Guidebook for the

PITTSBURGH GEOLOGICAL SOCIETY

Field Trip

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The Geology, Environmental Geology, and Engineering Geology Of Western Allegheny County, Pennsylvania

Trip Leaders:
Wendell L. Barner, IT Corporation
Brian H. Green, U.S. Army Corp of Engineers, Pittsburgh District
Chuck H. Shultz, Slippery Rock University

For additional copies of this field trip guidebook, contact:

Pittsburgh Geological Society
PO Box 58172
Pittsburgh, PA 15209

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GEOLOGY OF THE PITTSBURGH AREA

By John A. Harper
Pennsylvania Geological Survey

INTRODUCTION

The first references to geologic topics in the Pittsburgh area date from the early 1800s (Aigster, 1813; Meade, 1828; Daubeny, 1839). However, much of what we now considered to be the basic geologic knowledge of western Pennsylvania languished until Henry Darwin Rogers' First Geological Survey of Pennsylvania examined the entire state in the late 1830s and finally reached publication in the late 1850s (Rogers, 1858). The Second Geological Survey of Pennsylvania under J. P. Lesley provided even more information. Field investigations by Stevenson (1876, 1877 and 1878) and White (1878) produced seminal volumes of geological work during this period, including descriptions of coals, limestones, and other mineral commodities, and geologic maps of the areas studied. Both of these geologists later published syntheses of their work and the works of others (White, 1891; Stevenson, 1906) that helped establish much of the stratigraphic and structural nomenclature of western Pennsylvania. Follow-up studies during the third and fourth surveys provided a wealth of data and interpretations in geologic atlases and folios by Hughes (1933), Johnson (1928, 1929), Munn (1910, 1911a, 1911b), Shaw and Munn (1911), and Woolsey (1905, 1906) that stand, for the most part, as (if you'll pardon the pun) a rock solid foundation for later economic, geologic, geotechnical, and environmental investigations. It was these reports that provided the basis for the six-county Pittsburgh Regional Environmental Geologic Study (PREGS) during the 1970s (see the synopsis by Briggs, 1977) and many other more recent works.

PHYSIOGRAPHY

Most of southwestern Pennsylvania lies within the Appalachian Plateaus physiographic province, an area having a generally level surface at an altitude great enough to permit erosion of deep valleys by streams. This level surface results from the essentially flat-lying nature of the bedrock near the earth's surface. The area around Pittsburgh, including Allegheny, Beaver, and Washington counties, is further subdivided into the Pittsburgh Low Plateau Section. The topography in this section is characterized by a relatively small amount of relief (Sevon, 1996). The hilltops in this area stand at about equal elevations, approximately 1,100-1,300 ft (335-400 m) above sea level, or about 350-550 ft (105-165 m) above the level of the rivers. By comparison, relief in the neighboring Allegheny Mountain Section to the east (from Chestnut Ridge to the Allegheny Front) commonly exceeds 1,500 ft (460 m) in the vicinity of the prominent high ridges that characterize that section. Most of the hilltops of western Pennsylvania are the remnant of an ancient, relatively broad, relatively flat surface formed during the

Cenozoic. This surface has been highly dissected by numerous antecedent streams, at least some of which probably became established during the Mesozoic (Sevon, 1993). This surface sloped very gently toward the northwest, toward the area of the present-day Great Lakes. At least the larger streams probably meandered in broad, shallow valleys cut only a little deeper than the surrounding landscape. Once this drainage pattern was set, the topographic relief gradually increased as the streams cut down into the rocks of the plateau, until the present-day relief we see in western Pennsylvania became established.

STREAMS AND TERRACES

Southwestern Pennsylvania is drained by the Ohio River drainage system, which includes the Monongahela and Allegheny drainage systems and their tributaries. The pool elevations of the various rivers and streams is regulated by a set of US Army Corps of Engineers dams set at intervals along the three rivers. The pool elevation at Pittsburgh is 710 ft (216 m) above sea level.

The Ohio River drains into the Mississippi River, which eventually drains into the Gulf of Mexico. This was not always so, however. It has been explained many times by Leverette, 1902, 1934; Wagner and others, 1970; Harper, 1997, 2000; Kaktins and Delano, 1999). Suffice it to say that, prior to the first glacial advance into North America during the Pleistocene, all of the major rivers in western Pennsylvania drained northward into what is now Canada. The change to the current drainage probably occurred early during the Ice Age, perhaps during the first glacial advance (>770 ka BP).

The process of downcutting and lateral erosion by Pittsburgh's rivers during the Pleistocene excavated the bedrock bottoms and sides of their valleys to about 30 or 40 ft (9 to 12 m) below their present elevation. The river valleys became broad, flat-bottomed, and U-shaped valleys with steep walls (Figure 1). Glacial meltwaters filled the bottoms of the Allegheny and Ohio River valleys with silt, sand, and gravel outwash. In the Pittsburgh area, this sediment reaches thicknesses of up to 80 ft (24 m). The Monongahela and its tributaries, deprived of the outwash that flooded the Allegheny and Ohio valleys, built up their channels with sediments derived from their watersheds to the south.

After the retreat of the last glacier about 10 ka BP, the volume of water and sediment coming down the rivers decreased. The rivers cut new channels into the glacial valley-fill sediments, reworking the sediments in several areas. The results are low terraces, including the modern floodplain, about 10 to 30 ft (3 to 9 m) above present river level (Figure 1).

The porous gravelly valley-fill alluvium underlying the Allegheny and Ohio River is the primary source of ground water in Allegheny County. This aquifer recharges the rivers, and precipitation adds water to both.

STRATIGRAPHY

Figure 2 represents a very generalized columnar section of the rocks exposed in Allegheny County – changes in thickness and interval are too variable to be shown in a more specific illustration. Figure 3 illustrates the distribution of the main units in Allegheny County.

The Allegheny Group crops out at only three places in Allegheny County, in stream valleys in the northern part of the county. Only the upper 100 ft (30 m) or so of the Allegheny crops out in the Pittsburgh area; the rest of the group lies in the subsurface. The largest area of exposure is along the Allegheny River from Tarentum to Freeport. Exposures also occur along PA Route 8 where it crosses the Kellersburg anticline near Fall Run Park. The Allegheny Group forms most of the bedrock north of the Ohio River in Beaver County.

The Conemaugh Group is the thickest sequence in western Pennsylvania, commonly comprising between 600-700 ft (180-210 m) of sandstone, mudrocks, marine and freshwater limestones, and coal. The coals consist of a few thin seams that are mined only in limited areas. The top of the Upper Freeport coal and the base of the Pittsburgh coal form the boundaries of this group. The top of the Ames Limestone divides the Conemaugh Group into two formations, the older Glenshaw Formation and younger Casselman Formation (Flint, 1965). Each of these formations is about 300 ft (90 m) thick, dividing the group into two roughly equal subdivisions.

The Glenshaw Formation is characterized by up to six marine units in the Pittsburgh area. These range from thin argillaceous limestones sandwiched between organic-rich shale layers to even thinner calcareous siltstones and shales. The Casselman Formation typically contains no marine units, consisting instead of thick sandstones, red beds, shales, and thin coals.

The Conemaugh Group underlies almost all of the Pittsburgh area, but because of regional dip to the southwest, it crops out mostly in stream valleys in the southern half of Allegheny County (Figure 3). The Glenshaw Formation is best seen north of the two rivers and will be seen in detail on the second day of this field conference. The Casselman Formation is well exposed south and east of the Allegheny and Ohio Rivers. We will see a good chunk of it during the first day of the field conference. Although there are no complete, uninterrupted sections of the Conemaugh exposed in western Pennsylvania, there are places in Allegheny County where large portions of it can be seen in a single outcrop or roadcut.

The Monongahela Group lies almost entirely south of the Allegheny and Ohio Rivers. Its irregular outcrop pattern in Figure 3 is the result of stream erosion cutting deep valleys completely through the group and into the underlying Conemaugh. A few erosional remnants of the Monongahela Group remain on the tops of some high hills north of the rivers. The group is characterized by about 300 ft (90 m) of nonmarine carbonates, several highly productive coal seams, few red beds, a relative lack of sandstone, and prominent shales and siltstones.

The Dunkard Group contains the youngest sedimentary rocks in southwestern Pennsylvania. It is divided into three formations in southwestern Pennsylvania, the Waynesburg, Washington and Greene formations. However, only the Waynesburg and part of the Washington are present in the area of the Field Conference. The Waynesburg Formation, with the Waynesburg coal at its base, consists of about 180 feet of mixed claystones, shales, siltstone, and sandstones with minor amounts of coal and carbonates. The Washington Formation, with the Washington coal at its base, contains about 200 feet of mostly claystones and shales with minor amounts of other rock types.

STRUCTURE

The regional structure of southwestern Pennsylvania is shown in Figure 4. This area lies within the Pittsburgh-Huntingdon Synclinorium (also referred to as the Dunkard Basin), with the Dunkard Group occupying the center of the basin, and progressively older rocks cropping out towards the basin margins.

The strata of southwestern Pennsylvania are very gently folded, with axes trending approximately N35°E (Figure 5). The anticlines typically have flanks dipping less than 20 ft/mi (3.75 m/km), although some of the "more pronounced" folds in the Pittsburgh area have dips in the neighborhood of 200 ft/mi (38 m/km) – an amazing 2° slope! The typical fold tends to curve horizontally as well as vertically, resulting in serpentine structures marked by very gentle domes and saddles. Westward from the center of the Dunkard basin the folds become open and discontinuous. Eastward from the basin center the rocks become increasingly distorted by both folding and faulting, and the folds have more steeply dipping flanks and higher structural relief – for example, the Chestnut Ridge, Laurel Hill, and Negro Mountain anticlines. The principal surface fold axes in Allegheny County are shown in Figure 5, and a cross section illustrating the relation of structure to topography (at a GREATLY exaggerated scale) has been included as Figure 6.

Jointing is very common in southwestern Pennsylvania outcrops. The preferred orientations of the two principal joint sets, as measured in shales and sandstones, range from N10°E to N40°E and N50°W to N80°W (Nickelsen and Hough, 1967). In addition, two well-developed vertical and intersecting cleat sets have develop in the local coals. Western Pennsylvania joints play important roles in many aspects of regional geology. For example, they have affected surface drainage patterns by altering the predominantly dendritic pattern characteristic of essentially flat-lying strata to a trellis-modified dendritic pattern. Many of the streams in the region have long, straight segments that are oriented NW-SE or NE-SW as a result of the major joint sets (e.g. the Ohio River which flows in an almost straight channel from downtown Pittsburgh to Beaver). Joints create relatively easily eroded pathways that the antecedent streams followed as they cut down into the low folds of the Pittsburgh area. In addition to joints resulting from tectonic stresses, many joints also form approximately parallel to valleys, regardless of valley orientation, as a result of the release of stress (Ferguson, 1967). Many of these contribute to the plethora of landslide problems encountered in western Pennsylvania.

Faulting is not a common feature of the surface rocks of southwestern Pennsylvania, but faults do occur (Figure 7). Normal faults are the most common type present in the Pittsburgh area. Most, if not all, occurred penecontemporaneously with deposition as glide planes of slump blocks associated with stream-bank landslides. Reverse faults (Figure 7C) are far less common than normal faults in this area, but a good example can be seen in a roadcut along PA Route 28 at the Tarentum exit about 20 mi (32 km) north of Pittsburgh.

MINERAL RESOURCES

Geologic resources in the Pittsburgh area have included, at one time or another: coal; crude oil and natural gas; low-grade iron ores (primarily siderite); sand and gravel for glass and construction; sandstone used for construction (aggregate, foundations, flagging, and even dimension stone); limestone suitable for construction, agriculture, flux, and other products; clay and shale suitable for bricks, pottery, and refractories; brine for salt; and water. Even slag, the waste product of the steel making process, has become a major mineral resource within the last 20 years.

Fossil Fuels

Coal, historically the most important mineral resource to the Pittsburgh area's economy, has been mined in Allegheny County since the 1700s. It became most important in the 1800s and throughout the 1900s as a source of coke in the manufacture of steel. Particularly significant are the Pittsburgh and Redstone coals of the Monongahela Group and the Upper Freeport coal of the Allegheny Group (Figure 3). The Pittsburgh coal, arguably, is the best known and most valuable of all the bituminous coal beds of the Appalachian basin

Crude oil was far more important to the Pittsburgh area in the 1800s than it is today. Thousands of oil wells were drilled throughout the area in the latter half of the nineteenth century, for example the McDonald oil field in Figure 8, was discovered in 1890 and was second in volume of production only to the famous "giant" Bradford oil field in McKean County. Today few companies operate the old wells and no one is drilling new ones. In contrast, natural gas has increased in value through the 1900s. There has been a concerted effort to find economical quantities of gas in the last 20 years. The value of natural gas as a clean, environmentally "friendly" fuel will continue to make it a viable geologic resource in the near future, as well. Figure 10 shows the locations of the oil and gas fields of the county. Many of these fields are now long abandoned.

Iron Ore

The earliest iron furnaces in the Pittsburgh area, as elsewhere in western Pennsylvania, produced pig iron from local low-grade iron ores, primarily siderite nodules and bog iron. Siderite is a common, if not abundant, source of iron typically associated with marine limestones and shales in western Pennsylvania, including those of the Glenshaw Formation (Figure 3). The Buhrstone ore, a layer of bedded and/or nodular siderite typically capping the Vanport Limestone of the lower Allegheny Group, provided the primary source of ore throughout the area for many years. By the 1850s, however, western Pennsylvania saw a major change as the supply of wood for charcoal and the best siderite deposits both became exhausted. Cheap, plentiful, high-quality iron ore from the Lake Superior region flowed into the Pittsburgh area, where coke from the Pittsburgh coal supplanted the exhausted forests and the rivers provided cheap and efficient transportation corridors. Pittsburgh remained the primary iron- and steel-making city in the US until the last quarter of the 20th century. Most of the mills are gone now, and only the legacy of iron remains.

NON-METALLIC, NON-FUEL RESOURCES

The quantity of resources such as sandstone (Figure 9A) and limestone remains high in and around the Greater Pittsburgh area. However, the production of those resources has fallen off dramatically since the end of World War II. It is highly ironic that the same population pressures creating increased demands for such mineral resources in the region have also helped reduce the amount of annual production over the years. Much of the loss of mineral industries in this area can be blamed on competing landuse pressures such as the need for additional space for construction, zoning laws, increased taxes, and other factors. Where there used to be numerous sandstone, limestone, and clay quarries around the county, now there are very few or none at all.

Sand and gravel are available in large supply in stream deposits of both Pleistocene and Holocene age (Figure 9B). Sand and gravel dredged from the beds of the Allegheny and Ohio rivers are, to a large degree, reworked glacial materials containing durable rock; therefore, they are most suitable for construction aggregate. Some of the sand is suitable for glass manufacture as well and, in fact, Pittsburgh had a thriving glass manufacturing industry for many years. Sand and gravel also occur on the river terraces (such as in Figure 9B), but they contain a somewhat higher proportion of weathered pebbles than the deposits in the rivers and are, therefore, less useful and less valuable.

Local sandstones are abundant, but they typically exist as channel deposits that change thickness and lithology abruptly over short lateral distances. Internally, they often consist of thin beds and may contain enough iron minerals to limit their use as crushed rock and rough stonework. Where they are massively developed, even grained, and hard (as in Figure 9A), however, the sandstones have been quarried historically as dimension block for nearby use in bridge abutments, chimneys, and permanent building construction.

The majority of the carbonate rocks in the Pittsburgh area are impure, thin to nodular bedded, and irregularly distributed. The Sewickley Member of the Pittsburgh Formation (commonly called "Benwood Limestone") is the thickest and, arguably, most useful of these today. An average analysis for the thicker usable Pennsylvanian limestones in this area shows calcium 80% to 85%, magnesium carbonate 2.3% to 5.6%, silica 5% to 8%, and combined alumina and iron oxide 2.4% to 10.8% percent (Johnson, 1929). When Allegheny County was mostly agricultural, farmers used most of the local limestones for agricultural lime, burning and mixing them in homemade kilns, until commercial mixes became available.

Deposits of clay and clay products, once of fairly high importance in Allegheny County, currently are considered of minor consequence. This is due more to the steadily shrinking area of the county available for clay pits than to a decrease in either raw materials or demand for clay products. Local clay resources include surficial clays in the Carmichaels Formation, residual clays, and claystones and shales mined at or near the surface. The Carmichaels clays are very plastic, but erratic in occurrence and locally mixed with varying quantities of sand or silt. They were especially valuable in the 1800s for making stoneware, roofing tile, and brick. The largest deposits of residual clay occur where the Sewickley Member of the Pittsburgh Formation crops out on top of broad, flattopped hills, but these areas commonly are more valuable as farmland or housing developments (Johnson, 1929).

A more recent "mineral resource", slag has stopped being simply an eyesore in western Pennsylvania. Old slag dumps now command the attention of companies such as Lafarge and International Mill Service who are quarrying them for a number of purposes. The large slag dump in West Mifflin, Allegheny County (Figure 9C) now boasts a shopping mall and two shopping centers, as well as a "gravel" quarry. Quarrying operations in this dump, and others in Beaver and Westmoreland counties, provide fine aggregates, railroad ballast, PennDOT approved Type C coarse aggregates, PennDOT approved skid-resistant level H aggregates, and general fill (Barnes, 1997) (although, the District 11-0 office of PennDOT currently prohibits the use of slag for use beneath roads and road shoulders). Another slag dump on Nine Mile Run in the Squirrel Hill section of Pittsburgh is currently being graded and covered with topsoil in an effort to develop it as an upscale townhouse community (see Stop 4).

From about 1815 to 1870, the salt industry played a major role in the economy of the Pittsburgh area. Producers drilled or dug holes into shallow brine aquifers, typically less than 500 ft (150 m) deep, extracted salt from the produced brine by evaporation, and sold it all over the eastern United States. In the early 1800s Tarentum, the first place in western Pennsylvania where salt was produced from brine, became one of the more important salt producing areas of the country. Most of the brine came from sandstones of the Pottsville Group; oil and gas drillers still refer to these rocks as the Salt sands to this day. In fact, the technology of drilling wells began with the salt industry and soon spread to the oil and gas industry. "Uncle Billy" Smith, the man "Colonel" Edwin Drake hired to drill his famous well at Titusville, was working as a salt-well driller in Tarentum at that time.

WATER RESOURCES

The larger towns and cities of Allegheny County, which are located mainly along the three rivers, depend primarily on the rivers for their water supplies (Figure 10). Pittsburgh and much of the South Hills draw their drinking water directly from the Allegheny and Monongahela rivers. The rest pump water from the valley-fill deposits lining the river valleys to a depth of 60 or 70 ft (18 to 21 m). These deposits also constitute the chief source of water for air conditioning in the office buildings of downtown Pittsburgh, as well as the fountain at the Point.

The Allegheny and Ohio valley-fill deposits consist of clay, silt, sand, and gravel containing scattered cobbles and rare boulders. The Monongahela valley-fill is chiefly locally derived fine sand, silt, and clay with scattered large clasts of bedrock. The permeability of all these deposits can vary within relatively short distances owing to changes in sorting of the sediments. Thus, yield will fluctuate between otherwise similar wells. The alluvium ranges in thickness from 30 to 85 ft (9 to 26 m), averaging about 60 ft (18 m), and typically is overlain by Holocene fine sand and silt up to 25 ft (8 m) thick.

Recharge from the rivers supplies most of the water resources in the valley-fill, and fluctuations in the water level in the alluvium occur as the result of changes in withdrawal rate. Valley-fill groundwater is satisfactory for most ordinary uses – it contains less suspended matter, bacterial contaminants, and industrial wastes than the surface waters. Most contaminants are filtered out in whole or in part during movement of the water through the alluvium. However, chlorination and filtration are routinely used where the ground water is destined for human consumption. Unfortunately, this groundwater is harder and contains more iron and manganese than does the surface water.

The greatest proportion of groundwater used in Allegheny County, about 80% to 90%, comes from the alluvium. The remainder comes from bedrock aquifers, typically within the Conemaugh or Allegheny Groups. The better bedrock yields come from the sandstones, of which the Morgantown sandstone (Figure 2) is probably the best for consistent supply. The carbonate rocks of the Sewickley Member of the Pittsburgh Formation (Figure 2) is also a reliable source. In other units groundwater normally occurs at the top or base of an impermeable layer, or in communicating joint systems.

The Pittsburgh area was once blessed with an abundance of springs. The best tasting spring water came from the base of the Ames Limestone, the carbonate rocks of the Sewickley Member, and the Morgantown sandstone (Figure 2). The spread of residential areas following World War II, and mine subsidence problems, have led to local disruption and pollution of many springs. Only in the more rural areas of Allegheny and adjacent counties will one still find good quality springs.

Road Log

See Figure 11 for field stop location map.

Mileage		Description
Int	Cum	
0.0	0.0	Leave the Parkway Center Inn and turn right onto Parkway Center Drive.
0.1	0.1	At the traffic light turn left onto Green Tree Road.
0.2	0.3	At traffic light turn right onto Mansfield Road and continue past first traffic light.
0.4	0.7	At traffic light turn right onto Holiday Drive.
0.5	1.2	Turn left onto Anderson Drive.
0.2	1.4	Turn right into parking lot for Units 7, proceed to west end.

Stop 1. Overview of Chartiers Valley.

This will be a brief stop to discuss the history of Chartiers Valley, geomorphology, and land development in Chartiers Valley.

HISTORY

Long before there was a Crafton there was a creek snaking through a valley of unusual beauty and the history of Crafton is inseparable from that of this creek and valley. Harry Meredith, Crafton's Historian Emeritus, describes the vicinity as he remembered it in his boyhood in the 1860's:

> That once-beautiful valley was guarded on either side, first on one side then the other, by short ranges of high hills. Along the base of these hills the winding creek found its way, and by continually undermining and

> carrying away the soil, left huge bare rocks in natural formation, extending high in the air. These rocks and their many ledges and wonderful caves added much to the natural beauty of this part... The numerous large tributaries to Chartiers Creek, their deep ravines and miniature water falls, beautiful ferns and moss and wild flowers, all kinds of native trees and bushes, with their birds, squirrels, coon, rabbits and groundhogs, together with nearness of all to the big and beautiful Ohio River, no doubt made this part of the country an ideal neighborhood for the Indians in their day.

In 1749 a Frenchman, Celeron de Blainville, the first white man to record his impressions of the area, wrote *"It is the most beautiful I have seen until now on the Belle Riviere,"* as the French called the Ohio River. Celeron was sent to plant lead tablets between Canada and the Mississippi Valley to lay claim to the territory for King Louis XV. The creek was already familiar to one of his countrymen, the half-breed renegade Pierre Chartiers, who traded at the mouth of the creek and left it his name.

The creek appeared in 1750 on a map drawn by Lewis Evans, a Philadelphia cartographer commissioned by British Governor Hamilton to survey "the land across the mountains of the headwaters of the western flowing rivers." Evans wrote that "there was a village at the great rock a league below the forks on the left bank of the Ohio at the mouth of a creek called Alaquipa (though late known as Chartiers).

Young George Washington made his first appearance with his guide, Christopher Gist, in this part of the country in 1752 passing that great rock, later to be known as McKees Rocks, where the creek flows into the river. By then, the Seneca Queen Alaquipa had moved to the mouth of the Youghiogheny, and the Delawares under Chief Shingas occupied the mouth of the creek. Washington later referred to the creek as Shurtees when in 1770 he again visited land claimed by the Ohio Company farther upstream. Non-French speaking English and Native Americans early on corrupted Chartiers to Shurtees or Shurtees.

Native Americans in the Chartiers Valley

According to legend, the name given to the Crafton/Ingram area by the Seneca Chief Cornplanter was Killemun. There was a village, or more likely a camp, atop a rocky cliff on the east side of the creek (Backbone Road), a site that would have provided a commanding view of wildlife in the valley or enemies approaching by footpath or canoe from the north, south or west. The Seneca tribe was one of the six Iroquois Nations, which had been displaced farther east by the white man. The Iroquois who guarded their western hunting grounds were called Mingos. Cornplanter, from the upper Allegheny River, probably never lived in the area although he could have hunted and even camped here as the natives traveled great distances. Burial mounds containing artifacts from a much earlier pre-Colombian Adena Culture were discovered farther down the creek toward the "Rocks" in the vicinity of Fall Hole. The Indians who built the mounds were Algonquins and thought to be more sedentary, growing beans, squash, sunflowers, corn, and tobacco. They built houses, smoked tobacco and made pottery, tools of stone and even copper. The Fall Hole was a deep hole below a five-foot drop in the water level, which the Indians called Hinpoba. This placid backwater lake, formed where the creek made a sharp left curve, was destroyed when the Pennsylvania Railroad cut a channel through the hill diverting the course of the stream. The legend about it survived in **Tales of Pioneer Pittsburgh**, as told by William England, a former Crafton Borough Secretary:

When James Snowden, one of the first white settlers in the district, announced his intention of building his home at this site, he was warned against it by Cornplanter, the celebrated Seneca Chief. Cornplanter, whose Indian name was GY-ANY-WA-CHIA, told Snowden to beware the spirit of a dead Indian maiden, Incolala, which inhabited the waters of the lake:

> Incolala, whose name in the white man's tongue meant 'Silver Moonlight' had been the fairest maiden in all the tribes along the Ohio, and her bright eyes and smile had captured the heart of the stalwart young brave Cocohuha. But Incolala's happiness had been tinged with sadness over a former lover, Folcano, who kept to his teepee, brooding as her marriage to Cocohuha approached. This was related to Snowden in Cornplanter's deep sonorous voice.
>
> On the eve of the wedding, Folcano came to Silver Moonlight, expressing a desire to take her once again in his canoe on the placid waters of the lake. Pleased that Folcano seemingly had swallowed his bitterness, Incolala consented to go. As the evening chant of the tribe died away with the embers of the tribal fire, and forest shadows closed in upon the weakening flames, the two drifted upon the bosom of the waters. Only the pleading voice of Folcano broke the silence of the night.
>
> Then suddenly Folcano leaped from his place and seized his beautiful companion by the throat; A scream died in her breast and as she struggled like a terrified wild thing, the canoe upset and plunged them into the lake. Folcano swam back to shore, but Silver Moonlight never reappeared on the surface.

Snowden laughed at the superstition of the Indian, but oddly enough, he too fell into the lake the following spring and drowned.

On September 27, 1777, he again wrote to Jasper:

> "I parade two or three times a week between this place and my farm (where I have established a smallpox hospital) with 12 expert riflemen; the exercise is good and wholesome." The years of the three sevens, 1777, was an especially bloody one: Indians were on the warpath, taking advantage of the settlers' preoccupation with the Revolution, and they harassed the settlers unmercifully; but it was not the injured that were patients in his hospital.

The following is a somewhat embellished account from a newspaper article written 155 years later in September, 1932 at the time of the dedication of the memorial marker:

Using soldiers and farmers, Hand supervised the erection of the first Federal Hospital. It was made of logs, was 100 feet long and 30 feet wide; had two stories, no windows, but two doors on both of the long sides. It was surrounded by a porch and had a great stone furnace. Several small cannon are thought to have been used for protection from the hostile Indians... All that remains of the historic building is a stone well that is almost filled with debris. Relics have been found, including crude surgical instruments, guns, knives and implements used by Redskins.

Several large mounds in the immediate vicinity are believed to be the graves of frontiersmen... Nathaniel Bedford is known to have been the assistant doctor. Women whose husbands were killed in Indian raids, whose children were scalped and mutilated or kidnapped, worked as nurses. Flax that grew luxuriantly in the green valleys was spun and woven into bandages and the only medicine and carried over the mountains in pack trains or by wandering army scouts.

Friends in misery yet bitter enemies - savage Indians and stalwart bearded frontiersmen - lay side by side in the hospital, sociable in their helplessness. The wounded enemy, bitterly hated and feared, was treated with the same consideration as an ally.

And those humane principals established by General Hand in the first medical institution owned by the youthful government today are part of the traditions and rules of the Medical Corps of the Army.

The Stoops and Other Early Settlers

The following story survives of a local settler who lived in the area during the Indian troubles as related in the History of Allegheny County, 1753 - 1876. The author may have heard it from descendants of the Stoops family who still lived along the creek in his day and long afterwards:

> A family of Stoops built a cabin near Hand's Hospital, and planted a field of corn in the vicinity. During the Indian troubles of 1780 they were accustomed to leaving their family at Ft. Pitt, going out to the field in the morning and returning at evening.
>
> But on one occasion they remained at the cabin at night, having with them one child, William by name. Upon awaking in the morning they found the house surrounded by Indians. There was no opportunity for escape, however, and he thought that if no resistance were offered, his wife and child would be taken prisoners, while he would have time to make a diversion for their rescue.
>
> In this way he was forestalled: however, the Indian party was proceeding on its way, the boy bound to the brave, and mounted on a horse. Mrs. Stoops following on foot with a squaw when Samuel Brady of Brady's Leap, on his return from a journey to Sandusky, observed their movements from a place of concealment. And with the boldness, for which he is

celebrated, shot the Indian with whom William Stoops was riding and rescued his mother. The boy remained in captivity for three years.

The same history mentions John and James Bell, Joseph Hall, David Steele, and Jacob Day as among the other earliest settlers in the valley. Captain Alexander McKee, for whom the Rocks are named, owned 1400 acres at the mouth of the creek adjacent to Steele's. McKee and his neighbor, Matthew Elliott, were Tory sympathizers during the Revolution and they fled the area but continued to stir up the Indians here. The Bell brothers settled on both sides of the creek farther south, James on the Roslyn side and John on the Carnegie side. The Halls settled what is now Greentree.

Whether the Chartiers Valley was to be part of Virginia or part of Pennsylvania, was decided in favor of the latter by the Treaty of Baltimore, which extended the Mason-Dixon line westward. Washington County was established in 1781 and Allegheny County in 1789 when St. Clair Township, of which the Crafton section was a part, was also founded.

Warring with the Indians did not end with the successful conclusion of the revolution, but moved westward. The Indians were not decisively defeated until "MAD" Anthony Wayne's victory in 1792 at Fallen Timbers, near Toledo. By then Western Pennsylvania was no longer the frontier but the stopping place for those going west by wagon, keelboat, or flatboat. The population of Pittsburgh was 376 in 1790 at the time of the first census, about the same as it was in 1761 if soldiers were taken into account; but the census recorded 10,309 in Allegheny County. Life in the Chartiers Valley had become far less hazardous without the threat of raids by unfriendly natives, so that those farmers who settled here could concentrate on raising and selling their crops to those passing through or those living at the river forks. Thus began the age of road development and commercial river traffic in Western Pennsylvania.

Leave Stop 1. Proceed back to Holiday Drive.

0.3	1.7	Turn right onto Holiday Drive.
0.5	2.2	Holiday Drive and Mansfield Road. Turn left onto Mansfield Road.
0.2	2.4	Turn right onto Poplar Street.
0.1	2.5	Turn right onto I-279 (south) on-ramp.
2.7	5.2	Bear right onto I-79/Washington-Erie Exit.
2.0	7.2	Turn right at Exit 13/Carnegie and proceed west on Noblestown Road.
0.6	7.8	Turn right onto Noblestown Extension and turn around.
0.3	8.1	Turn left back onto Noblestown Road.
0.2	8.3	Pull over on right shoulder.

Stop 2. Monongahela Formation/Pittsburgh, Redstone and Sewickley Members
By James Kilburg, IT Corporation

Just south of the Carnegie Exit, Interstate 79 was constructed through a cut that exposed the lower half of the Monongahela Group. At the end of the exit ramp, the rocks immediately in view on the cut slope are the Pittsburgh Limestone of the Conemaugh Group up through the lower part of the Monongahela Group. The Pittsburgh Limestone member extends from the road up to the top of the bench about 15 to 20 feet above the road. Beds on non-marine limestone several feet thick are present on the slope. The top of the slope Geology at the I-79 South Carnegie Exit, where the bench was constructed coincides with the top of the Pittsburgh Limestone and the base of the Pittsburgh Coal.

The Pittsburgh Coal is exposed on the bench at the base of the cut slope. It is 6 to 8 feet thick and it was deep mined in this area. Collapsed mine workings and subsidence-induced slope failures are visible in the Pittsburgh Sandstone on the slope above the cut.

Eastward along the bench (toward I-79), one moves up the stratigraphic section through the Pittsburgh Sandstone and into the Redstone Coal interval. The Redstone Coal typically lies about 70 feet above the base of the Pittsburgh Coal. In the cut adjacent to I-79, the Redstone Coal is a carbonaceous shale that lies 6 to 8 feet above the base of the cut slope. Above the Redstone Coal interval is a buff colored limestone that is probably the Fishpot Limestone and above the Fishpot Limestone is a dark gray interval of shale with a thin coal bed that represents the Sewickley Coal. Viewed from across I-79, the dark gray shale layer contains two broad channels that were likely meander cuttoffs that were backfilled with fine-grained sediments. The Benwood Limestone crops out on the upper part of the cut slope. It is a non-marine limestone that is buff color on its weathered surface and dark brown on fresh surfaces. It is commonly interbedded with shale and claystone, and at some locations in Western Pennsylvania, bluish gray glauconitic sandstone is present.

Figure 2 illustrates the Generalized Columnar Section of the Exposed Rocks in Allegheny County.

Leave Stop 2

0.1	8.4	Turn left onto I-79 (South) on-ramp.
0.2	8.6	Passing road cut that exposes the Monongahela Formation
1.4	10.0	On right below the Chartiers Valley High School is the engineered flood-control channel.
0.5	10.5	Exit 12-Heidelberg, bear right.
0.6	11.1	Turn left onto Route 50.
0.3	11.4	Turn left onto Steen Road.
0.3	11.7	Turn left into entryway to Eichleay.

Stop 3

At this stop we will again look at the Pittsburgh Coal and the upper Conemaugh Formation (Little Pittsburgh Member) and discuss the modification to Chartiers Creek (The James Fulton Flood Control Project).

The flood control project (improvement) consisted of channel deepening, widening and realignment of the creek for an 11.2 mile reach which extends from the railroad bridge at Scully Yard located 3.4 miles above the mouth to the upstream end of Bridgeville Borough.

The project's design considerations were based on the maximum flood of record, 22,000 cubic feet per second (cfs) and 18,000 cfs at the respective downstream and upstream limits of the project. The maximum of record occurred on September 2, 1912.

The section of interest begins at a cutoff (meander loop) channel about 17,000 feet long. Both ends of the auxiliary channel were "improved". The original stream was transformed into two channels, the original channel, which is now an "auxiliary" channel and an engineered "cutoff" channel. Water entering these two channels does so by virtue of a multiplate arch and a three-foot high weir at the upstream end of the cutoff. The auxiliary channel carries a portion of the creek flood discharge around the existing alignment while the bulk of the normal flow and flood flow is diverted through the cutoff channel. Further use of the auxiliary channel is achieved by limited improvement in the upstream 6,000 feet to permit entry of low flows for water supply purposes and to discourage channel encroachment. The downstream 2,000 feet of auxiliary channel is transitioned from the natural to the excavated on a 0.66 percent grade. The cutoff channel, because of its abbreviated length, required a drop structure to overcome the change in grade needed to meet design considerations at both ends. Plan views of this project are provided at the end of this field log.

Leave Stop 3

0.0	11.7	Turn left out of Eichleay entrance and proceed to the Trader Jack's Flea Market on the left.
0.4	12.1	Turn left into flea market's main entrance and continue straight along the main road until it turns left. Continue straight and then bear right.
0.1	12.2	Pull off.

Stop 4

At this stop we will view the weir installed by the Corps of Engineers as part of the flood control efforts along Chartiers Creek. The auxiliary (original) channel is bypassed, and from this location, we can see the old entrance to the auxiliary channel by the culvert. A local resident indicated that during low flow conditions, the water in the bypass loop becomes stagnant. Here we will look at the arch and weir and discuss this flood control effort.

The multiplate arch was sized to discharge 2,200 cfs when the discharge elevation and flow are reached in the main channel. This discharge together with the tributaries of McLaughlin and Painter's Run would amount to 3,200 cfs at the lower end of the loop. The multiplate arch is set at 9 feet 2 inches high with a 20-foot span. The inlet is set in a vertical headwall and the outlet control is governed size of the culvert and gradient.

Leave Stop 4

0.1	12.3	Departing from the flea market lot, turn right onto Thomas Run Road.
0.5	12.8	Turn right into Chartiers Valley High School. Proceed up hill and turn around.
0.1	12.9	Depending on activities at the school, we will stop at the gashouse and depart the vans and walk down to the creek.

Stop 5

At this stop we will proceed down to the creek. Be very careful, since there is a precipitous drop to the creek. We will continue our discussion of the engineering efforts in channeling Chartiers Creek and also look at upper Conemaugh Formation. The bench we are standing on is the approximate base of the Monongahela Formation (Pittsburgh Coal).

Leave Stop 5

0.0	12.9	Turn right onto Thomas Run Road.
0.3	13.1	Turn right onto Route 50.
0.2	13.3	Turn left into ChemTech parking lot and proceed to the northeast corner. We will get out of the vans and proceed down to the creek.

Stop 6

At this stop we will observe the confluence of the auxiliary channel with the cutoff channel of Chartiers Creek. Notice the difference in flow between these two channels. If the water level is low, we can see a point bar near the confluence of these two creek beds. Also, look upstream of the cutoff channel and see the drop structure and change in gradient as the cutoff channel drops in elevation to meet the grade of the auxiliary channel. On the east side of the auxiliary channel, there are several outbreaks of abandoned mine drainage.

Rocks of the Casselman Formation of the Conemaugh Group underlie the valley floor. Just above the valley floor, at the base of the Monongahela Group, is the Pittsburgh Coal, which is overlain by the Pittsburgh sandstone. Structurally, the area is part of the Carnegie/Ninevah syncline, which has a meandering axis striking N/NE. Structure contours are amoeboid but generally strike NW and dip at a low angle toward the SW.

Leave Stop 6

As we leave the parking lot, note the historical marker for the Neville House across Route 50. This building was the second Neville House; the first, located nearby on Bower Hill was burned July 17, 1794 during the Whiskey Rebellion by angry farmers. Early settlers' and farmers' had only one cash crop-whiskey. Rye could not be easily transported down the river or over the mountains on a packhorse, but whiskey made from rye could be. It was the sale of whiskey that provided cash for farmers. Thus most of the citizens of Chartiers Valley and much of western Pennsylvania, except for the Quakers and teetotalers, opposed Hamilton's federal tax imposed on whiskey.

0.1	13.4	Turn right onto Route 50
0.4	13.8	Enter Scott Township.
0.2	14.0	Turn right onto Greentree Road. Structurally, this area is the center of a closed low.
0.2	14.2	Turn right onto Old Washington Pike.
0.2	14.4	Bear right and proceed down hill to Woodville, PA. a village built on a Quaternary-aged sand and gravel terrace. Also significant about this town is that it is not only crossed by the that the axis of the Carnegie syncline, but it is also home to a small abandoned oil field (#37).
0.2	14.6	Proceed to end of road.

Stop 7

At this location we will walk along the single-line railroad tracks and discuss the general geology of the area as well as observe several mine breakouts, a collapsed mine adit, and an exposure of the Pittsburgh Coal. We will then travel by foot to an upper-level train track to gain access to the Scrubgrass Run abandoned mine treatment ponds to discuss the origin and progress of the Scrubgrass Water Project.

Scrubgrass Water Project Overview

By Bob Hedin, Hedin Environmental

The Scrubgrass Project, originally begun in September, 1994, was a collaborative effort originating with students from Chartiers Valley High School. The students were tasked with understanding the history, the problems, the science, and the solutions (including funding) for the site.

The source of the mine water at Scrubgrass is the old Nixon Coal Mine (Consolidated Mine Company) that opened in 1907. In 1936, the mine closed and ultimately filled with water. Water from the mine, that used to discharge directly to Scrubgrass Run, is now intercepted by the Scrubgrass passive treatment system.

Discharge of the mine water is about 300gallons/minute. Water chemistry shows that the water is basically an alkaline iron discharge with a pH above 6.0. Untreated, the mine water is detrimental to healthy streams because 1) the iron hydroxide precipitates, coating the bottom of the stream and smothering the benthos and 2) the chemical reaction that produces the iron precipitate consumes oxygen in the stream. The raw mine water contains about 75 mg/L Fe and 200 mg/L alkalinity.

The treatment system, originally designed to be much larger, consists of two ponds. The treatment consists of 1) providing oxygen to promote the formation of iron hydroxides, 2) providing residence time to allow the iron hydroxide to precipitate, and 3) promoting the retention of oxygen in the discharge water. Although the system works, its efficiency is diminished, probably due to its reduced size. An aerator was installed to accelerate the reaction time of hydroxide formation but the system still lacks residence time to precipitate the hydroxides before discharging to Scrubgrass Run.

Plans for the site continue. A new grant will provide funding for removing the precipitate from the existing ponds and then deepening and lining them. Plans are also being made to market the sludge to pigment manufacturers where similar sludges are already being used. Lining these ponds will not only promote ease of sludge collection but will reduce contamination, rendering the sludge more marketable.

Return to vans and depart Stop 7

0.1	14.7	Intersection at Old Washington Pike.

Alternate Route and Field Trip Stops

0.0	00.0	Turn right onto Old Washington Pike.
0.2	00.2	Scrubgrass Road on right. Building on top of hill may have been the mining office building for the Nixon Coal Mine.
0.5	00.7	Turn left onto Vanadium Road.
0.4	01.3	Turn right onto Bower Hill Road.
0.3	01.6	Turn left onto Painters Run Road.
0.3	01.9	Mine adit on right that is closed with concrete wall.
0.1	02.0	Pull off on right side.

Stop A-1

At this stop we will briefly look at some mine subsidence problems and the overlying bedrock deformation. Because of the narrow road and heavy traffic, we will view from a distance. We will continue down Painters Run to allow viewing of the subsidence in the cut.

Leave Stop A-1

0.6	02.6	McMillan Road to right, note sandstone bed in cut on right.
0.1	02.7	Turn left into parking area and turn back onto Painters Run and head back west.
0.1	02.8	McMillan Road intersection and sandstone unit on left.
0.9	03.7	Turn right into the parking lot for the Sunoco gas station and mini mart and park on the right.

Stop A-2

At this stop we look at a couple of mine adits that have been closed and the Pittsburgh Coal. This valley is a structural trough and the Pittsburgh Coal is probably 10-12 feet thick in this area.

Leave Stop A-2

0.1	03.8	Turn right onto Bower Hill Road.
0.3	04.1	Turn left onto Vanadium Road.
0.6	04.7	Turn right onto Main Street (turns into Old Washington Pike).
0.0	04.7	Turn immediately into the Bower Hill Voluntary Fire Department parking lot.

Stop A-3

At this stop we can view the Pittsburgh Coal, a black carbonaceous shale that contains some plant fossils. Above the shale is the Pittsburgh Sandstone and one can look at the cross-bedding within this unit.

Leave Stop A-3

As we leave, turn right out of parking lot and proceed on Main Street (Old Washington Pike).

0.7	05.4	Woodville, PA and begin main road log.

Main Road Log

0.2	14.9	Turn left onto Greentree Road.
0.2	15.1	Intersection of Greentree Road and Route 50.

Alternative Road Log

0.0	00.0	Turn Right onto Route 50.
0.1	00.1	Turn Left onto Collier Street.
0.4	00.5	Pull off on the right side. We will get out and walk to the overpass.

Stop A-4

At this stop we can see the road cut from Stop 2 and get a nice view of the upper part of the road cut. The dark shaley unit is the Sewickly sandstone and shale. Above, the tan colored rocks are part of the Benwood Limestone.

Leave Stop A-4

0.3	00.8	Turn right into the Waterford/Nevillewood Townhouse unit and turn around and proceed back to Route 50.
0.7	01.3	Turn right onto Route 50.
0.1	01.4	Intersection of Greentree Road and Route 50 and continue south on Route 50.

Return to main road log.

1.2	16.3	Turn right onto I-79 onramp toward Erie.
3.2	19.5	Road cut on left, notice channel fill toward end of road cut. These are sandstones and siltstones of the upper Conemaugh Formation (Little Pittsburgh Member?).
2.2	21.7	Note new develop on left and coal outcrop underneath.
3.2	24.9	Pittsburgh Red Beds exposed on left.
2.2	25.1	Pittsburgh Red Beds on left and right.
2.6	27.7	Ohio river.
0.3	28.0	Bear right onto Emsworth/Sewickley Exit and follow route for Sewickley. Notice outcrops through this exit.
0.4	28.4	Connect with Route 65 North toward Sewickley.
0.5	28.9	Outcrop on right. Poison Park on the left on Neville Island.
2.1	31.0	Borough of Sewickley.
2.7	33.7	Turn left at traffic light onto Ferry Street, cross railroad tracks.
0.1	33.8	Turn right onto First Street.
0.3	34.1	Turn left onto West Park Road.
0.1	34.2	Bear right. Notice rubble on right side. This is debris from Three Rivers Stadium.
0.2	34.4	Turn right into gravel parking area and Leetsdale Dam Construction Site.

Stop 8

ENGINEERING GEOLOGY AND FOUNDATION DESIGN FOR THE BRADDOCK DAM IN-THE-WET CONSTRUCTION PROJECT

by Brian Greene and Kathleen Smyers, Geologists, Pittsburgh District

The new Braddock Dam (replacing existing Monongahela River Dam 2 located directly upstream from Pittsburgh, PA) will be built without the use of cofferdams using innovative in-the-wet construction techniques. In-the-wet technology was initially developed for offshore drilling platforms and immersed tube tunnels. The work at Braddock will be the first time that a navigation dam on an U.S. inland waterway will be constructed using in-the-wet technology. Pittsburgh is currently the largest inland waterways port in the United Sates and the new dam at Braddock will be an important component of the region's navigational infrastructure. The dam's construction procedure calls for fabricating two large concrete segments at an off-site location, which consist of reinforced concrete shells. The segments will then be floated from the fabrication site to the construction area. Once a segment is precisely positioned, it will be sunk onto a preinstalled drilled shaft foundation. After each dam segment is set into place, tremie concrete will be pumped into its internal hollow chambers, displacing water from the interior of the segment. The remainder of the work that is above water will be accomplished using traditional construction methods.

The foundation design for this project calls for eighty-nine reinforced concrete drilled shafts, which will carry the weight of the dam and transfer loads into the bedrock.

Each shaft will be 78-inches in diameter (with a 72-inch rock socket) and about 30 feet in length. Almost 16 feet of each shaft will be drilled into bedrock to assure a secure connection and transfer of loads. The rock units that encompass the rock socket include a soft clayshale and an underlying siltstone layer. The tips of all shafts will bear on the more competent siltstone.

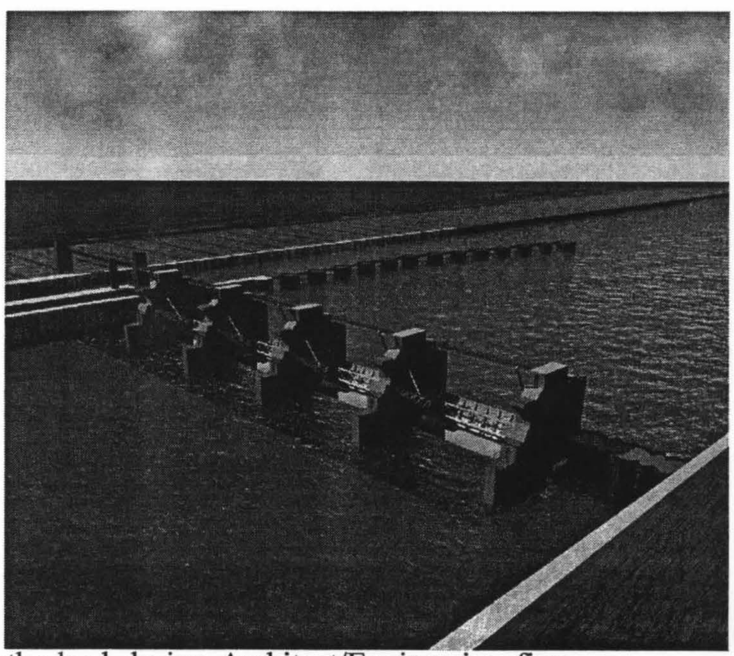
Prior to completion of the foundation design, a very innovative-drilled shaft load test of two prototype-drilled shafts was performed in the river along the axis of the proposed dam. The purpose of the test was to determine the axial and lateral loads that the shafts would be subjected to and validate the design size, spacing and proper depth of embedment of the production drilled shafts.

IN-THE-WET CONSTRUCTION OF A NEW GATED DAM BRADDOCK LOCKS AND DAM, MONONGAHELA RIVER, PA

By William R. Miles, P.E.,
Robert B. Bittner, P.E.
and
William Karaffa, P.E.

INTRODUCTION

A new navigation dam will be constructed at Braddock, Pennsylvania. This project represents a significant portion of a $705 million modernization of the lower Monongahela River navigation system. The new Braddock Dam is an important component of the Corps' Civil Works program not only for the benefits it will provide to the Nation's Inland Waterways System, but also for the innovations it will bring about in the way the Corps does business. The new Braddock Dam will be constructed using innovative in-the-wet construction techniques to achieve cost and time savings and improve quality. The Pittsburgh District made the decision to use in-the-wet construction for the Braddock Dam in July 1997. This pioneering effort involved the use of innovative design and acquisition processes to meet the challenges in-the-wet construction would present. The Corps of Engineers and its Architect/Engineer partners successfully completed plans and specifications for the new Braddock Dam in an aggressive fast track schedule. Bergmann Associates was the lead design Architect/Engineering firm,



with subconsultants Ben C. Gerwick and D'Appolonia, employed by the Pittsburgh District for the innovative features of the Braddock Dam construction. On July 1, 1999, the Corps awarded a $107 M construction contract to the Joint Venture of the J. A. Jones Construction Company and Traylor, Bros., Inc. to construct the new Braddock Dam. This paper presents technical information on the innovative in-the-wet aspects of the Braddock Dam construction.

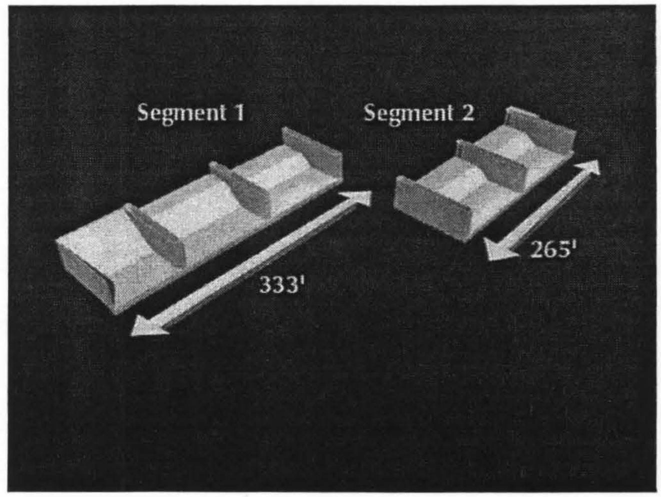
IN-THE-WET OVERVIEW

The new Braddock Dam will be constructed utilizing two large prefabricated concrete segments that will be floated into place above, and set down upon, a pre-constructed foundation system. The foundation system is composed of sheet-pile cut-off walls and large diameter drilled shafts installed in a pre-excavated area upstream of the existing dam. This innovative in-the-wet method of construction is a technique not previously utilized for inland navigation work by the Corps of Engineers. Local and internationally recognized firms specializing in this type of construction were used by the Corps to support the government's engineering team in completing the designs, plans and specifications for this project. Traditionally, inland navigation projects have been built in-the-dry using large temporary cofferdams to dewater the area where the structure was to be built. This permitted all work to be completed within a protected dry work area and when construction was completed, the cofferdam was flooded and removed. The materials used to build the cofferdam would normally be excessed, without salvage value to the Government. Though this method is reliable, the additional costs and construction time associated with this practice prompted the Corps of Engineers to explore better ways of building these large-scale projects faster and at a lower cost to the public, and without sacrificing the quality of the end product. The Corps of Engineers conducted an exhaustive research effort to evaluate successful in-the-wet techniques used for many years for bridge piers, offshore oil and gas facilities, and immersed tube tunnels. The conclusion of this investigation was that many proven techniques, with little or no adaptation, could be used for the construction of inland waterway projects including the new Braddock Dam.

The use of innovative in-the-wet construction for the new Braddock Dam takes advantage of performing many key features of the work concurrently, rather than building the entire dam in a linear fashion within a cofferdam. This maximum use of concurrent operations will save time and thus money. The dam segments will be fabricated at an off-site casting facility in Leetsdale, Pennsylvania using a combination of precast and conventional cast-in-place concrete. Simply put, the dam will be fabricated as two hollow concrete segments that resemble large concrete pontoons. To imagine the scale of these segments, each would nearly cover a (U.S) regulation-sized football field and will be tall enough to reach the tops of the end zone goal posts or both segments would almost cover a rugby field. When completed, these large scale floating structures will be launched and then towed to a staging pier located at Duquesne, Pennsylvania where it will be outfitted before being towed to the Braddock project site, floated into place above, and set down upon a pre-constructed foundation system.

FABRICATION OF FLOAT-IN DAM SEGMENTS

The new dam will be fabricated as two hollow concrete segments designed for flotation. These segments will be constructed off-site location while the dam and tailrace foundation systems are being completed at the project site. The float-in segments will comprise the gate sills, a portion of the stilling basin and a small portion of the pier bases. Dam Segment No. 1 will measure 333 feet along the axis of the dam and will include the fixed weir bay, the water quality gate bay and one of the standard gate bays. Dam Segment No. 2 will measure 265 feet along the axis of the dam and will include two standard gate bays. Each segment will measure 104'-6" upstream to downstream and all gate bays will be 110' wide. The joint between Segments 1 and 2 will occur at Pier 3, which was made 11 feet wider than adjoining piers to facilitate the connection. The float-in dam segments will be constructed with precast wall and diaphragm panels and cast-in-place bottom and top slabs. The individual wall panels of up to 21 feet by 30 feet and 80 tons, will be tied together with cast-in-place closure pours positioned at the intersection points of the walls.



Comprehensive evaluation of acquisition strategies to maximize the flexibility and efficiency of the construction contractor community lead to a District decision not to provide a mandated government-furnished fabrication site for the float-in concrete segments, since an array of options were possible. The contractors were allowed to determine the fabrication and assembly site and methods guided by the solicitation. Designers envisioned three possible options for assembly and launch of the float-in dam segments, including a conventional dry-dock or graving dock, a two level casting basin, and fabrication on a large offshore-type deck barge. Of the six proposals received by the Corps, four were land-based and two were barge-based.

The selected contractor elected to use the option of developing a two level casting basin. Using this method, a segment is assembled on an upper basin, a few feet above normal pool elevation in a protected area behind an earthen berm. A lower exit basin with additional depth is constructed adjacent to the upper basin. An exit structure is built at the end of the lower basin to separate the basins from the adjacent river. When the segment is ready for launch, the upper and lower basins are flooded to a common water level and the segment is floated over the lower basin. The water level in the lower basin is then returned to the outside river elevation, the exit structure is removed and the segment is towed out of the casting facility. Since the dam segments will be transported along the inland waterway navigation system, a maximum draft restriction of 10 feet was stipulated in the contract documents. The Contractor will implement a rigorous program to monitor and control the weight of each segment, as they are assembled, to ensure that the maximum draft requirement is met. Once assembled, each segment will be fitted with transport bulkheads and then launched. Following a commissioning trial, each segment will be towed up river to the outfitting facility located upstream of the immediate project site. All handling and transport will be by towboats.

DAM FOUNDATIONS

While the two float-in dam segments are being fabricated off-site, work at the project site will be progressing to complete the dam foundation system. The basic foundation system is comprised of upstream and downstream cut-off walls, a graded gravel base and a grid of reinforced concrete drill shafts that extend from the riverbed into bedrock.

Eighty-eight reinforced concrete drilled shafts will carry the weight of the dam and the operating loads experienced by the dam into the bedrock. Each shaft will be 78-inches in diameter and about 40 feet in length. Almost 15 feet of each shaft will be drilled into the bedrock to assure a secure connection and transfer of loads. Pre-excavation for the dam foundation will occur first and will consist of excavating the river bed from the existing lock river wall to the left bank abutment toe. This will also provide the necessary draft for the delivery, positioning and set-down of the float-in segments.

After pre-excavation is completed, steel sheet piling will be installed to provide both upstream and downstream cut-off walls and to serve as retaining walls for various stages of work on the dam. A prerequisite pile driving program will be used to determine the elevation of rock to which sheets are to be driven. Sheets may be ordered to accurate lengths once these elevations are determined. Using a barge mounted pile driver, steel sheets approximately 45 feet in length will be installed in 35 feet of water between H-piles with pre-installed interlocks at a 19 foot spacing. The H-piles will be pre-driven to provide for better placement tolerance of the sheets and to shake out the sheets prior to final driving. Followers will be utilized to drive the sheets to the required top elevation without underwater cutting.

The downstream cut-off wall will then be installed in a similar manner. The downstream cut-off wall will be composed of a structural system of 24-inch diameter pipe piles and sheet piles. This system is required to resist the loads imposed by the retained alluvium when the downstream face of the wall is later excavated to rock in order to install downstream scour protection. The pipe piles will be driven to rock from which point a reinforced concrete rock socket will be extended 6 feet into the rock. The pipe piles will also be strategically positioned to support the downstream edge of the tailrace structure. Then a 12-inch layer of 1 ½ inch crushed stone will be placed over the footprint of the foundation area. Once these operations are completed, the dam foundation system will be constructed, comprised of a drilled shaft foundation system for the dam piers and gate sills and a H-pile foundation system for the tailrace.

For the dam foundation system, two types of drilled shafts will be installed. Set-down drilled shafts will support the dead weight of the float-in dam segments at set-down and foundation drilled shafts will support the completed dam. Six (6) set-down drilled shafts will be provided for each dam segment. Seventy seven foundation drilled shafts will be provided beneath the dam piers and gates sills. All drilled shafts will be steptapered with a 72" diameter rock socket drilled through a 78" diameter permanent steel casing. Permanent casings will be driven and seated into the top of the claystone rock layer. Drilling will remove all material from within the casing and the 72" diameter rock socket will then be drilled roughly 6 feet into the lower siltstone rock layer. Drilled shafts will be precisely located using a pile-anchored two-level floating guide template. A 14-inch steel wide flange tension anchor and 36 inch diameter pipe shear pin assembly, which extends above the final concrete top elevation will be installed and the final concrete placement performed in-the-dry. The casing will be refilled and the top of each casing will be cut-off to its final elevation using remotely controlled cutting tools. Two rows of 14-inch H-piles will be driven within the footprint of the tailrace. Piles will be spaced on 8 foot centers and will be driven to the claystone rock layer. The tops of these piles will extend up into the first tremie concrete pour below the tailrace slabs.

TRANSPORT OF FLOAT-IN DAM SEGMENTS

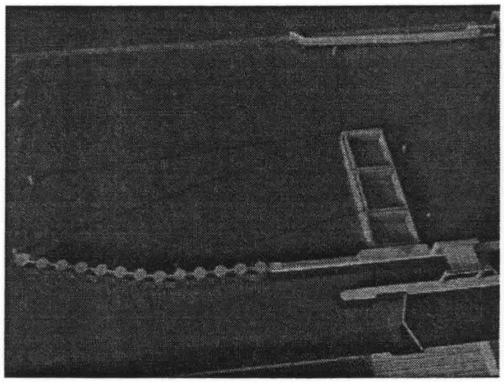
Once each dam segment is completed at the casting basin, it will be towed to the outfitting facility upstream of the Braddock dam site where the segments will be outfitted with positioning equipment and prepared for set-down onto the prepared foundations. The contractor's fabrication site is located approximately 26 river miles downstream of the project site. Transport time is expected to be well within the 48 hour safe period. The segments will require lockage through three locks en route to the site. Prior to actually transporting the segment, all Contractor personnel assigned to the transport operation will perform a simulated towing run using the actual tugboats that will do the towing and a barge tow made up to simulate the size of the segment. By doing this, the Contractor will be better able to understand and adjust procedures and possibly discover any unique difficulties prior to transporting the actual segments.

OUTFITTING OF FLOAT-IN DAM SEGMENTS

Once the segment arrives at the outfitting facility, it will be moored to the outfitting pier and connected to an anchor pile located approximately 500 feet upstream of the outfitting facility. The anchor pile will consist of a 78-inch diameter steel casing with a 72-inch rock socket and is a contingency to protect the segment in the event of a flood while it is moored at the facility. The outfitting facility will also include a braced fendering system at least 15 feet high to keep the segment on the face of the pier during high water. Dam segments will be completed and readied for set-down at the outfitting facility. Piers of each segment will be extended approximately 21 feet. Temporary bulkheads for immersion will be added both upstream and downstream on each gate and the fixed weir bay. Work platforms and vertical tremie pipes for post-set-down underbase and concrete infilling operations will also be installed. Once completely outfitted, each segment will draw 14 feet. Additional water ballast will be added as necessary to trim the segment before transport to the project site.

POSITIONING AND SET-DOWN OF DAM SEGMENTS

Once outfitted, each segment will be transported to the dam site for set-down onto the prepared foundation system. Simulated towing runs will be completed prior to transporting the actual segments. On arrival at the dam site, positioning equipment and cables will be connected to mooring piles and the dam segment will be accurately positioned over its foundations. The mooring and alignment equipment, consisting of cables and mechanical winches, will allow the contractor to make final adjustments to the alignment of each segment as they are lowered down onto their foundations. Then the hollow compartments of the segment will be gradually filled with water to add weight, causing it to sink slowly down onto its drilled shaft foundations.



After each outfitted segment is maneuvered into position, it will be ballasted onto its 6 set-down drilled shafts. A hydraulic flat jack and steel piston is located at each reaction beam. By adding water to the gate bays and using horn guides, mooring lines, snubbing towboats, and land based survey control for guidance, each segment will be accurately lowered into position. Ballast water will be fed into the dry piers until the hydraulic jacks register the required initial loading. A hydraulic ram on the downstream corner of Segment 1, adjacent to the lock wall and in conjunction with the mooring lines, will be used to align the segment on its longitudinal axis. Leveling of Segment 1 to the specified vertical tolerance will be accomplished with the hydraulic flat jack which will be operated to engage the steel piston and reaction beam. The flat jacks will be "locked-off" once the segment is fully aligned and leveled.

UNDERBASE GROUTING AND INFILL OPERATIONS

Following set-down, the 9 to 15 inch area between the underside of each dam segment and the pre-leveled, stone-covered river bottom will be filled with grout to eliminate flow below the dam. To make the underbase grout placement more manageable, the area under segment 1 will be divided into five 70-foot wide transverse strips by using inflatable bags attached to the underside of the float-in segment prior to set-down. Underbase grouting will start at the downstream row of grout pipes and continue upstream until the space below the dam is completely filled. After underbase grouting is completed, the dam segment will be "locked" onto the foundation drilled shafts by pumping a sand/cement grout into the hollow recesses surrounding the shear pin and tension anchor assemblies. Once the segments have been locked onto the foundation, the dam will be infilled with concrete, which will act compositely with the hollow segment. Thirty two compartments in Segment 1 will each be filled with concrete in a two stage operation. The first stage placement consists of filling the bottom 8 to 16 feet of each compartment continuously with tremie concrete while the segment is fully flooded. The second stage placement will be placed in-the-dry after the first stage tremie concrete has cured and the compartment dewatered.

TAILRACE CONSTRUCTION

Following completion of the underbase grouting and concrete infill operations, the dam tailrace will be constructed in-the-wet utilizing precast panels that are 30'-6" wide by 20'-0" long by 15" thick. These panels will be supported on the upstream side by the float-in dam segments and on the downstream side by the row of previously installed pipe piles, which are integral with the downstream cut-off wall. The area below the tailrace panels will be filled with tremie concrete to create a mass concrete tailrace section supported by the previously installed H-pile foundation system. Each panel will be cast with stainless steel armor angles on top surface edges and three stainless steel pipe sleeves through the panel at a 10 foot spacing for tremie placement ports. To limit tremie concrete placements to 500 cubic yards or less, a stiffened steel plate cut-off wall will be attached to the bottom of approximately every fourth tailrace panel. To provide closure to the top of downstream cut-off walls, compressible wire brushes will be connected to the downstream ends of each panel. To minimize uplift pressures on the panels during tremie placement, the tremie will be performed in two stages. The first placement will be limited to an approximate 5 foot lift. Extending into this tremie placement will be a grid of rebar dowels, which will be connected to the underside of the tailrace panels. The dowels will act to provide additional support to the tailrace panels to withstand the uplift pressures caused by the second tremie concrete placement to the underside of the panels.

COMPLETION OF DAM PIERS

The upper 39-feet of each dam pier will be placed in-the-dry after the float-in dam segments have been set-down and filled with concrete. This portion of the dam piers will be completed with a combination of precast concrete and conventional cast-in-place concrete. Eight inch thick precast panels will complete the upstream portion of each pier and will function as a stay-in-place forming system. These panels will be temporarily braced in-place with sacrificial steel bracing until the infill concrete is placed. The downstream third of the pier will be conventionally formed cast-in-place concrete to facilitate the installation of the trunnion girder and cylinder girder anchorages. A pier building will be erected on the top of each pier and will house the mechanical and electrical equipment for the tainter gates and enclose the entrance into the piers interior passageways. Trunnion girders will be attached to the downstream face of each of the five piers for support of the four tainter gates. On the sides facing the gate bays, the trunnion girders will cantilever out from the face of the pier approximately 5'-6". The trunnion girders will be post-tensioned longitudinally and will also be strengthened with conventional reinforcing bars for resisting design stresses. Post-tensioned trunnion anchorage rods will extend from the girder down into the infill concrete to tie each trunnion girder to its pier.

INSTALLATION OF TAINTER GATES

Following the completion of the pier and tailrace structures and prior to installation of the left closure weir, the upstream and downstream bulkheads will be installed using a floating plant for one gate bay at a time. That bay will be completely de-watered and the tainter gate installation begun. After each bay is de-watered, a temporary steel frame will be erected to support individual tainter gate sections while they are assembled. Gate arms will be connected to their anchorages at the trunnion girders anchored to the piers . Once the assembled gate is located in its final position, two hydraulic operating cylinders will be installed and connected to each end of the tainter gate. The cylinders will also be anchored to the upper pier structure. The bay will be flooded and the bulkhead structures will be moved to another bay. All three standard gates and the one shorter and lighter water quality gate will be installed in a similar manner.

CONSTRUCTION OF LEFT CLOSURE WEIR

Following completion of the dam piers and installation of the tainter gates, the left closure weir will be constructed. This structure will connect the left end of the new dam to the existing left abutment wall providing a fixed weir crest at elevation 725. The closure weir will be comprised of two 52 foot diameter sheet pile cells driven down to the top of rock. A connecting arc will be located between each cell and two closure sections between the cells, the new dam and the left abutment will complete the closure. Existing alluvium in the cells will be removed or leveled off and jet grouted and tremie concrete will be placed to the top of the sheet pile cells. After the existing dam has been breached, the lowered water level over the intervening space will allow for the construction of a variable thickness reinforced cast-in-place concrete cap to complete the weir crest.

CLOSING

The advantages of using innovative in-the-wet construction are two-fold. Eliminating the need to build and then remove a cofferdam yield time and cost savings. Construction of the dam foundation system at the same time the dam components are being constructed offsite brings efficiency as well as cost savings to the project. As a bonus, offsite work will not be slowed by flow conditions that would normally affect in-river construction. In addition, with extensive use of high quality precasting operations, the finished dam will be more durable and have an extended useful life.

Leave Stop 8

Follow route back to Ferry Street. NOTE: follow one-way signs to return to Route 65. Note wetland on left.

0.4	34.8	Turn right onto First Street.
0.3	35.1	Turn Right onto Center Street.
0.1	35.2	Turn left at stop sign.
0.1	35.3	Turn left onto Ferry Street.
0.1	35.4	Intersection of Ferry Street and Route 65 (Ohio River Boulevard). Turn left onto Route 65 North.
1.2	37.1	Turn right onto Cross Street (Red Belt). Continue along Red Belt (Big Sewickley Creek Road.

2.2	39.3	Turn right onto Turkeyfoot Road.
0.1	39.4	Pull off at outcrop.

Stop 9

At this location we have an opportunity to collect plant fossils. Be very careful at this site because of the loose overhanging rock.

Leave Stop 9

0.1	39.6	Turn left onto Sewickley Creek Road (Red Belt).
2.2	41.8	Turn left onto Cross Street and continue on the Red Belt. Just before intersection pull off and notice rock slide on adjacent hillside. Road leads up to single residence.
1.2	43.0	Turn left onto Route 65 South (Ohio River Boulevard).
6.3	49.3	Ramp to I-79 North.
0.3	49.6	Turn left, Glenfield exit.
0.7	50.3	Bear right onto I-79 North on-ramp.
0.4	50.7	End of on-ramp.

Alternate Field Stop A-5

Depending on the amount of time available we may stop and hike to the top of the hill and look at some of the slumping and "rock city". See end of guidebook for additional information on some of the engineering problems associated with the construction of I-79.

Leave Stop A-5

0.9	51.6	Pittsburgh Red Beds.
0.4	52.0	Exit 20, bear right.
0.3	52.3	At end of exit ramp, turn right onto Mt. Nebo Road.
3.8	56.1	Bear right.
0.1	56.2	Turn right onto Camp Horne Road.
0.2	56.4	Turn left and proceed up to Giant Eagle/Home Depot.
0.3	56.7	Park near end of lot next to slope and slide area.

Stop 10

At this stop, we will witness active slope failure in colluvium associated with the "Pittsburgh redbeds" owing to both incompetence of the material and man-induced errors. Bedrock ranges from upper Glenshaw Formation to lower Casselman Formation of the Pennsylvanian Conemaugh Group. The thin, highly fossiliferous Ames limestone marks the boundary between the formations; the bulk of the "Pittsburgh redbeds" lies below the Ames in the Glenshaw Formation.

Construction at this site began in fall of 1997. Prior to that, the slope was forested and contained a terrace that was used as a drive-in theatre. Visible along the slope situated between Giant Eagle and Allstate Financial is a series of nested slumps with a debris flow or earthflow toe that is currently saturated and moving. Slump scarps are low, no more than a few feet at most. The toe, roughly 2-3 feet high, is convex in plan and cross section. The crown is ill-defined owing to man-made excavations, but is situated roughly at or just below the Ames limestone. There are numerous open cracks and holes in the crown area and the upper part of the landslide in general. Walk with care!

Contributing factors initiating the landslide and promoting its continuance are as follows:

- Oversteepening of the slope by excavation to make space for a roadway between Giant Eagle and the slope. This may have inadvertently reactivated an ancient pre-existing landslide.
- Dumping waste earth material (largely redbeds) in the crown area at the head of the landslide.
- Inadequate or ineffective surface run-off controls at the head of the slide, contributing to the general wetness of the mass.

At the head of the long northeast-southwest cut-slope between Giant Eagle and Home Depot near the edge of the forest is a lined drainage trough filled with large blocks of Loyalhanna Limestone. Water in the trough is supposed to feed into downslope drains that presumably feed into the storm-water collection system at parking-lot level. This drainage trough has failed in at least two places. The northeast end has been damaged by the Giant Eagle landslide complex. About halfway between Giant Eagle and Home Depot at the top the cut slope, there is a shallow slump-and-flow failure ("spoon"?) that has damaged the limestone-rock drain. Any water in this trough feeds into the head of the slope failure. Which came first: trough failure or slope failure? A small perennial stream intersects the trough very near this point and perhaps is involved in the slope failure. The toe of the "spoon" is totally saturated; a dewatering event produced a very small, yet classic, mudflow beyond the toe. A similar "spoon" about 100 yards or so to the southwest toward Home Depot can be observed.

Leave Stop 10

Depart site and head back to Pittsburgh and Parkway Inn.

0.2	56.9	Turn left out off Giant Eagle/Home Depot.
0.1	57.0	Turn right onto Camp Horne Road.
0.1	57.1	Turn right onto I-279 South.
7.7	64.8	On left is the new PNC baseball park.
0.2	65.0	On right is the new football stadium under construction.
0.2	65.2	Allegheny River.
0.4	65.6	Point State Park.

0.1	65.7	Monongahela River.
0.2	65.9	Fort Pitt Tunnel.
1.8	67.7	Bear right onto Parkway Center Mall exit ramp.
0.4	68.1	Turn right into Parkway Center Inn parking lot.

The End

Thank you for travelling with the Pittsburgh Geological Society. We hope you have enjoyed your trip. Please come back.

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FIGURES

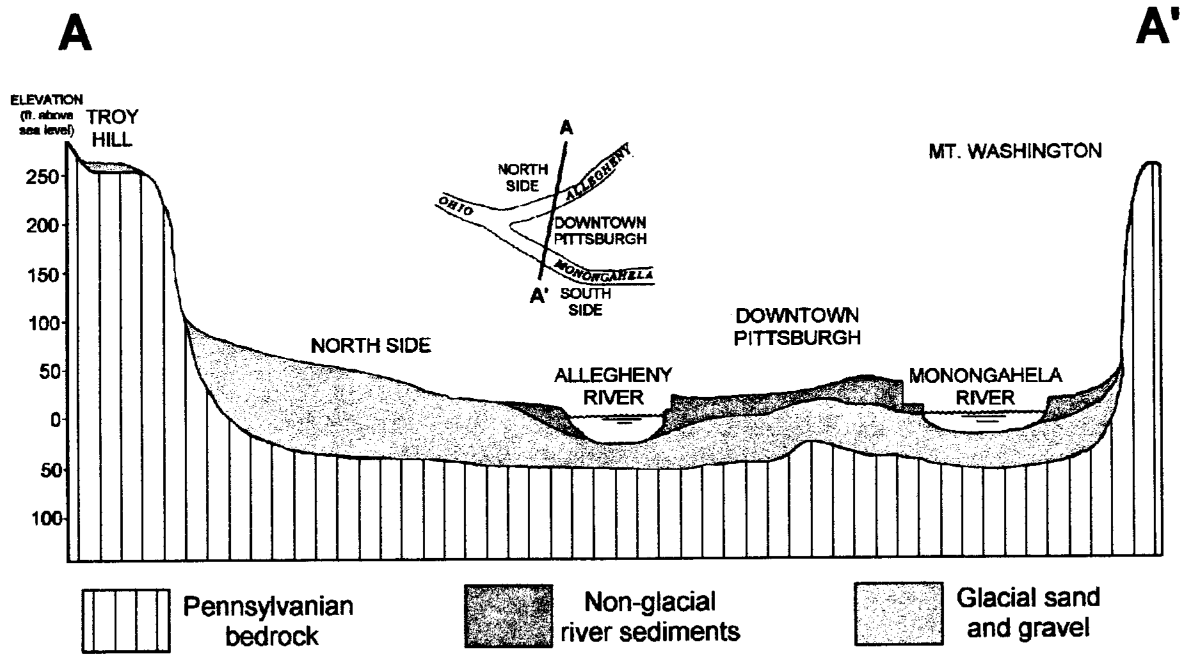


Figure 1. Cross section of the Allegheny and Monongahela river valley across downtown Pittsburgh (modified from Wagner and others, 1970).

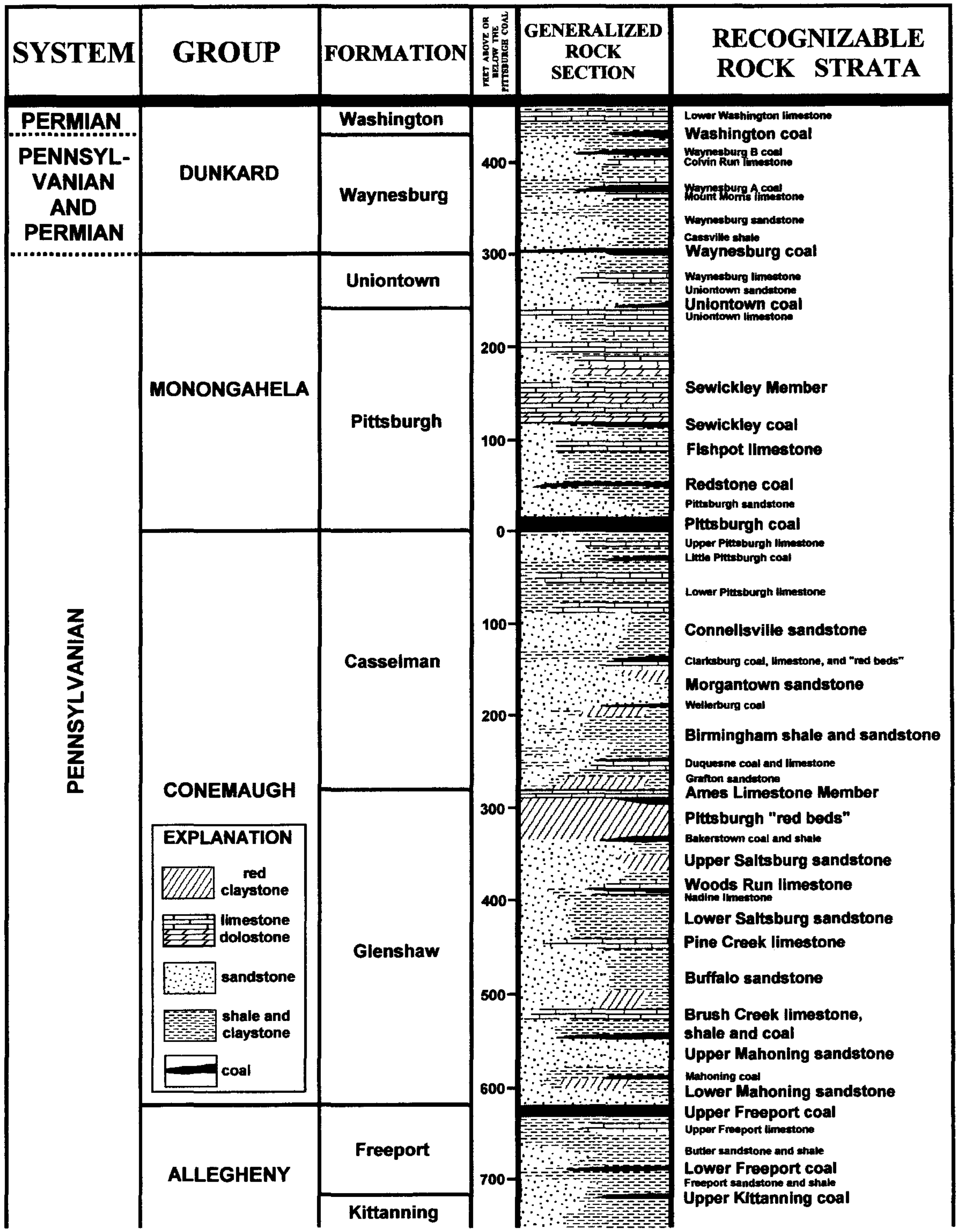


Figure 2. Generalized geologic column of the exposed rocks of Allegheny County.

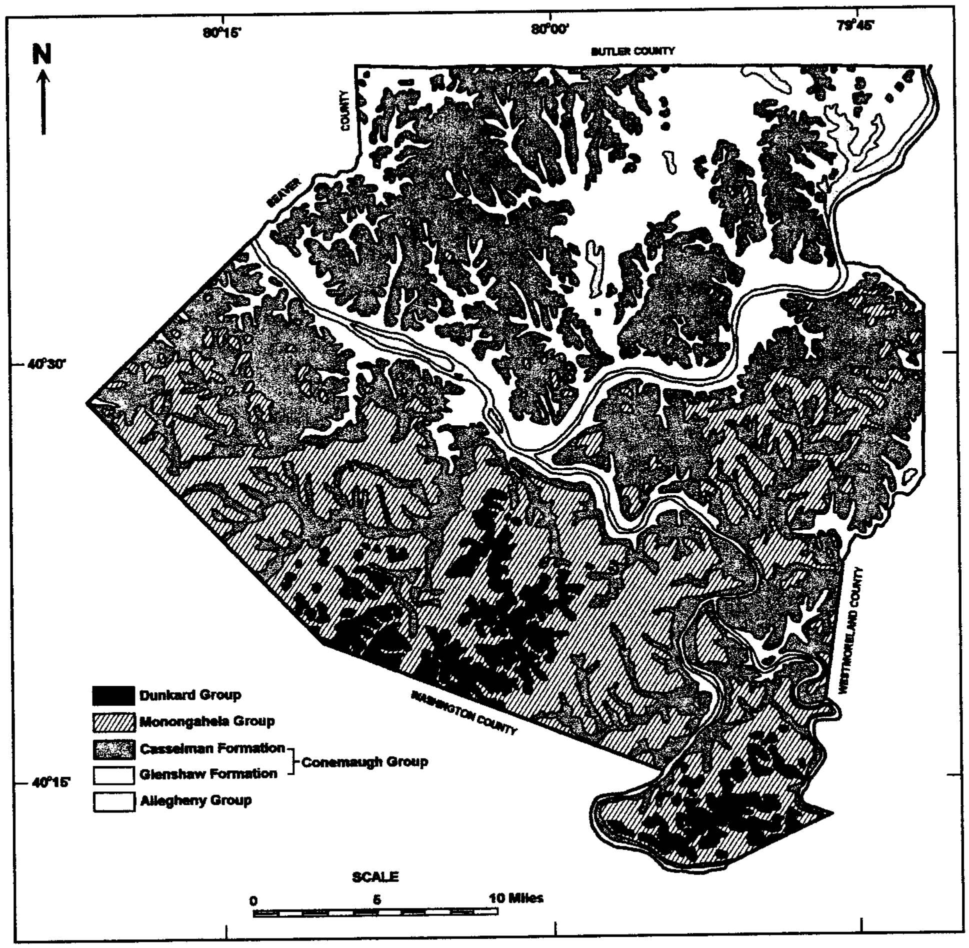


Figure 3. Geologic map of Allegheny County.

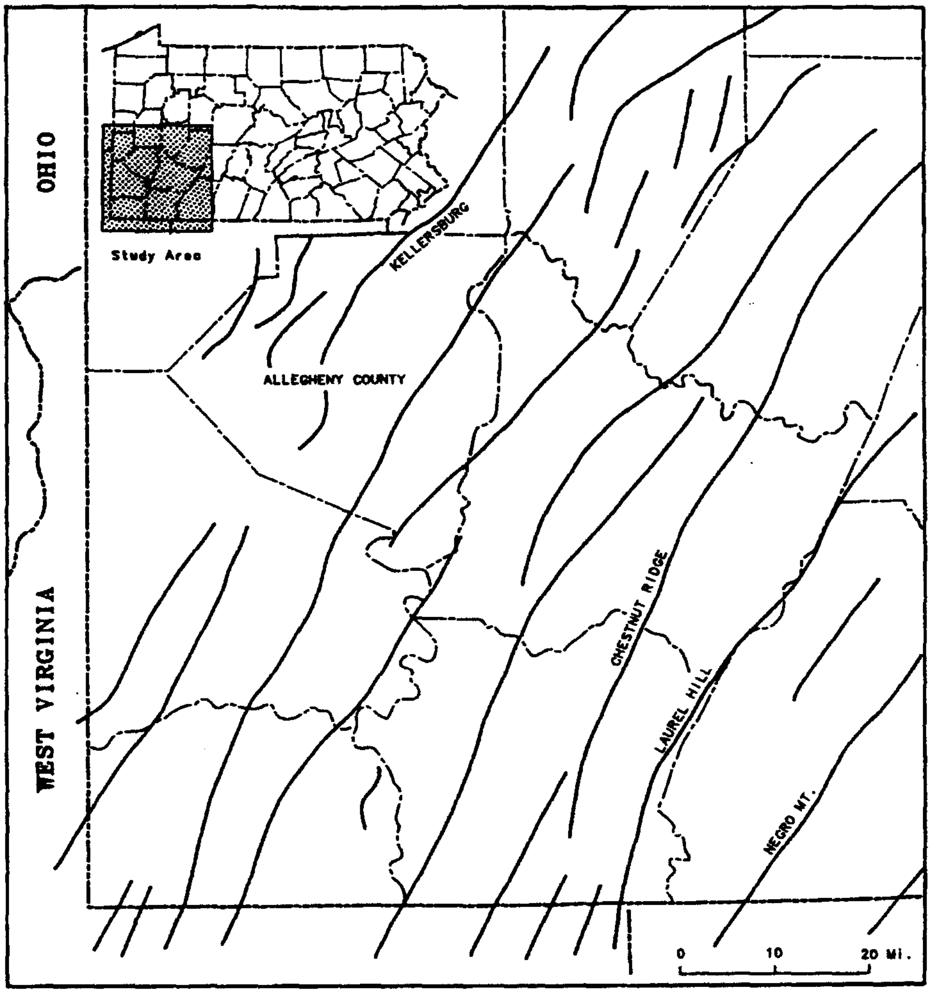


Figure 4. Map of southwestern Pennsylvania showing the trends of the major anticlinal axes. The three most prominent, Chestnut Ridge, Laurel Hill, and Negro Mountain anticlines, form high, deeply dissected ridges. From Harper (1995).

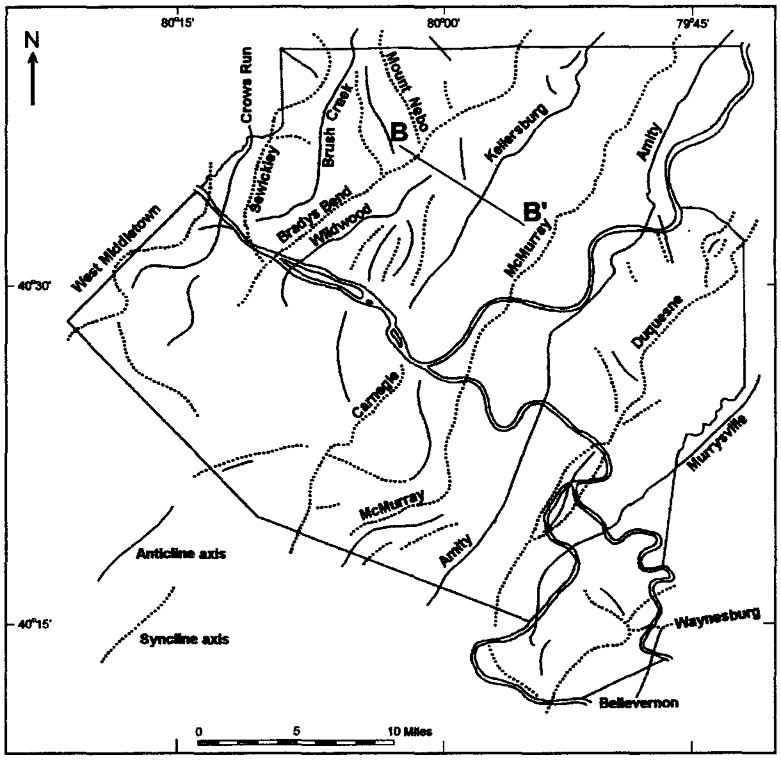


Figure 5. Structure map of Allegheny County (from Harper, 1995). Cross section B-B' is shown in Figure 6.

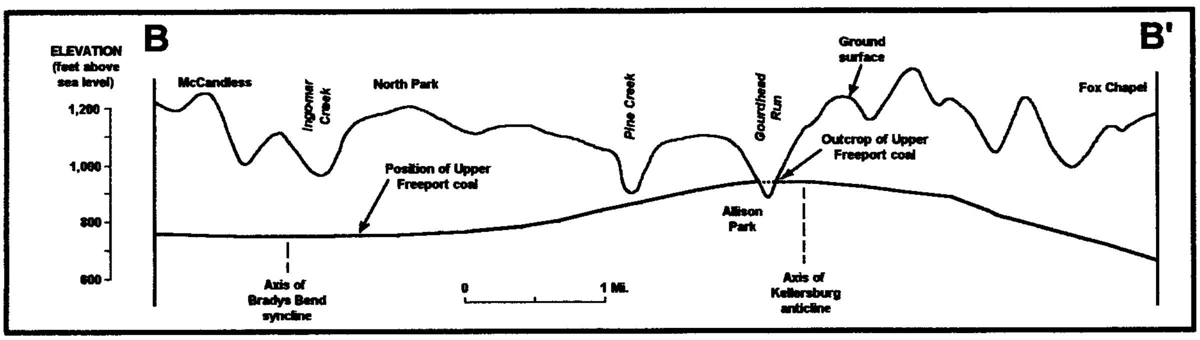


Figure 6. Cross section B-B' across the Bradys Bend syncline and Kellersburg anticline in northern Allegheny County (from Harper, 1995). See Figure 5 for location.

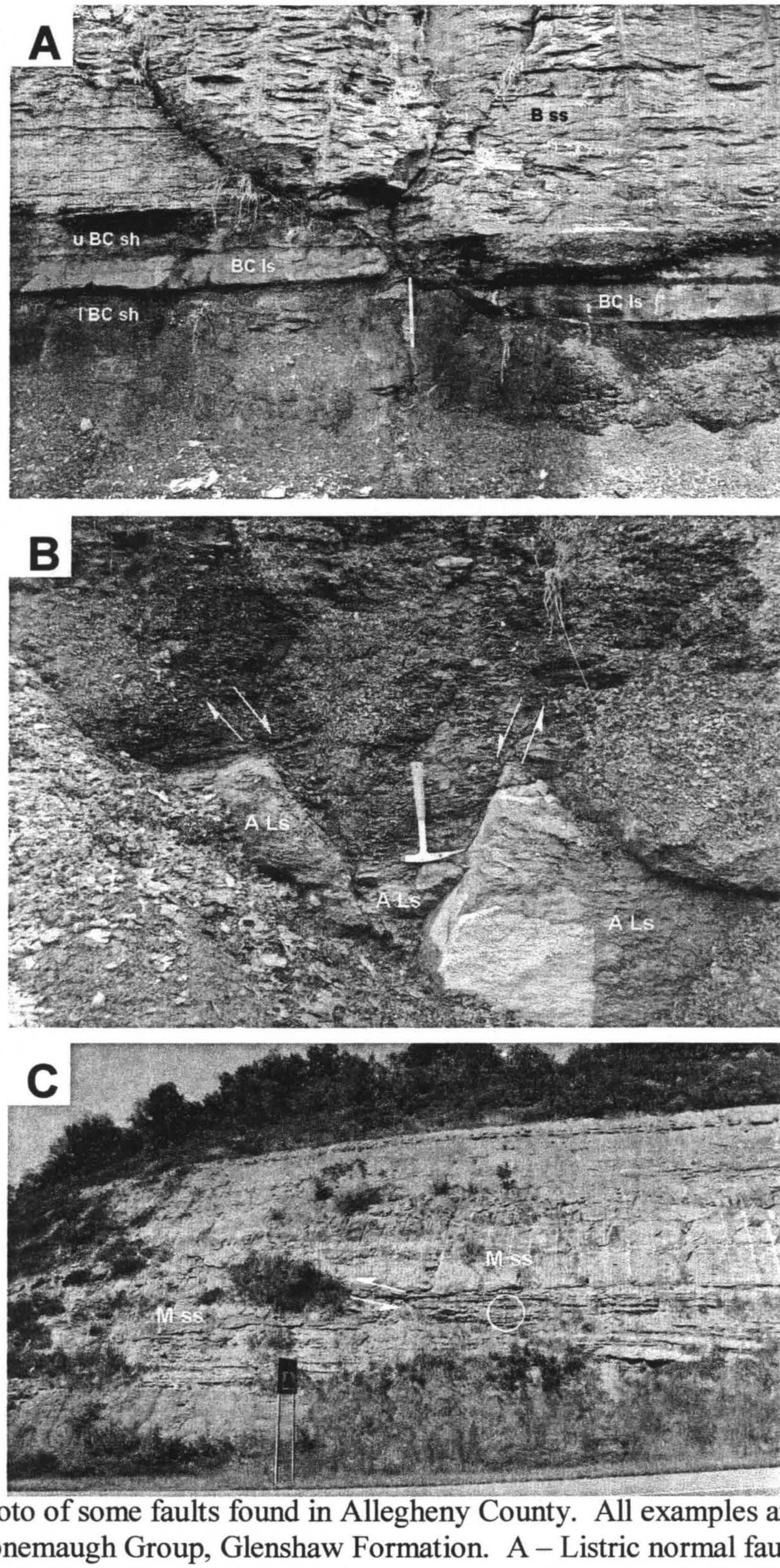


Figure 7. Photo of some faults found in Allegheny County. All examples are from the Conemaugh Group, Glenshaw Formation. A – Listric normal fault in the Brush Creek limestone (BC ls) and upper and lower Brush Creek shales (u BC sh and l BC sh) below the Buffalo sandstone (B ss) along PA Route 51 near the Sewickley Bridge. Scale is one meter. B – Graben formed in the Ames Limestone (A Ls) and adjacent shales along PA Route 28 near Creighton. Rock hammer for scale. C – Thrust fault in the Mahoning sandstone (M ss) at the Tarentum exit of PA Route 28. Scale (in circle) is one meter.

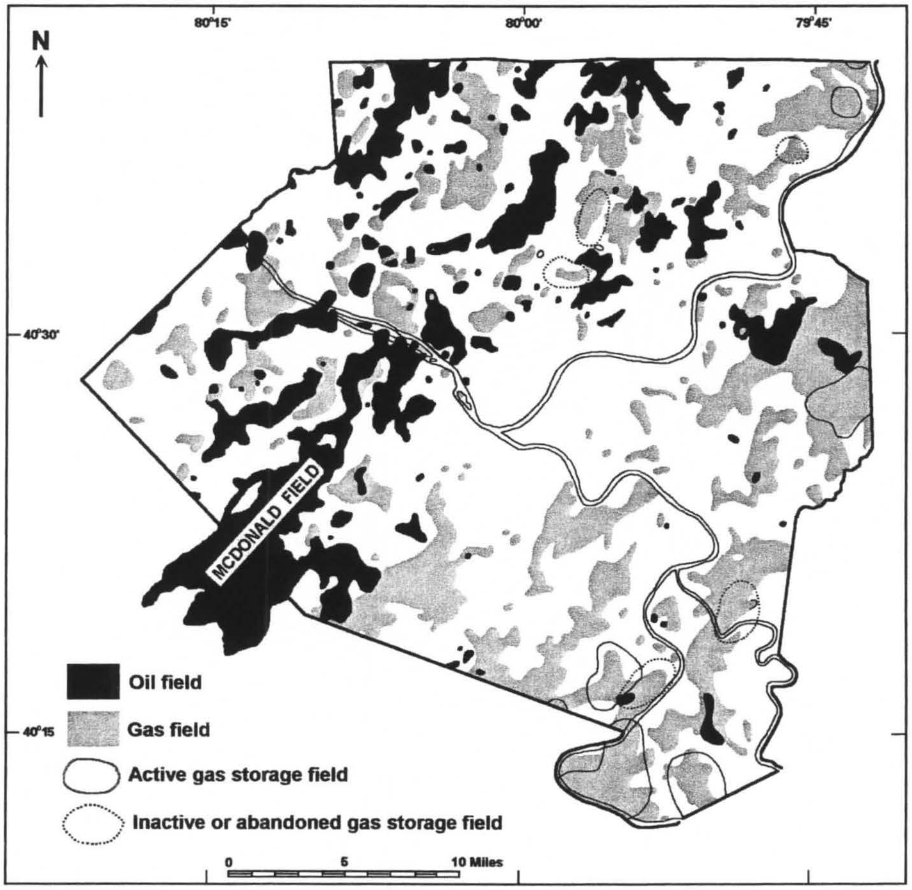


Figure 8. Oil and gas fields of Allegheny County. McDonald field is the second largest oil field in southwestern Pennsylvania. Adapted from Harper and others, 1982.

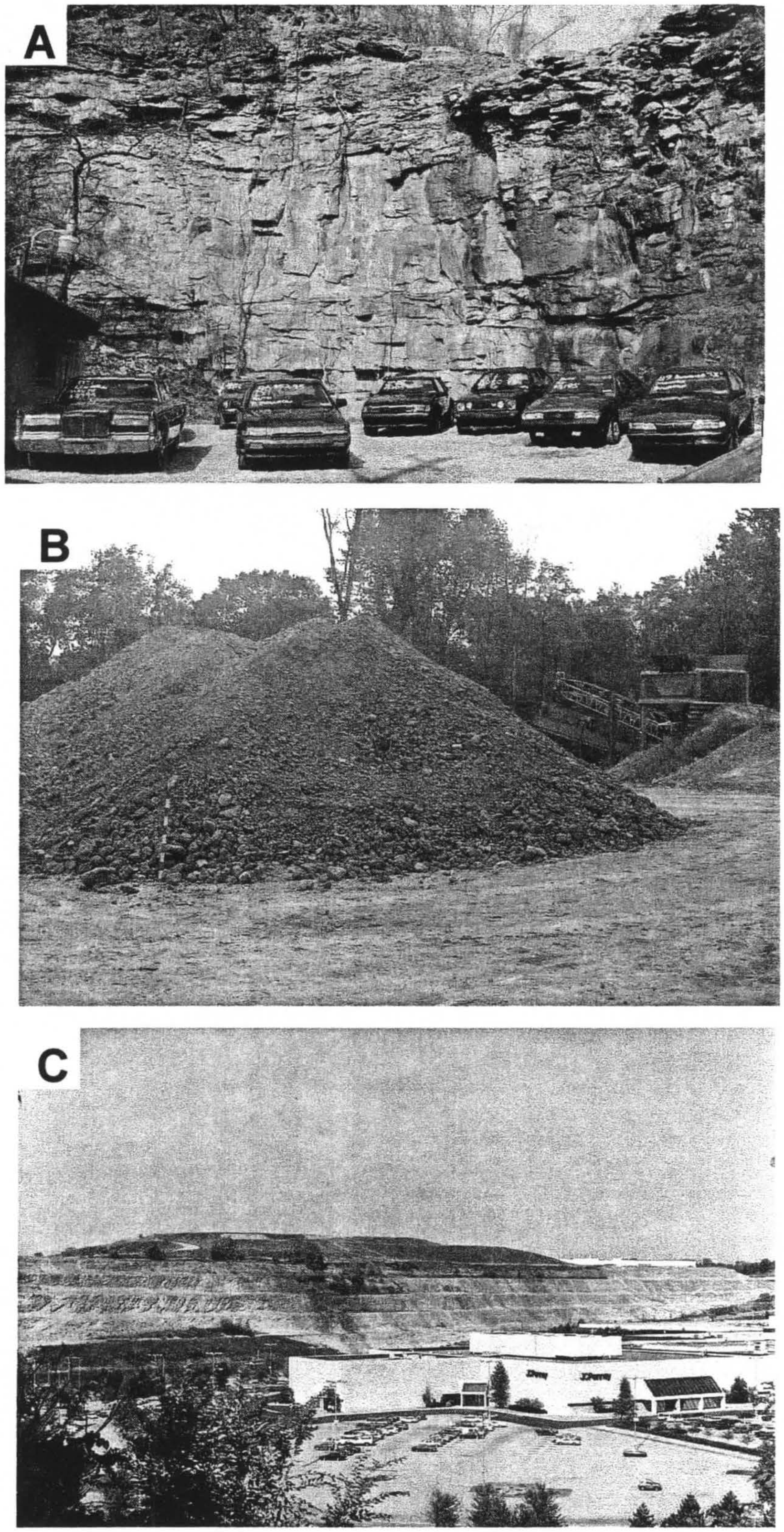


Figure 9. Photos of some of the non-fuel mineral resources found in Allegheny County. A. Sandstone – this particular quarry has been long abandoned, but others are still in operation. B. Sand and gravel – most resources are dredged from the rivers, but some terrace deposits still exist to be exploited. C. Slag – the latest "rock" type to gain acceptance as a commercially viable mineral resource.

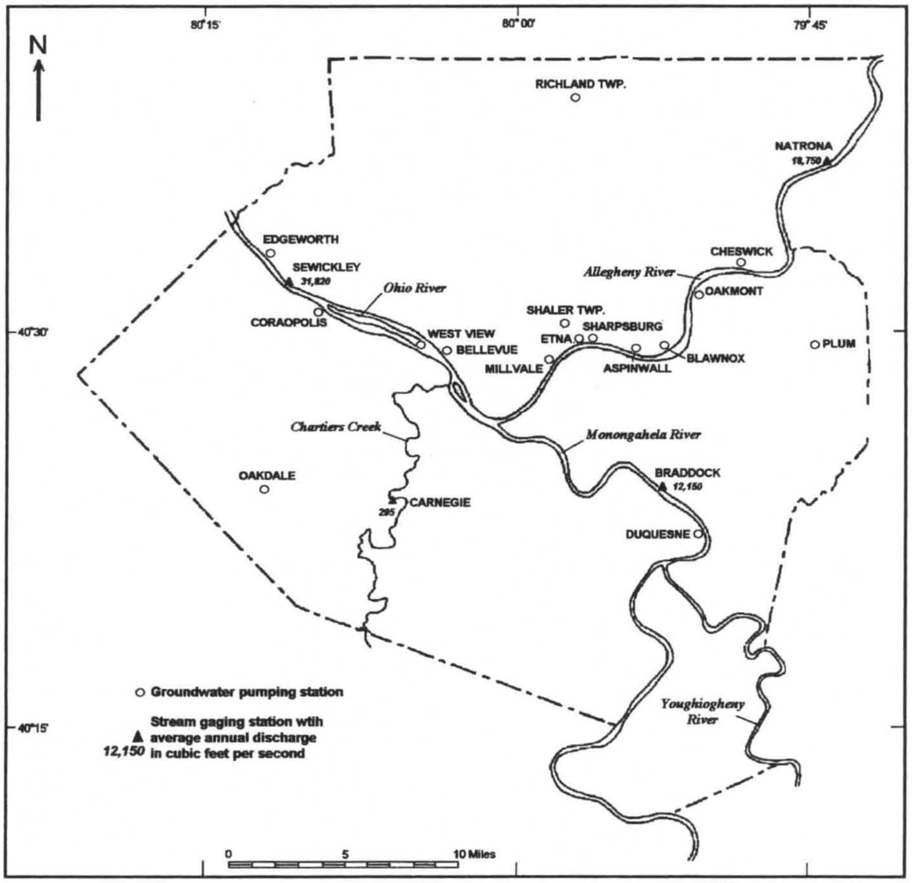


Figure 10. Map of Allegheny County showing locations of stream-gauging stations and of major areas of ground-water pumping by municipalities and townships (modified from Gallaher, 1973).

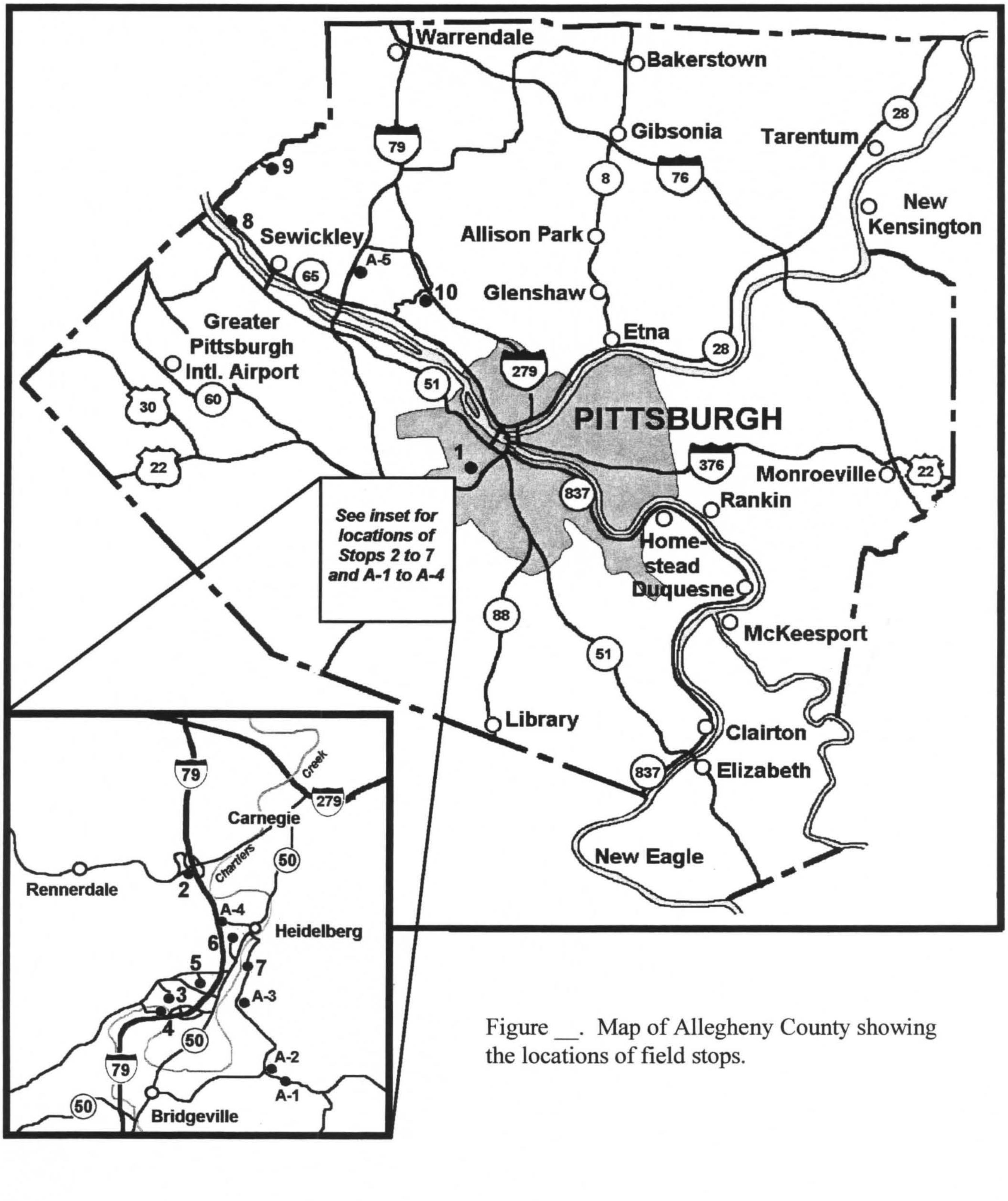
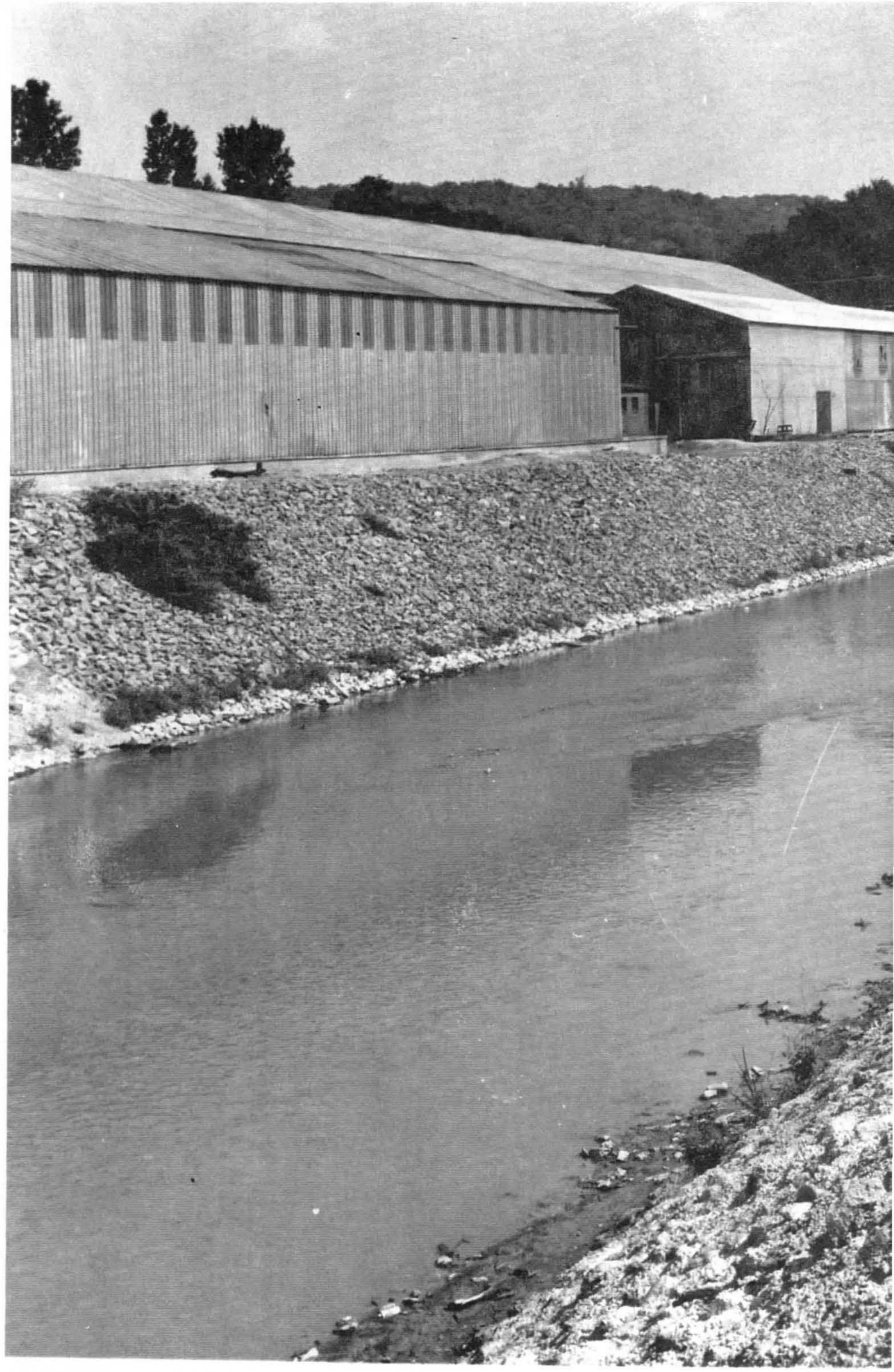
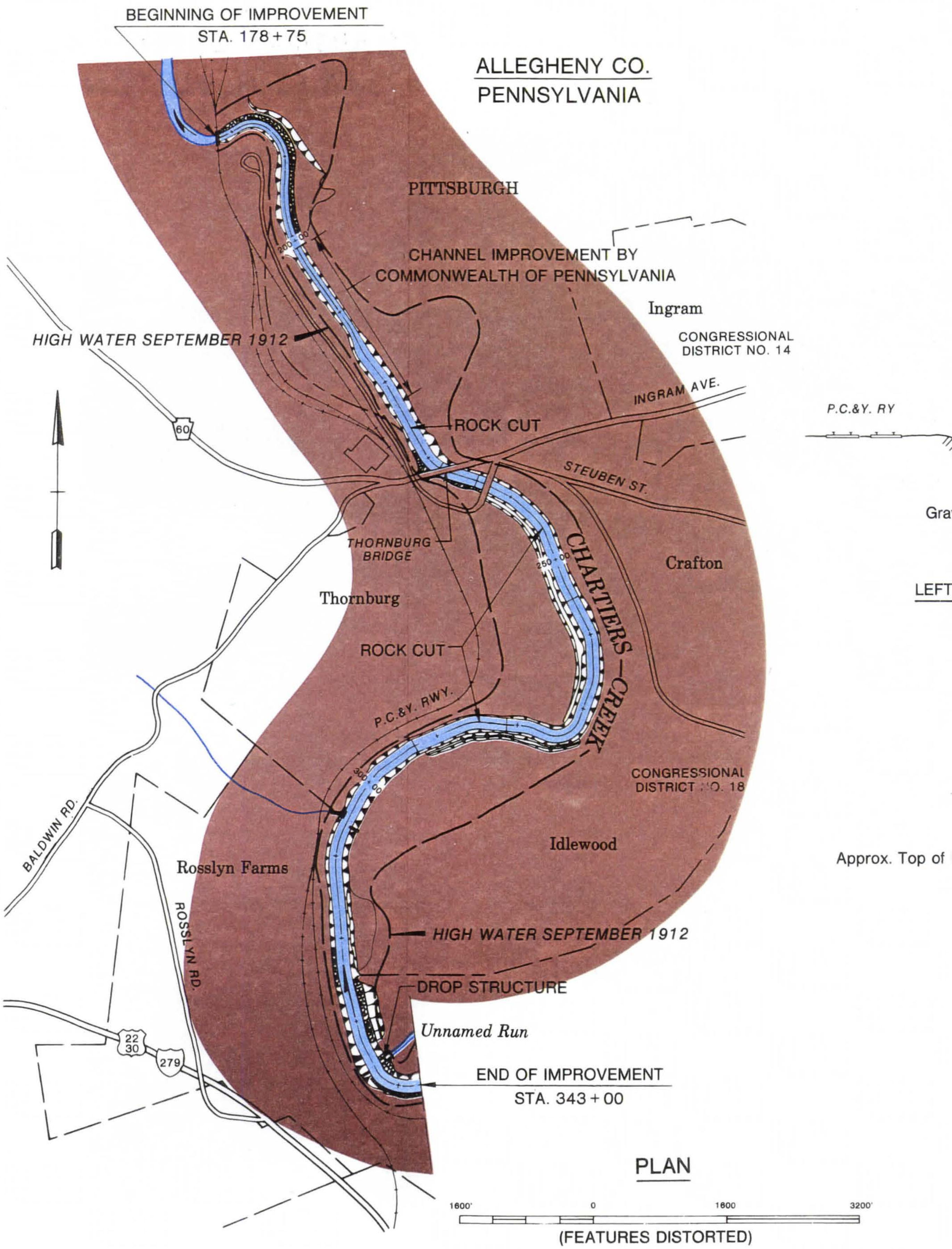


Figure __. Map of Allegheny County showing the locations of field stops.



F/C 94
CHARTIERS CREEK, PA.
JAMES G. FULTON



BEGINNING OF IMPROVEMENT
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ALLEGHENY CO.
PENNSYLVANIA
PITTSBURGH
CHANNEL IMPROVEMENT BY
COMMONWEALTH OF PENNSYLVANIA
Ingram
HIGH WATER SEPTEMBER 1912
CONGRESSIONAL
DISTRICT NO. 14
INGRAM AVE.
P.C.&Y. RY
60
ROCK CUT
STEUBEN ST.
THORNBURG
BRIDGE
CHARTIERS—CREEK
Crafton
Thornburg
LEFT
ROCK CUT
P.C.&Y. RWY.
CONGRESSIONAL
DISTRICT NO. 18
Idlewood
BALDWIN RD.
Rosslyn Farms
Approx. Top of
ROSSLYN RD.
HIGH WATER SEPTEMBER 1912
DROP STRUCTURE
Unnamed Run
22
30
279
END OF IMPROVEMENT
STA. 343+00
PLAN
1600'
0
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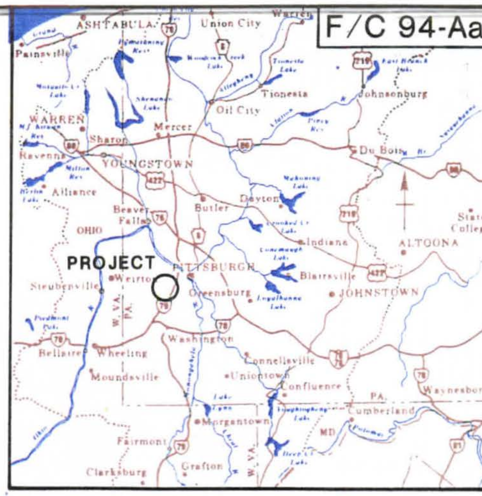
3200'
(FEATURES DISTORTED)

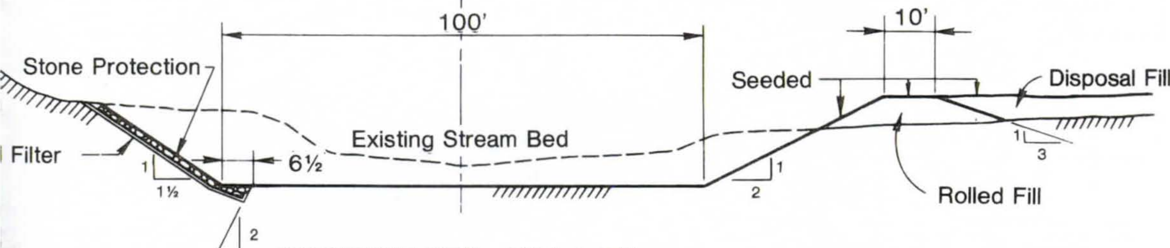
F/C 94-Aa



VICINITY MAP

SCALE OF MILES

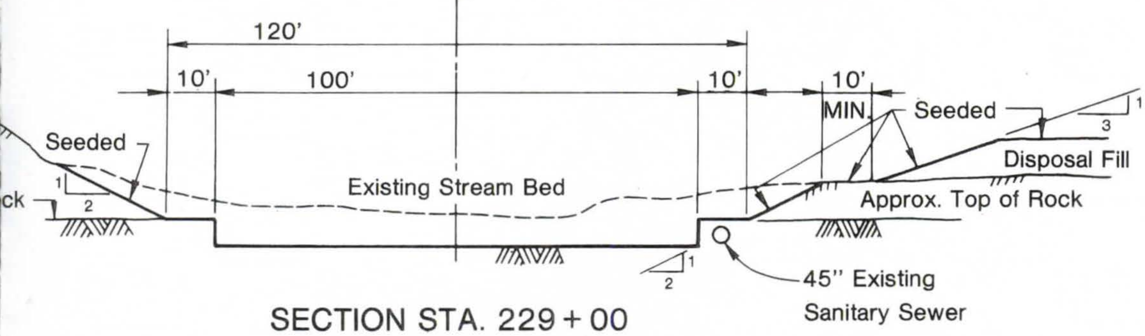
35 0 35 70



SECTION STA. 321 + 95

RIGHT BANK

20' 0 40'



SECTION STA. 229 + 00

LEFT BANK

RIGHT BANK

20' 0 40'

CHARTIERS CREEK, PA.

JAMES G. FULTON

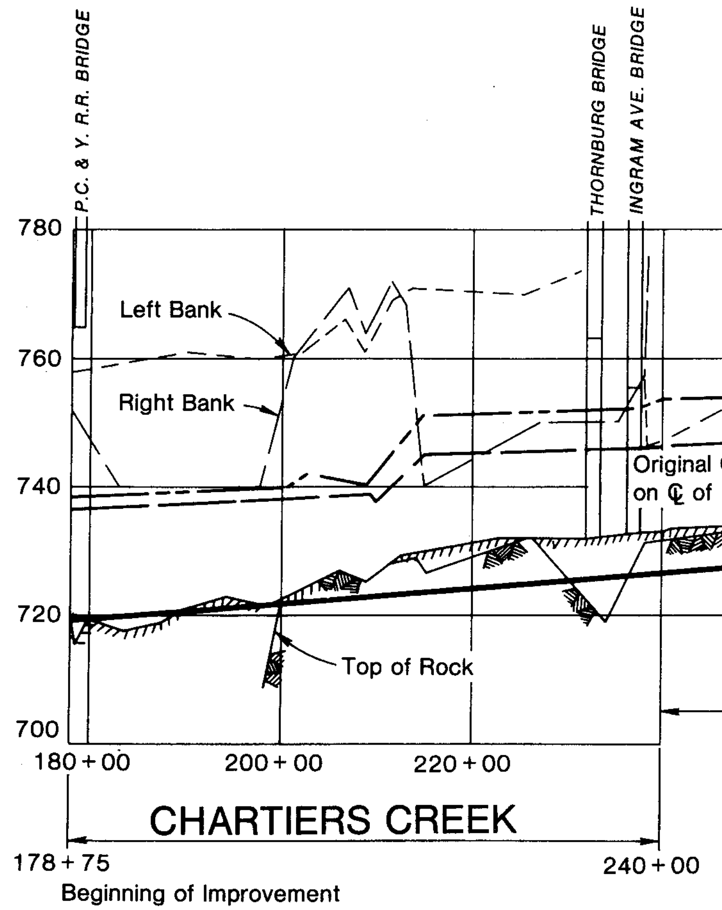
UNIT 1

LOCAL PROTECTION

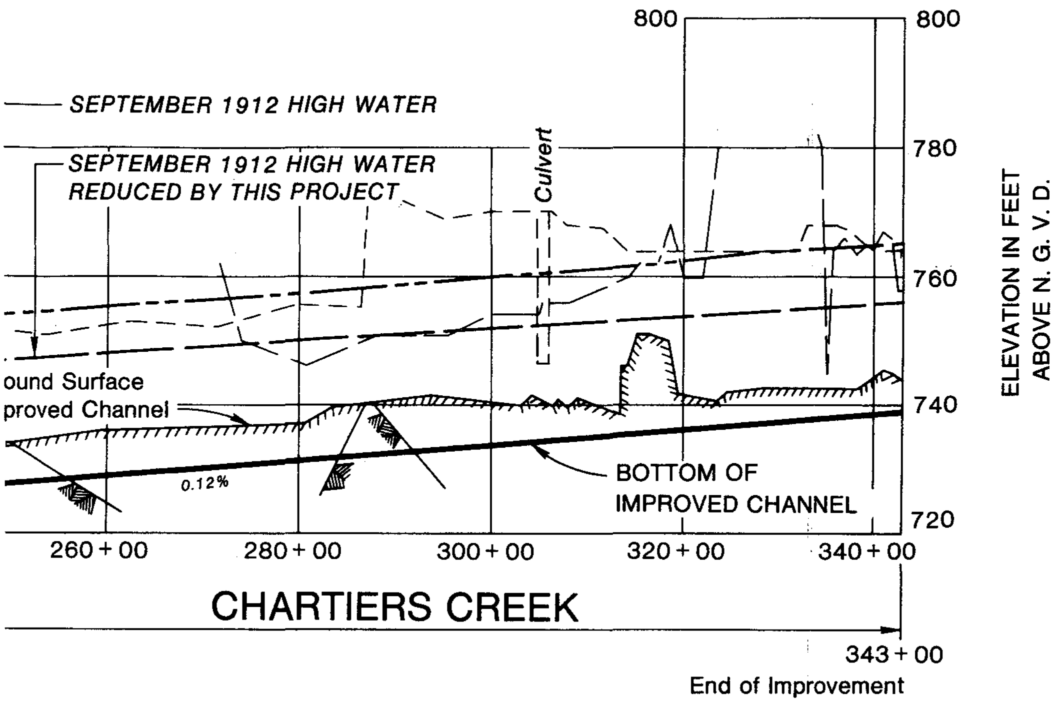
PLAN AND SECTIONS

PITTSBURGH DISTRICT, PITTSBURGH, PA.

Revised: 30 September 1986



P.C. & Y. R.R. BRIDGE
THORNBURG BRIDGE
INGRAM AVE. BRIDGE
780
760
740
720
700
Left Bank
Right Bank
Original
on ℄ of
Top of Rock
180+00
200+00
220+00
CHARTIERS CREEK
178+75
Beginning of Improvement
240+00



CHARTIERS CREEK, PA.

JAMES G. FULTON

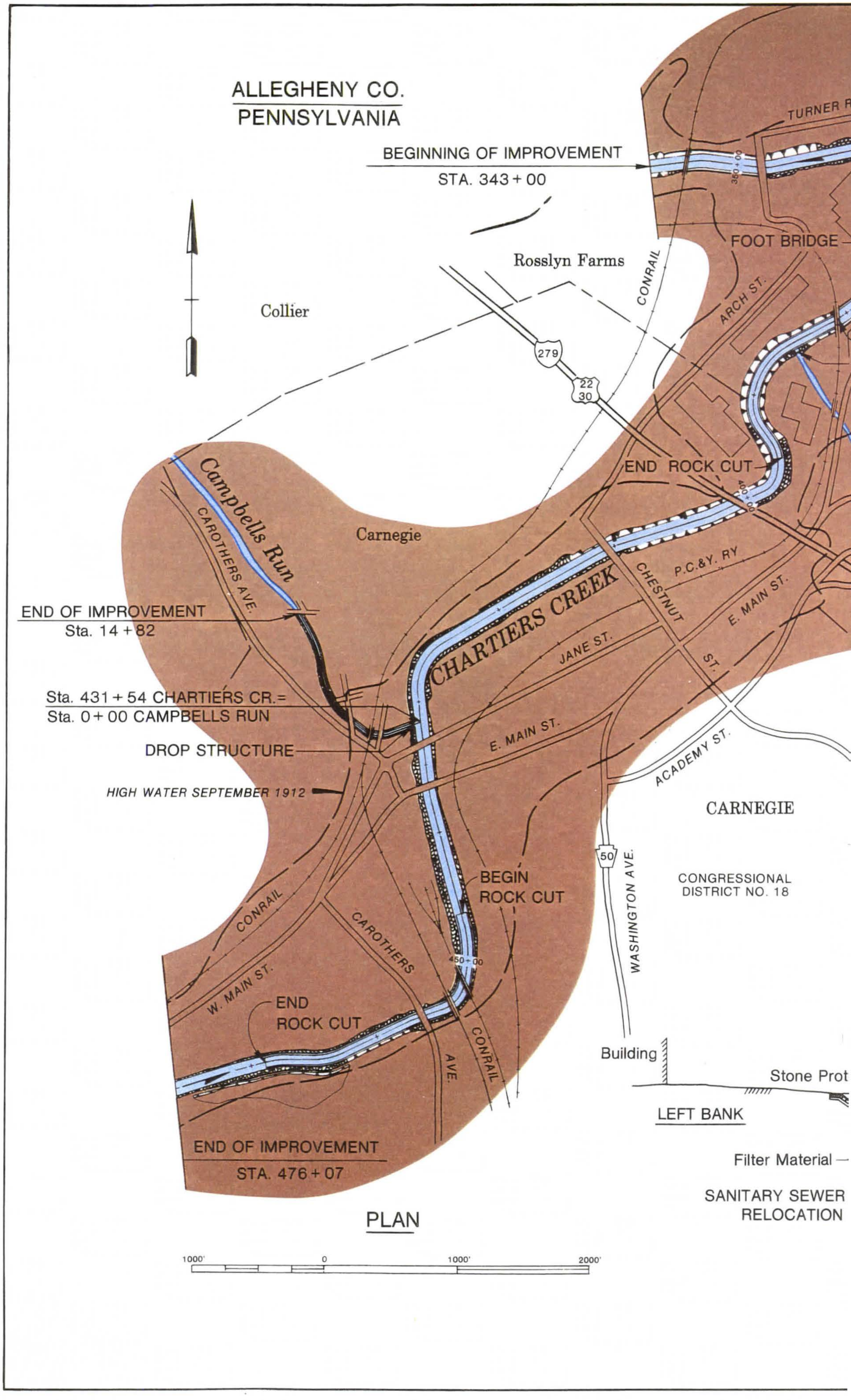
UNIT 1

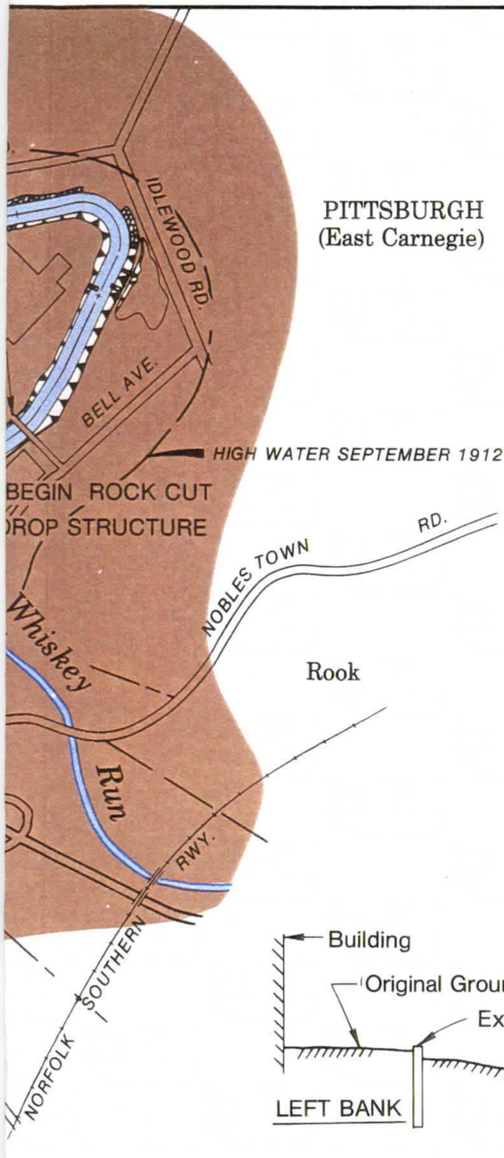
LOCAL PROTECTION

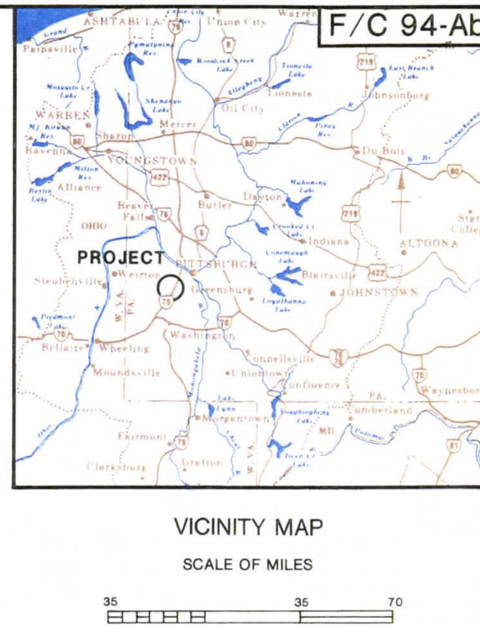
PROFILE

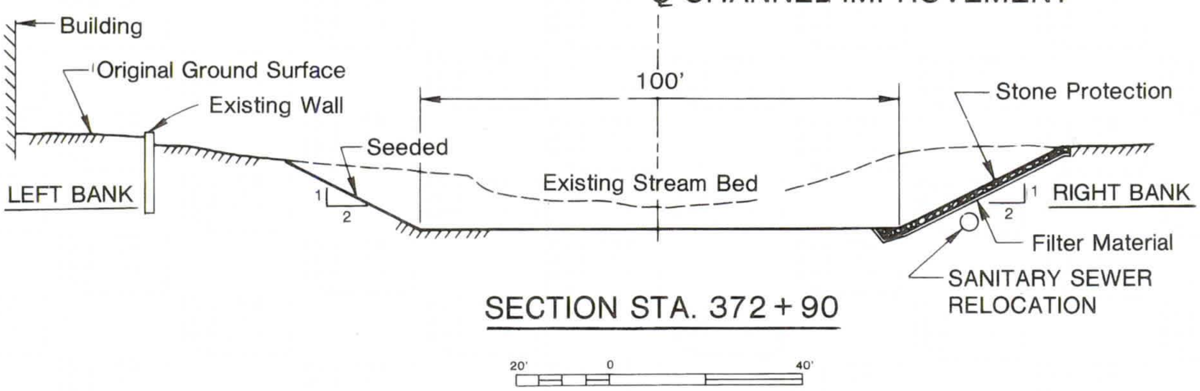
PITTSBURGH DISTRICT, PITTSBURGH, PA.

Revised: 30 September 1986

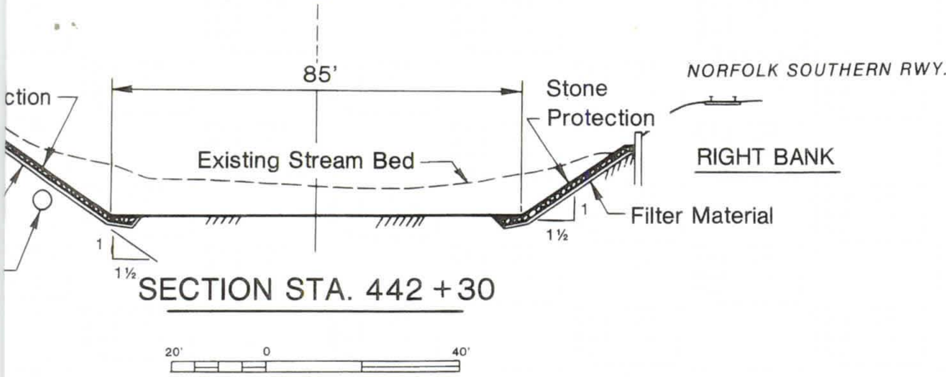

ALLEGHENY CO.
PENNSYLVANIA
BEGINNING OF IMPROVEMENT
STA. 343 + 00
TURNER R
FOOT BRIDGE
Rosslyn Farms
Collier
CONRAIL
ARCH ST.
279
22
30
END ROCK CUT
Campbells Run
CAROTHERS AVE.
Carnegie
CHARTIERS CREEK
CHESTNUT ST.
P.C.&Y. RY
E. MAIN ST.
JANE ST.
END OF IMPROVEMENT
Sta. 14 + 82
Sta. 431 + 54 CHARTIERS CR.=
Sta. 0 + 00 CAMPBELLS RUN
DROP STRUCTURE
E. MAIN ST.
ACADEMY ST.
HIGH WATER SEPTEMBER 1912
CARNEGIE
50
WASHINGTON AVE.
CONGRESSIONAL
DISTRICT NO. 18
BEGIN
ROCK CUT
CONRAIL
CAROTHERS
AVE.
W. MAIN ST.
450+00
END
ROCK CUT
CONRAIL
Building
Stone Prot
LEFT BANK
Filter Material
END OF IMPROVEMENT
STA. 476 + 07
SANITARY SEWER
RELOCATION
PLAN
1000'
0
1000'
2000'







SECTION STA. 372 + 90



SECTION STA. 442 + 30

CHARTIERS CREEK, PA.

JAMES G. FULTON

UNIT 2

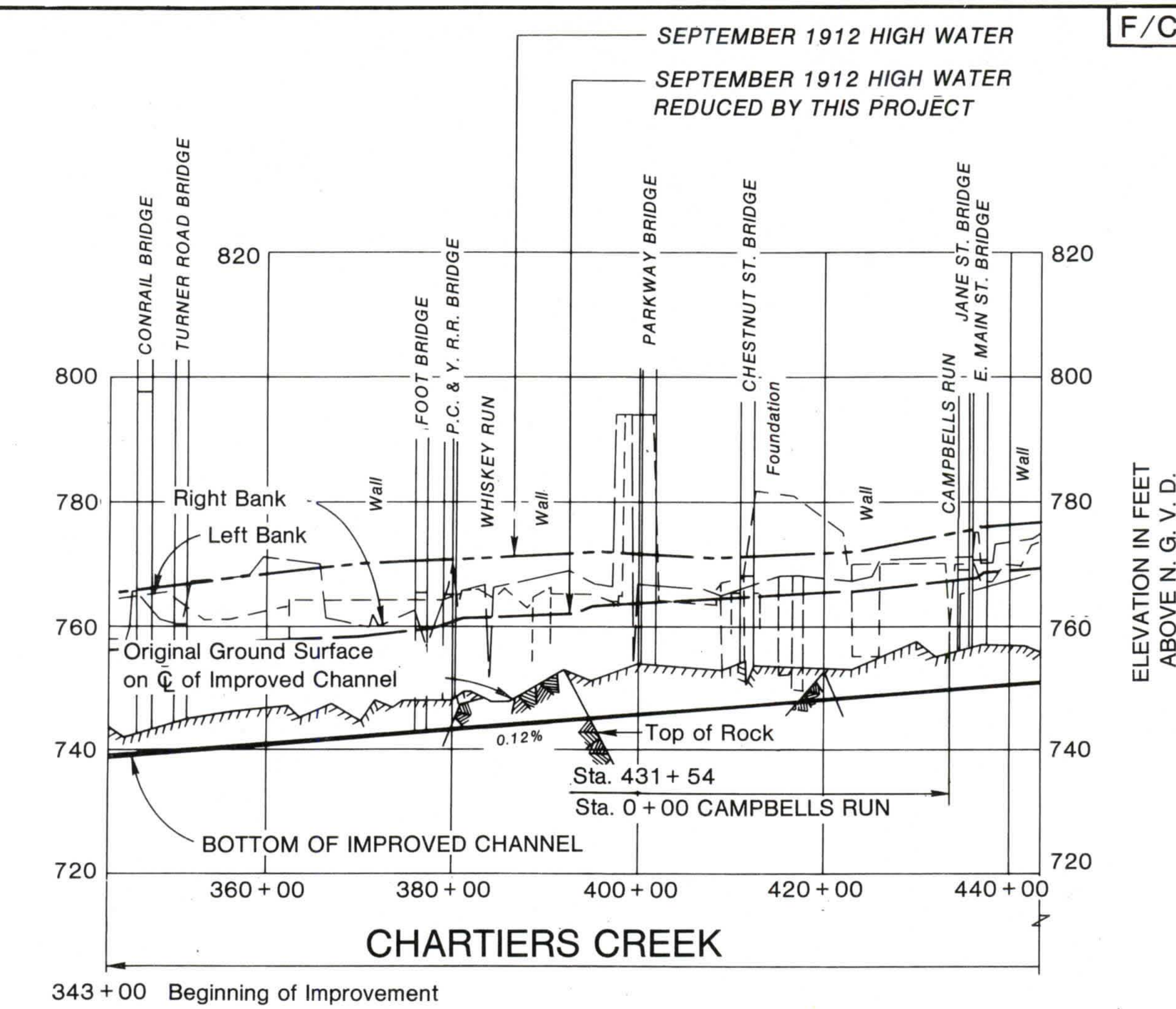
LOCAL PROTECTION

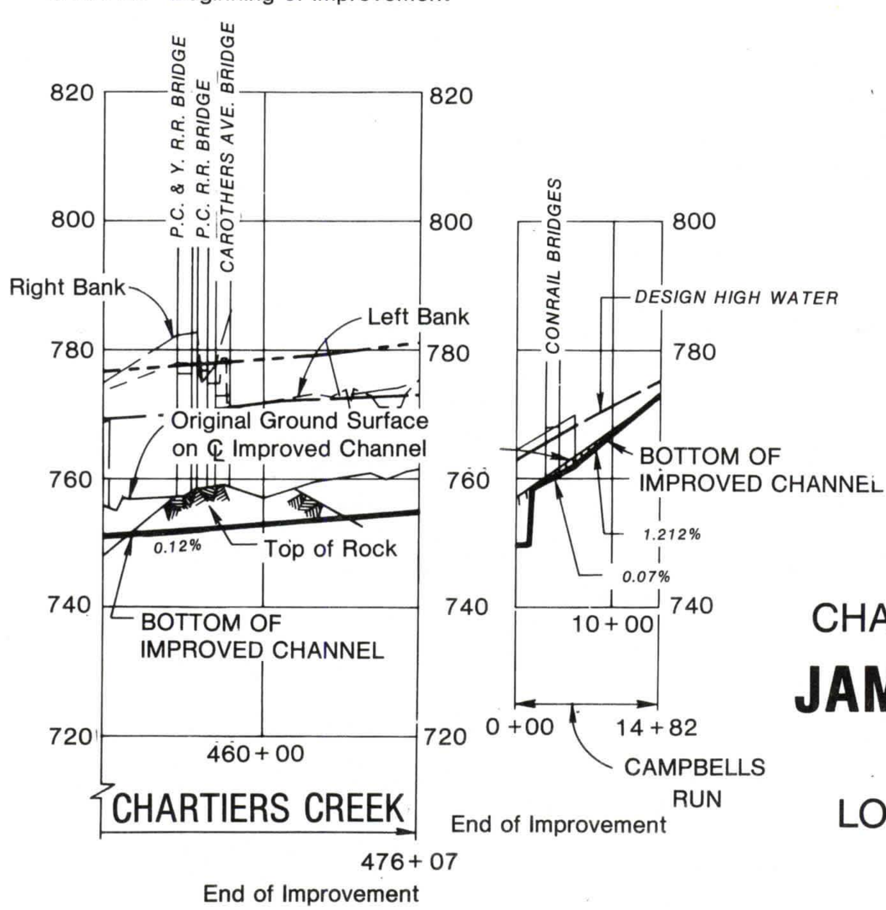
PLAN AND SECTIONS

PITTSBURGH DISTRICT, PITTSBURGH, PA.

Revised: 30 September 1988

F/C 94-Cb





CHARTIERS CREEK, PA.

JAMES G. FULTON

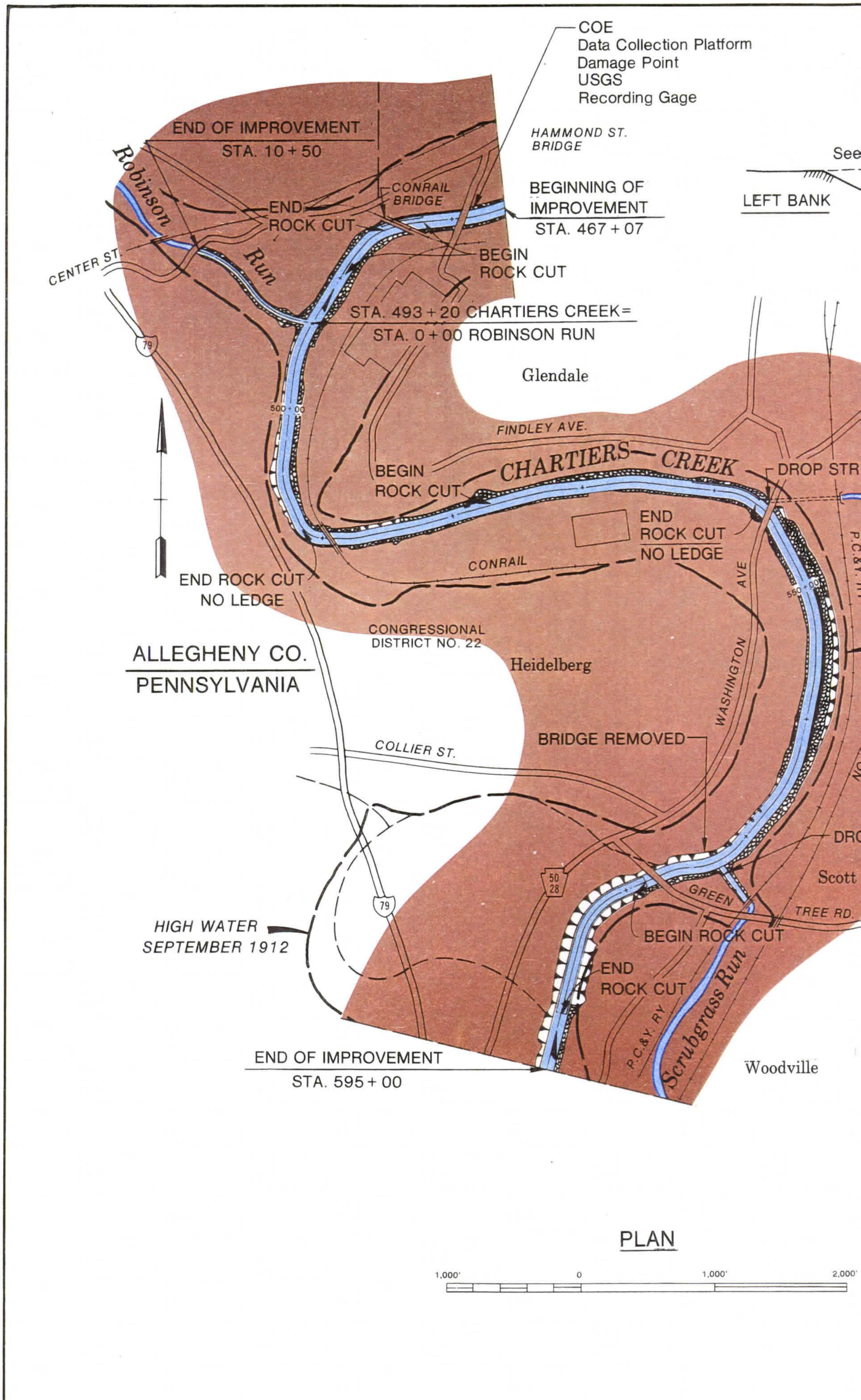
UNIT 2

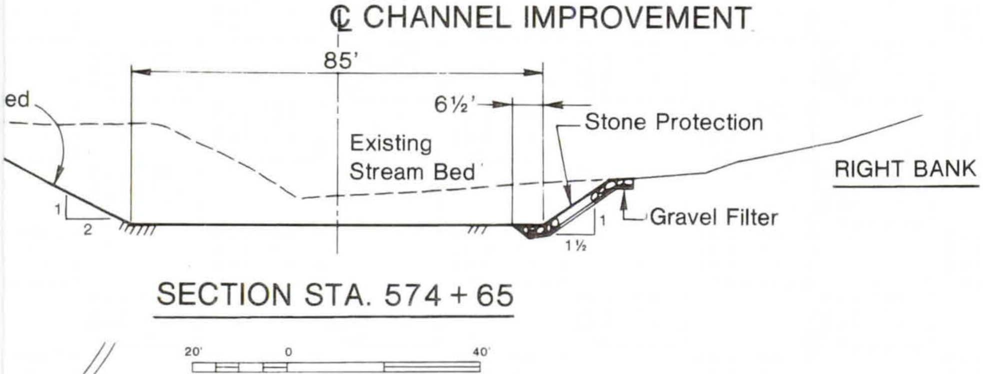
LOCAL PROTECTION

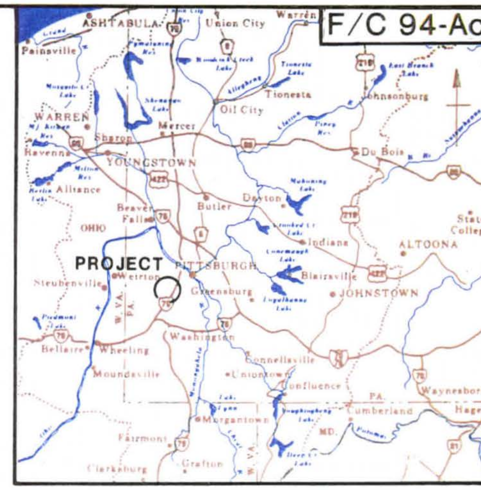
PROFILE

PITTSBURGH DISTRICT, PITTSBURGH, PA.

Revised: 30 September 1986


COE
Data Collection Platform
Damage Point
USGS
Recording Gage
HAMMOND ST. BRIDGE
See
LEFT BANK
END OF IMPROVEMENT
STA. 10 + 50
Robinson Run
CONRAIL BRIDGE
END ROCK CUT
BEGINNING OF IMPROVEMENT
STA. 467 + 07
BEGIN ROCK CUT
CENTER ST.
STA. 493 + 20 CHARTIERS CREEK=
STA. 0 + 00 ROBINSON RUN
79
Glendale
FINDLEY AVE.
500 + 00
BEGIN ROCK CUT
CHARTIERS CREEK
DROP STR
END ROCK CUT NO LEDGE
CONRAIL
END ROCK CUT
NO LEDGE
AVE
550 + 00
P.C.&Y. RY
CONGRESSIONAL DISTRICT NO. 22
ALLEGHENY CO.
PENNSYLVANIA
Heidelberg
WASHINGTON
BRIDGE REMOVED
COLLIER ST.
DRO
Scott
50
28
GREEN TREE RD.
HIGH WATER
SEPTEMBER 1912
BEGIN ROCK CUT
END ROCK CUT
Scrubgrass Run
P.C.&Y. RY
END OF IMPROVEMENT
STA. 595 + 00
Woodville
PLAN
1,000'
0
1,000'
2,000'

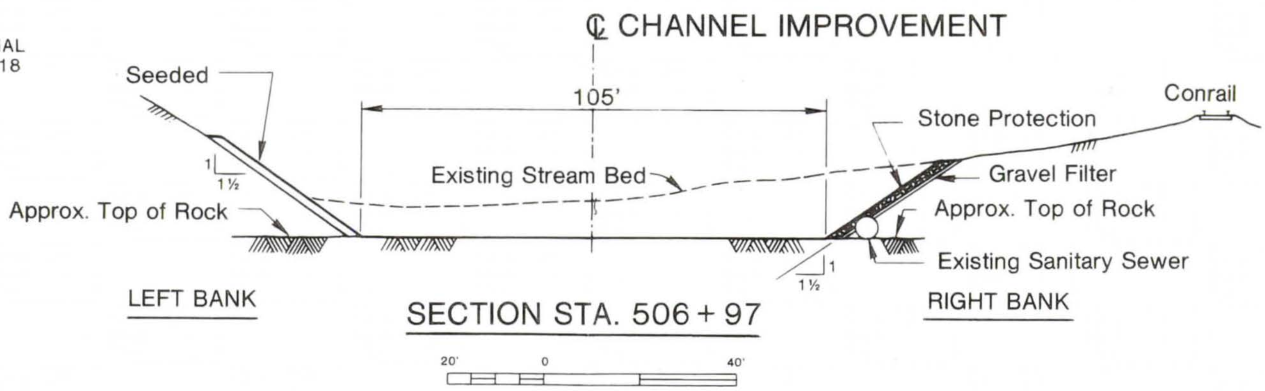




VICINITY MAP

SCALE OF MILES

35 0 35 70



CHARTIERS CREEK, PA.

JAMES G. FULTON

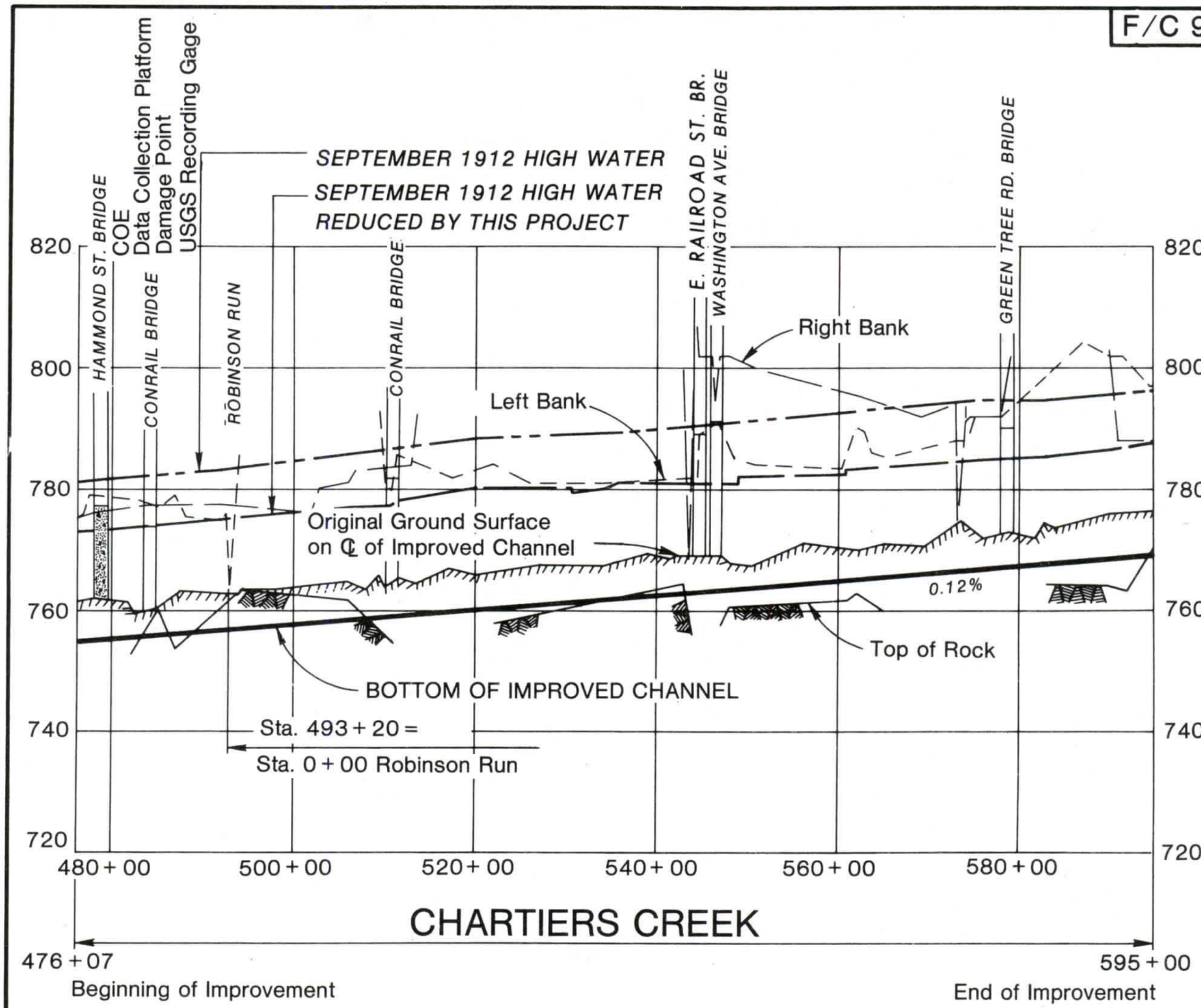
UNIT 3

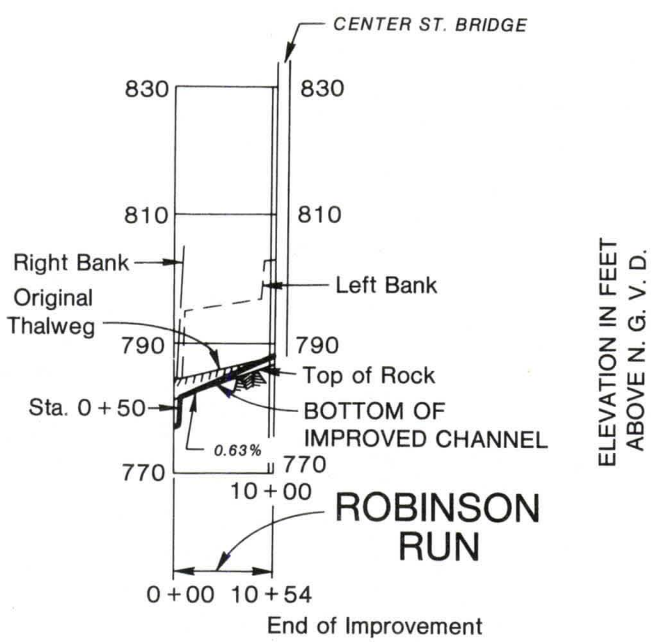
LOCAL PROTECTION

PLAN AND SECTIONS

PITTSBURGH DISTRICT, PITTSBURGH, PA.

Revision: 1 April 1992





CHARTIERS CREEK, PA.

JAMES G. FULTON

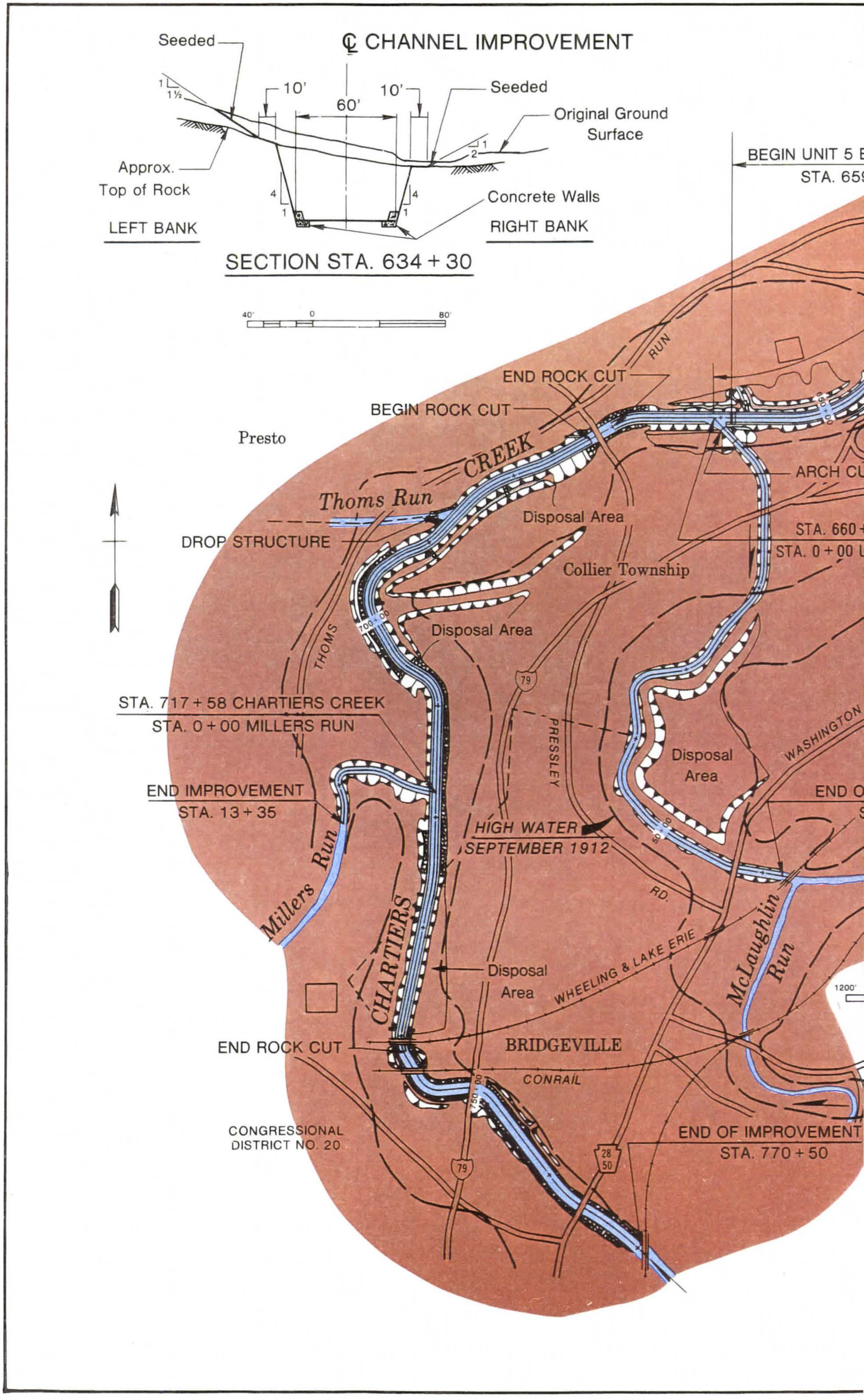
UNIT 3

LOCAL PROTECTION

PROFILE

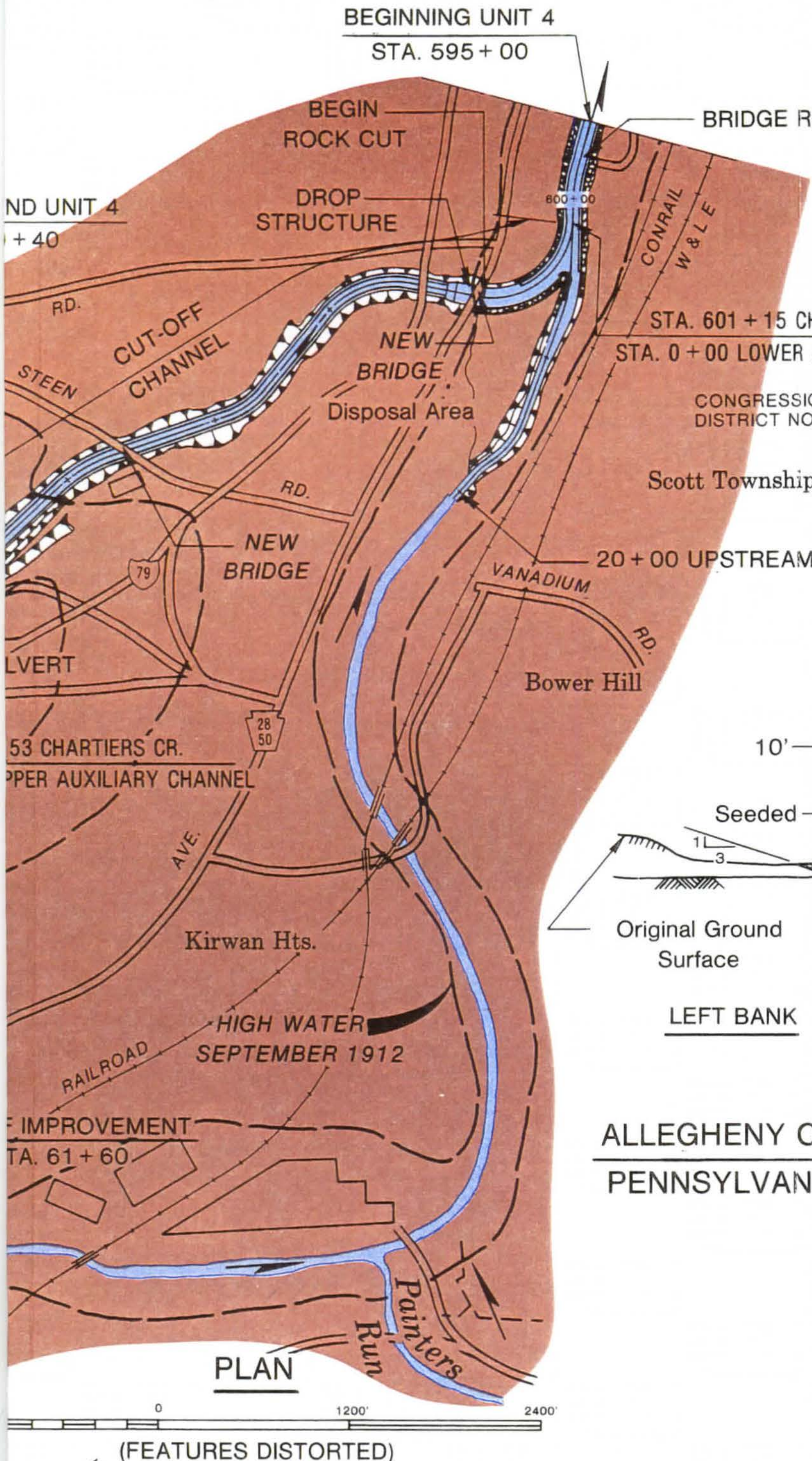
PITTSBURGH DISTRICT, PITTSBURGH, PA.

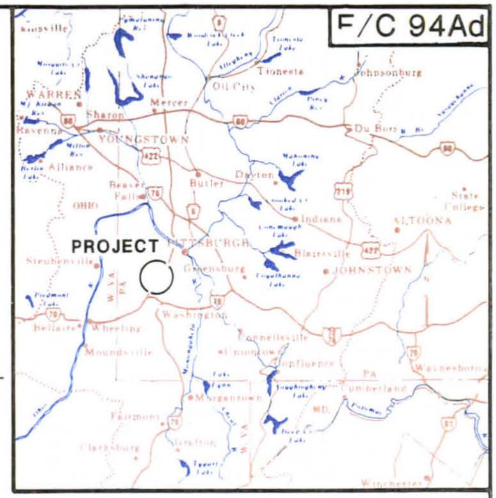
Revised: 1 April 1992



℄ CHANNEL IMPROVEMENT
Seeded
10'
60'
10'
Seeded
Original Ground Surface
Approx. Top of Rock
Concrete Walls
LEFT BANK
RIGHT BANK
SECTION STA. 634 + 30
BEGIN UNIT 5
STA. 65
END ROCK CUT
BEGIN ROCK CUT
RUN
Presto
CREEK
ARCH CU
Thoms Run
Disposal Area
DROP STRUCTURE
Collier Township
STA. 660
STA. 0 + 00
THOMS
Disposal Area
79
PRESSLEY
STA. 717 + 58 CHARTIERS CREEK
STA. 0 + 00 MILLERS RUN
Disposal Area
WASHINGTON
END IMPROVEMENT
STA. 13 + 35
END O
HIGH WATER
SEPTEMBER 1912
Millers Run
RD.
McLaughlin Run
CHARTIERS
Disposal Area
WHEELING & LAKE ERIE
END ROCK CUT
BRIDGEVILLE
CONRAIL
CONGRESSIONAL DISTRICT NO. 20
79
28 50
END OF IMPROVEMENT
STA. 770 + 50

F/C 94Ad

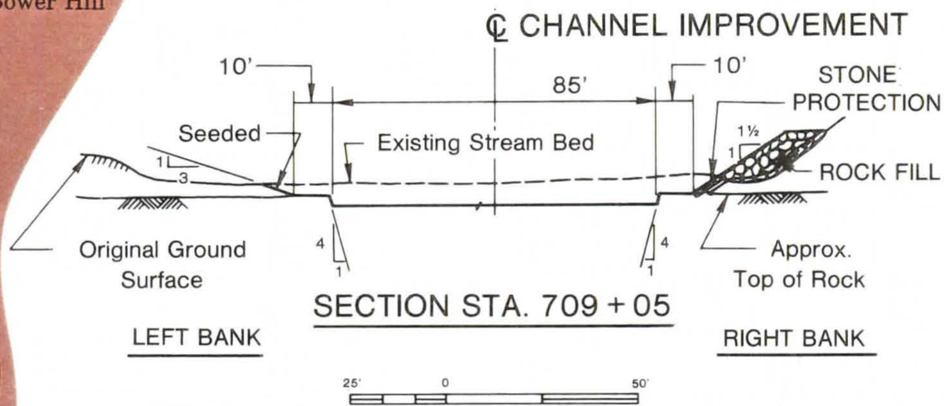


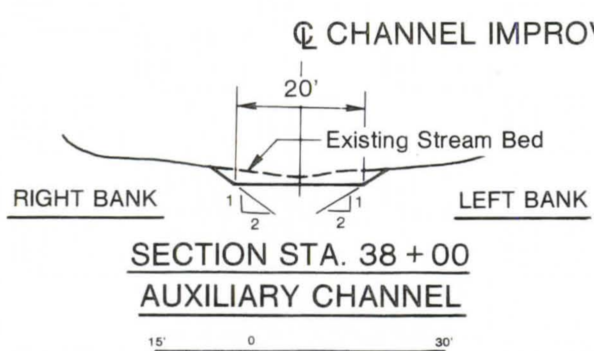


VICINITY MAP

SCALE OF MILES

35 0 35 70





CHARTIERS CREEK, PA.

JAMES G. FULTON

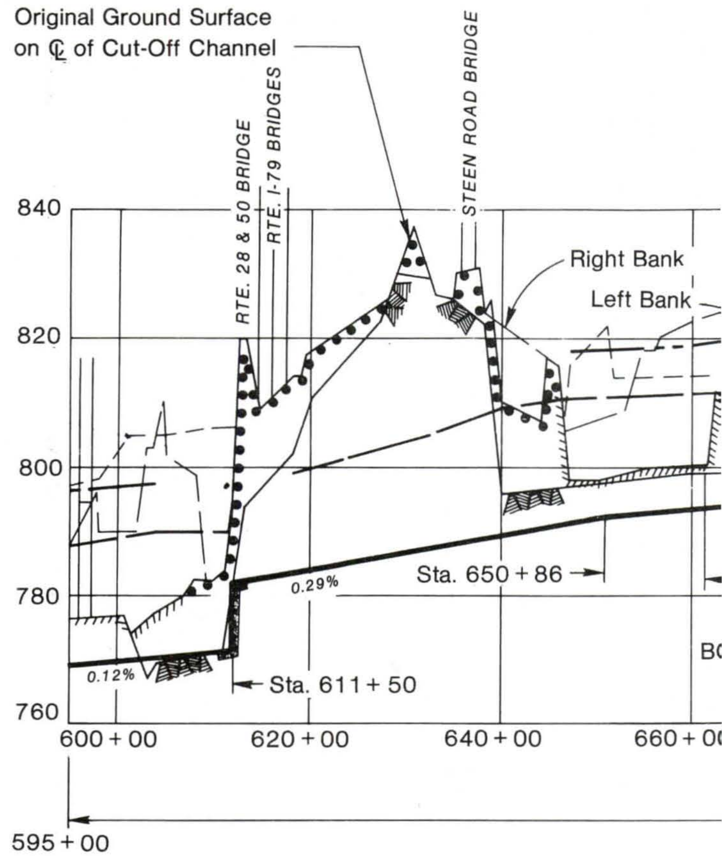
UNIT 4 & 5 AND AUXILIARY CHANNEL

LOCAL PROTECTION

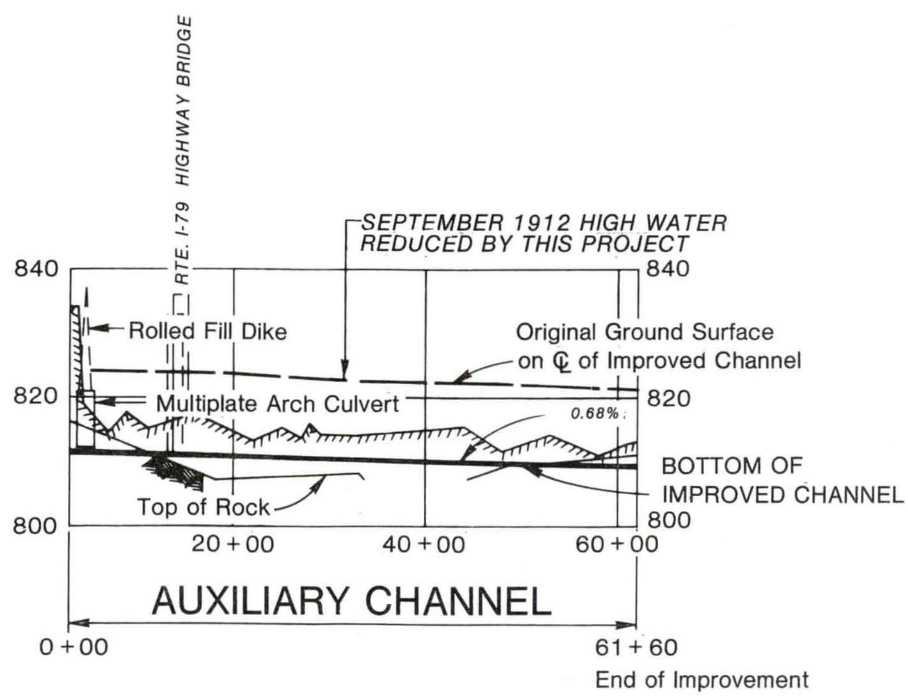
PLAN AND SECTIONS

PITTSBURGH DISTRICT, PITTSBURGH, PA.

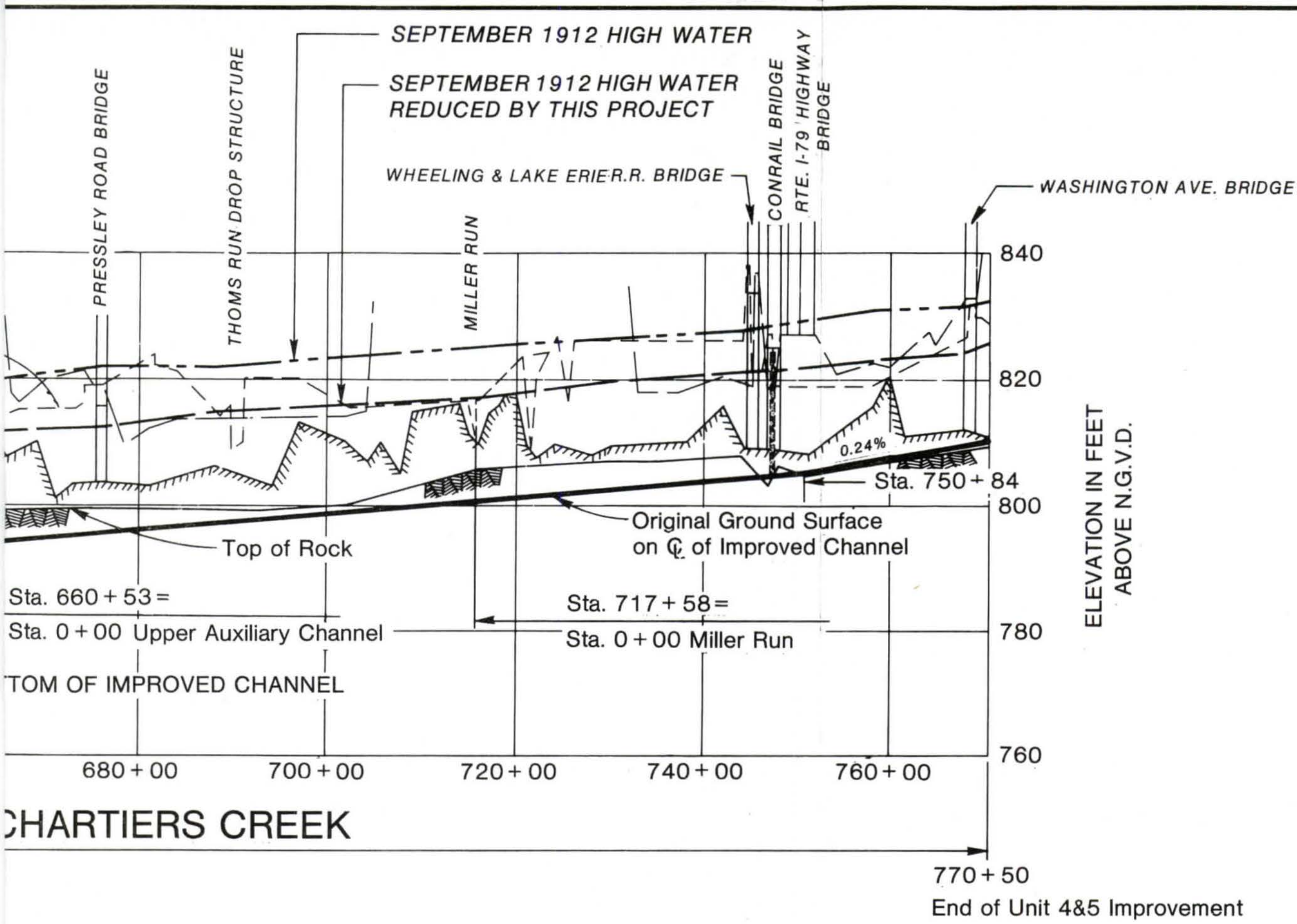
Revised: 30 September 1986

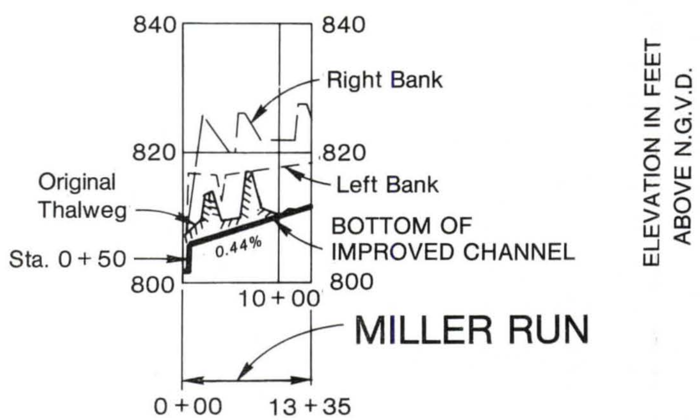


Original Ground Surface
on ℄ of Cut-Off Channel
RTE. 28 & 50 BRIDGE
RTE. I-79 BRIDGES
STEEN ROAD BRIDGE
Right Bank
Left Bank
840
820
800
780
760
0.29%
0.12%
Sta. 650 + 86
Sta. 611 + 50
600 + 00
620 + 00
640 + 00
660 + 00
595 + 00



RTE. I-79 HIGHWAY BRIDGE
SEPTEMBER 1912 HIGH WATER
REDUCED BY THIS PROJECT
Rolled Fill Dike
Original Ground Surface
on ℄ of Improved Channel
Multiplate Arch Culvert
0.68%
BOTTOM OF
IMPROVED CHANNEL
Top of Rock
840
820
800
20 + 00
40 + 00
60 + 00
AUXILIARY CHANNEL
0 + 00
61 + 60
End of Improvement





CHARTIERS CREEK, PA.

JAMES G. FULTON

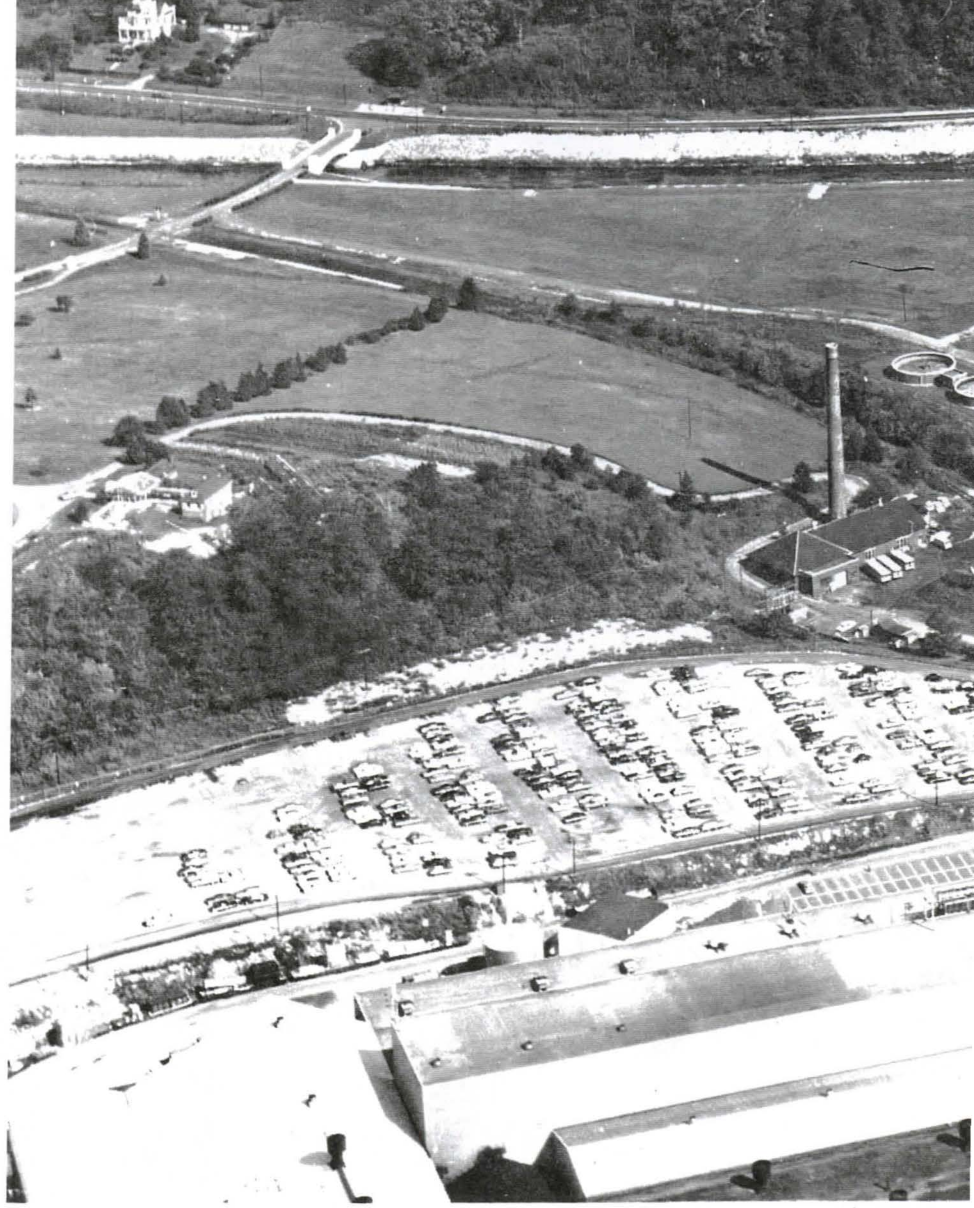
UNIT 4&5

LOCAL PROTECTION

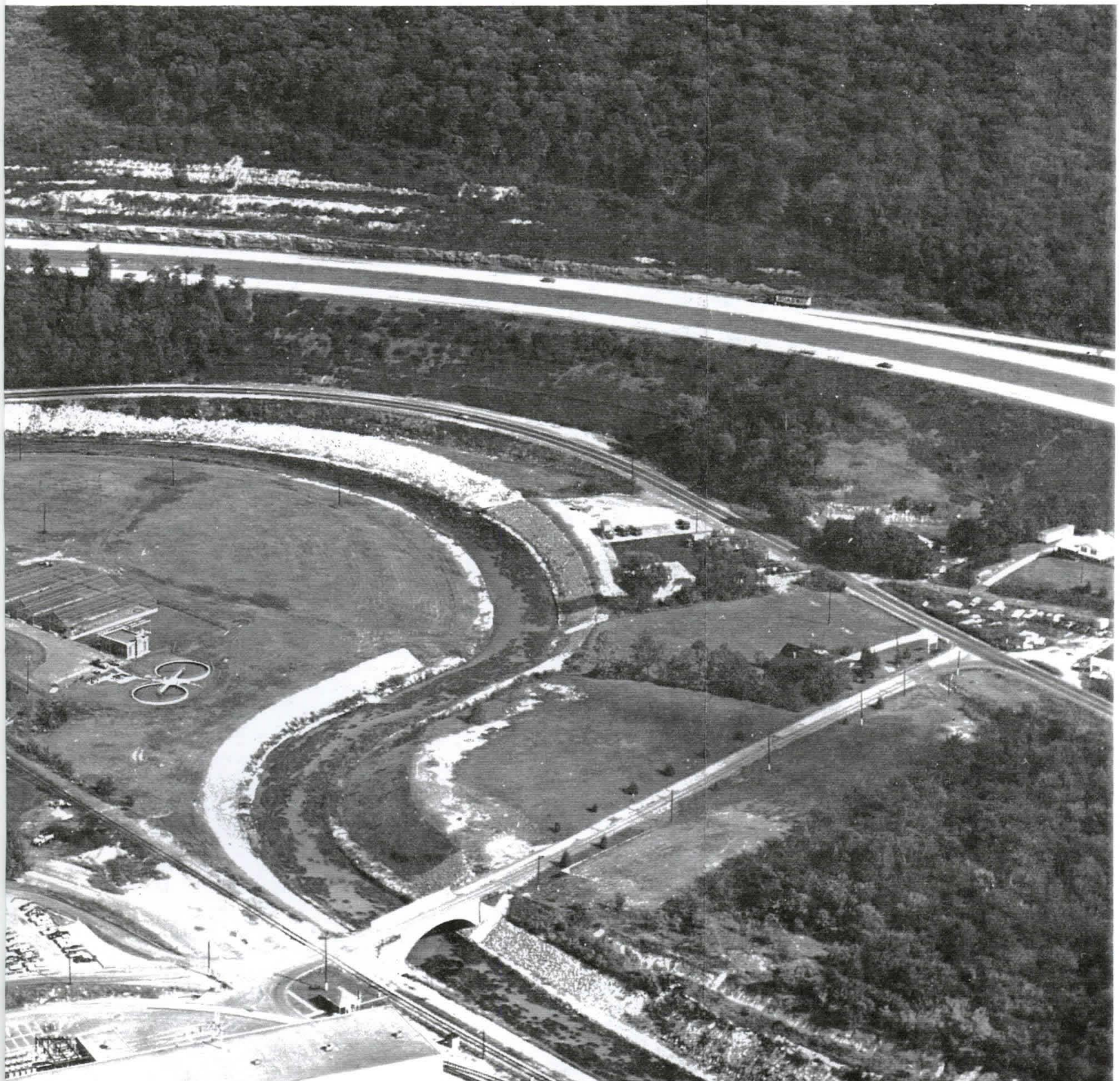
PROFILE

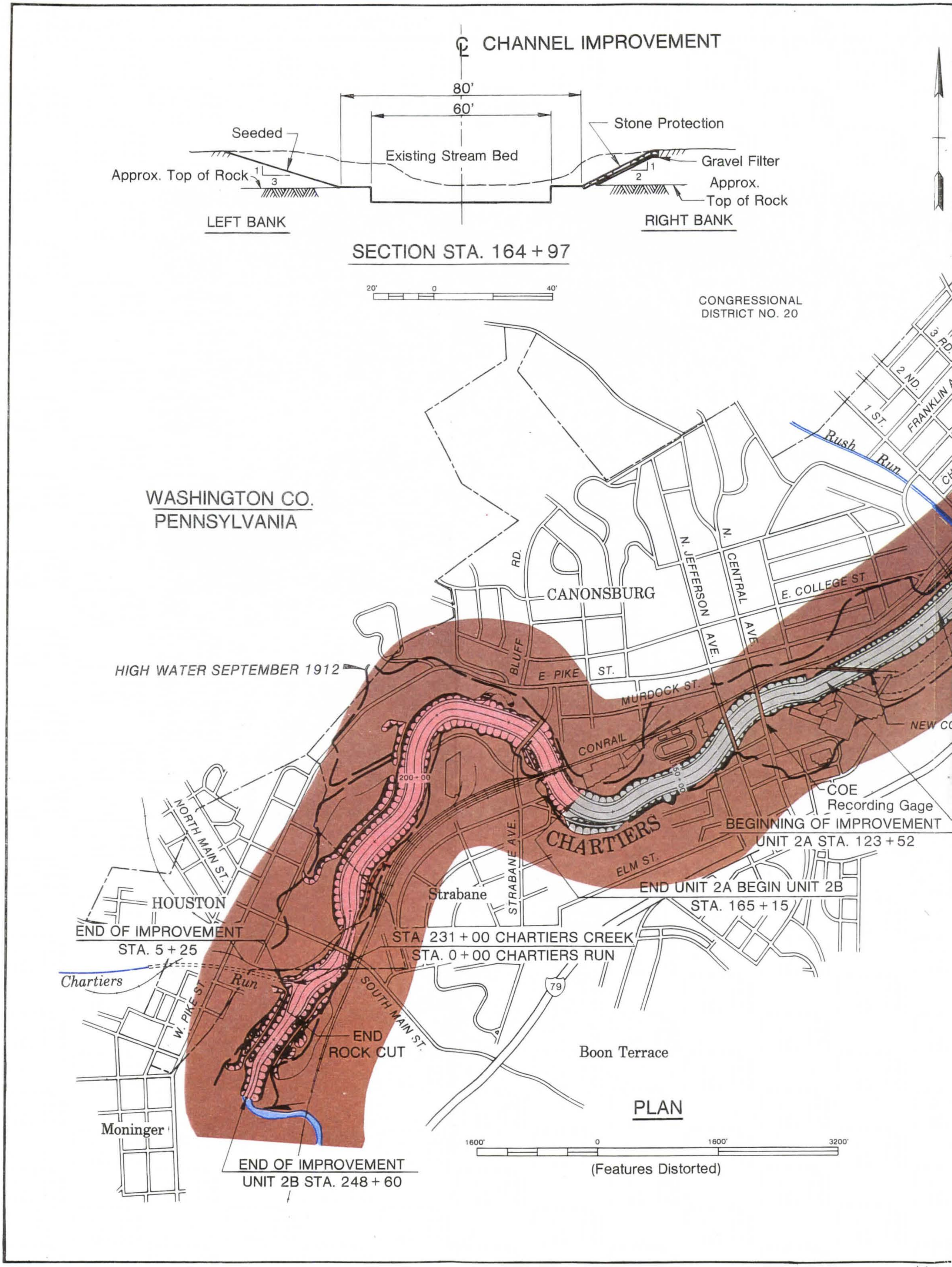
PITTSBURGH DISTRICT, PITTSBURGH, PA.

Revised: 30 September 1986



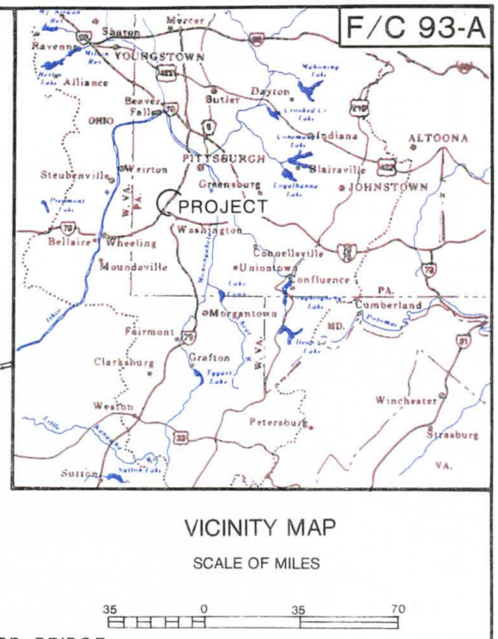
F/C 93
CANONSBURG - HOUSTON, PA.

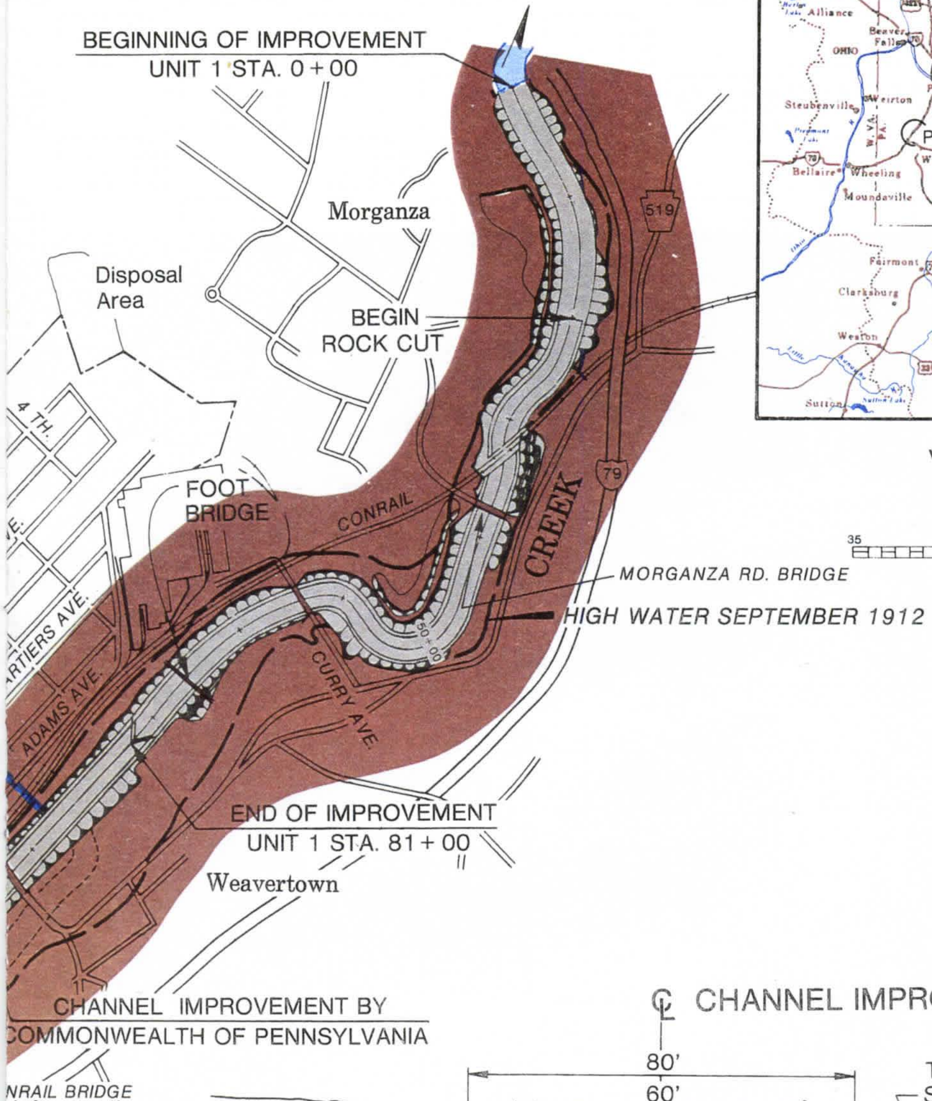


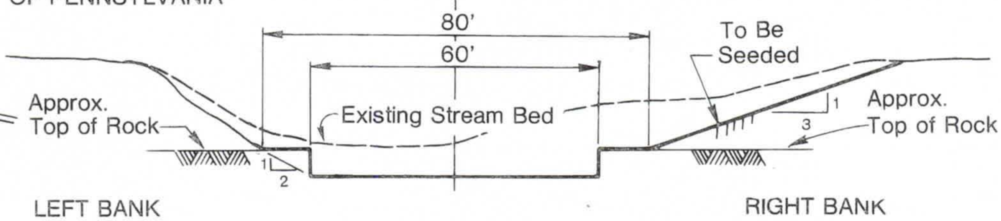


℄ CHANNEL IMPROVEMENT
80'
60'
Seeded
Existing Stream Bed
Stone Protection
Gravel Filter
Approx. Top of Rock
Approx.
Top of Rock
LEFT BANK
RIGHT BANK
SECTION STA. 164 + 97
20'
0
40'
CONGRESSIONAL
DISTRICT NO. 20
WASHINGTON CO.
PENNSYLVANIA
CANONSBURG
N. JEFFERSON AVE.
N. CENTRAL AVE.
E. COLLEGE ST
E. PIKE ST.
MURDOCK ST.
BLUFF RD.
CONRAIL
Rush Run
HIGH WATER SEPTEMBER 1912
COE
Recording Gage
BEGINNING OF IMPROVEMENT
UNIT 2A STA. 123 + 52
CHARTIERS
ELM ST.
STRABANE AVE.
Strabane
END UNIT 2A BEGIN UNIT 2B
STA. 165 + 15
HOUSTON
NORTH MAIN ST.
END OF IMPROVEMENT
STA. 5 + 25
STA. 231 + 00 CHARTIERS CREEK
STA. 0 + 00 CHARTIERS RUN
Chartiers
Run
W. PIKE ST.
SOUTH MAIN ST.
79
END
ROCK CUT
Boon Terrace
Moninger
PLAN
1600'
0
1600'
3200'
(Features Distorted)
END OF IMPROVEMENT
UNIT 2B STA. 248 + 60

F/C 93-A







SECTION STA. 80 + 50

20' 0 40'

STATUS

Completed

Not Under Construction

CHARTIERS CREEK

CANONSBURG-HOUSTON, PA.

LOCAL PROTECTION

PLAN AND SECTIONS

PITTSBURGH DISTRICT, PITTSBURGH, PA.

Revision: 1 April 1992

CHARTIERS CREEK
CANONSBURG-HOUSTON, PENNSYLVANIA
LOCAL PROTECTION PROJECT
SUMMARY OF PERTINENT DATA
1 APRIL 1992

AUTHORIZATION. Section 204 of the Flood Control Act of 1965.

LOCATION. The project is located on Chartiers Creek in the vicinity of Canonsburg and Houston, Washington County, Pennsylvania.

LOCAL COOPERATION. Washington County, Pa., being the local cooperating agency, is required to, without cost to the United States: provide all lands, easements, rights-of-way and disposal areas necessary for construction; adjust utilities and bridges; hold and save the United States free from damages due to construction; maintain and operate the improvement after completion.

PRINCIPLE FEATURES OF PROJECT PLAN. Project consists mainly of widening, deepening and realigning approximately 20,600 feet of Chartiers Creek, the reconstruction of one highway and one railroad bridge, the protection of bridge and building foundations, stone protection on bank slopes and the adjustment of railroad tracks, utility lines and drainage facilities.

HYDROLOGY AND HYDRAULIC DESIGN. Drainage area: Chartiers Creek upstream of downstream limit of Canonsburg-Houston project 87.3 square miles.
Maximum flood of record in the Canonsburg-Houston area occurred 2 September 1912, with peak flow of 8,500 c.f.s. at the Central Avenue Bridge in Canonsburg, Pa.
The improvement is designed to provide complete protection against a discharge up to 8,500 c.f.s.

ECONOMIC EVALUATION. Based on October 1978 price levels, average annual benefits and charges are estimated to be $644,800 and $353,700 respectively, indicating a favorable economic ratio of 1.8.

PROGRESS. Construction of unit 1, consisting of 1.5 miles of channel improvement, was started in December 1968 and completed in March 1970. To expedite construction, Unit 2 was sub-divided into Units 2A and 2B. Unit 2A, consisting of approximately 0.8 mile of channel improvements, started in January 1976 and was completed in December 1976. Construction plans and specifications for Unit 2B are completed and approved, awaiting rights-of-entry and assurances from local interests. Length of Unit 2B is 1.6 miles.

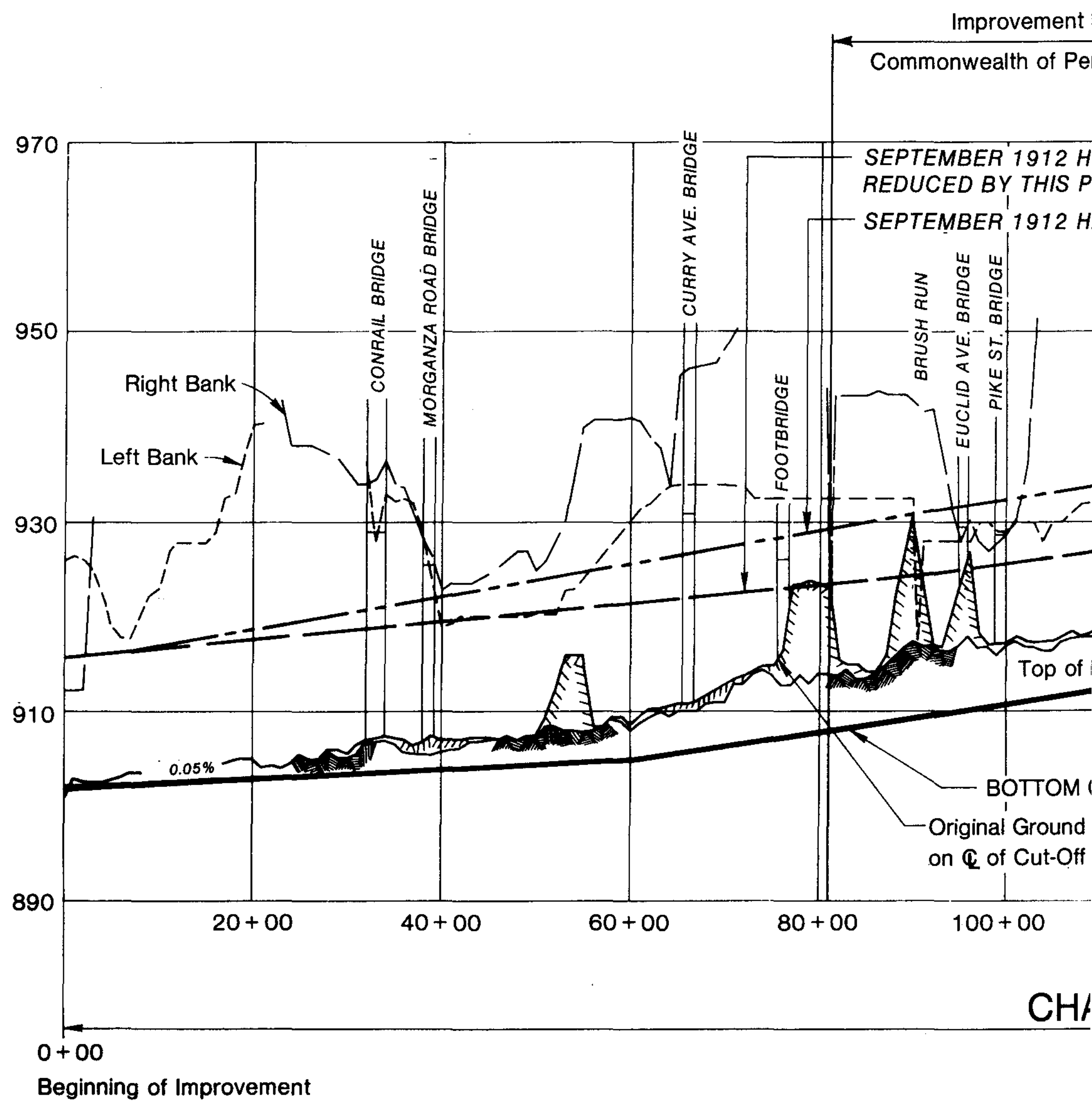
COSTS.

Total Costs, estimated 1 October 1983

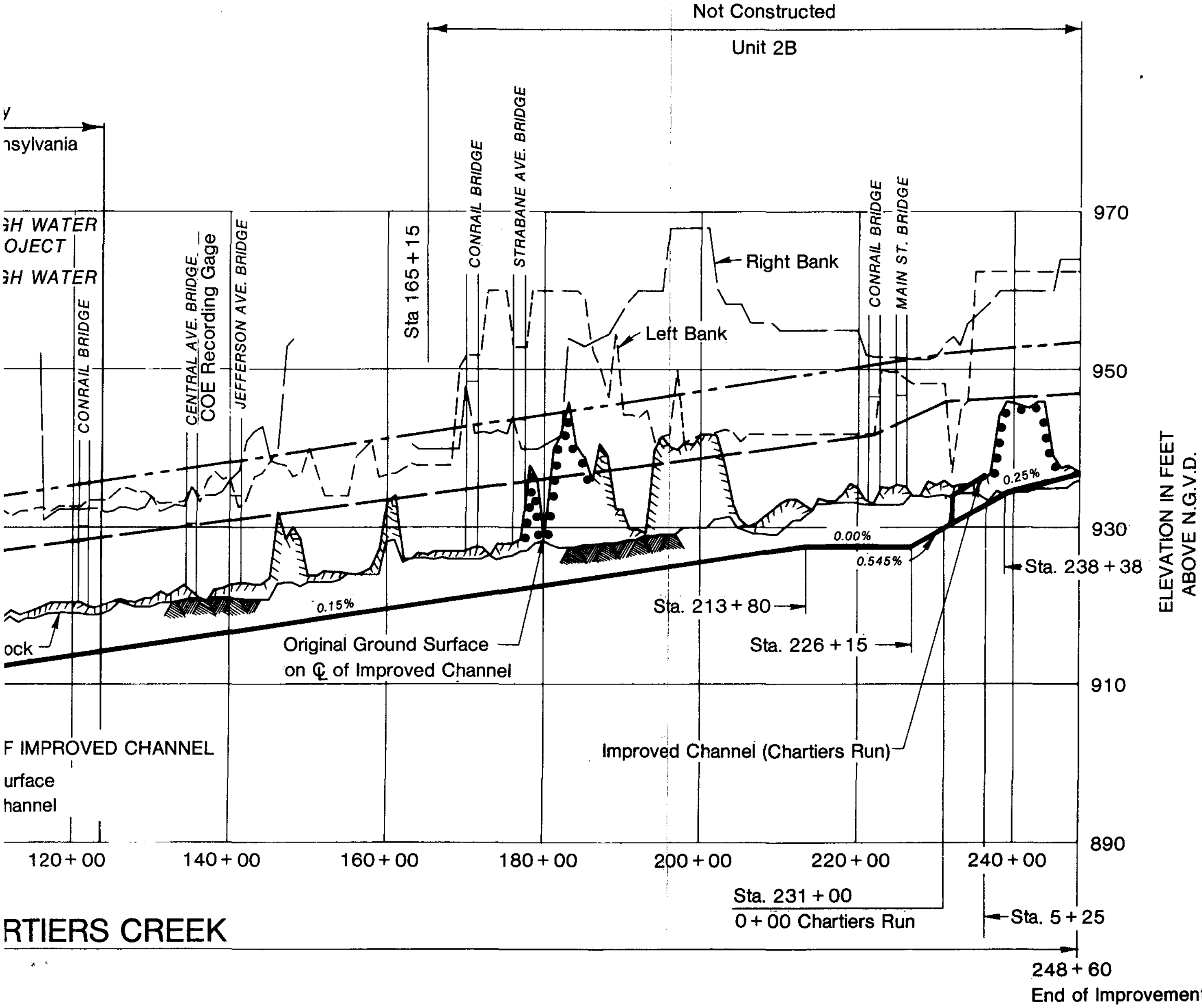
Federal	$8,600,000
Non-Federal	$2,660,000
Federal Costs to 30 September 1983	$2,680,052
Federal Allotments to 30 September 1983	$2,687,728

SURVEY DATA. Control surveys were made 1958-1959. Horizontal positions are controlled by third-order traverses tied to USGS Stations TT15CWH and TT16CWH and are computed on the Pennsylvania South-Zone Lambert Coordinate System. Elevations are tied to USC&GS Level Line, Pa. No. 98, and refer to Sea Level Datum, 1929 Gen. Adj. Topography was obtained by plane table in 1959, supplement and/or revised in 1966.

CONGRESSIONAL DISTRICT. The project lies within the 20th Congressional District of Pennsylvania.



Improvement
Commonwealth of Pe
970
950
930
910
890
SEPTEMBER 1912 H
REDUCED BY THIS P
SEPTEMBER 1912 H
CONRAIL BRIDGE
MORGANZA ROAD BRIDGE
CURRY AVE. BRIDGE
FOOTBRIDGE
BRUSH RUN
EUCLID AVE. BRIDGE
PIKE ST. BRIDGE
Right Bank
Left Bank
0.05%
Top of
BOTTOM
Original Ground
on ℄ of Cut-Off
20+00
40+00
60+00
80+00
100+00
CHA
0+00
Beginning of Improvement



CHARTIERS CREEK

CANNONSBURG-HOUSTON, PA.

LOCAL PROTECTION

PROFILE

PITTSBURGH DISTRICT, PITTSBURGH, PA.

Revised: 1 April 1992

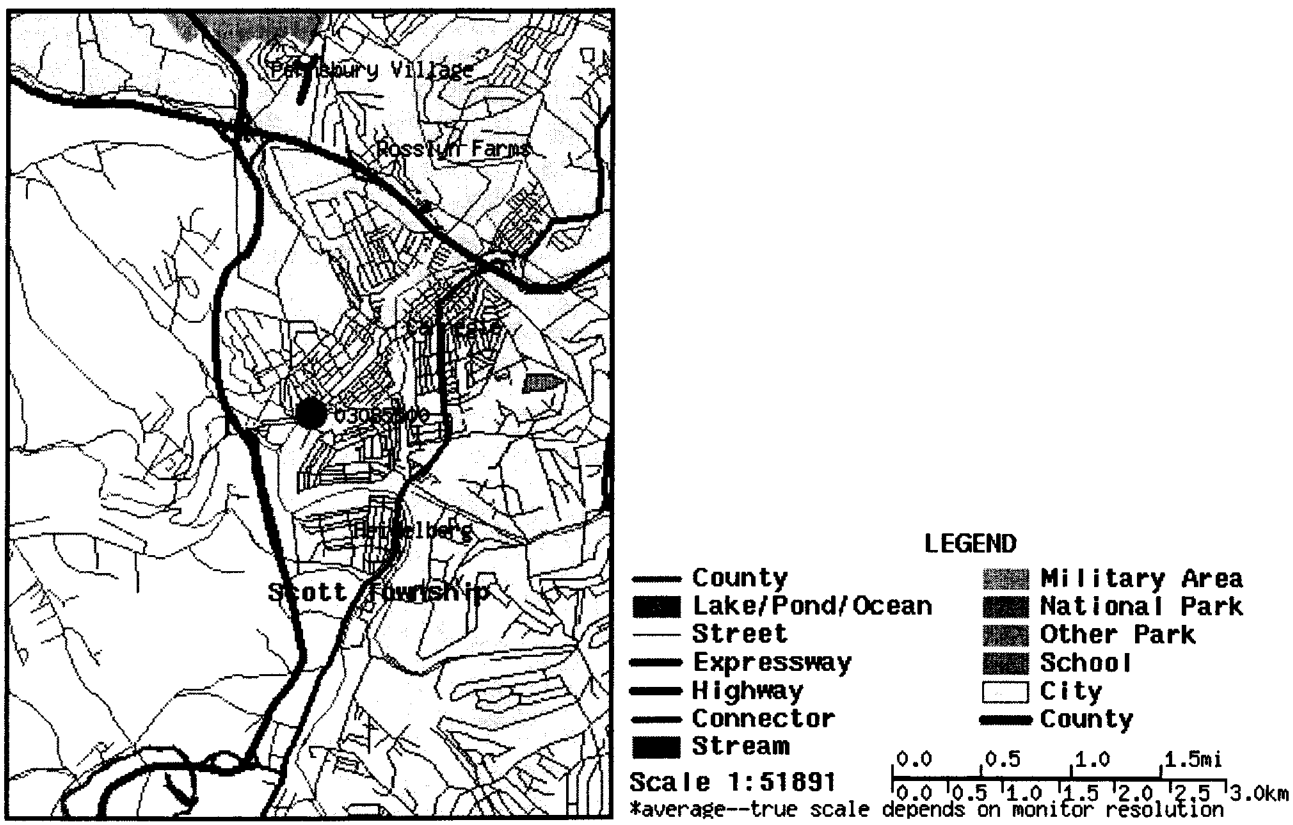
APPENDIX

USGS science for a changing world

Map of region surrounding Chartiers Creek At Carnegie, Pa.

This map is provided by the US Census Tiger Mapping Server.

Another interface to this service is provided by USGS Mapping Information server.



USGS

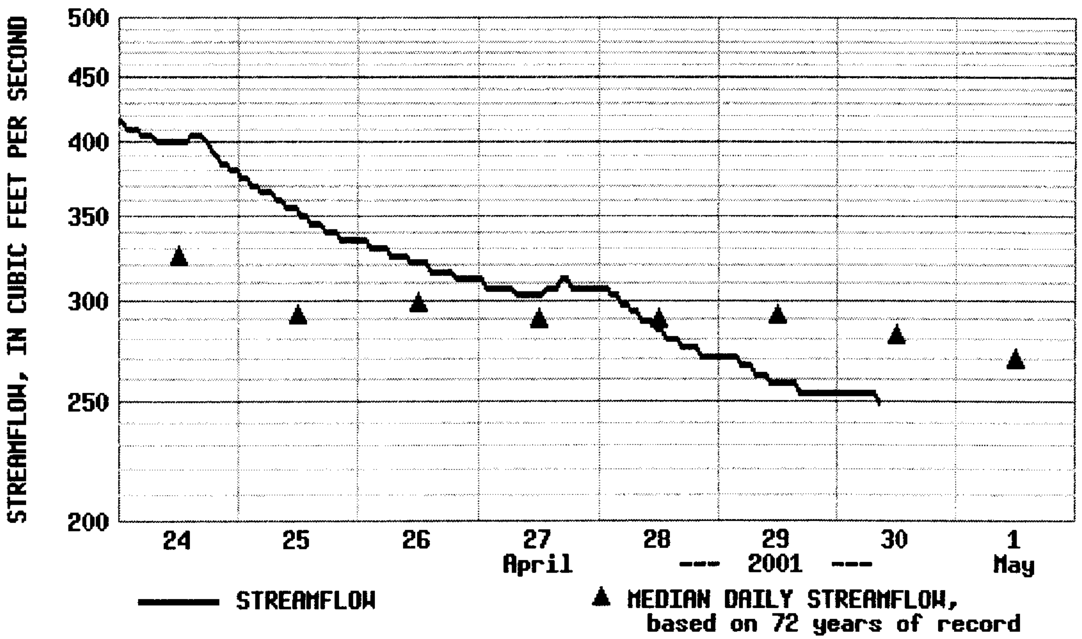
PROVISIONAL DATA SUBJECT TO REVISION

03085500-- CHARTIERS CREEK AT CARNEGIE, PA

Current Conditions

Flow (ft^3/s)	Stage (ft)	Date	Time
249	1.62	04/30	08:44

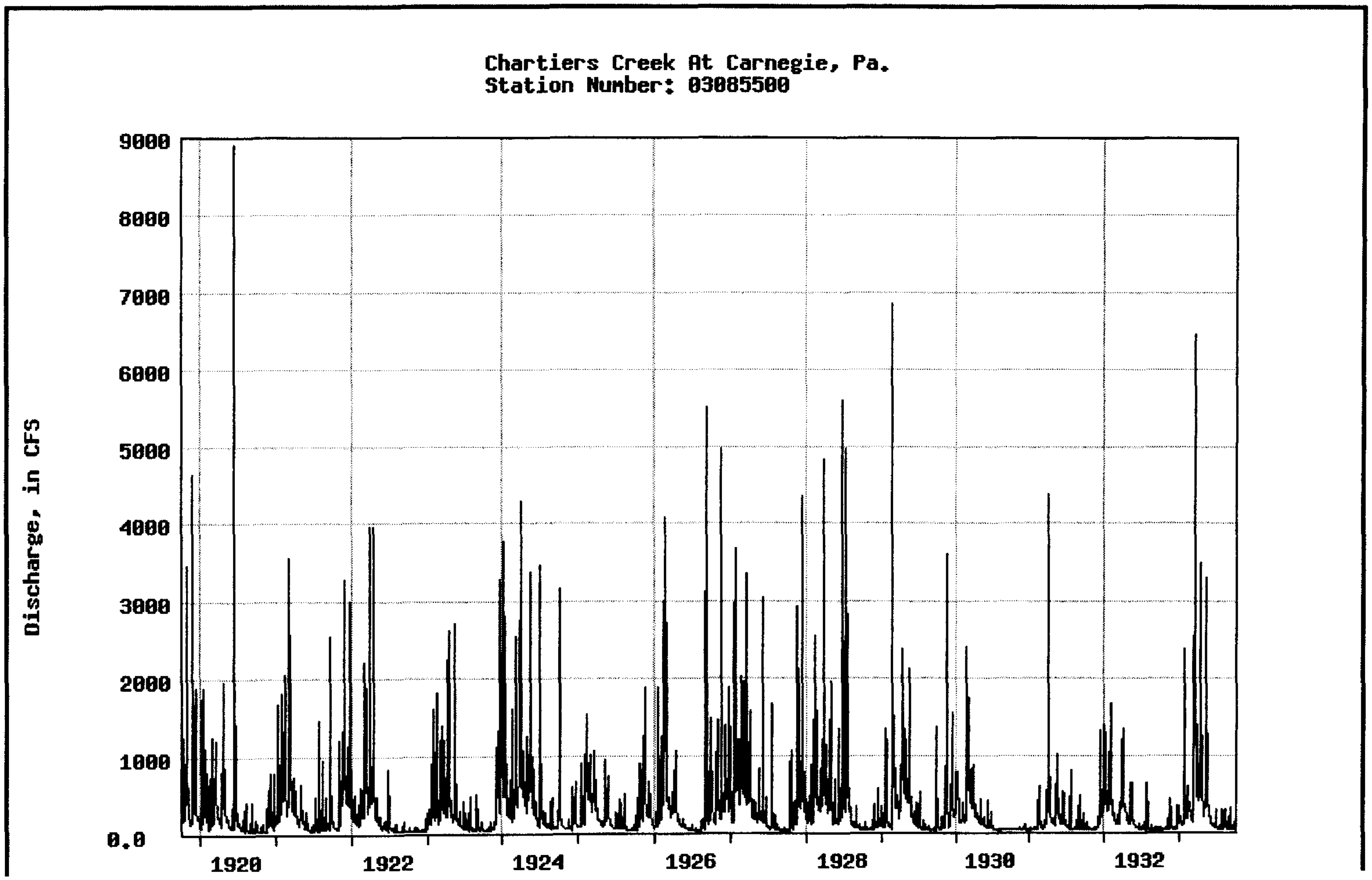
Streamflow -- updated Mon Apr 30 08:44 2001 -- download presentation-quality graph



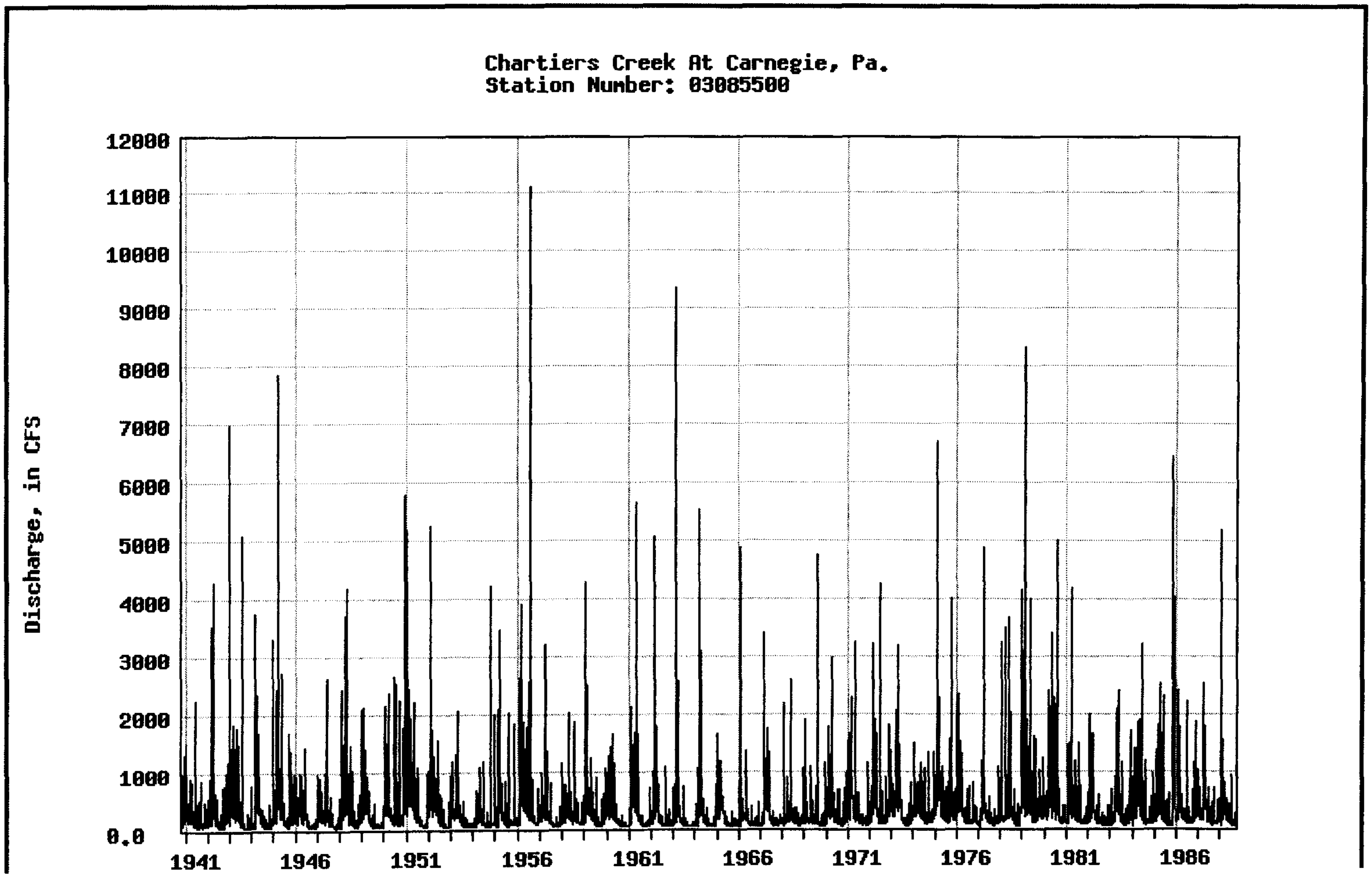
Stage -- updated Mon Apr 30 08:44 2001 -- download presentation-quality graph

USGS
science for a changing world

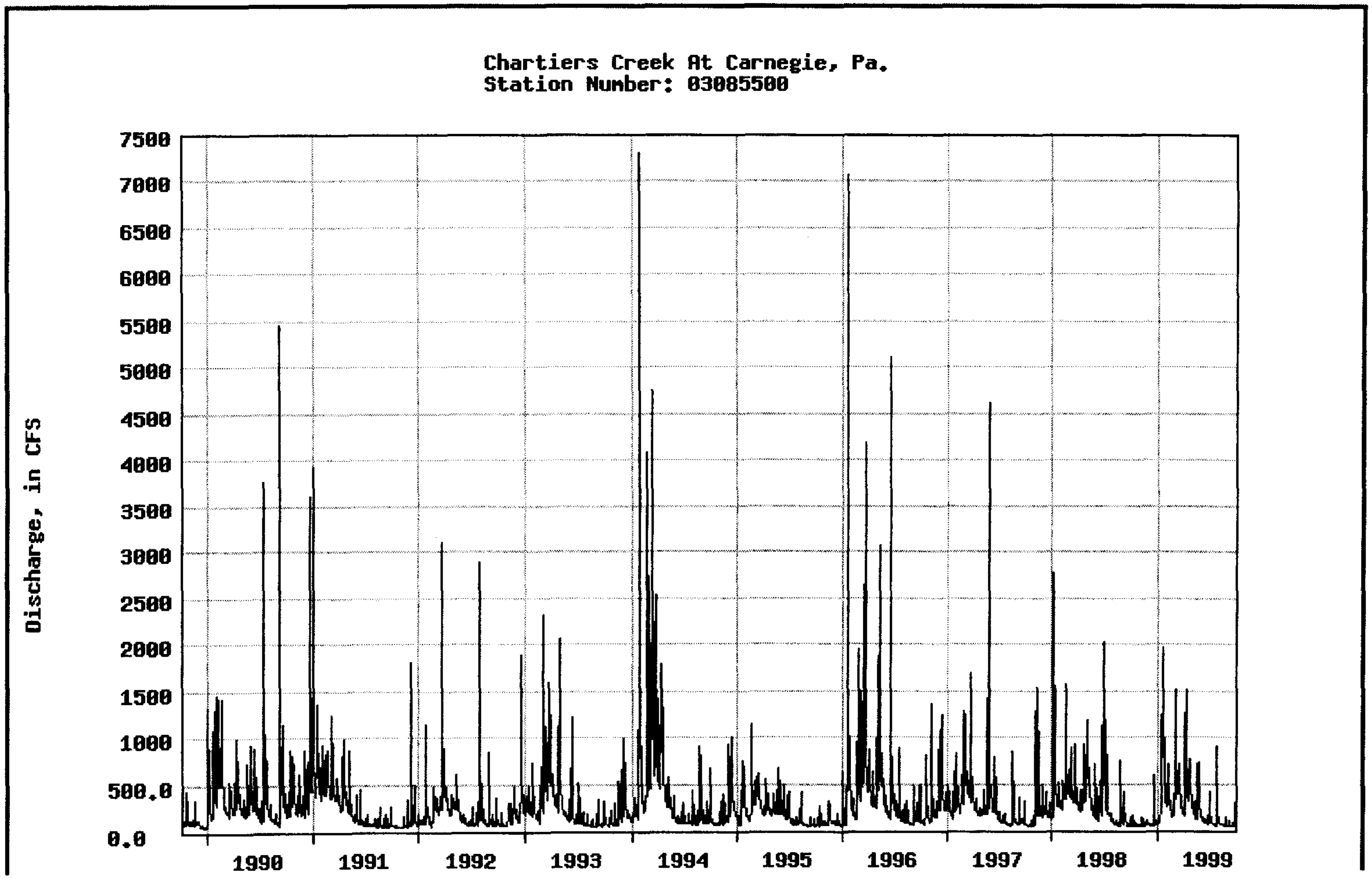
Historical Streamflow Daily Values Graph for Chartiers Creek At Carnegie, Pa. (03085500)



Historical Streamflow Daily Values Graph for Chartiers Creek At Carnegie, Pa. (03085500)



Historical Streamflow Daily Values Graph for Chartiers Creek At Carnegie, Pa. (03085500)



8731 February, 1972 SM 2

Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

FAILURE OF COLLUVIAL SLOPE

By James V. Hamel,[1] A. M. ASCE and Norman K. Flint[2]

INTRODUCTION

Colluvium is geological material which has moved downslope under the influence of gravity, i.e., landslide or creep debris. Colluvial slopes are natural slopes which have a geological history of landslides or creep, or both. The zone of colluvium along these slopes is potentially unstable because the shearing displacements associated with past movements have reduced the shear strength along the surface (or surfaces) of sliding or creep, or, both. When a cut is made in a colluvial slope, failure is frequently initiated along the existing surface (or surfaces) of sliding in the slope.

A section of Interstate Route 279 near Pittsburgh, Pa. passes through a zone of colluvium in the wall of a tributary valley of the Ohio River. When construction began on this section late in 1968, several slides were initiated along ancient landslide surfaces in the colluvium. These slides were investigated in a research project sponsored by the Pennsylvania Department of Highways and the United States Department of Transportation, Bureau of Public Roads, and they were described in a report by Hamel and Flint (4).

The location and geology of the slide site are described herein along with engineering and geological features of the slides. Shear strength parameters calculated for limiting equilibrium of a typical slide mass with the Morgenstern-Price method of slope stability analysis are presented and compared with shear strength parameters measured in laboratory tests on the

Note.—Discussion open until July 1, 1972. To extend the closing date one month, a written request must be filed with the Executive Director, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 98, No. SM2, February, 1972. Manuscript was submitted for review for possible publication on September 2, 1971.

[1] Asst. Prof. of Civ. Engrg., South Dakota School of Mines and Technology, Rapid City, S.D. and Principal, Hamel Geotechnical Consultants, Rapid City, S.D.

[2] Prof. of Geology, Univ. of Pittsburgh, Pittsburgh, Pa.

failure surface material. Conclusions are drawn concerning the level of shear strength mobilization in this typical slide.

DESCRIPTION OF SLIDE SITE

Location.—Interstate Route 279 (I-279) crosses the Ohio River 9 miles northwest of Pittsburgh on a bridge at Neville Island (see Fig. 1). This bridge carries the highway into the west wall of the valley of Kilbuck Run, a small stream which flows south into the Ohio River. The highway extends along the

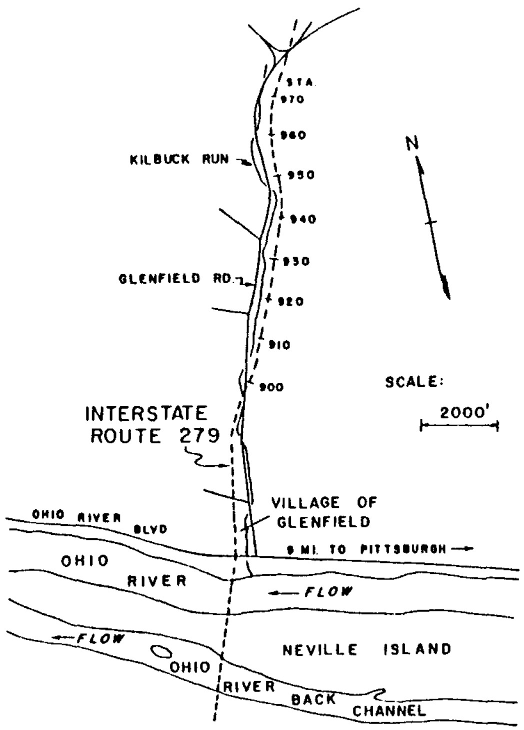


FIG. 1.—LOCATION MAP, INTEREST ROUTE 279

west wall of the valley above the village of Glenfield for approximately 0.9 miles north of the Ohio River where it crosses to the east wall of the valley for approximately 1.6 miles and then crosses back to the west wall on a third bridge. This highway alinement was chosen to avoid as much as possible the existing houses, roads, and stream on the valley floor.

Construction of this section of I-279 began in the autumn of 1968. Slides began soon after slope excavation commenced at several sidehill cut sections on the east wall of the valley between Station 899 and Station 955. Slide A, a typical slide described herein, was located between Station 906+50 and Station 909+50 (see Fig. 2).

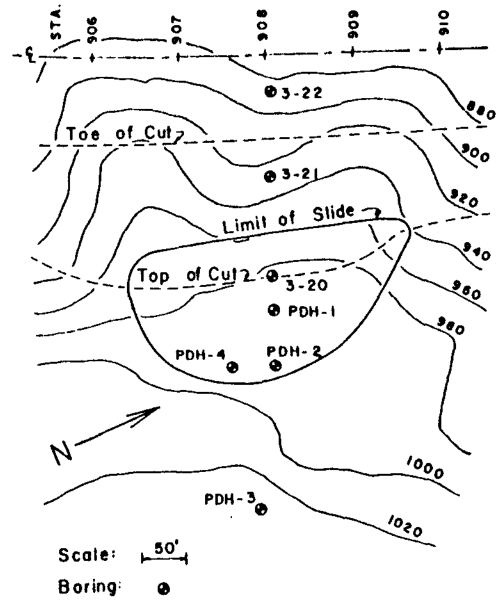


FIG. 2.—PLAN OF SLIDE A

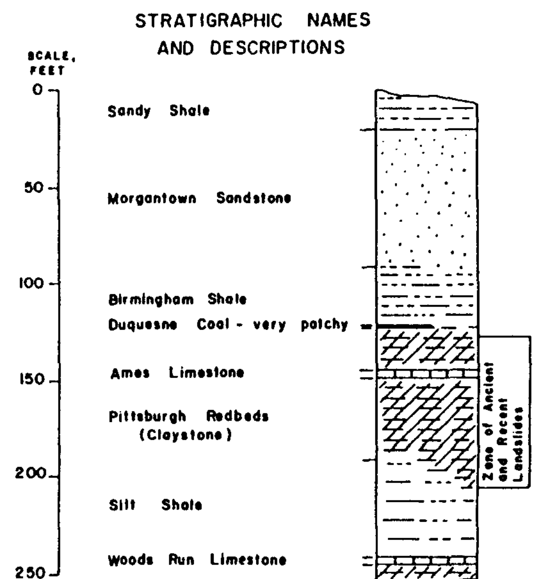


FIG. 3.—STRATIGRAPHIC SECTION

Other major slides occurred from Stations 899 to 904, 916 to 919, 920 to 922, 926 to 932, and 950 to 955. Not all of these slides were studied in detail, but they were inspected periodically and their significant features were noted. These features are reviewed in later sections of this paper in connection with the significant features of Slide A.

Stratigraphy and Structure.—The rocks at the site are typical cyclic sedimentary rocks of the Conemaugh Group (mid-Pennsylvanian age) of western Pennsylvania, West Virginia, and eastern Ohio. These cyclic sediments have been described in detail by Philbrick (7,8). They are essentially flat-lying strata consisting of repeated beds of sandstone, shale, and claystone with occasional thin beds of limestone and coal. Mixed lithologies such as silty shale, shaly sandstone, sandy shale, and calcareous claystone are common. The thickness of stratigraphic units ranges from a few inches to a few feet or tens of feet. Vertical changes in lithology are numerous. Even though there is an overall lateral continuity of rock type within a given stratum, there are local changes in most beds that may make generalizations of their lithology misleading.

The part of the stratigraphic section of particular interest at the slide site is shown in Fig. 3. An unnamed silt shale (above the Woods Run Limestone) lies at the base of this sequence. This silt shale ranges from 35 ft to 50 ft in thickness. The lower part contains thin interbedded sandstone layers; the upper part is composed of thin-bedded silt shale. The Pittsburgh Redbeds lying directly above this silt shale represent a weak zone of claystone 40 ft to 60 ft thick which has participated in extensive landsliding (both ancient and recent) in the study area. The failure surfaces of the landslides along I-279 all occurred at or slightly above the contact between these redbeds and the silt shale.

The Pittsburgh Redbeds are predominantly clayey but they also contain silt and fine sand fractions. Small limestone nodules are scattered through the redbeds in some places. Although dull red is the dominant color of the unit, pale green, gray, greenish gray, and purple colors are also common. The redbeds show no fissility. Overall, they are a unit of massive claystone containing a myriad of apparently randomly oriented, closely spaced fractures along which slickensiding is common. Extensive raveling of the claystone begins after only a few days exposure. With continued exposure, the claystone weathers into a red-brown sandy silty clay soil of medium plasticity.

The Ames Limestone lies immediately above the Pittsburgh Redbeds in the study area. The Ames is a fossiliferous limestone about 2 ft thick. It is generally a single bed but it may occur in two beds separated by a few inches of calcareous shale. Because the Ames Limestone is laterally persistent and because it can be rather easily recognized, it is a good marker bed.

Another redbed-type unit 10 ft to 15 ft thick overlies the Ames Limestone. This unnamed unit is composed of claystone of predominantly drab green and greeenish gray color with patches of dull red. The upper part grades into poorly bedded clay shale locally containing limestone nodules. The entire zone of 55 ft average thickness extending downward from the top of this unnamed claystone through the Ames Limestone and Pittsburgh Redbeds is a weak zone in which ancient and recent landsliding has occurred. Colluvial benches that are rather conspicuous features of the hillside topography indicate localities of ancient sliding in this zone.

The Duquesne Limestone and Duquesne Coal locally overlie this weak zone,

though they are generally absent in the study area. The weak zone is thus overlain by the Birmingham Shale, a unit of interbedded sandy shale and shaly sandstone with an average thickness of 30 ft. The Birmingham Shale contains well developed vertical joints which generally trend parallel to the valley walls.

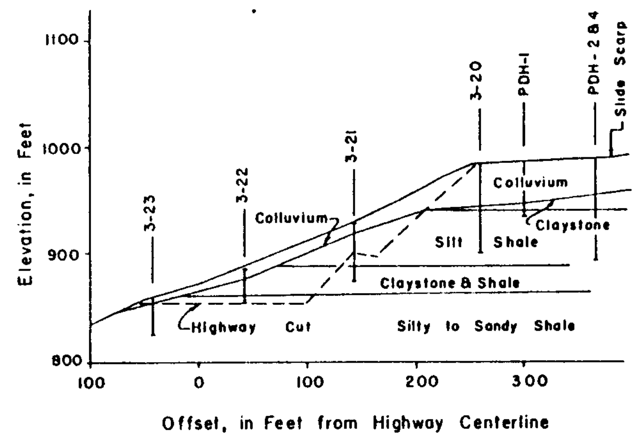


FIG. 4.—GEOLOGIC SECTION THROUGH SLIDE A

FIG. 5.—PHOTOGRAPH OF SHEAR ZONE AT STATION 928, MAY 20, 1969

The Birmingham Shale is overlain by the Morgantown Sandstone, a massive to thick-bedded unit whose base is disconformable. This irregular base truncates the Birmingham to different extents regionally so that in some places the base of the Morgantown Sandstone is in direct contact with the top of the previously mentioned weak zone.

The Morgantown Sandstone occurs at or near the level of ridge tops in the study area and, like the Birmingham Shale, contains numerous vertical joints. This is an ideal condition for the development of contact springs. Surface water infiltrates the Morgantown Sandstone and seeps down through it (and through the Birmingham Shale, where present) to the contact with the relatively impermeable claystone of the weak zone. The water then moves laterally along this contact to the ground surface where it emerges as a spring or as a line of springs. Spring water flowing over the surface of the slope below the level of the Morgantown Sandstone then permeates the colluvium produced by ancient landsliding in the redbeds. This seepage increases water pressures in the colluvium and contributes to alteration of the claystone. Both of these effects contribute to further landsliding.

Colluvium.—The material above about El. 940 is colluvium as shown in Fig. 4, a section through Slide A. The upper part of this colluvium was derived largely from the Morgantown Sandstone. It consists of a heterogeneous mixture of angular, gravel to boulder size sandstone fragments with variable amounts of and, silt, and clay. Some localized zones of this material consist almost exclusively of highly interlocked sandstone boulders; other localized zones are predominantly clayey. The lower part of the colluvium consists of clay and claystone derived from the Pittsburgh Redbeds. Infiltration of surface water and of spring water flowing from the base of the Morgantown Sandstone has produced a perched ground-water table on this relatively impervious clay-claystone colluvium.

The rocks below about El. 940 are in place. They consist of silty to sandy shales with some thin beds of claystone and sandstone. There was a relatively thin zone of colluvium and weathered rock along the valley wall below above El. 940 before slope excavation began. The colluvium in this zone is believed to have spilled over the edge of in-place shale at approximately El. 940 during the ancient landsliding. The boundaries between colluvium and weathered rock and between weathered rock and unweathered rock in the valley wall below about El. 940 were difficult to determine from the available boring information. The colluvium-rock boundary shown along the valley wall below El. 940 in Fig. 4 is therefore only approximate.

It should be noted that ground surface profiles along both walls of the valley of Kilbuck Run have the characteristic shape of landslide terrain. This is shown by the ground surface profile in Fig. 4. The surface of the colluvium near the top of the cut slope is nearly level. This colluvial bench or terrace is quite consistent on both sides of the valley. It occurs at approximately the stratigraphic level of the Ames Limestone (top of Pittsburgh Redbeds) and marks the upslope extremity of the ancient landslide masses.

The surface of the colluvium along the lower parts of the valley walls is hummocky and generally convex upward. This colluvium surface had a mean inclination of 23° along the valley walls of Kilbuck Run. The hummocky ground profile along with the numerous tilted trees on the valley walls indicates that surface creep is active in the colluvium.

DETAILS OF SLIDES

Shear Zones.—The failure surfaces of Slide A and other slides studied are located in clay-claystone colluvium at or slightly above the base of the Pitts-

burgh Redbeds. These failure surfaces are all believed to coincide with the failure surfaces of ancient landslides. Outcrops of these failure surfaces were studied in test pits excavated in the slope faces and in surface exposures.

Each of the failure surfaces was located in a shear zone from 1 in. to 12 in. thick. Most shear zones were located at the top of in-place silt shale and were generally overlain by claystone colluvium. The exact nature of the shear zone different from location to location. At Station 928, for example, the failure surface was a 1/4-in. to 1/2-in. thick seam of damp, medium-stiff, slickensided gray clay (see Fig. 5). The clay, which was underlain by weathered fissile silt shale, graded upward into relatively intact red and gray claystone. The failure surface at Station 909 (the north end of Slide A) consisted of a 2-in. thick seam of wet, soft, gray silty clay (see Fig. 6). It was underlain by a 3-in. zone of silt shale and claystone fragments in a silty clay matrix and then by in-place silt shale. This failure surface was overlain by above 6 in. of claystone fragments and silty clay and above that by fractured claystone.

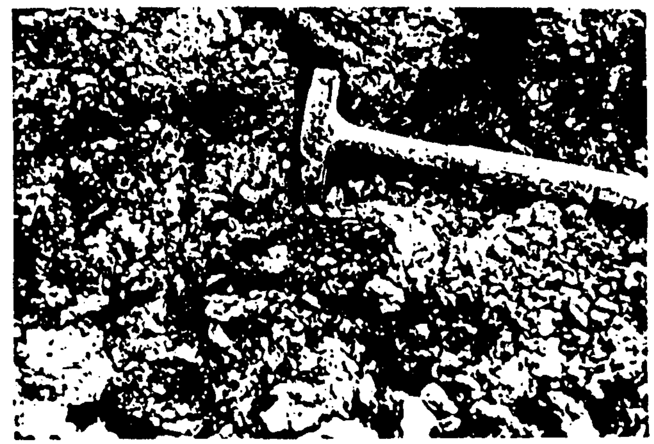


FIG. 6.—PHOTOGRAPH OF SHEAR ZONE AT STATION 909, MAY 13, 1969

Most of the shear zones studied along this section of I-279 were similar to the one at Station 909. They had three definite parts. The actual surface of sliding was generally a 1/4-in. to 2-in. thick seam of damp to wet, soft to medium-stiff, gray silty clay with variable amounts of sand. This seam was usually located near midheight of the shear zone. The parts of the shear zone above and below the clay seam consisted of a mixture of silty clay and angular, sand to gravel size claystone and shale fragments. Though the thicknesses of these upper and lower parts varied considerably from place to place, they were typically 2 in. to 3 in. The platy shaped claystone and shale fragments in the upper and lower parts of the shear zones were commonly alined parallel to the direction of movement. This is considered a macroscopic manifestation of the parallel particle arrangement reported by Skempton (9) for residual strength behavior. The shear zones were usually damp to wet and frequently showed appreciable seepage.

The preliminary X-ray diffraction study of the minerology of shear zone materials indicates that they contain quartz, kaolinite, illite, and expandable-lattice clay minerals. These expandable-lattice minerals (possibly vermiculite and one or more minerals of the smectite group) occur preferentially along the failure surfaces of the slides. Ground-water flow through the relatively permeable shear zones may have caused geochemical changes that resulted in the formation or concentration of these expandable clay minerals, or both, along the failure surfaces. This possibility is being studied further.

Index properties were determined for samples of clayey failure surface materials obtained from 11 locations between Station 899 and Station 953. Ranges and average values of these index properties are given in Table 1.

TABLE 1.—INDEX PROPERTIES OF CLAYEY FAILURE SURFACE MATERIALS

Property (1)	Range of values (2)	Average value (3)
Natural water content, as a percentage	17-31	24
Liquid limit, as a percentage	27-41	35
Plastic limit, as a percentage	19-29	24
Plasticity index, as a percentage	8-13	11
Clay fraction (minus 2 μ), as a percentage	14-29	21
Specific gravity	2.74-2.80	2.77

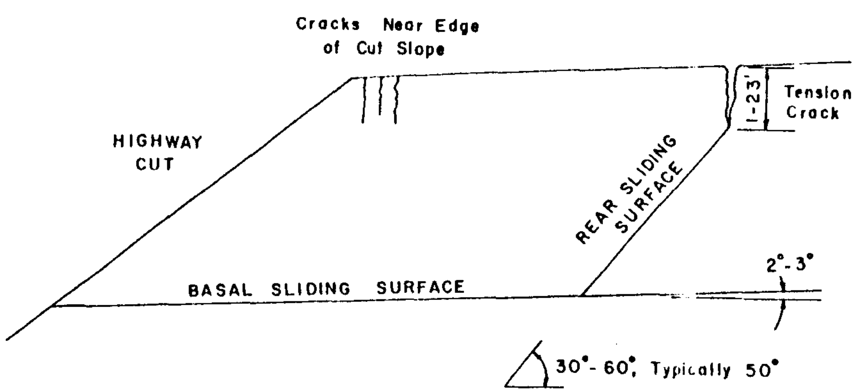


FIG. 7.—SCHEMATIC CROSS SECTION OF TYPICAL FAILURE MASS

The failure surface materials are well-graded mixtures of sand, silt, and clay size particles. There was considerable variation in particle size distribution from sample to sample. About half the particles in each sample were of fine sand and silt size; 40 % to 80 % of each sample passed the No. 200 sieve. It is difficult to classify these materials according to either the Unified Soil Classification System or the Revised Bureau of Public Roads Soil Classification System. As about half their particles are in the fine sand to silt size range, these materials generally fall at or near the border between the SM-SC and ML-CL soil classifications of the Unified System. They are A-4 or A-6 soils in the Revised Bureau of Public Roads System.

Geometric Details.—All slides observed along this section of I-279 were of the sliding wedge type. Each failure surface consisted of three parts as shown in Fig. 7. These parts are a basal surface of sliding, a rear surface of sliding and a tension crack at the ground surface. The basal surfaces of sliding typically dipped 2° or 3° and, as mentioned previously, probably coincided with ancient landslide surfaces near the top of in-place rock.

The rear surfaces of sliding dipped 30° to 60° along their upper portions where they crossed clay-claystone colluvium. Most of these rear sliding surface dips were on the order of 45° to 55°; a 50° dip is considered typical. There was commonly a 1/4-in. to 1/2-in. thick layer of soft to stiff, slickensided clay along these rear surfaces of sliding. It is not certain whether the rear sliding surfaces coincided with segments of ancient landslide surfaces, though it is suspected that some did.

The details of failure surface geometry at the intersections of the rear sliding surfaces with the basal sliding surfaces are not well known. It is considered likely that the rear sliding surfaces flattened somewhat or became curved above these intersections but this was not verified.

Tension cracks occurred at the rear of each failure mass observed along I-279. These cracks began to open in the early stages of sliding. The depth of tension cracks depended primarily on the nature of the colluvium near the ground surface. Deeper tension cracks generally formed in sand-sandstone colluvium than in clay-claystone colluvium. Tension crack depths of 1 ft to 10 ft were measured at the rears of slides in clayey colluvium; tension crack depths of 6 ft to 23 ft were measured at the rear of slides in sandstone colluvium.

Where the failure mass moved out into the highway cut, the toe of the mass was sometimes cantilevered as much as 1 ft over the edge of in-place rock. Where the toe material was relatively clayey and plastic, it sometimes bent under its own weight and flowed several inches down over the face of the cut slope before breaking off and falling to the bottom of the cut slope. Definite streamlines were observed in this flowing clay; rock particles in it were generally alined parallel to these streamlines. Nonplastic toe material simply tumbled over the edge of in-place rock and rolled or slid down the slope. This sliding over the edge of the cut induced flexural-type tensile stresses in the upper parts of the failure masses. Additional tension cracks then formed near the edge of the original cut slope as shown in Fig. 7.

ANALYSIS OF SLIDE A

History of Slide.—Excavation began in the vicinity of Slide A on November 26, 1968. The colluvium was excavated at an inclination of 1.25:1 or 39°. Tension cracks were reported above the cut slope between Stations 906 + 50 and 908 + 50 on December 4. This slide was first inspected by the senior writer on December 17 when excavation was down to about El. 920. On December 17 when excavation was down to about El. 920. On December 17, the slide extended from Station 906 + 50 to Station 908 + 50. There was a 3 ft to 5 ft vertical scarp about 100 ft back from the edge of the cut slope between Station 907 and Station 908 and horizontal movement of 3 ft to 4 ft had occurred. A relatively planar sliding surface was exposed at the base of the scarp at Station 908. This surface consisted of red clayey colluvium slickensided from the slide movement. The surface dipped 35° to 45° west (in the direction of the slide movement). There were also many open fissures in the top of the slide mass parallel to the edge of the cut slope. Fig. 8 is a photograph of the cut slope at Slide A.

Surface creep indications and new cracks were observed at the rear of the slide mass after a relatively warm period at the end of January, 1969.

FIG. 8.—PHOTOGRAPH OF CUT SLOPE AT SLIDE A, MARCH 22, 1969

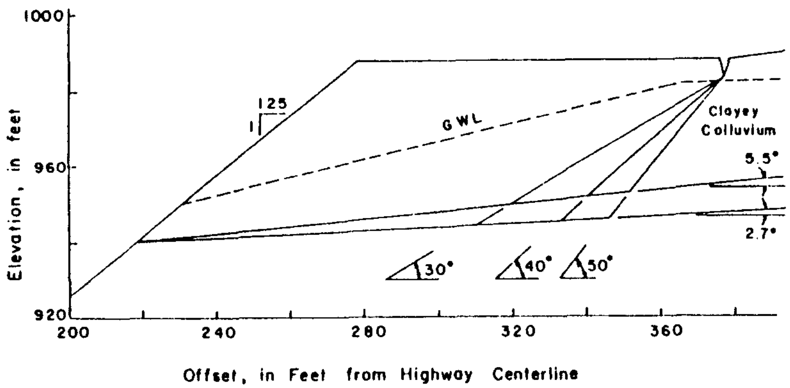


FIG. 9.—IDEALIZED CROSS SECTION THROUGH SLIDE A

The slide outline shown in Fig. 2 is that of February, 1969. Slide material began falling to the bench at El. 900 in appreciable quantities at that time. Movement of Slide A continued at least through November, 1969.

Idealized Slope Cross Section.—Before calculating sets of Mohr-Coulomb shear strength parameters for limiting equilibrium of the failure mass in Slide A, it was necessary to define an idealized cross section through the slide. This idealized slope cross section is shown in Fig. 9.

Two basal surfaces of sliding were assumed for the stability analyses. Both intersect the slope face at El. 940, the observed elevation of the failure surface outcrop. The lower basal surface of sliding has an inclination of 2.7°. It follows what is believed to be an ancient landslide surface at the base of the clay-claystone colluvium. The upper basal sliding surface has an inclination of 5.5°. It is located near the top of the clay-claystone colluvium at what may also be an ancient landslide surface.

A 5-ft tension crack was assumed to exist at the rear of the slide mass. Rear sliding surfaces of three different inclinations (30°, 40°, and 50°) were drawn from the bottom of this tension crack to the basal surfaces of sliding.

The colluvium in the failure masses analyzed for Slide A was assumed to have an average total unit weight of 145 pcf. Sandstone boulders and the relatively intact claystone in the colluvium typically have total unit weights of 155 pcf to 160 pcf; the more soil-like parts of the colluvium typically have total unit weights of 120 pcf to 130 pcf. The average unit weight of 145 pcf was therefore considered reasonable for the total failure mass.

The colluvium along each failure surface was also assumed to have the same average shear strength parameters. Inspection of samples from borings PDH—2 and PDH—4 and of exposures in open cracks indicated that the colluvium at the rear of Slide A was predominantly clayey. This clayey colluvium at the rear of the failure mass was, of course, derived from the same Pittsburgh Redbeds claystone material as the clayey colluvium along the base of the failure mass.

The ground-water level (GWL) shown in Fig. 9 is that corresponding to steady state seepage. It is based on springs observed at El. 950 in the slope face and the average water elevation of 982 observed in boring PDH-2 over the period from February to June, 1969. Boring PDH-2 pinched off near the failure surface of Slide A so water levels observed in it correspond to the perched water table above the failure surface. This ground-water level shown in Fig. 9 is considered a good estimate of ground-water conditions in the failure mass at the time of failure.

Calculation of Shear Strength Parameters.—The Morgenstern-Price (5,6) method of slope stability analysis was used to calculate effective stress Mohr-Coulomb strength parameters ϕ' and c' required for limiting equilibrium along each of the failure surfaces described in the previous section and shown in Fig. 9. The Morgenstern-Price method is a two-dimensional limiting equilibrium method of stability analysis which treats a failure surface of general shape and requires the failure mass to be in complete static equilibrium. Calculations were performed with the IBM 7090 computer at the University of Pittsburgh Computer Center using a Fortran II version (2) of the MGSTRN program developed by Bailey (1).

The values of ϕ' required for limiting equilibrium with $c' = 0$ and the values of c' required with $\phi' = 0$ were calculated along with the values of c' required with $\phi' = 10°$. These calculated values are given in Table 2. Values of tan ϕ' versus c' for the 2.7° basal sliding surface are plotted in Fig. 10. The tan ϕ' versus c' values for the 5.5° basal sliding surface are not plotted in that figure as they fall within the range of tan ϕ' versus c' plotted for the 2.7° basal sliding surface.

The effective shear strength of the failure surface material is believed to have been predominantly frictional in nature. The ϕ' values of 12.5° to 15.5°

calculated for limiting equilibrium with $c' = 0$ are therefore considered to represent the in situ strength of the failure surface material. It is impossible to calculate the exact value of friction angle mobilized in situ because of minor uncertainties concerning the geometry of the failure mass and the water forces acting on it. The writers believe, however, that the ϕ' value of 14° calculated for limiting equilibrium with the 2.7° basal sliding surface and the 40° rear sliding surface is a reasonable estimate of the friction angle mobilized along the failure surface of Slide A.

Comparison of Calculated and Measured Shear Strength Parameters.—A series of consolidated drained direct shear tests was performed on samples of material from the failure surfaces of ancient and recent landslides at 11 locations on the site. The index properties of these samples are summarized

TABLE 2.—STRENGTH PARAMETERS CALCULATED FOR LIMITING EQUILIBRIUM

Inclination of basal sliding surface, in degrees (1)	Inclination of rear sliding surface, in degrees (2)	Cohesion, c', in pounds per square foot (3)	Friction Angle, ϕ', in degrees (4)	Tangent ϕ' (5)
2.7	30	0	15.5	0.277
		320	10.0	0.176
		850	0.0	0.000
	40	0	14.0	0.249
		230	10.0	0.176
		770	0.0	0.000
	50	0	12.5	0.222
		150	10.0	0.176
		720	0.0	0.000
5.5	30	0	15.5	0.277
		300	10.0	0.176
		800	0.0	0.000
	40	0	14.5	0.259
		250	10.0	0.176
		750	0.0	0.000
	50	0	13.5	0.240
		200	10.0	0.176
		720	0.0	0.000

in Table 1. The direct shear test specimens were 2 in. square by 1/2 in. to 1 in. thick; they were sheared at a rate of 0.0045 in. per min. Each specimen was sheared repeatedly on the same failure surface using the shear box reversal technique suggested by Skempton (9) in order to obtain residual shear strengths. The sampling and testing of these materials were described in detail by Hamel (3) and Hamel and Flint (4).

The residual strengths of the failure surface materials were found to depend on the amount of sand size claystone and shale fragments present. Specimens containing appreciable quantities of these fragments had residual cohesion intercepts of 50 psf to 100 psf and residual friction angles of 20° to 25°. Specimens containing few sand size fragments had residual cohesion intercepts of essentially zero and residual friction angles of 8° to 18°. The residual friction angles of most of these latter specimens were in the range of

11° to 16°. Residual strength parameters $c' = 0, \phi' = 13.5°$ to 16° were determined from tests on three samples from the failure surface of Slide A.

The friction angles of 12.5° to 15.5° calculated for Slide A are in excellent agreement with the measured residual friction angles of the failure surface material (see Fig. 10). The fact that a residual-level strength was mobilized in the slide is consistent with the geologic history of the site. Movements in the ancient landslides were of sufficient magnitudes to reduce the strengths of the failure surface materials to residual levels. It is believed that the failure surfaces did not heal following the ancient landslides and that residual or near

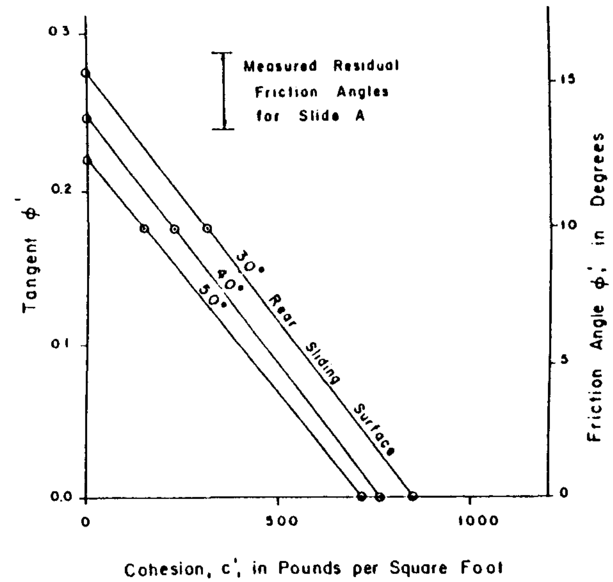


FIG. 10.—CALCULATED TAN ϕ' VERSUS c' FOR SLIDE A

residual strengths existed along the failure surfaces when slope excavation began.

CONCLUSIONS

Landslides encountered during construction to I-279 were caused by excavation of the toes of ancient landslide masses along the valley wall. This removal of toe support reactivated the ancient landslides. Both ancient and recent landslides occurred in a weak zone of the stratigraphic section composed primarily of the Pittsburgh Redbeds claystone. The ancient landslide masses were recognizable in the preconstruction topography of the valley wall.

Failure surfaces of the ancient and recent landslides were located at or near the base of the Pittsburgh Redbeds claystone. Movements in the ancient landslides reduced shear strengths along these failure surfaces to residual

8731 FAILURE OF COLLUVIAL SLOPE

KEY WORDS: Claystones; Colluvium; Cut slopes; Displacement; **Engineering geology; Landslides; Pennsylvania; Rock mechanics; Shear strength; Slope stability; Soil mechanics**

ABSTRACT: Interstate Route 279 crosses the Ohio River 9 miles northwest of Pittsburgh, Pa., and continues northward along Kilbuck Run tributary valley. One section of the highway passes through a zone of colluvium on the wall of the tributary valley. When construction of this section began in 1968, several slides were initiated along ancient landslide surfaces in the colluvium. The relationship among the landslides, a weak claystone zone in the stratigraphic dequence (the Pittsburgh Redbeds), and the valley wall topography are analyzed. One typical landslide which was studied in detail is described. Friction angles of 12.5° to 15.5° were calculated for limiting equilibrium of the failure mass and residual friction angles of 13.5° to 16° were determined from repeated direct shear tests on failure surface materials. It is concluded that the average shear strength mobilized along the failure surface material. This residual strength condition is attributed to the shearing displacement of the ancient landslide.

REFERENCE: Hamel, James V., and Flint, Norman K., "Failure of Colluvial Slope," **Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 98, No. SM2, Proc. Paper 8731**, February, 1972, pp. 167-180

FAILURE OF COLLUVIAL SLOPE[a]

Errata

The following corrections should be made in the original paper:

Information Retrieval Abstract, line 6: change "... stratigraphic dequence ..." to read "... stratigraphic sequence ..."

Information Retrieval Abstract, line 11: change "... the average shear strength mobilized along the failure surface material." to read "... the average shear strength mobilized along the failure surface of this typical slide was the residual strength of the failure surface material."

Page 168, Fig. 1 caption: change "Fig. 1.--Location Map, Interest Route 279" to read "Fig. 1.--Location Map, Interstate Route 279"

Page 172, paragraph 2, line 5: change "... and, silt, and clay." to read "... sand, silt, and clay."

Page 172, paragraph 3, line 4: change "... above El. 940 ..." to read "... about El. 940 ..."

Page 173, paragraph 1, line 4: change "... different from location to location." to read "... differed from location to location."

Page 173, paragraph 1, line 11: change "... above 6 in. ..." to read "... about 6 in. ..."

Page 173, last line: change "The preliminary X-ray diffraction study ..." to read "A preliminary X-ray diffraction study ..."

Page 174, line 4 below Fig. 7: change "... 40% to 80% of each sample ..." to read "... 50% to 80% of each sample ..."

Page 175, paragraph 2, line 6: change "... surfaces conincided with ..." to read "... surfaces coincided with ..."

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[a] February, 1972, by James V. Hamel and Norman K. Flint (Proc. Paper 8731).

Page 175, paragraph 4, line 7: change "... rear of slides ..." to read "... rears of slides ..."

Page 175, paragraph 6, lines 5 and 6: Delete second "On December 17 when excavation was down to about El. 920 ..."

Page 179, paragraph 2, line 1: change "... construction to I-279 ..." to read "... construction of I-279 ..."

Page 180, last paragraph, last line: change "... Pennsylvania Department of Highways of the Bureau of Public Roads." to read "... Pennsylvania Department of Highways or the Bureau of Public Roads."

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Claystone slides, Interstate Route 79, Pittsburgh, Pennsylvania, USA

JAMES V. HAMEL
Hamel Geotechnical Consultants, Monroeville, Pennsylvania, USA

WILLIAM R. ADAMS, Jr
Allegheny County Department of Engineering & Construction, Pittsburgh, Pennsylvania, USA

1 INTRODUCTION

Landsliding is a significant geologic process and geotechnical consideration in the northern Appalachian Plateau of the eastern United States, especially in the vicinity of Pittsburgh, Pennsylvania (Fig. 1). Flat-lying interbedded strong and weak sedimentary rocks have been acted upon by erosion, stress relief, weathering, creep and sliding processes to produce masses of marginally stable colluvium on many of the steep hillsides common to the area. The precarious equilibrium of colluvial masses is frequently upset by heavy precipitation and by construction activities, e.g., removal of toe support, loading the slope, changing surface and subsurface drainage (Gray et al. 1979, Hamel 1980).

Some of the largest colluvial landslides reported in this region occurred in 1968 and 1969 during construction of Interstate Route 79(I-79) 16 km northwest of Pittsburgh (Fig. 1). The recent landslides were initiated by excavating toes of ancient landslides which had occurred in a thick sequence of weak claystones. These recent landslides were studied in detail; clay samples from failure surfaces were obtained and tested in the laboratory to determine index properties and peak and residual shear strengths (Hamel 1969, Hamel & Flint 1969, 1972).

The purpose of this paper is to describe these landslides with emphasis on sampling and testing of failure surface clays.

2 GEOLOGIC SETTING

A section of I-79 extends from Sta. 900 to Sta. 970 along the east wall of the valley of Kilbuck Run, a tributary of the Ohio River (Fig. 1). I-79 was located here in sidehill cuts to avoid the pre-existing

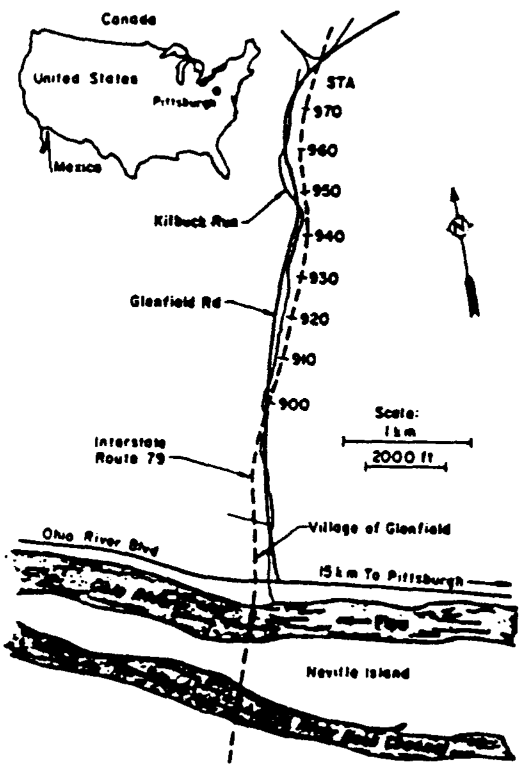

Fig. 1 Location Map

road and dwellings along the stream in the valley bottom.

Site geology has been described by Hamel & Flint (1969, 1972). Rock strata are flat-lying cyclothemic, coal measure types of the Pennsylvanian (Carboniferous) age Conemaugh Group. Sandstones, shales and claystones are predominant in the valley walls. Strata dip west, i.e., out of the east valley wall, at approximately 2%.

For purposes of this paper, the geologic setting can be characterized by the generalized slope cross-section in Fig. 2. The lower portion of the slope consists primarily of silty to sandy shale. This is overlain by a 20 m thick weak rock interval consisting principally of claystone. The weak rock interval is overlain by 20 to 30 m of massive sandstone which extends to the ridge top.

Deep seated ancient landsliding occurred in the weak rock interval and extended up to the ridge top (Fig. 2). This ancient landsliding appears to have involved lateral translation (block sliding) with some rotation (slumping) at the rears of slide masses. Conspicuous landslide benches developed on both valley walls at the level of the weak rock interval (Figs. 2 & 3). These benches had maximum widths of 60 m perpendicular to the slope.

The ancient landsliding is inferred to have occurred in Pleistocene time. Continental glaciers stopped 70 km north of Pittsburgh but the area was subjected to a periglacial climate with associated heavy precipitation. Details of Pleistocene history are sparse but two facts are known. Early in Pleistocene time, ice sheets blocked north flowing streams of the region, causing ponding in many valleys until new southerly drainage outlets were developed. Later in Pleistocene time, meltwater from retreating glaciers caused major downcutting in many valleys, including that of the Ohio River and hence that of Kilbuck Run.

Lateral stress release accompanying valley downcutting (Ferguson & Hamel 1981), probably in conjunction with high pore and joint water pressures in valley wall rock due to periglacial precipitation, apparently caused rock sliding along the valley walls. The basal failure surface of these slides was located at or near the base of the weak rock interval, probably along a bedding plane shear zone due to valley stress relief. Sliding extended up through the massive sandstone to the ridge top; open vertical stress relief joints allowed sandstone blocks to detach from the valley wall (Fig. 2).

This sliding probably occurred before the end of the Pleistocene Epoch some 10,000 years ago and the maximum thickness of colluvium probably exceeded 30 m. Over the years, slide masses achieved states of quasi-equilibrium and were acted upon by weathering, erosion and creep. Colluvium in the old slide masses was highly variable in composition; lower portions generally consisted of decomposed claystone while upper portions generally consisted of gravel to boulder size sandstone fragments with locally variable sand, silt and clay contents. Colluvium thicknesses ranged from about 10 to 30 m at and above the weak rock interval and 1 to 3 m on the valley wall below the weak rock interval (Figs. 2 & 3).

3 RECENT SLIDING

Ancient slide masses on the valley walls were not recognized during highway design. Slopes of 1 (vertical) on 1-1/4 (horizontal) were designed for sidehill cuts in colluvium (Fig. 3). Sidehill excavations for I-79 began in late 1968 and continued through 1969. These excavations removed the toes of old slide masses and initiated progressive failures which propagated upslope. By mid-1969, re-activated ancient slide masses extended discontinuously for a distance of 1.6 km along the valley wall from approximately Sta. 900 to Sta. 955 (Fig. 1).

Most of the re-activated slides involved translational movements of well-defined blocks extending as much as 100 m along the valley wall. The failure surface of a typical block slide had three parts: basal sliding surface, rear sliding surface and tension crack. Basal sliding surfaces were located at or near the base of the weak rock interval (Fig. 2) and typically dipped 2° to 3° out of the slope, essentially parallel to bedding. Rear sliding surfaces commonly dipped 45° to 55°. Vertical to sub-vertical tension cracks on the order of 1 to 7 m deep extended from the ground surface to rear sliding surfaces.

Because of the magnitude of the slide problem, its occurrence during a major highway construction project, and the low strengths of the failure surface materials, the most economical solution involved removal of substantial portions of the slide masses. Slopes were flattened to 1 on 3 from a bench at or below the base of colluvium (Fig. 3).

Slope excavation was completed in 1970. Since then, movements have progressed further into colluvial remnants on the valley wall and, in some locations, into previously stable rock which had been loosened by valley stress relief (Ferguson & Hamel 1981). These continuing movements have not been studied in detail but they have been observed by the writers intermittently over the past decade. During this period, sandstone blocks 20 m thick have detached from the valley wall and begun to move slowly downslope. These movements pose no threat to highway use.

4 SAMPLING FAILURE SURFACE CLAYS

Most of the basal failure surfaces consist-

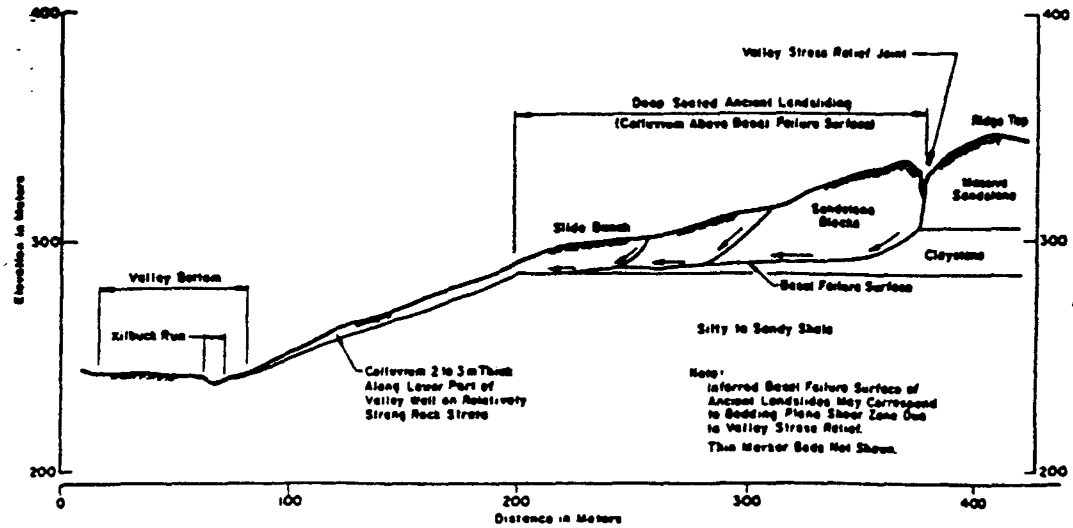

Fig. 2 Generalized Slope Cross-Section, Sta. 908

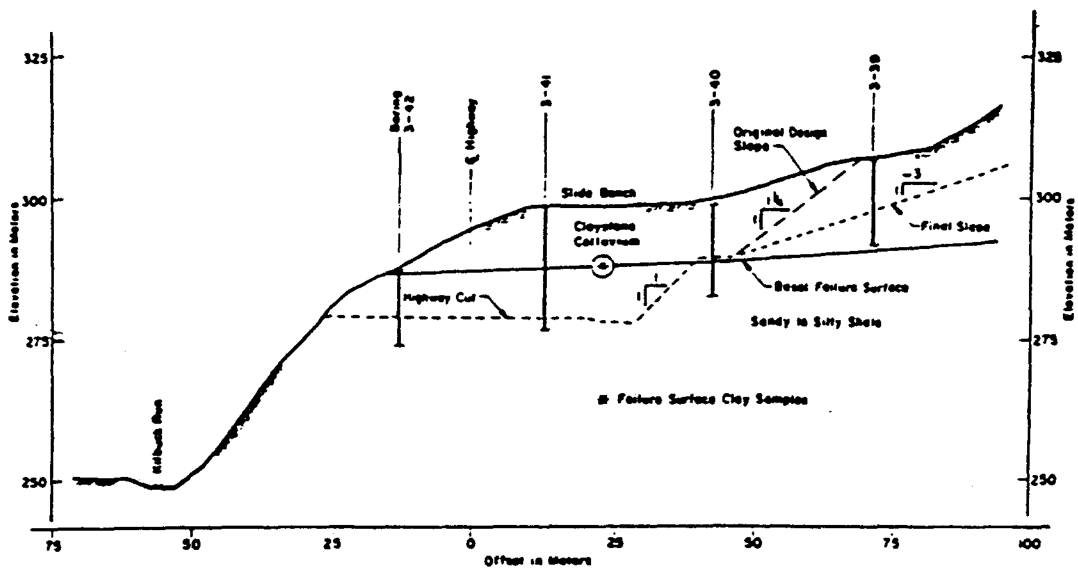

Fig. 3 Slope Cross-Section, Sta. 928

ed of 5 to 50 mm thick seams of damp to wet, soft to medium stiff, gray silty clay with variable sand content in a shear zone of silty clay and angular, sand to gravel size claystone and shale fragments. At Sta. 928, the basal failure surface was a 3 to 12 mm thick seam of damp, medium stiff, gray silty clay which graded up into soft but relatively intact red and gray claystone (Figs. 3 & 4).

Samples of failure surface clays were obtained in 1969 at 11 locations along the 1.6 km section of active landsliding. Ten of the samples were from basal failure surfaces in the interval from Sta. 900 to Sta. 928; the eleventh was from a rear sliding surface at Sta. 952.

All of these samples were taken from failure surfaces exposed by slide movements or test pits. Representative disturbed samples

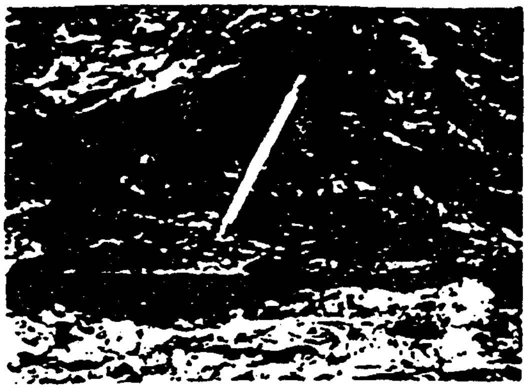

Fig. 4 Failure Surface Clay Seam, Sta. 928; Pencil 120 mm Long

were carefully scraped from thin failure surface clay seams. Where possible, relatively undisturbed block samples were obtained.

The situation at Sta. 928 was unique. Undisturbed samples of failure surface clay and overlying claystone were cut from blocks of claystone that broke off as the slide mass moved out and cantilevered over underlying shale. The slide mass was moving at an estimated rate of 100 mm/hr at that time and the very fresh failure surface clay was slickensided and glistening with moisture.

Observations during sampling in 1969 indicated that failure surface clay seams of these types can not generally be sampled with standard soil and rock drilling techniques, i.e., 35 to 51 mm diameter split barrel soil samplers or 54 mm diameter rock core barrels. The writers' experience to date indicates that test pits, trenches or large diameter drill holes are necessary for reliable location and sampling of such failure surface clay seams.

5 TESTING AND TEST RESULTS

Index properties were determined for all samples. Failure surface clays had natural water contents of 17%-31%, liquid limits of 27%-41%, plactic limits of 19%-29% and clay size fractions (-2μm) of 14%-29%. Soft claystone 40 mm above the failure surface of Sta. 928 had a natural water content of 10%, a liquid limit of 27%, a plastic limit of 22% and a clay size fraction of 10%.

The failure surface clays were well graded mixtures of clay, silt and sand size particles with many of the latter being claystone fragments. About half the particles in each sample were of fine sand and silt sizes; samples had fractions of 40% to 80% finer than 74μm. The failure surface clays were typically CL or ML soils in the Unified Soil Classification System.

Multiple reversal direct shear tests were performed to determine peak and residual shear strengths. The shear box was 50 mm square and specimen heights ranged from 12 to 25 mm. Specimens were submerged in water, consolidated in the shear box and sheared at an average displacement rate of 0.11 mm/min.

Nominal normal stresses of 80 to 270 kPa were used along with multi-stage testing techniques. Where several specimens from a sample were tested, each test began with a different normal stress so that peak as well as residual strengths could be determined. After residual strength was obtained under the initial normal stress, the normal stress was varied to obtain additional residual strength data. Where only one specimen (typically a remolded specimen from a disturbed sample) was tested, an initial normal stress of about 270 kPa was generally used.

Typical test results are shown in Figs. 5 & 6 for claystone and failure surface clay from Sta. 928 (Fig. 3 & 4). The claystone had peak cohesion intercept $c' = 0$-12 kPa and peak friction angle $\phi' = 26.5°$-30°. The failure surface clay had peak $c' = 7$ kPa, $\phi' = 13°$; maximum residual strength essentially equal to peak strength, and minimum residual $c' = 0$, $\phi' = 11°$. These results indicate that field shearing displacements had reduced the strength of the failure surface clay to a residual or near-residual condition and also that both peak and residual strengths of failure surface clay were significantly less than respective strengths of claystone from above the failure surface. This points out the need to sample failure surfaces in weak rock like the claystone, rather than just sample the rock itself, if appropriate strength values are to be obtained in situations like that at Sta. 928.

Tests on failure surface clays from other locations along this section of I-79 gave peak $c' = 8$-38 kPa, $\phi' = 19°$-25° and residual $c' = 0$-3 kPa, $\phi' = 8°$-25°. The higher peak cohesion intercepts are attributad to the presence of appreciable quantities of sand size quartz and claystone particles and, in some cases, to the effects of specimen remolding and reconsolidation in the laboratory. The higher residual friction angles are attributed to the presence of appreciable quantities of sand size particles. Peak $c' = 7$-14 kPa, $\phi' = 13°$-24° and residual $c' = 0$, $\phi' = 8°$-18° are considered typical for the I-79 failure surface clays with effective normal stresses of 80 to 270 kPa.

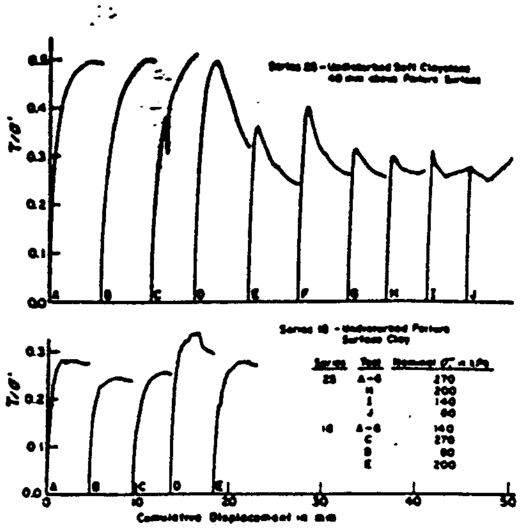

Fig. 5 Stress Ratio vs. Displacement

The variation of peak and residual strengths of failure surface clays along this section of I-79 may be typical for extensive landslides in weak rock and in colluvium derived from such rock. This variation reinforces the need for careful sampling and testing of such materials.

6 SUMMARY AND CONCLUSIONS

Extensive landsliding occurred in a thick interval of weak rock (mainly claystone) high on the east wall of Kilbuck Run valley, probably during Pleistocene time. Construction of I-79 in 1968 and 1969 removed the toes of ancient slide masses and initiated new movements along existing failure surfaces. Most of the colluvium was removed but colluvial remnants and loosened rock at the rears of ancient slide masses have continued to move slowly over the past decade. These continuing movements pose no threat to the highway.

Failure surfaces of the re-activated slides were studied and clays from them were sampled and tested in 1969. This work indicated the need for extreme care in locating and sampling such failure surfaces. Peak and residual strengths of failure surface clays were generally much lower than respective strengths of clays and claystones short distances above and below the failure surfaces. Typical values of peak and residual strengths were presented above.

7 ACKNOWLEDGEMENTS

The 1969 investigation of I-79 slides was supported by the Pennsylvania Dept. of Highways and U.S. Dept. of Transportation under a research grant to the University of Pittsburgh. Numerous individuals from the Dept. of Highways and University of Pittsburgh assisted in this investigation (Hamel & Flint 1969, 1972).

James H. Wylie drew the figures in this paper which was typed by Barbara Susalla. Elizabeth A. Hamel also assisted in preparation of this paper.

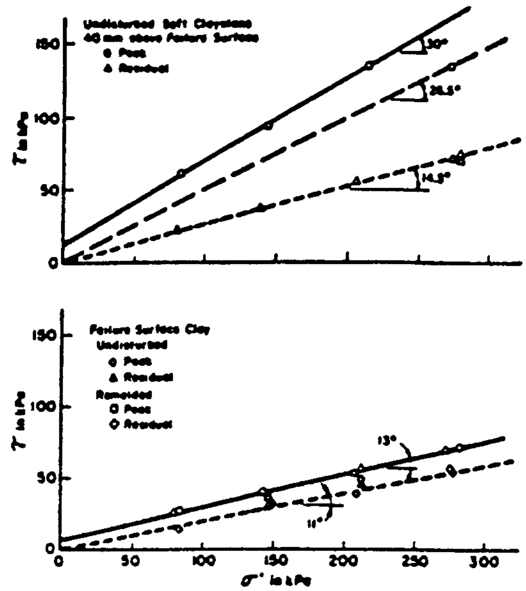

Fig. 6 Mohr-Coulmb Strength Envelopes

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higher one. Therefore porepressure is mainly decided by the presence and the capacity of water-pathes.

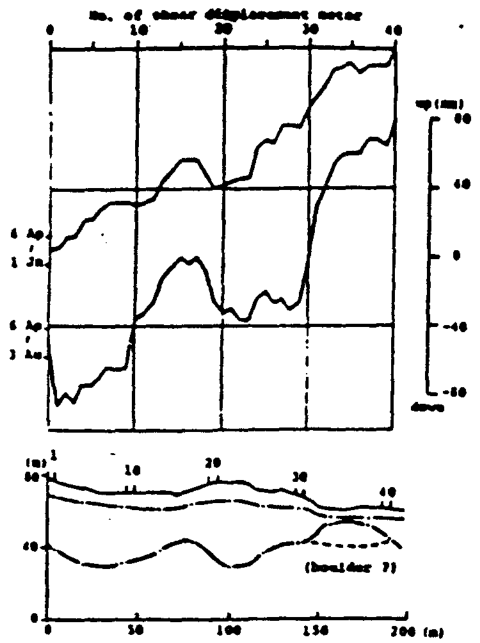

Fig. A The influence of the underground valley on the vertical movement

Abb. A. Der Einfluß des unterirdischen Tales auf die Vertikal-Bewegung

Discussion on:

J.V. Hamel and W.R. Adams Jr. (U.S.A.)
CLAYSTONE SLIDES, INTERSTATE ROUTE 79, PITTSBURGH, PENNSYLVANIA, U.S.A. (Vol. I, pp. 549 – 553)

Question by:
R. SANCIO (Venezuela)

I have a short question and a comment. Did the slope finally stabilize after the 3 (horizontal) on 1 (vertical) cut was made?

If the slide continued to move after taking the corrective measures, it would confirm the theory was well as the experience. Any slope steeper than the slip surface would fail after the cohesion has been lost, and any material resting on a cohesioness surface, steeper than the angle of friction, would continue moving downhill.

Authors' reply:

Dr. R.T. Sancio asked whether flattening the excavation slope solved the I-79 land-slide problem. Dr. B.A. Chappell had a similar question and he also asked about the variation in friction angle with location along this section of I-79.

The excavation slope of 1 (vertical) on 3 (horizontal) was a practical compromise based on economics and right of way considerations during highway construction. This excavation extended from a bench at or below the base of colluvium (Fig. 3) upslope to the top of colluvium or, more commonly, to a near-vertical joint face in the massive sandstone overlying the weak claystone (Fig. 2).

Excavation of colluvium to the inclination of 1 on 3 did not stop slope movements unless this excavation removed all (or essentially all) of the colluvium at a given location. Where sizable remnants of colluvium were left beneath or upslope from the 1 on 3 excavation, these remnants have typically continued to creep or slide intermittently toward the highway. At some locations, movements have progressed upslope into previously stable rock which had been loosened by valley stress relief. Creeping or sliding masses of colluvium and loosened rock are sufficiently far from the highway that they pose no threat to its use. Moreover, experience at this site and elsewhere indicates that re-activated ancient landslide masses of these types seldom move rapidly. The probability of rapid and catastrophic downslope movement of colluvial remnants at the I-79 site is considered essentially nil.

It is not surprising that colluvial remnants at the I-79 site continue to creep or slide intermittently downslope in view of the low residual strengths of the failure surface materials. Surface water infiltration and groundwater flow from the massive sandstone to the colluvium (Fig. 2) also contribute to the continuing movements of the latter (Hamel & Flint 1972).

Peak and residual shear-strengths, and especially the respective friction angles, of failure surface clays varied considerably with location along the valley wall at the I-79 site. Important factors in variaction of the measured friction angles are:

1. Depositional changes in the claystone which is far from homogeneous.
2. Different movement histories of ancient landslide masses.
3. Extreme differences in shear zone and failure surface geometry and composition.
4. Localized geochemical alteration (including chemical weathering) of shear zone materials and failure surface clays.
5. Inevitable difficulties in sampling failure surface clay seams without contamination by adjacent soil particles.

Experience in Western Pennsylvania and elsewhere indicates that the peak and residual shear strengths of failure surface clay seams like those at the I-79 site depend primarily on three inter-related factors: amount of particles of sand size and larger (mainly rock fragments of various types); soil structure, including fabric, composition, and inter particle forces; and degree of weathering or alteration (Hamel 1980). All of these factors varied considerably with location at the I-79 site, as noted above, so the friction angles also varied with location.

Results shown in the paper for samples from 11 locations along the 1.6 km section of active landsliding are insufficient for statistical analysis but the following points are significant from a practical viewpoint. Soil mechanics index properties displayed much less variation than did peak and residual friction angles of the failure surface clays. Separate analyses disclosed no apparent correlation of residual friction angles with clay size fraction, liquid limit, plastic limit, or plasticity index. Many of the fine sand and silt size particles in the failure surface clays were probably play claystone or shale fragments. When these particles became oriented parallel to the surface of shearing, whether in the field or in the laboratory, the resultant residual friction angle was probably close to that of the clay mineral constituents.

The I-79 case history emphasizes the need for recognition of ancient landslides in connection with planning, design, and construction of engineering works. Although this section of I-79 would probably have had essentially the same alignment even if the ancient landslides had been recognized in planning and design, such recognition would have substantially reduced problems and cost increases during construction.

Criteria and procedures for recognition of ancient landslides have improved as a result of the I-79 experience (Hamel 1969, Hamel & Flint 1969, 1972) and other work over the past decade (Gray et al. 1979, Hamel 1980) but geotechnical practice in Western Pennsylvania, for the most part, fails to reflect these advances. Many land-slide problems in this region continue to result from construction-related disturbances of un-recognized ancient land-slide masses derived in large part from weak rock strata. These disturbances includes excavation of slide toes, loading slide masses (commonly with fill), and diverting surface and subsurface drainage into slide masses.

Experience of the writes and others in dealing with ancient landslides of the region can be summarized as follows. The first and most important aspect is recognition of the landslide mass and estimation of its size. Smaller ancient landslide messes can sometimes be removed entirely or stabilized by some combination of drainage, buttess fills, or retaining structures. If the ancient landslide mass is large (as at the I-79) site), it is best avoided if at all possible. Stabilization of a large reactivated ancient landslide mass is seldom easy or inexpensive. Such stabilization commonly involves extensive excavation (as at the I-79 site) or provision of very substantial buttress fills or retaining structures. Experience with drainage measures for stabilization of large re-activated ancient landslide masses is limited and the results to date suggest that drainage alone will not effectively stabilize such landslide masses.

ERRATA

Preprint Vol. 2,p. 192

colum 1, line 3 should read:

tively undisturbed block samples were ob-

column, 2, paragraph 3, lines 6-9 should read:

30° along with residual $c' = 0$, $\phi' = 14.5°$.
The failure surface clay had peak $c' = 7$ kPa, $\phi' = 13°$; maximum residual strength essentially equal to peak strength: and minimum residual $c' = 0$, $\phi' = 11°$.

Discussion on:

K. Sugawara, H. Okamura, K. Kaneko and O. Kimura (Japan)
MECHANICAL BEHAVIOR OF COAL SEAM SANDWICHED BY RIGID STRATA (Vol. I, pp. 561 – 566)

Question by:
Y. HIRAMATSU (Japan)

I appreciate very much the presentation of Mr. Sugawara et al, but I'd like to ask Mr. Sugawara some questions.

(1) Have the coal bursts been prevented only by the stress-relief boring of proper depth?
(2) What do you think about the mechanism of the coal bursts of the longwall coal seam in Miike coal mine? Are there any differences in general between the mechanism of coal bursts of pillars and those of longwall coal seams?

Authors' reply:

Thank you very much for your interesting question for our research. I believe, for the safety, the careful destressing is indispensable and the parallel destress drilling is effective. But only by the destress drilling, we cannot completely prevent the hazard of coal bursts. The most effective measure is to control the rate of face advance with judging the proper value of the factor α. Also, the combination of destress blasting and water injection is useful.

As to the mechanism of the coal bursts at longwall face, I think, it is closely concerned with the stress concentration as well as the brittleness of coal seams and the magnitude of stress concentration has the close relation with the stiffness of strata. It is clear that in the case of hard and dry coal seam with discontinuities of only horizontal direction and which is sandwiched by rigid sandstones as in the Miike Yothuyama mine, the stress concentration draws near the face with cutting and the hazard of coal burst increases.

Finally, as to the difference of bursts of pillars and those of longwall face, I think, no difference in the mechanism of occurrence, but concerning with the scale of stress concentrated area, we can find out a clear difference.