA flows to obtain the 100-year flood water elevations for the existing and proposed conditions of the project site. These results are given in Table 10.

Overall results of the hydraulic model indicate that there will be a reduction of up to 0.35 feet in the currently accepted 100-year water surface elevations with the completion of the project. The overall floodplain will be reduced upstream of the bridge due to the project. Figure 7, provided in the “Other 100-year Floodplain Map” section of the JPA ECMS submittal, shows the existing and proposed 100-year floodplain maps, based on FEMA flows.

TABLE 10
100-YEAR WATER SURFACE ELEVATIONS
EXISTING AND PROPOSED
FEMA FLOWS

X-Sn. Location				Water Surface Elevations (ft)		
X-Sn. #		River Station (ft)	Distance to Next X-Sn. (ft)	Existing Bridge	Proposed Bridge	Difference
Upstream	14	1382.67	150	439.17	439.17	0.00
	13	1232.67	75	438.65	438.65	0.00
	12	1157.64	75	437.99	437.98	-0.01
	11	1082.67	50	437.27	437.22	-0.05
	10	1032.67	50	436.97	436.78	-0.19
	9	982.67	50	436.71	436.36	-0.35
	8	946.44	58.49	436.50	436.23	-0.27
	Existing and Proposed Bridge @ River Station 930					
	7	887.95	17.95	435.80	435.77	-0.03
	6	875	54.15	435.89	435.89	0.00
	5	820.85	66.85	435.72	435.72	0.00
	4	754	50	435.04	435.04	0.00
	3	704	100	434.46	434.44	-0.02
	2	604	100	433.69	433.69	0.00
	1	504		433.58	433.58	0.00

NOTE: Cross section locations are shown in Attachments G and H.

VII. RISK ASSESSMENT

The hydraulic analysis reveals that there will continue to be a risk of overtopping the SR 3001 bridge near approach during the 100-year flood events. Since there will be an overall reduction in water surface elevations, risk associated with overtopping will be decreased by completion of the proposed project.

During construction, stringent measures will be in place to protect the Swabia Creek from sediment and other pollutants. An approved erosion and sediment pollution control plan will be in place during all construction activities. The erosion and sediment pollution control plan is provided as part of the GP-11 submittal materials.

The proposed bridge replacement will have no adverse impacts on public safety, public property, or the environment. Public safety will be enhanced by replacing the old bridge with a new structure meeting current design standards. Flooding conditions will be improved at the site due to overall reduction in water surface elevations. Therefore, the proposed bridge replacement will decrease the associated risks to the environment and the public.

VIII. SUMMARY DATA

1. Highway Route Number: SR 3001, Section 01B

2. Station: 14+88

3. Name of County: Lehigh County

4. Name of Township: Lower Macungie Township and Borough of Alburtis

5. Name of Stream: Swabia Creek

6. Drainage Area: 6.57 sq. km (2.538 sq. miles)

7. Location: **USGS Quadrangle:**
Allentown West, East Greenville, Manatawny
Latitude: 40E 30' 08.1";
Longitude: 75E 35' 49.5

8. Stream Widths and Depths:

Average Top Width: 23 – 38 ft
Average Bottom Width: 10 – 16.5 ft
Average Channel Depth: 2.48 – 4.27 ft
Average Channel Slope 0.00615

9. Normal Flow Depth: 2.6 ft (1.2-yr storm as normal)

10. Streambed and Water Surface Elevations 500 ft+/- up and down stream:

Streambed Elevation 500 ft Upstream: 436.13 ft
Water Surface Elevation 500 ft Upstream 436.79 ft
Streambed Elevation 500 ft Downstream 430.51 ft
Water Surface Elevation 500 ft Downstream 431.50 ft

11. Dimensions of Structure:

a. Structure:

	Existing Bridge	Proposed Bridge
1. Structure Type	Steel I-Beam	Concrete Box
2. Number of Spans	1	1
3. Clear Span	16.5 ft	20 ft
4. Normal Span	16.5 ft	20 ft
5. Underclearance	Varies 4.55 ft – 5.05 ft	4.08 ft
6. Skew Angle(hydraulic)	0^0	0^0
7. Low Chord Elevation	437.72 ft	437.08 ft

b. Channel: No changes are recommended to the channel.

1. **Type of Channel:** Natural with sand and cobble bottom
2. **Bottom width of Channel:** 10 – 16.5 ft.
3. **Side Slopes:** 2:1 both sides upstream, 3:1 both sides downstream.

12. Permit Items: PASPGP – 3

13. Impact to Wetlands: None

14. Quantity of Fill Below Ordinary High Water:

Volume of Temporary Fill: None
Volume of Permanent Fill: None

*Hydraulic Data:

	Design: 25-Yr	100-Yr
Existing		
Flood Magnitude(ft^3/s)	350.6	670
Velocity(ft/s)	2.51	2.41
WSEL(ft)	437.59	438.39
Proposed		
Flood Magnitude(ft^3/s)	350.6	670
Velocity(ft/s)	3.68	2.85
WSEL(ft)	436.82	438.20

* Note: Data at the cross-section just upstream of the existing and proposed bridges.

IX. PRELIMINARY COST ESTIMATE

The estimated cost estimate for each bridge alternative is as follows.

- Alternative 1 (Prestressed Concrete, Spread Box Beam): $242,131
- Alternative 2 (Box Culvert): $156,442
- Alternative 3 (Concrete Arch): $215,479

X. RECOMMENDATIONS AND CONCLUSIONS

The existing bridge should be replaced by a Pre-cast Reinforced Concrete Box Culvert that has a single clear span of 20 ft. The proposed alignment will be the same as the existing structure. The hydraulic efficiency of the bridge will be increased and enhanced due to the larger clear opening. The upstream low chord elevation will be at an elevation of 437.20 ft with an under-clearance of 5.50 ft. to the top of the bottom slab, which is expected to silt in over a 6 month period to an average under-clearance of 4.08 ft. The curb-to-curb width of the bridge will be increased to 32 ft to better accommodate two lanes of traffic. The information provided supports the proposed minimum clear span and details the resulting backwater conditions. There is no increase in backwater conditions and while velocities have been increased in the bridge area, they remain below the velocity needed to cause scour and erosion in the channel.

Hydraulic conditions at the site will be improved by replacing the existing structure. There will be an overall reduction in water surface elevations. Riprap will be provided for undercutting protection of the entrance and exit aprons.

An Individual Section 404 permit will not be required for the construction activities associated with the proposed project. The Pennsylvania State Programmatic General Permit No. 3 (PASPGP-3) applies to the proposed activities.