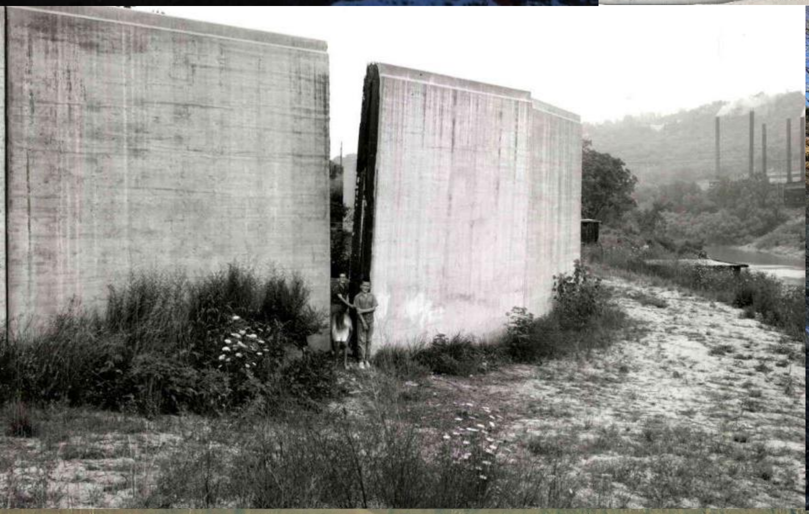
Ohio River Valley Soils Seminar LI

Geohazards: Challenges to Geotechnical Engineering



November 2, 2021
Cincinnati, Ohio

Proceedings of the 51st Annual Ohio River Valley Soils Seminar

Geohazards: Challenges to Geotechnical Engineering

November 2, 2021
Hilton Netherland Plaza
35 West 5th Street
Cincinnati, Ohio

ORVSS LI Planning Committee:

Russ Gatermann, P.E.
Terracon

David Mueller
Goettle

Rebecca Scherzinger, P.E.
S&ME

Donald B. Thelen, P.E.
Geotechnology, Inc. (retired)

Mike Abruzzo
Goettle

Agenda – Tuesday, November 2, 2021

6:30-7:15 am	Exhibitor Setup
7:15-8:00 am	**Attendee Sign-In/Registration**
8:00-8:10 am	**Welcome Remarks**
8:10-8:25 am	"Short Summary of Cincinnati's Glacial Geology" – Nancy Dendramis, P.E., Terracon Consultants, Inc.
8:25-8:55 am	"The Recognized Geotechnical Engineers List of Greater Cincinnati" – Mark T. Bowers, Ph.D., P.E., University of Cincinnati
8:55-9:40 am	"Earthquake Drains to Mitigate Liquefaction Hazards: Theory and Implementation" – Sarah Ramp, P.E., Menard Group USA
9:40-9:55 am	**Break**
9:55-10:25 am	Roundtable Discussion: Government Agencies and Geohazards – Rich Pohana, P.E., City of Cincinnati; Paul Gruner, P.E., P.S., Montgomery County Engineer; Ryan Jeffries, P.E., USACE
10:25-11:10 am	"Emergency & Limited Access Geohazard Mitigation: Design-Build Solutions" – Dylan Jones and Jose LuQuin, P.E., GeoStabilization International
11:10-11:35 am	"History of Tilting Floodwalls in Covington, Kentucky" – Terry M. Sullivan, P.E., USACE
11:35-12:25 pm	**Lunch**
12:25-12:30 pm	Keynote Speaker Introduction
12:30-1:30 pm	**Keynote Speaker:** "Lessons Learned from Failures – Walls and Slopes" – Liz Smith, P.E., G.E., Terracon Consultants, Inc.
1:30-2:15 pm	"Drilled Displacement Elements for Liquefaction Mitigation" – W. Morgan NeSmith, P.E., Berkel
2:15-3:00 pm	"Case Study for Exploratory Drilling and Grouting in Karst Geology at Rough River Dam" – Steven Shifflett, P.E., USACE
3:00-3:15 pm	**Break**
3:15-4:00 pm	"Columbia Parkway Landslide Stabilization" – Joe Hauber, P.E., Geotechnology, Inc. and Rich Pohana, P.E., City of Cincinnati
4:00-4:45 pm	"Liquefaction, Lateral Spreading, and Pile Foundations in the Ohio Valley" – Enrique Farfan, Ph.D., P.E., Stantec
4:45-4:50 pm	**Closing Remarks**

In Remembrance

Dr. Andrew Bodocsi
1931-2021

Andrew Bodocsi was born July 11, 1931 in Budapest, Hungary. He passed away in Cincinnati, Ohio on March 12, 2021 at the age of 89. Andy met his wife-to-be, Jean, in April 1974. They were married August 30, 1975 and made Cincinnati their home. Jean passed away on January 4, 2018. Andy had an extraordinary youth, surviving the Siege of Budapest in World War II. Andy graduated in 1956 with his undergraduate degree in Civil Engineering from the Technical University of Budapest. He lived through the 1956 Revolution in Hungary and escaped through the "Iron Curtain" into Austria in November 1956. From Austria he emigrated to the United States and settled in Cincinnati in the Spring of 1957. Andy began his academic career as an Instructor in the Civil Engineering program at the University of Cincinnati in 1960. He completed a Masters degree in 1961 and his doctorate in 1966. Andy changed his emphasis from bridge engineering to Soil Mechanics and Foundation Engineering during his graduate studies. Andy was promoted to Assistant Professor in 1967, to Associate Professor in 1972, then to Professor in 1996. He served as Assistant Department Head for eight years. He developed a strong research program with funding from the U.S. Environmental Protection Agency, the Ohio Department of Transportation, and other agencies. He was an author of seven conference papers, eight refereed journal articles, two books, and numerous research reports. He was a gifted teacher. Upon retirement from the University of Cincinnati in 1998, he joined the H.C. Nutting Company (now Terracon) as Senior Consultant and served in that capacity until early 2020. He served as a consultant on hundreds of geotechnical engineering investigations. Some of Andy's design projects include the foundation design of the Great American Insurance Building in downtown Cincinnati; the lowering of the playing field of Ohio State University football stadium to increase seating capacity; and the railroad overpass bridge foundation design of State Route 747 in Cincinnati. Andy was a man of integrity and wisdom. He had a wonderful laugh, an extraordinary memory, and an amazing talent for engineering. He will be missed by friends, family, and the engineering community.

George J. Thelen
1936-2020

It is with a sad heart that we acknowledge the passing of one of our own from the engineering community during this past year. George Jay Thelen passed away on September 2, 2020 at the age of 84 at the Saint Elizabeth Hospital in Ft. Thomas, Kentucky from complications associated with Covid 19. George started his engineering career at the H. C. Nutting Company and later became the Cincinnati branch manager for the American Testing and Engineering Company. In 1971, he founded Thelen Associates, a geotechnical engineering and construction materials testing firm in Northern Kentucky. George was a mentor to many engineers and engineering technicians as the firm grew from a start of three people to a group of ten engineers and fifty support personnel.

He was married to Mary Susan Hoppenjans Thelen with whom he raised four children, Maribeth Harper, Jennifer Regan, Becky Cipollone and Jay Thelen. George had three brothers; Gerald, James and Donald: twenty-six grandchildren: and ten great grandchildren. George lost his wife

Mary Susan to cancer in 1996. He later remarried Judith Ann Thelen with whom he enjoyed another 22 years of marriage.

George graduated from Thomas More College with a bachelor of science degree as well as a bachelor of science degree and a masters of science degree in Civil Engineering from the University of Notre Dame. He was a member of the St. Pius X Church in Edgewood, Kentucky and the St. Mary’s Church in Longboat Key, Florida. He was very active in the community and served on the Board of Directors at the Thomas More College, was committed to the Evans Scholarship fund, and started a scholarship fund at the Notre Dame Academy in honor of his deceased parents, George and Maureen Thelen. George was also a former chairman of the Northern Kentucky Area Planning Commission and active in the American Society of Civil Engineers, the Kentucky Society of Professional Engineers, and the Northern Kentucky Homebuilders Association. He was a tribute to the geotechnical engineering profession and will be missed.

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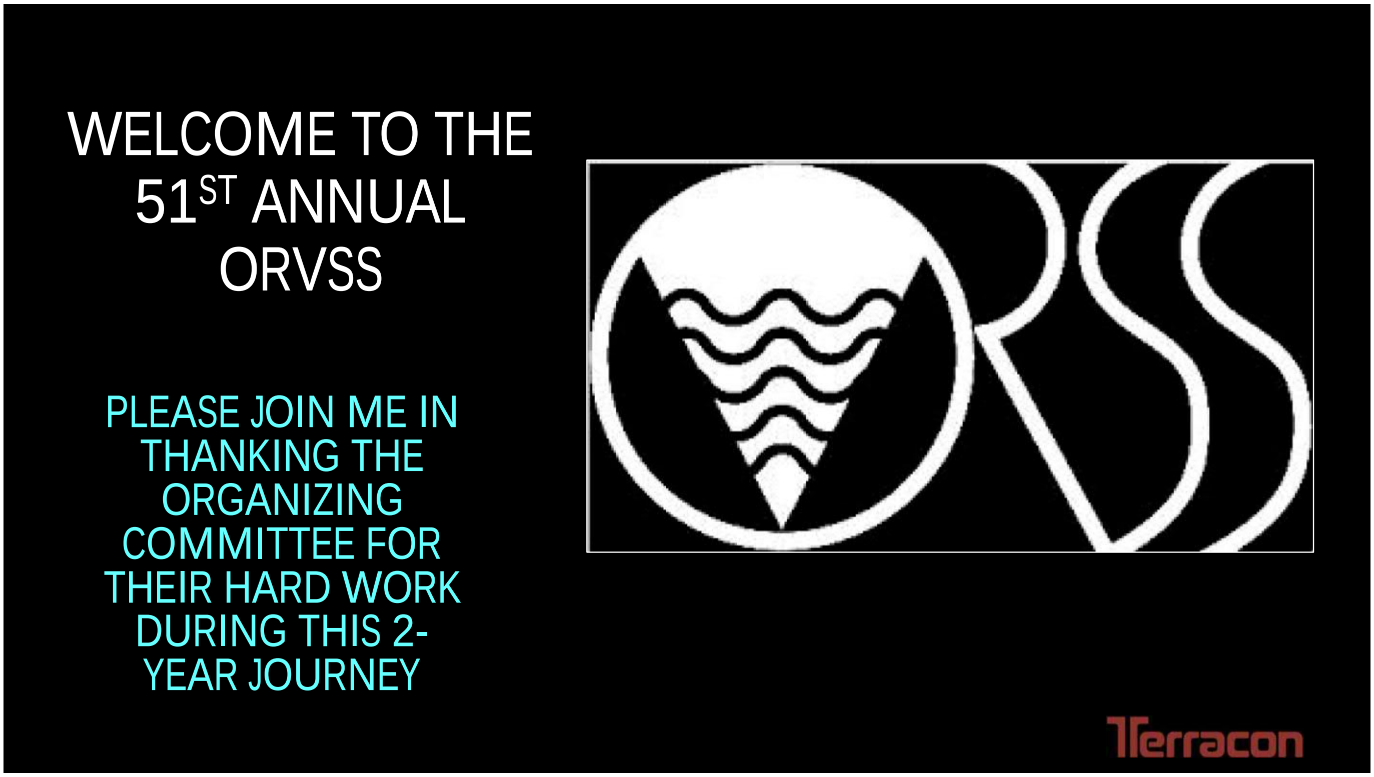
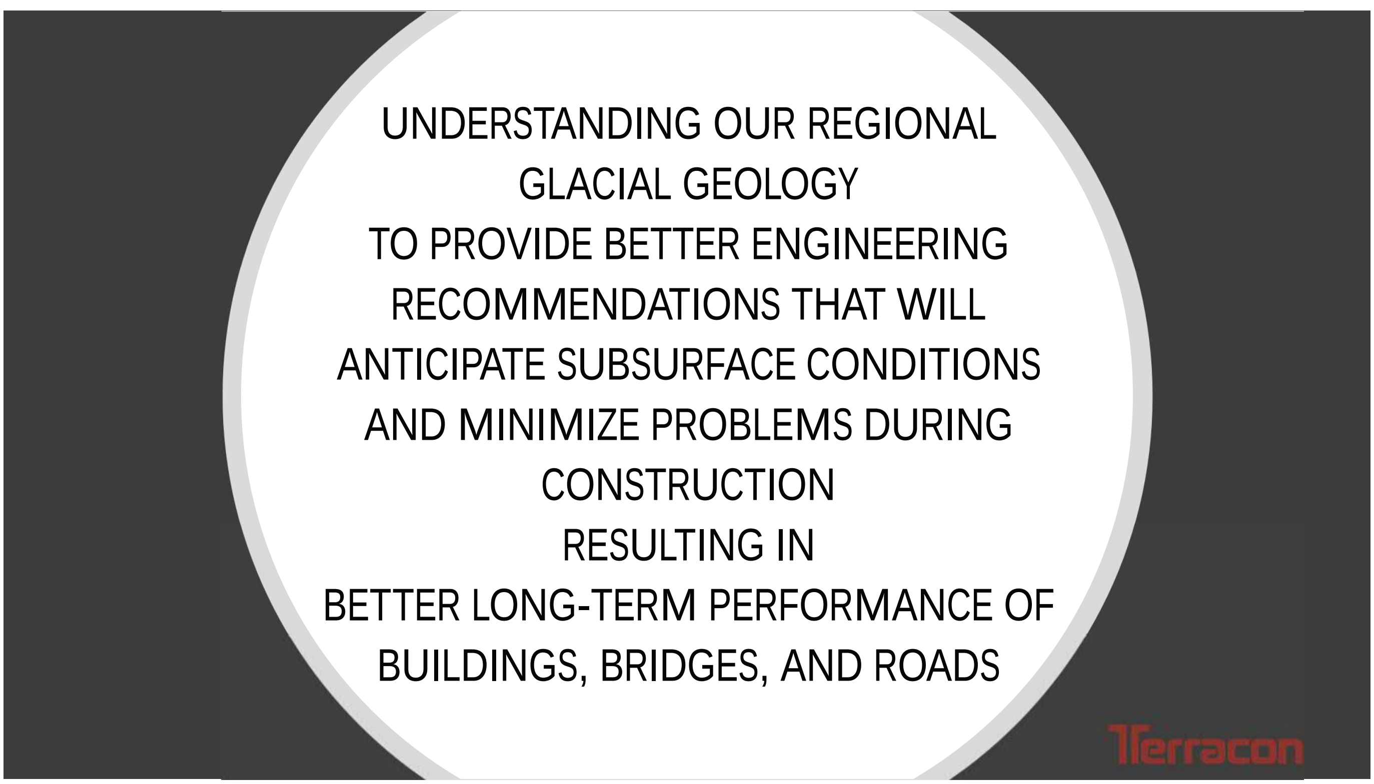

WELCOME TO THE 51ST ANNUAL ORVSS
PLEASE JOIN ME IN THANKING THE ORGANIZING COMMITTEE FOR THEIR HARD WORK DURING THIS 2-YEAR JOURNEY
ORSS
Terracon


UNDERSTANDING OUR REGIONAL GLACIAL GEOLOGY
TO PROVIDE BETTER ENGINEERING RECOMMENDATIONS THAT WILL ANTICIPATE SUBSURFACE CONDITIONS AND MINIMIZE PROBLEMS DURING CONSTRUCTION
RESULTING IN
BETTER LONG-TERM PERFORMANCE OF BUILDINGS, BRIDGES, AND ROADS
Terracon

THESE BOXES HOLD SAMPLES OF GLACIAL DEPOSITS FORMED LONG BEFORE OUR TIME.

WHAT ARE THESE DEPOSITS AND HOW DID THEY GET THERE?

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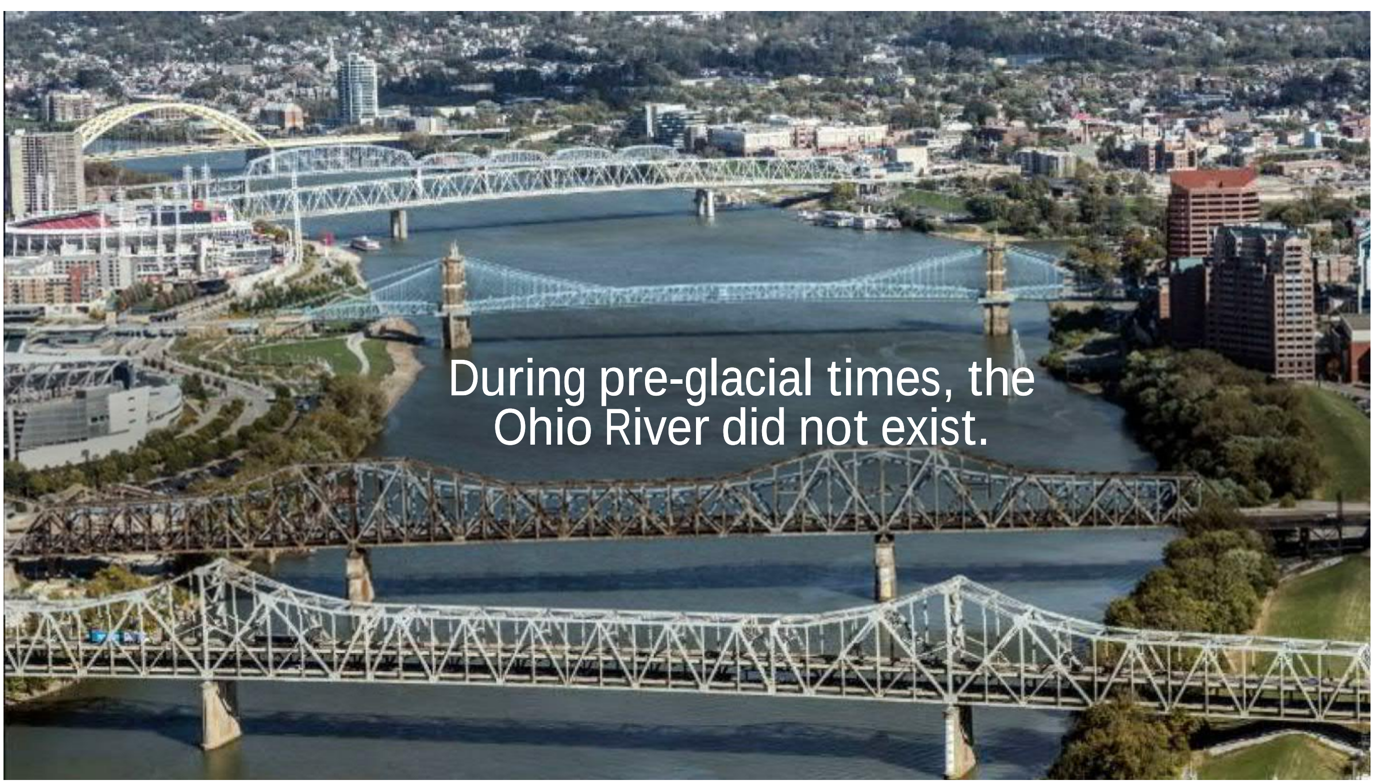
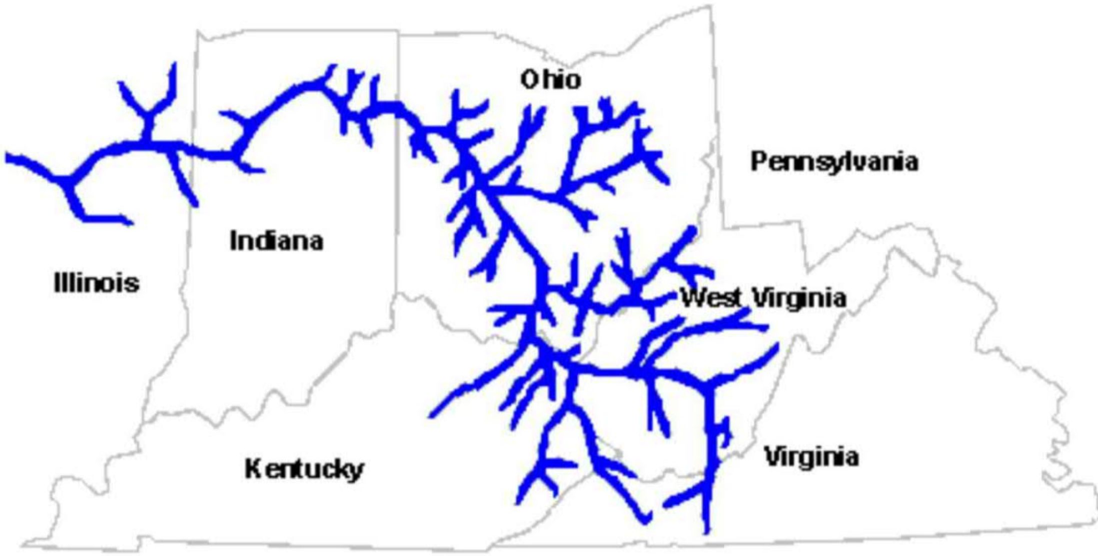
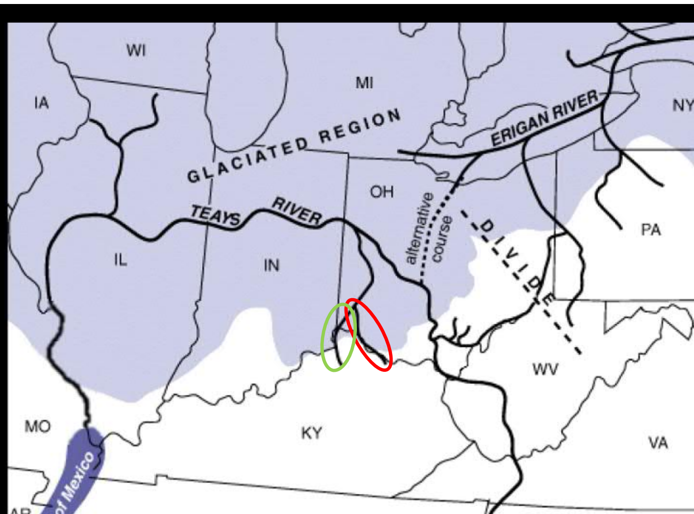




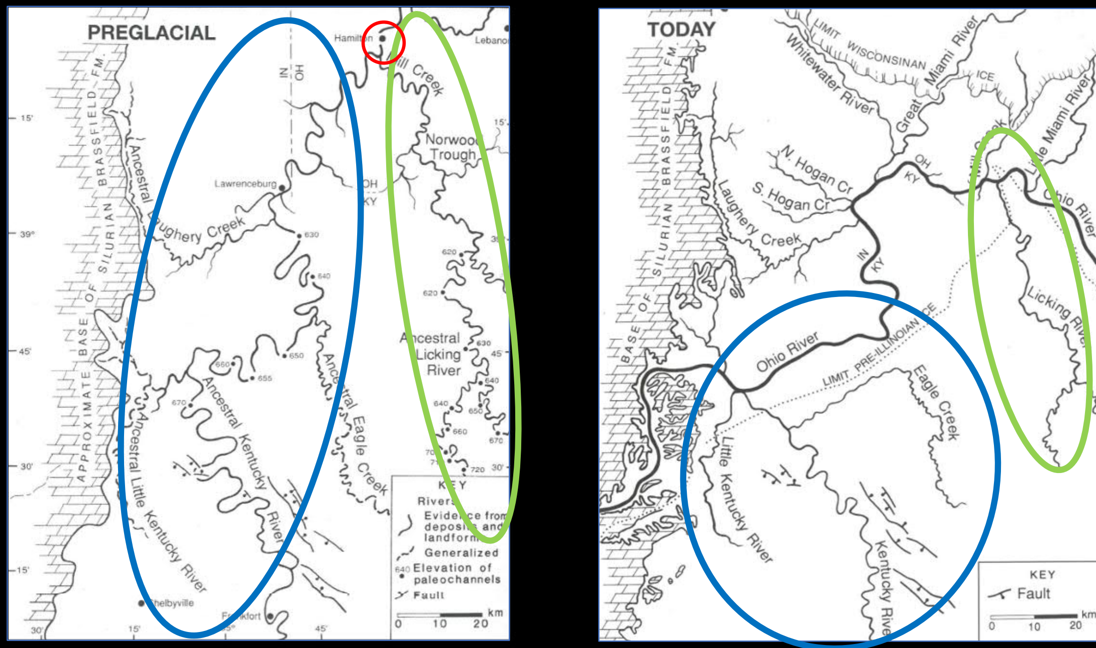
Imagine a time when our local, relatively flat, region was drained by the broad and shallow north-flowing streams of the Teays River basin watershed.



The Teays system included:

The ancestral Licking River that flowed generally northward from Alexandria, KY through Sharonville, OH.

And the Kentucky River that flowed generally northward from Harrison, OH and joining with the ancestral Licking in Hamilton, OH.



Cincinnati/ Northern Kentucky Region, Dr. Paul Potter (2nd edition, 2007).

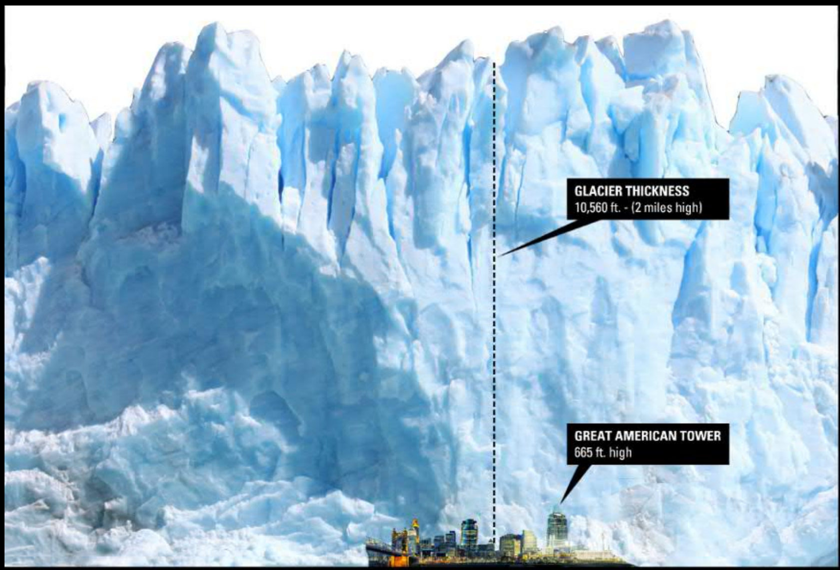
Pleistocene Age ice built to a great thickness as it began to creep southward into the northern United States.

The classic interpretation of these advances and retreats includes four major glacial advances named after states in which their deposits are prominent. They are, from oldest to youngest: Nebraskan, Kansan, Illinoian, and Wisconsinan.

Geologists now recognize that the Pleistocene was more complex than implied by this four-fold division. In our local region, the earlier glaciations, before the Illinoian, are lumped together as "pre-Illinoian."

https://ohiohistorycentral.org/w/Ice_Age_Ohio

REGIONS FURTHER NORTH OF CINCINNATI WERE UNDER UP TO A 2-MILE THICKNESS OF GLACIAL ICE. THE IMAGE BELOW PROVIDES A COMPARISON OF THAT 2-MILE THICKNESS AND THE CINCINNATI SKYLINE.



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As the pre-Illinoian ice advanced, north-flowing streams were dammed forming glacial lakes.

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IN THE QUIET WATERS OF GLACIAL LAKES, SUSPENDED CLAY PARTICLES EVENTIALLY SETTLED TO FORM LAKEBED DEPOSITS

Eventually, the lake waters rose and overflowed a divide near Madison, Indiana allowing westward drainage flow.

When the ice sheet melted, a new river was formed called the Deep Stage Ancestral Ohio.

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The tremendous amount of high-velocity meltwater caused a deep, wide channel to form cutting into bedrock some 300 feet lower than the Teays Age River valley, removing much of the pre-Illinoian lakebed clay above the elevation of 650 feet.

The Deep Stage erosion extended some 100 feet lower than the present-day Ohio and Miami Rivers.

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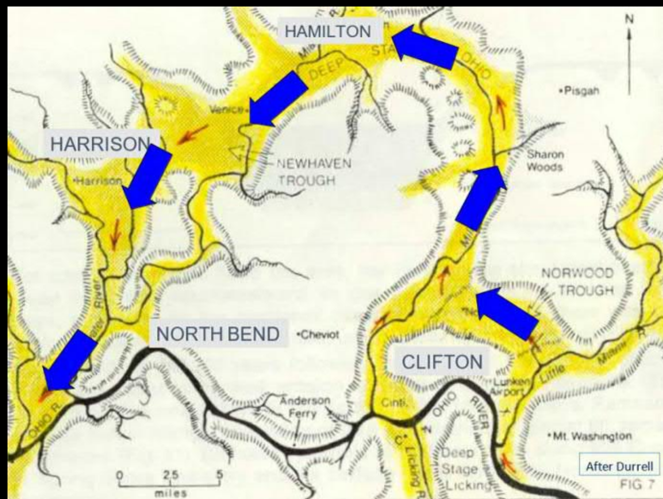
DEEP STAGE ANCESTRAL OHIO RIVER PATHWAY



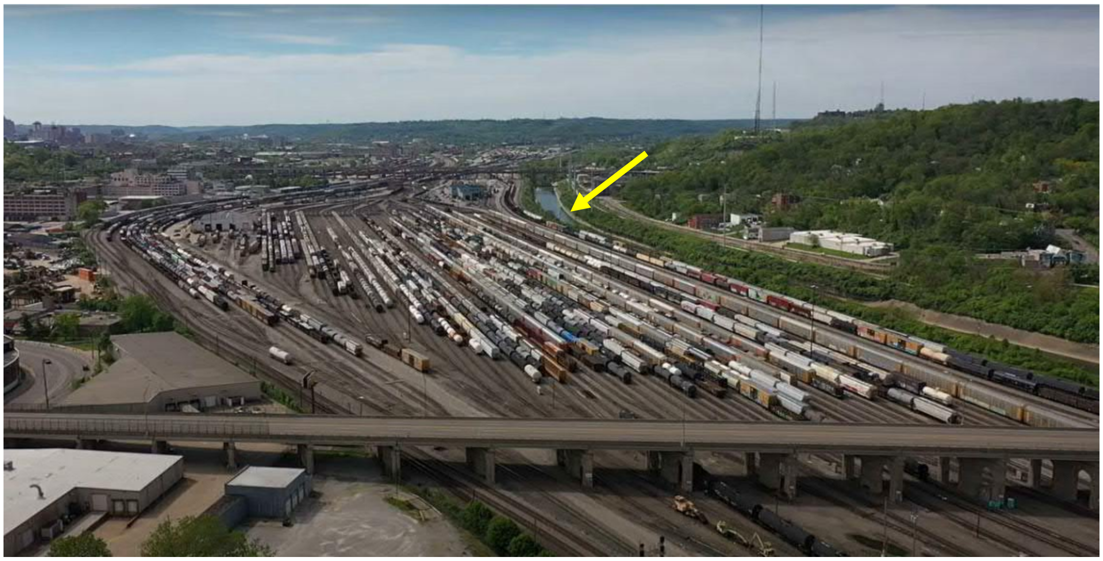
The newly-formed river followed the present-day Little Miami River westward through the Norwood Trough over to the Mill Creek Valley and back southward along the Great Miami River valley.

DEEP STAGE ANCESTRAL OHIO RIVER

During deep-stage time, the Teays-Age Licking River abandoned its course and shifted westward continuing northward across the basin of downtown Cincinnati and up the present-day Mill Creek to join the Deep Stage near Norwood.

This valley is over 15,000 feet wide between Mt. Adams and Price Hill in downtown Cincinnati.

Evidence of the wide Deep Stage channel is shown in the following aerial images of the Queensgate Intermodal CSX Railyard.



VIEW OF THE QUEENSGATE INTERMODAL CSX RAILYARD LOOKING NORTH.
THE MILL CREEK VALLEY IS AN OVER-SIZED VALLEY.
THE PRESENT-DAY MILL CREEK IS VISIBLE IN THE BACKGROUND

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Terracon

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The Deep Stage Ohio River flow carried fine-grained silt and clay-size particles away and left outwash deposits of sand and gravels.

Outwash deposits, forming the groundwater aquifers along the Mill Creek and the gravel pits along the Great Miami River, were the result of this Deep Stage drainage.

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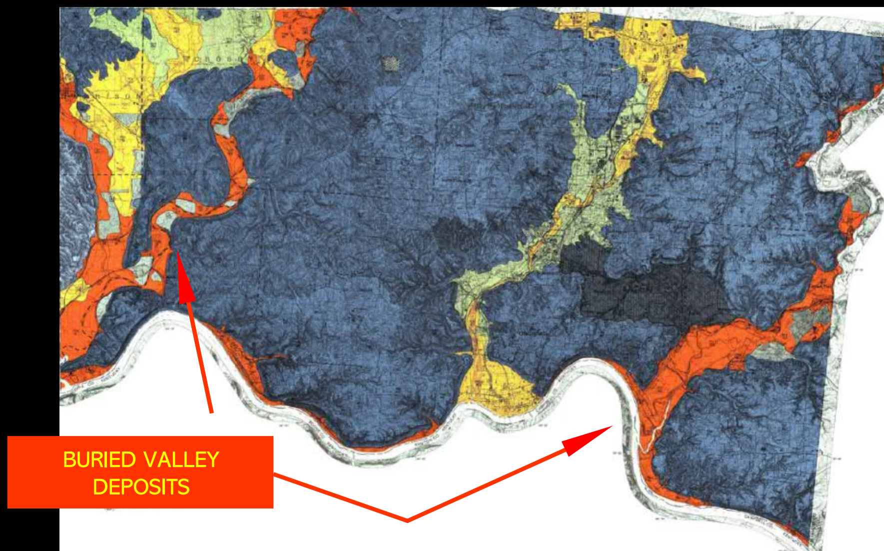

SOURCE: ODNR GROUND-WATER POLLUTION POTENTIAL MAP OF HAMILTON COUNTY (UNIVERSITY OF CINCINNATI GROUNDWATER RESEARCH CENTER)

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So, what happened to the Deep Stage Ohio River?

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The Deep Stage period ended when the Illinoian glaciation advanced into our area. The Illinoian ice sheet dammed the north flowing Deep Stage Ohio River and created a glacial lake within the Deep Stage valleys where new lakebed clays were deposited above the Deep Stage sand and gravel outwash.

With time, water levels in the glacial lake rose and breached a divide approximately 3 miles west of Cincinnati near Anderson Ferry.

The Illinoian ice sheet continued to creep south, pushing the deep-stage flow into the approximate present-day location of the Ohio River. With this advancement of the Illinoian ice sheet, the lakebed clays were covered over with glacial till deposits.

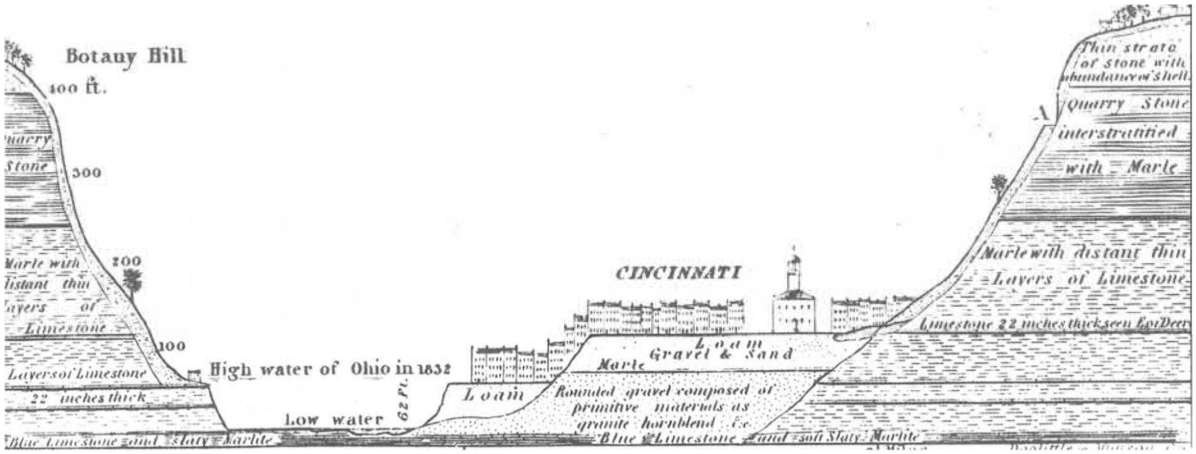
Illinoian glacial till encountered in our subsurface exploration is typically a stiff to very stiff, stable lean clay that is generally over-consolidated due to glacial ice load exposure.

Engineering recommendations should take into consideration the possibility that weak and compressible lakebed clay layers may be present underlying the glacial till.

The Wisconsinan glacier advancing into the region following the Illinoian was the last continental ice sheet to invade the Cincinnati area. This last glacier pushed an accumulation of glacial drift which was deposited in the higher elevations. When the glacier retreated, great braided streams of meltwater eroded portions of the Illinoian glacial soil.

Terraces of outwash sand and gravel were formed in the major valley systems at lower elevations. Many of our present-day valleys which are lower than an approximate elevation 540 feet were filled with this outwash.

The first detailed geologic cross section published for the Cincinnati region shows both bedrock geology and its overlying unconsolidated glacial outwash much as we know it today (Locke, 1838, Plate 2). Keys Hill in Cincinnati is now called Mount Auburn and Botany Hill is Devou Park in Covington. Today different names are used for these rock units, but Locke, a professor of Chemistry, correctly saw their essential features much as geologists see them some 170 years later. The Quarry Stone Beds, today's Fairview and Belle Formations, were the principal material for stone walls, foundations, and piers. The blue limestone is today's Point Pleasant Formation. Notice Locke's recognition of colluvium mantling the bedrock. (Reprinted with the permission of the Ohio Division of Geological Survey.)



(Locke, 1838, Plate 2). Botany Hill (Devou Park in Covington) and Keys Hill (Mount Auburn in Cincinnati)

The Recognized Geotechnical Engineers List of Greater Cincinnati

Mark T. Bowers, PhD, PE, Life Member ASCE

Abstract: This paper reviews the regulations and ordinances that have been developed in response to geotechnical problems in the Cincinnati area. Resources to assist young engineers and engineers new to the area are presented. In addition to the development of ordinances, the institution of a List of Recognized Geotechnical Engineers in the Greater Cincinnati Area is discussed. Requirements for inclusion on the List are presented.

Introduction

The previous speaker has given a very worthwhile presentation on the geology of Cincinnati. We thank all those who have contributed to the development of that paper. It was necessary to present that paper on the geology of Cincinnati first today in order to place this paper in its proper context. The subject matter of this seminar concerns geohazards. Cincinnati has many geohazards. The successful geotechnical engineer in the Greater Cincinnati area must have an appreciation for these geohazards. A geotechnical engineer new to the area, even if seasoned by many years in a different locale, may be surprised by the breadth of geotechnical problems to be dealt with here. We desire to eliminate as much of that "surprise" as we can. We feel it is imperative that geotechnical engineers know the geology of their service area very well. They need to know what to look for, how to investigate the site, what testing to complete, what to consider in the analyses, and be able to design new works given the circumstances.

Very often we are called upon when a problem has occurred. The list of potential problems is long but the major issues involve landslides. The Greater Cincinnati area has a long history of dealing with landslides, some of which have been very costly and some of which continue to plague us to this day. We will hear of the landslide inventory and repairs to a major landslide along Columbia Parkway later in the program today.

Some of our geohazards are natural but many are man-generated. Our hills are underlain by limestone interbedded with shales. The limestones were quarried for years with the spoils simply being dumped over the slopes. These waste shale dumps are deceptive. One might think they are just steep natural slopes but they consist of loose, porous waste rock that has weathered, slaked, and degraded. Many of these areas are resting at incipient failure.

Emeritus Professor of Civil Engineering, University of Cincinnati, Cincinnati, Ohio

Email: marktbowers@hotmail.com

Other problems arise when we attempt to modify building lots. Perhaps the owner wants more flat ground so fill is brought in. It could be that a homeowner wants to have a swimming pool built in the backyard. It may be thought that the excavated soil could assist in providing more flat ground near the slope so it is placed there. Time and time again when this occurs we see movement of the slope as we have taken a situation where the natural slope was sitting at incipient failure and we have overloaded the slope with fill.

Historical Development of Ordinances and Resources

Bob Sheets, a Geotechnical Engineer with the Hamilton County Soil and Water Conservation District (now with ATC Group Services), wrote an undated article entitled “Landslides in Hamilton County.” He sets forth discussion on historical perspective, answers “Just what is a landslide?”, poses the question “Why Hamilton County?”, informs us “How are we affected?”, and concludes with “So what do we do?”. Sheets notes that the City of Cincinnati adopted earthwork regulations in 1974 following the massive 1973 landslide in Mount Adams that was associated with the construction of I-471. Hamilton County, after much study and debate, passed a comprehensive set of earthwork regulations in 1990 which is similar to the ordinance developed by the City of Cincinnati in 1974. The landmark Cut-and-Fill ordinance for Hamilton County and the City of Cincinnati insists on geotechnical involvement in landslide prone areas and the projects requiring more than five feet of fill and requires all hillside developments to install erosion control measures that protect deforested slopes from sliding and eroding. This ordinance was shepherded by The Hillside Trust.

Two publications of note concerning landslides of the Cincinnati, Ohio area were published by the U.S Geological Survey as Bulletins 2059A and 2059B. Bulletin 2059B reached publication first in 1994. It is entitled “Landslides in Colluvium” and was authored by Robert Fleming and Arvid Johnson. Bulletin 2059A was published in 1996 and is entitled “Overview of Landslide Problems, Research, and Mitigation, Cincinnati, Ohio Area.” It was authored by Rex Baum and Arvid Johnson. In the abstract to the latter Bulletin, we find the following:

> Landslides cause much damage to property throughout the metropolitan area of Cincinnati, Ohio. Most landslides occur in unconsolidated deposits, including colluvium, till, glacial lake clays, and man-made fill derived from colluvium and glacial deposits. Landslides in thin colluvium are widespread on steeper slopes that wall the valleys of the Ohio River and its tributaries. Abundant landslides also form in thick colluvium on flatter slopes, especially where the colluvium has been disturbed by earthwork. Unusual block glides and block-extrusion glides form where till rests on lake clay. Through the years, knowledge of the distribution and causes of landslides has increased as a result of many investigations. This knowledge became part of the basis for landslide mitigation programs adopted by the City of Cincinnati and Hamilton County, Ohio…. In 1989 following much additional study, Cincinnati created a geotechnical office within its Department of Public Works…. Since 1989, members of the geotechnical staff have worked in several ways to reduce landslide damage in the city; their work includes engineering-geologic mapping of selected parts of the city, inspection of retaining walls that impact public right-of-way, review of proposed construction in hillside areas,

inspecting and arranging for repair of landslide areas that affect city property, and compiling geologic and geotechnical data on landslide areas within the city.

What a great overview! We will hear more about the work of the Geotechnical Office with the City of Cincinnati's Department of Public Works later in today'

Some twenty-five years ago yet another idea was instituted—the development of a Recognized Geotechnical Engineers List of Greater Cincinnati. Working with the City of Cincinnati in both Engineering and Planning, it was felt important that geotechnical work be completed by engineers with more than the Professional Engineers license. In committee work members of the Geotechnical Engineering Group of the Cincinnati Section of the American Society of Civil Engineers developed a platform of requirements for those geotechnical engineers who wished to be considered for City and County work. The basic requirements for inclusion on the List of Recognized Geotechnical Engineers of Greater Cincinnati are as follows:

1) The Engineer must hold an active Professional Engineers (PE) license in the State of Ohio.
2) The Engineer must have earned a Bachelor of Science degree in Civil Engineering and have at least three years of experience, one year of which must involve experience with slope stability investigation, analysis, and remediation in the Cincinnati area; or have earned a Masters degree in Civil Engineering and have at least one year in the Cincinnati area with experience in slope stability investigation, analysis, and remediation.

Those who wish to be considered for inclusion on the List of Recognized Geotechnical Engineers apply to a committee expressing their desire to be included on the List. They are instructed to provide a current resume that clearly shows how they meet the requirements listed above. The application might be thought of as an expanded resume. The committee hopes to see job assignments that reflect the applicant's role in the investigation, site reconnaissance, testing, data analysis, and recommendations for construction and/or remediation of a geotechnical problem, primarily concerning landslides and slope stability issues. Appropriate experience would involve both deep-seated rotational failures, and shallow translational failures. The committee also looks for breadth of experience with local geohazards including expansive clays, karst foundations, loose floodplain deposits, earth retention problems and solutions, the impact of floodwaters on the stability of valley wall stability, and the interbedded nature of the bedrock of limestone and shales. The local shales have a tendency to slake upon exposure to wetting and drying becoming muds in mere hours. Also, some of the local shales experience secondary mineral growth, causing heave problems for a building or pavement.

The review committee consists of at least two and usually three engineers who independently review the applications and report to one of the engineers who serves as committee chairman. The committee chairman writes the response to applicants. The committee members have long consulting experience in the Greater Cincinnati area and represent the companies established in the area. To avoid the appearance of bias, one of the engineers on the review committee is a member of the Civil Engineering faculty of the University of Cincinnati. Some 70 engineers are now on the List and applications continue to be submitted. The List has now grown to encompass governmental agencies and geotechnical firms across the river in Kentucky. Further questions about the List of Recognized Geotechnical Engineers may be directed to our current Chairman, George C. Webb of Terracon.

This cooperative effort has been useful in providing guidelines for the design and construction of geotechnical works in the Greater Cincinnati area. We have all benefitted by having individuals who drove the cut-and-fill ordinance through, who pioneered and continue to inventory the landslides in the City and County, and who prepare themselves by education, experience, and professional licensure to execute safe and economic solutions to our local geohazards.

Additional Resources

The Hillside Trust consults with various municipalities and government planning commissions on land use regulations that pertain to hillside development and preservation. The Hillside Trust acts with the City of Cincinnati, the Hamilton County Regional Planning Office, and the Northern Kentucky Area Planning Commission.

The City of Cincinnati Building and Inspections Office handles excavation and fill permits. Under "How do I obtain a permit?", Item S: "A report from a Geotechnical Engineer showing the results of surface and subsurface exploration conditions of the land and procedures for performing the operation. Finished slopes steeper than 3 horizontal to 1 vertical must be designed by a Geotechnical Engineer. The Engineer must inspect the work in progress and submit a summary report to the City upon completion of the work."

The City of Cincinnati Building and Inspections Office also handles "permits for work in landslide areas." This office refers the applicant to "Landslide Susceptibility Maps," a list of active landslides, or field inspections. In Item III—Additional Requirements, Part A we find the following:

> Geotechnical Engineers (a list of recognized Geotechnical Engineers is available through the Cincinnati Chapter of the American Society of Civil Engineers) are required to submit Soils Reports, make recommendations on methods of performing the earthwork, and to field supervise the following types of projects:

1) Sites with active landslides. This determination shall be made from the list of active landslides that is maintained by the Department of Buildings and Inspections or by a field inspection.
2) Sites where existing or proposed slopes are greater than 3 horizontal to 1 vertical.
3) Sites where any excavation is greater than 12 feet in depth or located next to a existing structure where a possibility of undermining exists.
4) Sites where a structural fill in excess of five feet is required.
5) An earthwork area is greater than two acres.
6) Projects located in the Hillside District Zoning Overlay.

The Code of Ordinances for the City of Cincinnati may be found on-line. Go to "Municipal Code City of Cincinnati, Ohio." Next scroll to "Title XI-Cincinnati Building Code." Scroll down to Chapter 1113-Excavation or Filling of Land."

A Landslide Susceptibility Map of Cincinnati was prepared by Sowers and Dalrymple, Consulting Engineers, in 1980. One can contact the City of Cincinnati Division of Engineering. A copy of the map is also available in the Langsam Library on the Main Campus of the University of Cincinnati.

I have appreciated the collegiality of my colleagues in Cincinnati. In 2013 a number of us put together a brochure entitled "Landslides and Your Property." The authors included Paul Potter and Barry Maynard from the Department of Geology, University of Cincinnati, myself from the Department of Civil and Architectural Engineering and Construction Management at UC, Matthew Crawford and Gerald Weisenfluh of the Kentucky Geological Survey, and Tim Agnello of Ohio Valley Landslides, LLC, Cincinnati. The brochure is replete with photographs of the types and examples of landslides, sections on inspecting a property, avoidance of slides, remediation, lake bed clays, the Kope Formation, and additional sources of information and assistance. I brought some of those brochures which you may freely take. I acknowledge publication support which we received from Duke Energy.

There is much information on-line. For example, Ohio Valley Landslides serves as a repository for basic information on geohazards. The site is ohiovalleylandslides.com.

I have shared this information with the hope that our young geotechnical engineers maneuver successfully through their early years of experience and to assist new geotechnical engineers to our area that they may know that there are many resources for learning about our geohazards and the efforts and guidelines that are readily available. We do not have to learn by sad experience. This is a great place to live and work. It is immensely challenging and satisfying. I cannot thank my colleagues enough for the impact they have had on me. Thank you for sharing your experiences, your files, your passion, and your enthusiasm.

Earthquake Drains to Mitigate Liquefaction Hazards: Theory and Implementation

Sarah Ramp, P.E.[1]

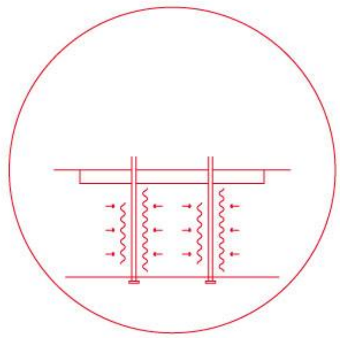
Abstract: Earthquake Drains are increasingly being installed to mitigate liquefaction hazards during seismic events. Earthquake drains are believed to expedite drainage during excitation, which shortens the time of sustained excess pore water pressures (Δu), lowers the magnitude of excess pore water pressures, and thus, reduces the potential for liquefaction. Many researchers have demonstrated the performance of earthquake drains in controlled laboratory and field tests. It has been demonstrated that earthquake drains increase the rate of pore water pressure dissipation through instantaneous drainage, and that the installation process can contribute to densifying the native soil. However, the true effectiveness of earthquake drains on the response of the soil and structure is not fully understood because no installed earthquake drains have been subjected to liquefaction-triggering seismic events to date. This presentation covers the theory of earthquake drains for mitigation of liquefaction hazards as well as three case histories to describe the engineering design and installation of earthquake drains on projects in seismically active regions. Abstract The importance of seismic design considerations continues to increase in areas of the U.S. where, traditionally, they have not been considered. Liquefaction-induced settlement or structure movement due to lateral spread are two significant design challenges. In deep liquefiable sands (depths of 30 to 40 ft [9.1 to 12.2 m] and greater), traditional vibration or soil mixing techniques may prove to be financially and/or operationally inefficient. Drilled displacement (DD) systems that densify coarse-grained soils by mechanically displacing them laterally can be an efficient alternative in this scenario. This paper provides background on the development of DD tools in North America, the research and development of the ground improvement provided by DD tool installation, and the subsequent use of DD tools to install structural piles or ground improvement elements to mitigate potential liquefaction as a seismic hazard.

[1] Senior Design Engineer, Menard Group USA, Carnegie, PA. Email: sramp@menardgroupusa.com

Sarah Ramp

Senior Design Engineer

Menard USA



Earthquake Drains to Mitigate Liquefaction Hazards: Theory and Implementation

Earthquake Drains are Ground Improvement

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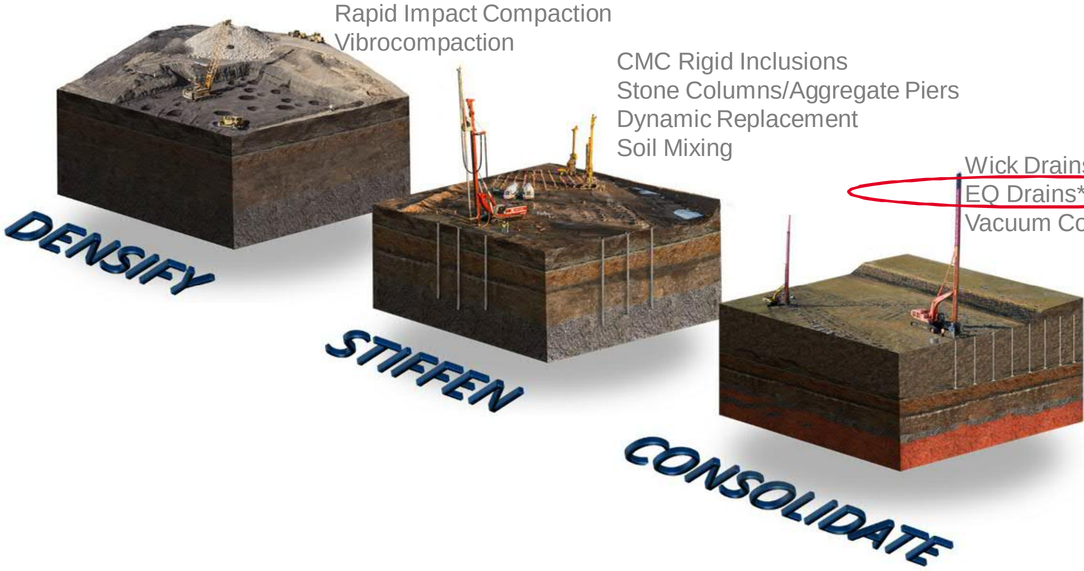
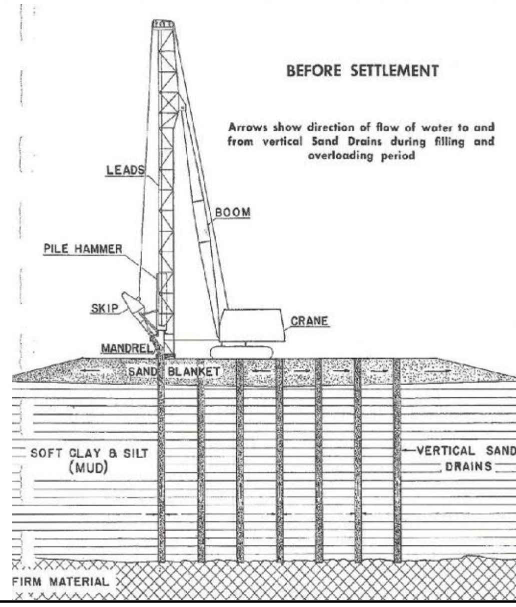
Dynamic Compaction
Rapid Impact Compaction
Vibrocompaction

CMC Rigid Inclusions
Stone Columns/Aggregate Piers
Dynamic Replacement
Soil Mixing

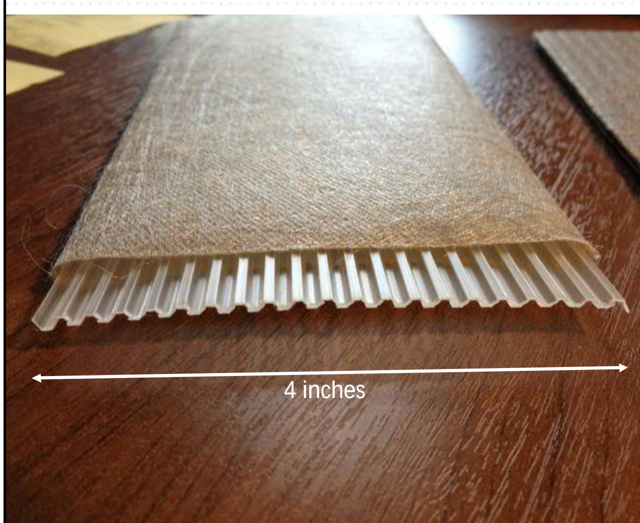
Wick Drains
EQ Drains*
Vacuum Consolidation



History of Drains as Ground Improvement



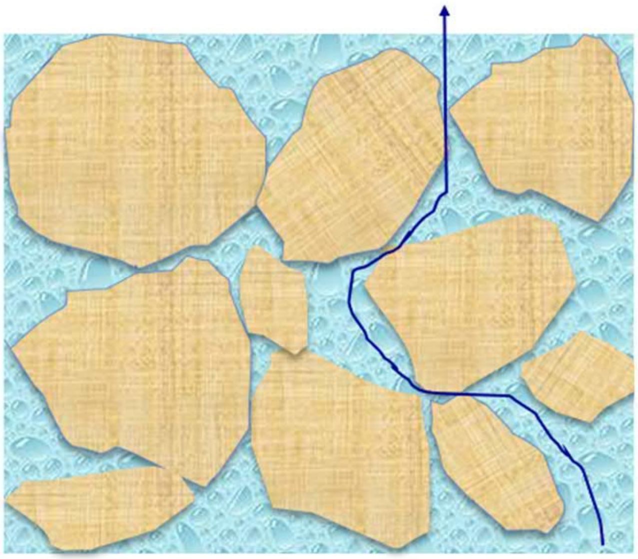
History of Drains as Ground Improvement



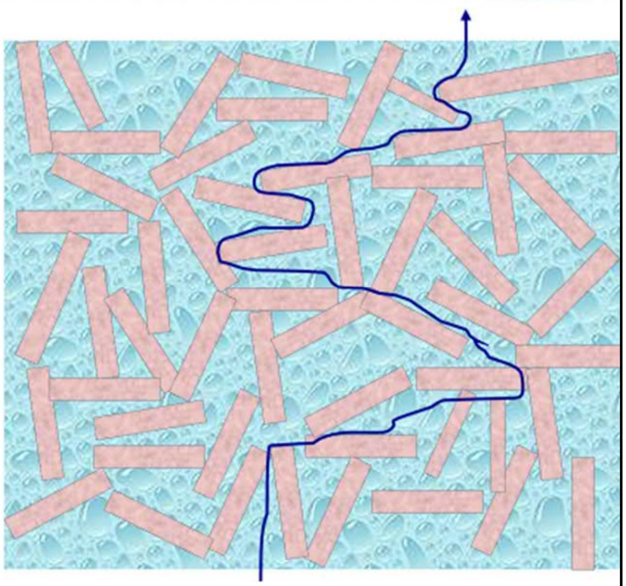
Two-Part Drains

Composite Drains

Sand vs. Clay Drainage



Sand

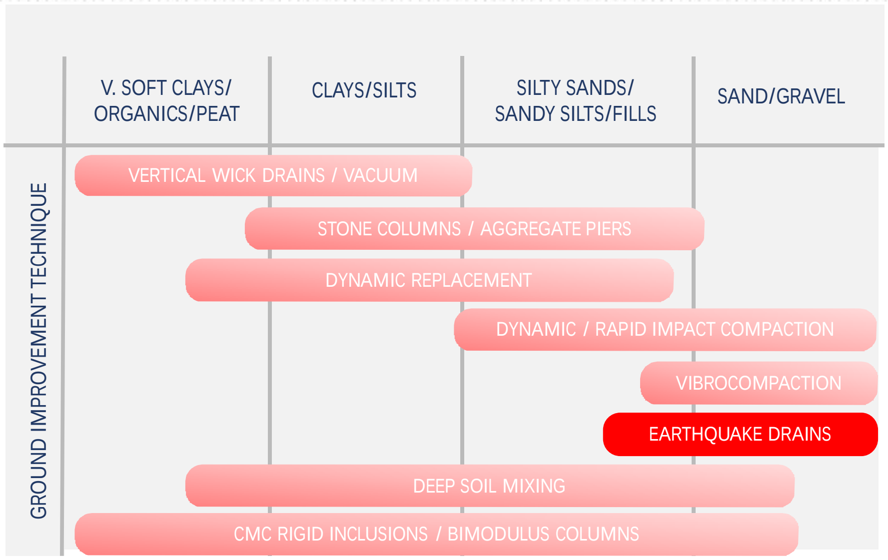


Clay

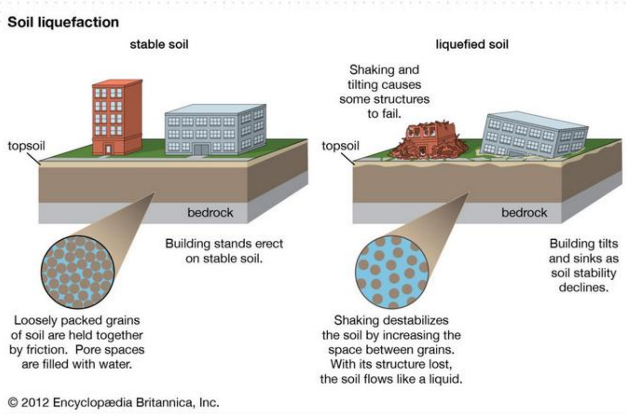
What Are Earthquake Drains?

Corrugated Pipe wrapped in a filter fabric with sufficient discharge capacity (design) installed in a grid pattern. Diameter varies, typically 4 inches.

Applicable Soil Types



Soil Liquefaction



- Liquefaction triggering: $\Delta u = \sigma'_{v0}$
  - Δu is the change in pore water pressure (excess of hydrostatic cond.)
  - σ'_{v0} is the initial effective stress

Devastating Events

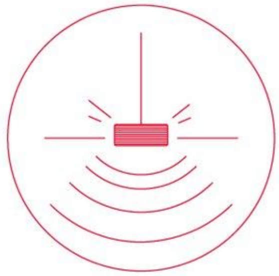
Cyclic Softening & Excess Pore Pressure Ratio "r_u"

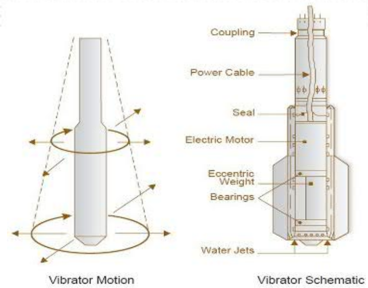
- Cyclic Softening: $\Delta u = \sigma'_{v0} - \sigma'_{v}$
  - σ'_{v0} is the initial vertical effective stress
  - σ'_{v} is final vertical effective stress
- Excess Pore Pressure Ratio: $r_u = \frac{\Delta u}{\sigma'_{v0}}$
  - $0 < r_u < 1$

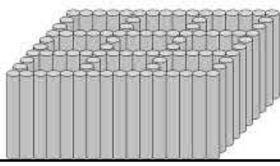
Principles of Liquefaction Mitigation

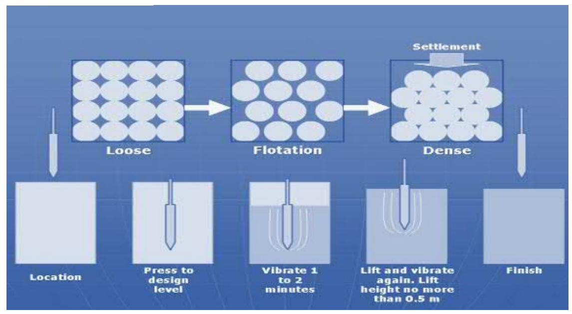
- Increase Strength i.e. Increase Cyclic Resistance Ratio (CRR)
  - Densification
    - Vibro compaction
    - Compaction grouting
    - Dynamic compaction
- Decrease driving shear stress i.e. Decrease Cyclic Stress Ratio (CSR)
  - Shear Reinforcement
    - Stone Columns
    - Rigid Inclusions/soil mixing columns
    - Soil Mixing panels
- Dissipate Excess pore water pressure generated by Earthquake loading quickly
  - Drains reduce drainage path

Examples of Liquefaction Mitigation

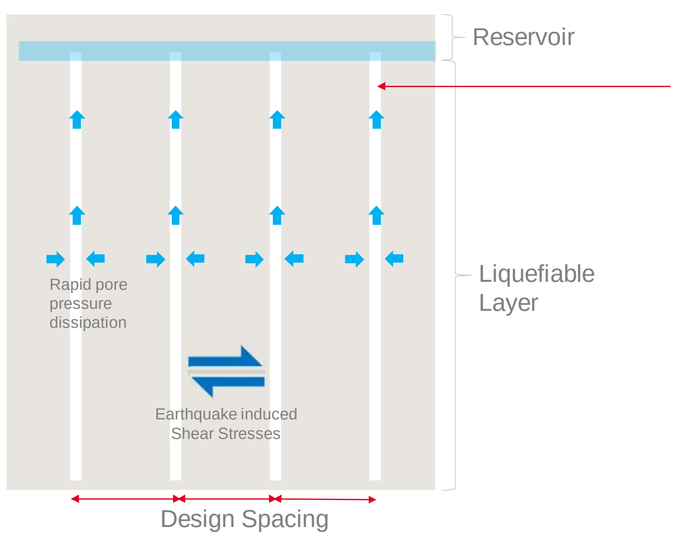








Principle of Earthquake Drains



Drains are Typically installed on triangular patterns 5 to 8 ft on center

Diameter and spacing are linked and in practice, 4 inch diameter drains are typically used – best compromise between flow, spacing and cost

Performance Under Controlled Conditions

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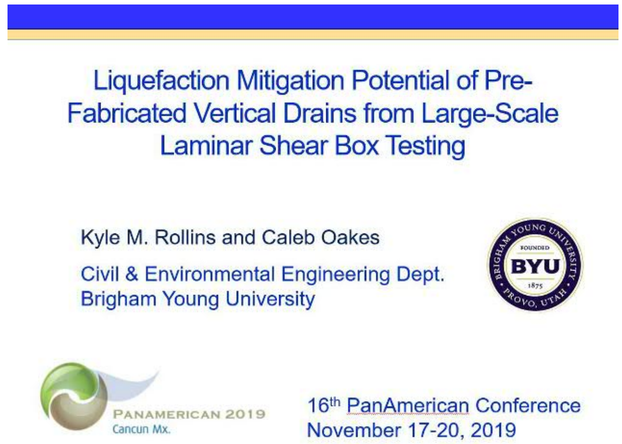
- Centrifuge Testing
  - 30-50% reduction in settlement with drains as compared to undrained areas
- Controlled Blast Full-Scale Testing
  - Settlement with drains reduced to 20-60% of settlement in undrained areas
- In both tests, time of elevated pore pressures was reduced to a few seconds (with drains) compared to 1-2 minutes without drains


Liquefaction Mitigation Potential of Pre-Fabricated Vertical Drains from Large-Scale Laminar Shear Box Testing

Kyle M. Rollins and Caleb Oakes

Civil & Environmental Engineering Dept.
Brigham Young University

PANAMERICAN 2019
Cancun Mx.

16th PanAmerican Conference
November 17-20, 2019

Rollins, K.M. (2004). "Liquefaction mitigation using vertical composite drains: full scale testing." Transportation Research Board of the National Academies. *Highway IDEA Project*.

Earthquake Drain Installation

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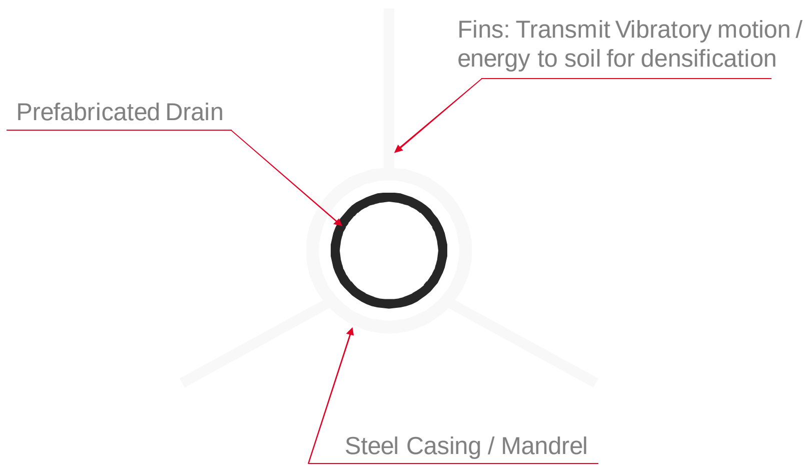
Earthquake Drain Installation

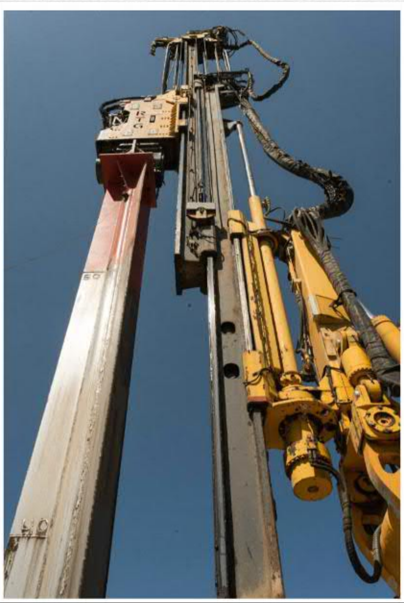
Cross-Section of Installation Mandrel



Earthquake Drain Installation

Drains are typically pre-cut prior to being pushed into mandrel and installed

Earthquake Drain Installation

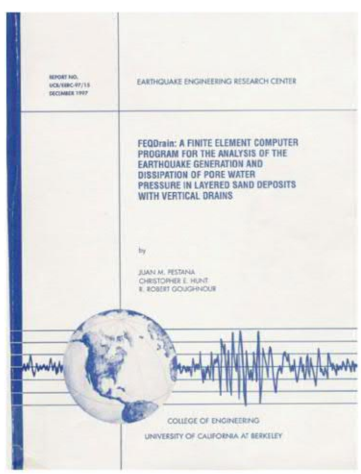


Completed Installation

Earthquake Drain Design Concepts


```
"LIQUEFACTION Potential CPT 23
7,12,62.4,6,0,1,0 !nlayers,nrinc,gammaw,depwat,effob,isurf,iexcess
2,2,0.001,0.00066,0.00000833,120,20,0.8,0.7 !linc,thick,kx,ky,mv,gammat,nl,dr,theta
4,4,0.001,0.00066,0.00000833,120,20,0.8,0.7 !linc,thick,kx,ky,mv,gammat,nl,dr,theta
8,8,0.00018,0.0001188,0.000002,110,5,0.75,0.7 !linc,thick,kx,ky,mv,gammat,nl,dr,theta
3,3,0.00049,0.00003234,0.0000024,110,3,0.55,0.7 !linc,thick,kx,ky,mv,gammat,nl,dr,theta
4,4,0.0017,0.0001122,0.0000024,120,16,0.8,0.7 !linc,thick,kx,ky,mv,gammat,nl,dr,theta
3,3,0.00000014,0.00000000924,0.000015,110,20,0.35,0.7 !linc,thick,kx,ky,mv,gammat,nl,dr,theta
1,1,0.00000014,0.00000000924,0.000005,110,20,0.35,0.7 !linc,thick,kx,ky,mv,gammat,nl,dr,theta

12,40,2,2400 !nq,td,numstep,fintim
1000,0.1,1 !itertime,timestep,prnres
1000,1,100 !itertime,timestep,prnres
2,2,4,3 !iopt,jopt,kopt,noprint
0.16667,2.89,0.0873 !rw,rout,aread
0.3676,2,1,0.021875,2.92 !c1,c2,corf,orf,permit
7,2,0,1 !arear,depres,c3,c4
end !eflag"
```



Pestana, Hunt, and Goughnor (1997)

No easy analytical equation (unlike traditional wick drains)

Complex finite element soil modeling program from University of California, Berkeley (1997)

Earthquake Drain Design Concepts

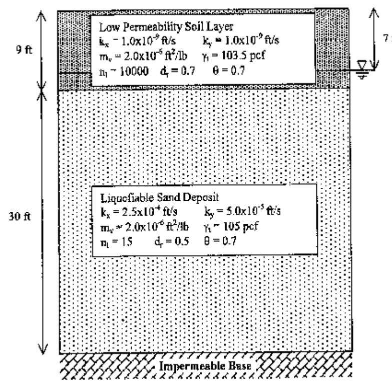


Figure 4.7: Soil profile and properties

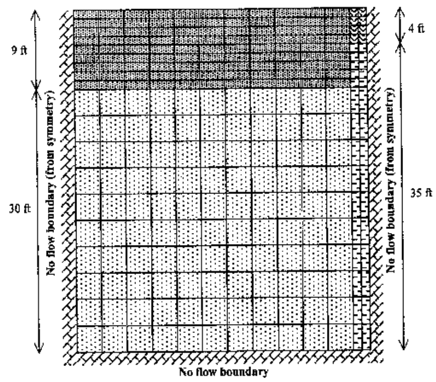


Figure 4.8: Generated mesh

Pestana, Hunt, and Goughnor (1997)

- No easy analytical equation (unlike traditional wick drains)
- Complex finite element soil modeling program from University of California, Berkeley (1997)

Earthquake Drain Design Concepts

$$\frac{m_v}{m_{vo}} = \frac{\exp(y)}{1 + y + y^2/2} \geq 1 \qquad (10)$$

$$y = a \cdot r_u^b; \quad a = 5(1.5 - Dr); \quad b = 3(4)^{-Dr}$$

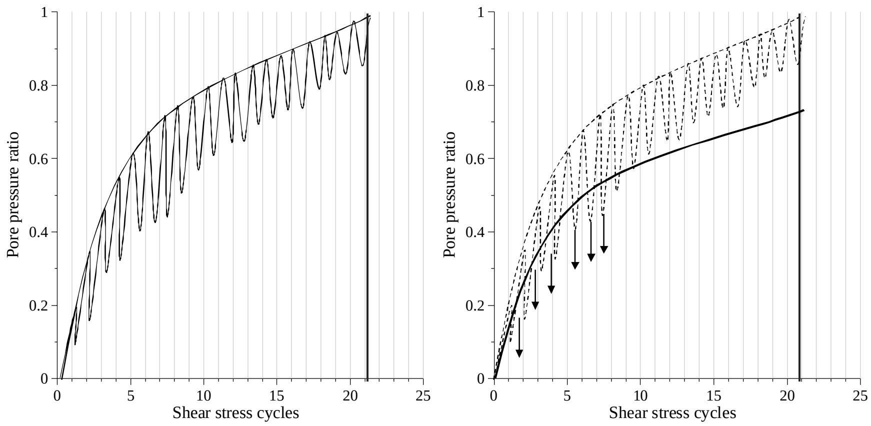
The head loss, Δh, for water entering the drain inside an EQ drain (horizontal flow) is given by:

$$\Delta h = \frac{v_r}{\psi} + \frac{c_{orf}}{a_{orf}} \cdot \frac{v_r^2}{2g} \qquad (2)$$

The hydraulic gradient in the vertical direction, for flow beneath the reservoir, is given by the modified Manning equation:

$$\frac{dh}{dz}|_{z,t} = c_1\left(Q_d(z,t)\right)^{c_2} \qquad (3)$$

Principle of EQ Drain design



§ Reduction of excess pore pressure accumulation during Earthquake

Principle of EQ Drain design

$$\delta_{layer} = (H_{layer})(m_{v-New})(\Delta\sigma'_v)$$
$$\Delta\sigma'_v = \Delta u$$
$$\Delta u = R_u \sigma'_{vo}$$
$$\delta_{layer} = (H_{layer})(m_{v-New})(R_u \sigma'_{vo})$$

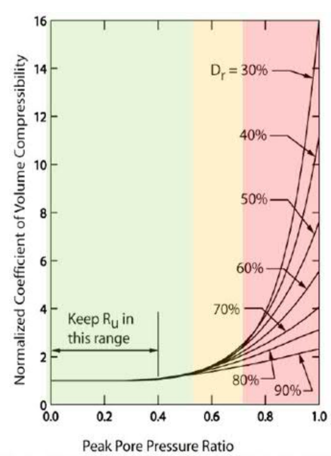
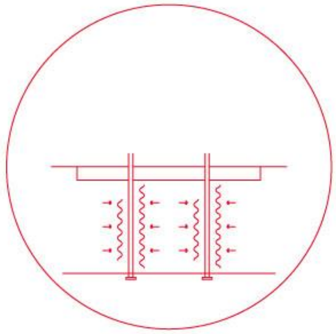


Figure 1.6 Normalized coefficient of volume compressibility versus pore pressure ratio for sands at various relative densities. Settlement is negligible when peak pore pressure ratio remains below approximately 0.40 (after Seed *at al.*, 1975a).

§ If r_u stays low, settlement stays low



Case History – Port Access Road
N. Charleston, SC

Case History: Port Access Road

Case History: Port Access Road

Owner: **SCDOT**

General Contractor (JV): **Fluor-Lane South Carolina**

Highway EOR: **Davis & Floyd, Inc.**

Structural Engineer: **Jacobs**

Geotechnical EOR: **S&ME**

Ground Improvement EOR/Installer: **Menard USA**

Case History: Port Access Road

CMC

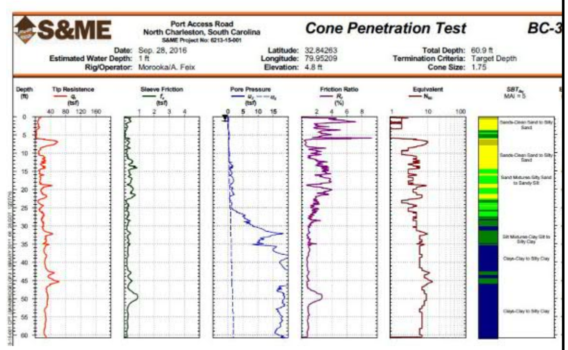
CMC/ EQD

EQD

Case History: Port Access Road

Table 3-1: Generalized Soil Stratification Table

Soil Unit	Elevation of Top of Layer (NAVD 88)	Depth to top of Layer (ft)	USCS Soil Type	Average SPT-N Values (bpf)	Average CPT Tip Resistance (tsf)	Average DMT p1 Pressure (tsf)
1A – Artificial Fill	+7	0	SP/SM/SC CL	10	80	24
1C – Holocene Marsh Deposits (west of I-26)	+4	0	MH/PT CH/SM	3	<5	2
2A – Recent to Pleistocene "Barrier" Sands	+0	10	SP/SM/SC	7	70	n/a
2B – Pleistocene Fossiliferous Shell Sand, Silt, and Clay	+2	5	n/a	n/a	10	5
2C – Pleistocene Clays and Silts	-10	19	SP/SM/CH SC/MH/CL PT	1	8	6
3 – Pleistocene Lower Sands	-32	40	SM/SP/SC	8	110	11
5 – Cooper Marl	-50	57	MH/ML	12	50	20

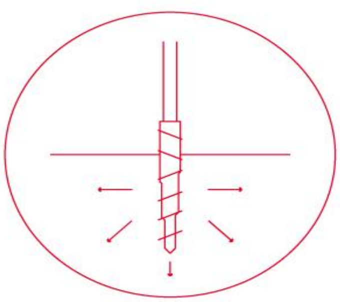


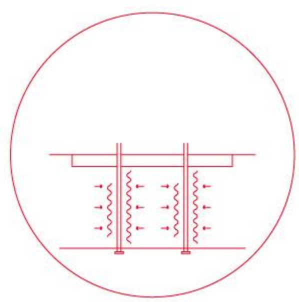
Case History: Port Access Road

Area	Extreme Event Limit State PGA	Estimated Liquefaction Vertical Settlements Without Drains
Ramp A	0.27 g	4-6 inches
Ramp B	0.27 g	4-16 inches
Ramp C	0.27 g	2-13 inches
Ramp G	0.27 g	2-10 inches
Ramp GH "Gore"	0.27 g	2-10 inches
Stromboli East	0.44 g	2-6 inches
Tidewater Bluff	0.32 g	< 4 inches*
Bainbridge Connector Bridge North Approach	0.44 g	2-7 inches
Bainbridge Connector Bridge South Approach	0.44 g	2-5 inches

*Although estimated vertical settlements were low, Earthquake Drains were specified for global stability near bridge foundations

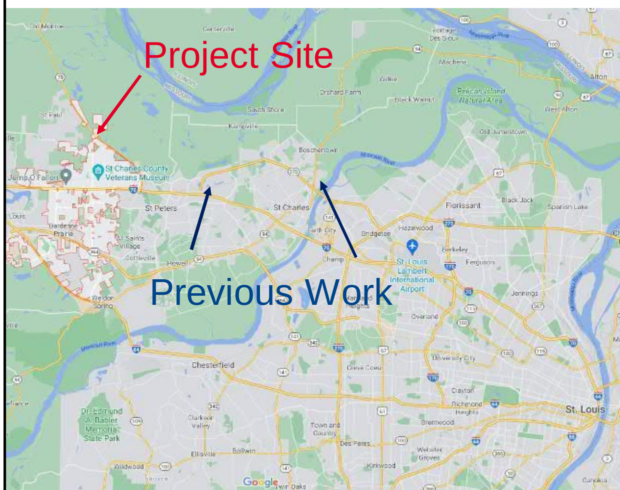
Case History: Port Access Road





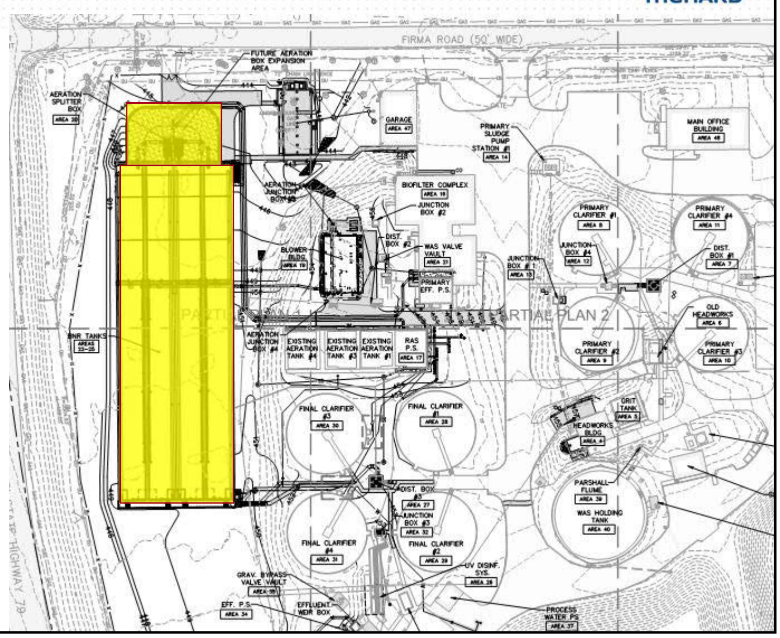
Case History – O'Fallon WWTP

O'Fallon WWTP - Overview

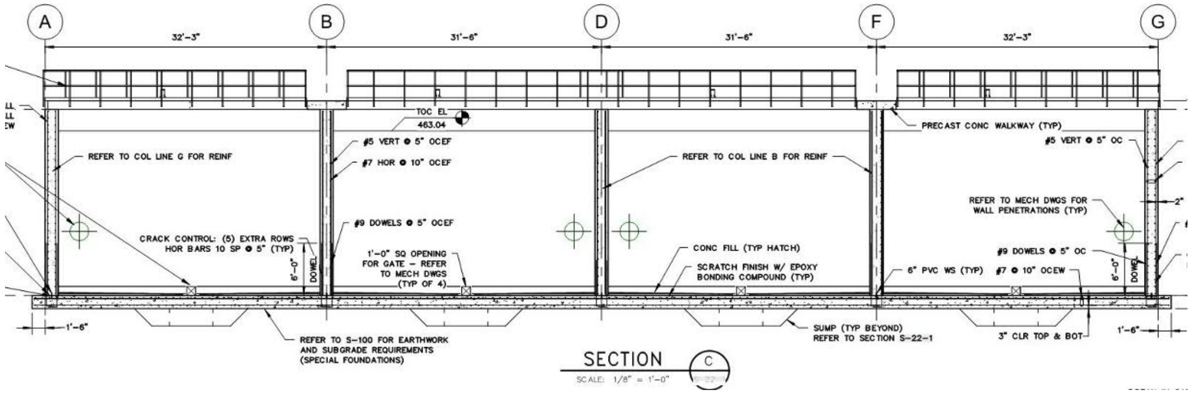


O'Fallon WWTP - Overview

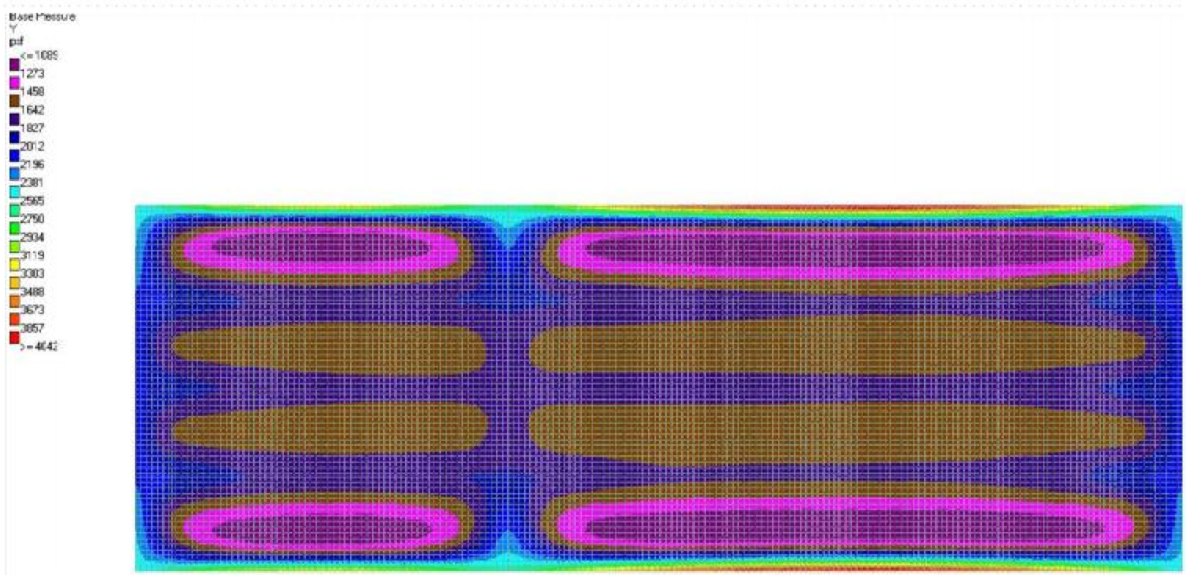
- **Project Team**
  - *Owner*: City of O'Fallon
  - *Engineer*: Woodard & Curran
  - *GC*: River City Construction
  - *Geotech*: Geotechnology
  - *Ground Improvement EOR/Installer:* Menard USA
- **Scope of Support**
  - Four (4) Biological Nutrient Removal Tanks
  - Aeration Splitter Box



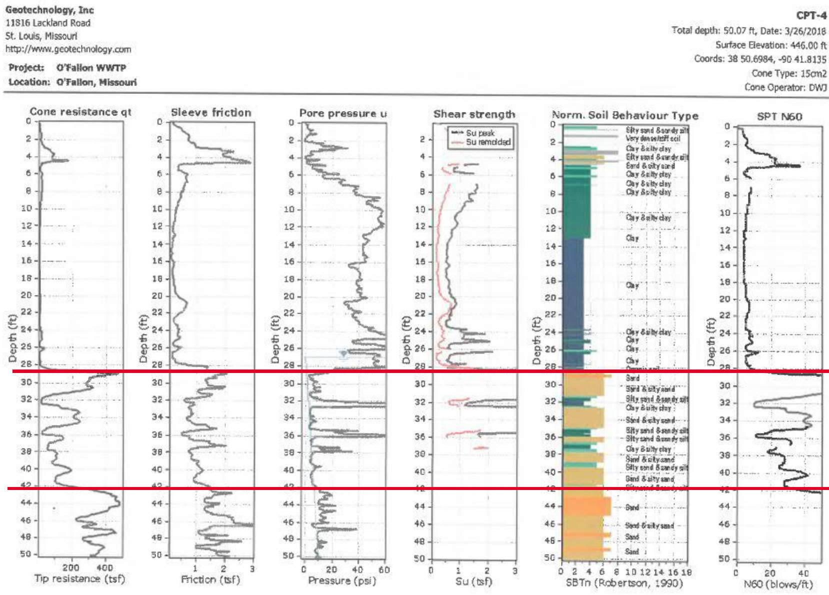
O’Fallon WWTP - Structure



O’Fallon WWTP – Bearing Pressures



O'Fallon WWTP - Soils



- **Soft Clay**
  - Static settlement concerns
  - Bearing capacity concerns
- **Loose to Medium Sand**
  - Liquefiable
- **Medium to Dense Sand**
  - Not Liquefiable

O'Fallon WWTP - Specifications

4. Site soils are potentially liquefiable. Special foundations shall limit total deflection to 3 inches during design seismic event and/or mitigate the liquefaction potential.

5. The Geotechnical Report suggests the installation of aggregate piers to achieve the performance requirements outlined herein. Alternatively, other Special Foundation alternatives will be considered upon review by the Engineer. These include but are not limited to earthquake drains, soil mixing, compaction grouting, rigid inclusions/Controlled Modulus Columns or any combination thereof.

This note was clarified to mean "total settlement of 3 inches, including seismic settlement"

O’Fallon WWTP – Menard Concept

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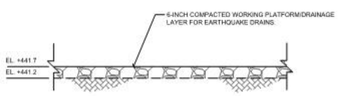
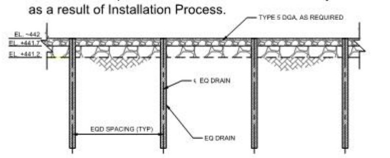
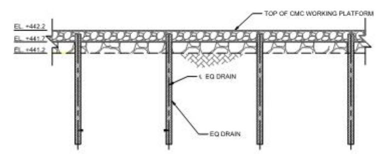
1 Excavate, as required, and place Minimum 6-Inch Compacted Working Platform/Drainage Layer for Earth Quake Drains to EL. 441.7-FT per Menard's recomendations.



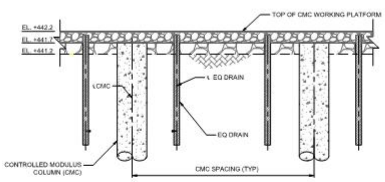
2 a. Place up to 6-inches of Type 5 DGA, as required, as topping to complete Working Platform.
b. Install Earthquake Drains. Some Settlement may occur as a result of Installation Process.



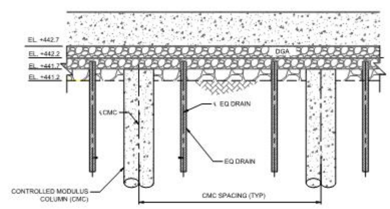
3 Add Type 5 DGA, as required, and re-level / re-compact surface at El. 442.2-FT per Menard Requirements.



4 a. Install Controlled Modulus Columns (CMCs)
b. Cutoff CMCs at El. 441.7-FT



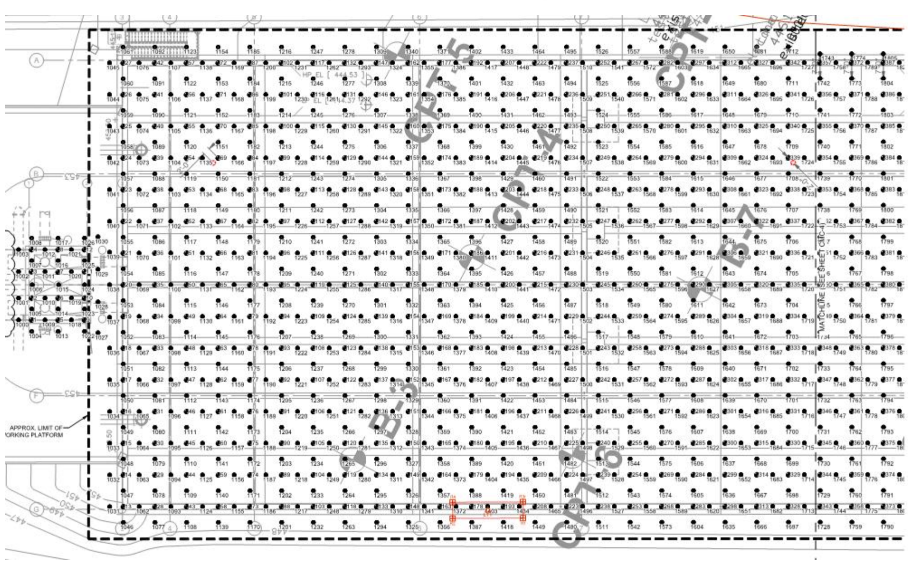
5 a. Place and compact additional Type 5 DGA to bottom of mat elevation per Menard's requirements
b. Pour Mat Foundation.



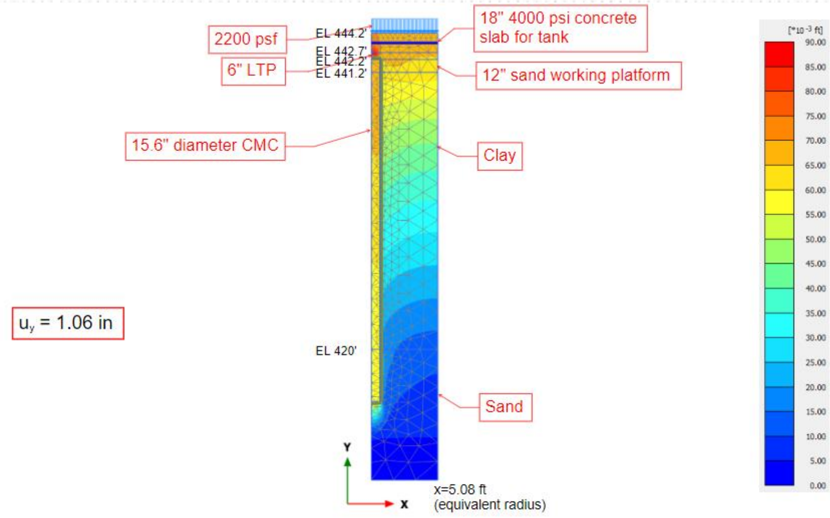
O’Fallon WWTP – Menard Concept

menard

- **Alternate Approved**
  - Design started in June 2020
- **Design Quantities**
  - 1,355 EQDs
    - Avg. 42 ft
  - 646 CMCs
    - Avg. 36 ft



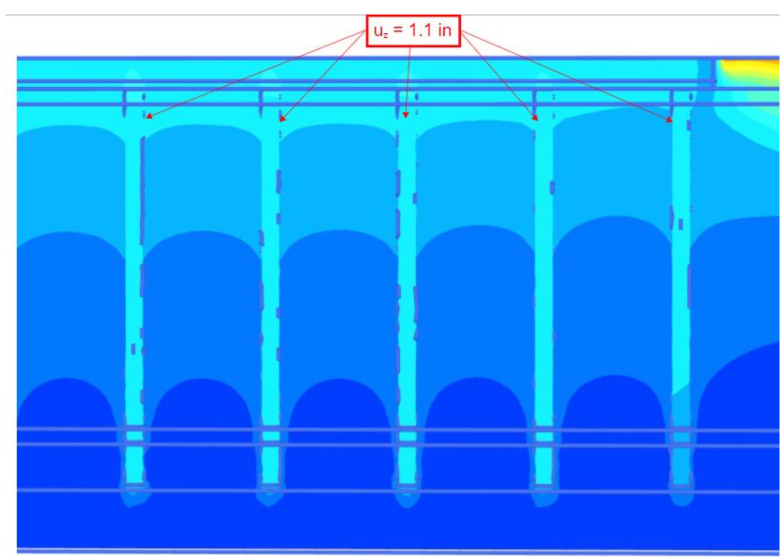
O’Fallon WWTP – Design Analysis



Total displacements |u| (scaled up 10.0 times)

Maximum value = 0.08831 ft (Element 169 at Node 797)

O’Fallon WWTP – Design Analysis



O'Fallon WWTP – Design Analysis

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ANALYSIS TYPE:

Axisymmetric case (iopt = 2)
Variable compressibility (jopt = 2)
Composite drain (kopt = 4)

EARTHQUAKE DATA:

Equivalent No of cycles = 16.32
Duration of shaking = 7.12 seconds

CONTROL DATA:

Number of layers = 23
Radial soil increments = 15
Radial drain increments = 2
Vertical increments = 39
Effective Overburden = 2200.000 lb/ft²
Depth to water table = 22.700 feet
Unit weight of water = 62.400 lb/ft³
Print results to a limit of = 2400.00 seconds
No of different time steps = 2

Number of Steps	Step Size (s)	Print Resolution
1000	0.100	10.000
2300	1.000	20.000

DRAIN DATA:

Radius of tributary area = 3.6200 feet
Equivalent radius of drain = 0.1667 feet
Internal drain area = 0.0873 sq ft
Vertical head loss, dh/dz = C1*Q^C2 (Manning)
Coefficient C1 = 0.3676
Coefficient C2 = 2.0000
Horizontal head loss, dh/dx = corf*V²/2g+V/permit
Coefficient of head loss, corf = 1.00000
Orifice Area, per unit length = 0.02187 sq ft / feet
Geotextile Permittivity = 2.92000 ft/sec / feet of head

RESERVOIR DATA:

Area of reservoir = 0.0873 sq ft
Depth to bottom of reservoir = 22.7000 feet
Vertical head loss, dh/dz = C3*Q^C4 (Manning)
Coefficient C3 = 0.0000
Coefficient C4 = 1.0000

FEQDRAIN
ANALYSIS OF PORE PRESSURE GENERATION AND DISSIPATION
DURING EARTHQUAKE LOADING FOR A SOIL PROFILE TREATED
WITH VERTICAL DRAINS
Version 2.1 December 1997

Table 1: Layer Values

Soil Properties										Cumulative Values				
layer number	layer thickness	kx (ft/s)	ky (ft/s)	mv (psf⁻¹)	unit weight (pcf)	nl	linc	theta	dr (%)	max	node	layAvg	DepAvg	time
1	2.5	3.29E-07	8.10E-08	4.38E-05	115.08	1000	2	0.786	0.375	0.0464	23	0.0347	0.0464	7.1
2	2.5	3.29E-07	8.10E-08	4.38E-05	114.11	1000	2	0.781	0.343	0.046	82	0.0458	0.046	7.1
3	2.5	3.29E-07	8.10E-08	4.38E-05	112.54	1000	2	0.786	0.271	0.0485	112	0.0468	0.0464	121
4	2.5	3.29E-07	8.10E-08	4.38E-05	111.39	1000	2	0.812	0.2	0.0513	154	0.0502	0.0504	7.1
5	2.5	3.29E-07	8.10E-08	4.38E-05	111.39	1000	2	0.823	0.2	0.0522	168	0.0519	0.0522	7.1
6	2.5	3.29E-07	8.10E-08	4.38E-05	111.39	1000	2	0.817	0.2	0.0517	190	0.0512	0.0517	7.1
7	1.74	3.29E-07	8.10E-08	4.38E-05	111.63	1000	2	0.81	0.2	0.0508	219	0.0501	0.0508	7.1
8	2.5	3.29E-07	8.10E-08	4.38E-05	114.58	1000	2	0.803	0.2	0.0495	256	0.0488	0.0495	138
9	2.5	3.29E-07	8.10E-08	4.38E-05	112.33	1000	2	0.786	0.207	0.0533	328	0.0466	0.0477	150
10	0.96	3.29E-07	8.10E-08	4.38E-05	113.08	1000	1	0.777	0.2	0.4537	345	0.0524	0.0459	150
11	0.19	3.29E-07	8.10E-08	4.38E-05	115.91	1000	1	0.684	0.362	0.4537	345	0.236	0.0582	150
12	0.94	3.24E-04	8.22E-05	1.00E-06	120.76	33	1	1.098	0.791	0.452	363	0.4077	0.4128	150
13	2.5	3.29E-07	8.10E-08	4.38E-05	116.51	1000	2	0.854	0.314	0.4427	381	0.1459	0.4023	150
14	0.71	3.29E-07	8.10E-08	4.38E-05	119.39	1000	1	0.811	0.319	0.4984	450	0.268	0.0591	7.1
15	2.5	3.24E-04	8.22E-05	1.00E-06	118.06	23	2	1.156	0.832	0.5806	486	0.4906	0.4809	7.1
16	0.97	3.24E-04	8.22E-05	1.00E-06	111.73	17	1	1.002	0.613	0.7416	504	0.6144	0.5434	7.1
17	0.6	3.24E-05	8.22E-06	6.25E-06	111.47	8	1	0.741	0.265	0.7416	504	0.6638	0.6872	7.1
18	1.95	3.29E-07	8.10E-08	4.38E-05	111.39	1000	2	0.782	0.9	0.6476	522	0.2931	0.6423	8.3
19	1.51	3.24E-04	8.22E-05	1.00E-06	112.42	17	2	1.017	0.618	0.4504	558	0.4033	0.4384	7.1
20	2.5	3.24E-04	8.22E-05	1.00E-06	120.41	34	2	1.171	0.729	0.3907	594	0.3556	0.3712	7.1
21	2.5	3.24E-04	8.22E-05	1.00E-06	120.94	32	2	1.171	0.878	0.3698	666	0.3408	0.3481	7.1
22	1.97	3.24E-04	8.22E-05	1.00E-06	118.06	24	2	1.1	0.71	0.4433	702	0.3808	0.3774	7.1
23	0.9	3.24E-05	8.22E-06	6.25E-06	120.94	39	1	0.963	0.586	0.449	720	0.4283	0.4195	7.1

O'Fallon WWTP – Design Analysis

menard

FEQDRAIN
ANALYSIS OF PORE PRESSURE GENERATION AND DISSIPATION
DURING EARTHQUAKE LOADING FOR A SOIL PROFILE TREATED
WITH VERTICAL DRAINS
Version 2.1 December 1997

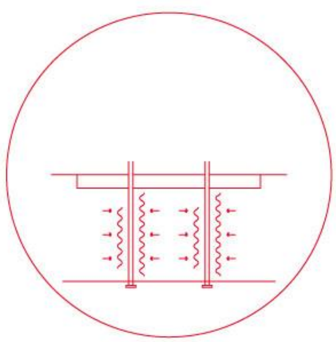
Table 2: Results at Each Print Out

Current time (s)	Max. Pressure Ratio	Max. Average ru		Water level in drain (ft)	Volume Discharged (ft³)	Settlement (ft)
		Value	Depth (ft)			
10	0.6443	0.6381	31.11	1.811	1.8237	0.0444
20	0.6353	0.6235	31.11	0.000	2.5385	0.0618
30	0.6158	0.6050	31.11	0.000	3.0254	0.0736
40	0.5984	0.5864	31.11	0.000	3.3978	0.0827
50	0.5813	0.5683	31.11	0.000	3.6949	0.0899
60	0.5647	0.5511	31.11	0.000	3.9375	0.0958
100	0.5051	0.4906	31.11	0.000	4.5822	0.1115
500	0.2974	0.2918	22.890	0.000	5.8110	0.1415
1000	0.2908	0.2870	22.890	0.506	5.7726	0.1405
1500	0.2826	0.2795	22.890	1.130	5.7203	0.1392
2000	0.2753	0.2726	22.890	1.688	5.6738	0.1381
2400	$\boxed{0.2700}$	0.2676	22.890	2.091	5.6404	$\boxed{0.1373}$

O’Fallon WWTP – Installation

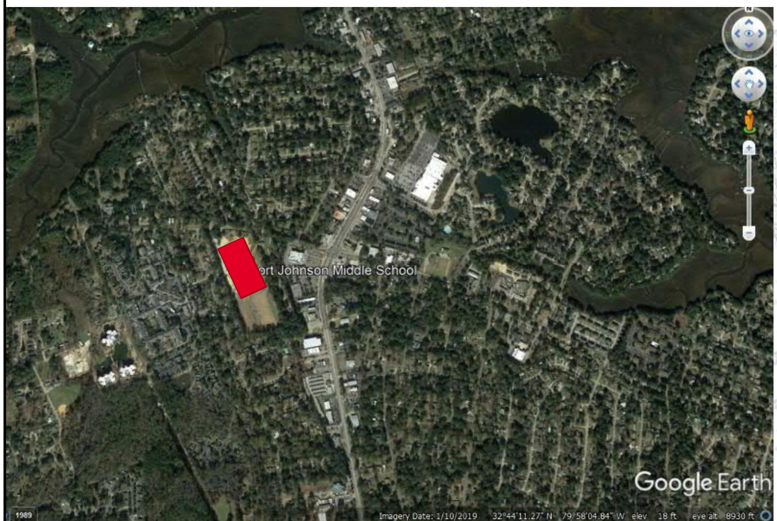
O’Fallon WWTP – Installation

O'Fallon WWTP – Installation

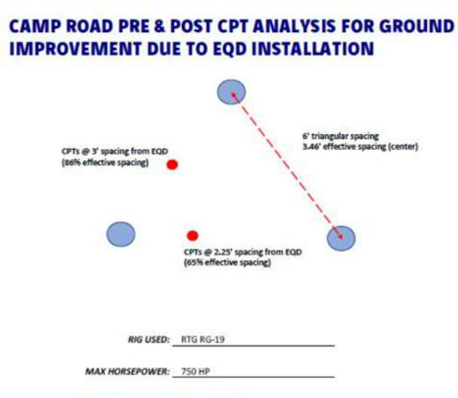


Case History – Camp Road Middle School Charleston, SC

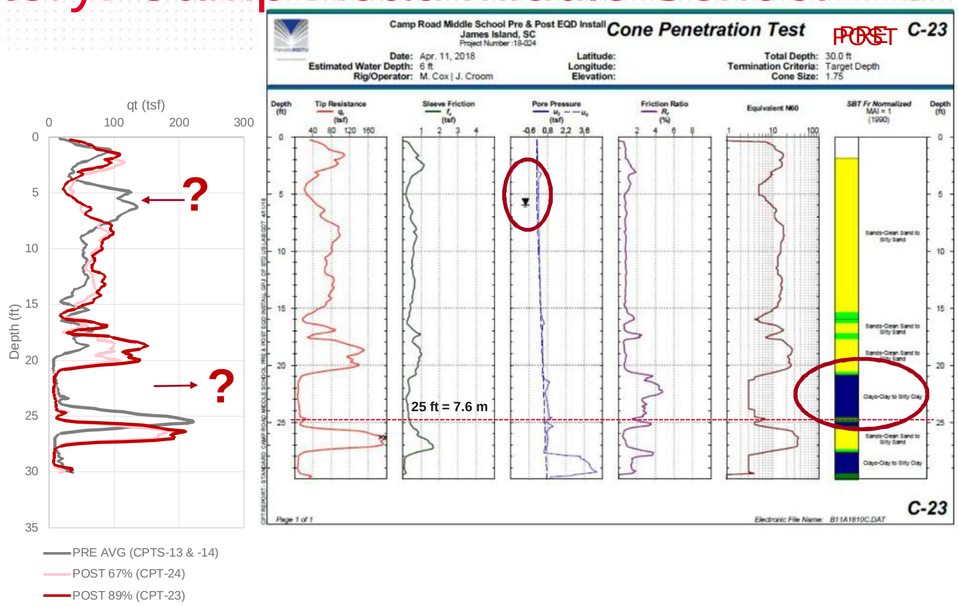
Case History: Camp Road Middle School



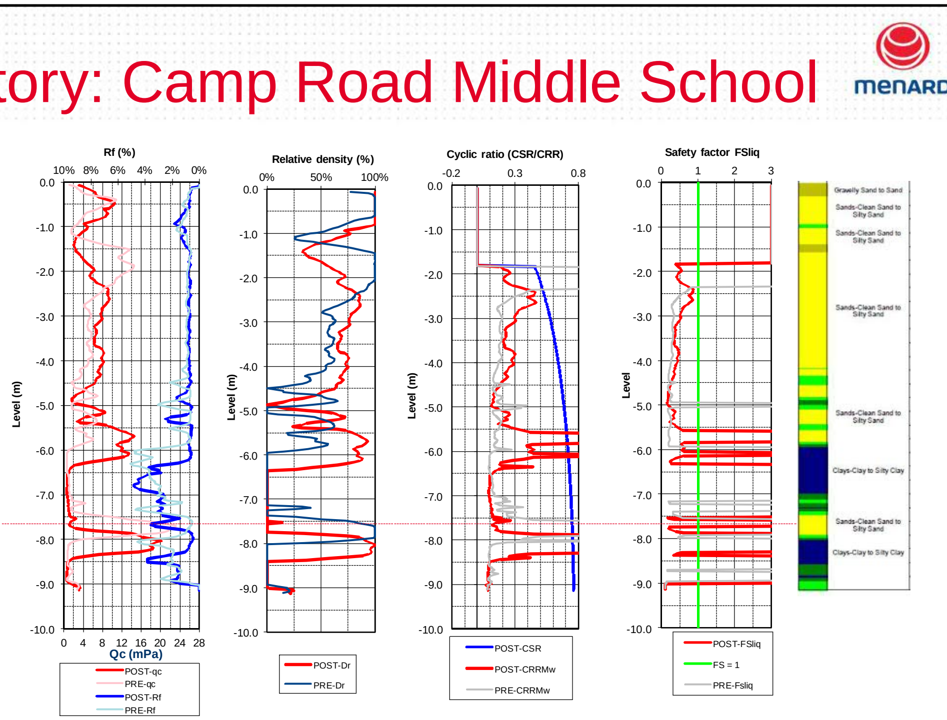
Case History: Camp Road Middle School



Case History: Camp Road Middle School

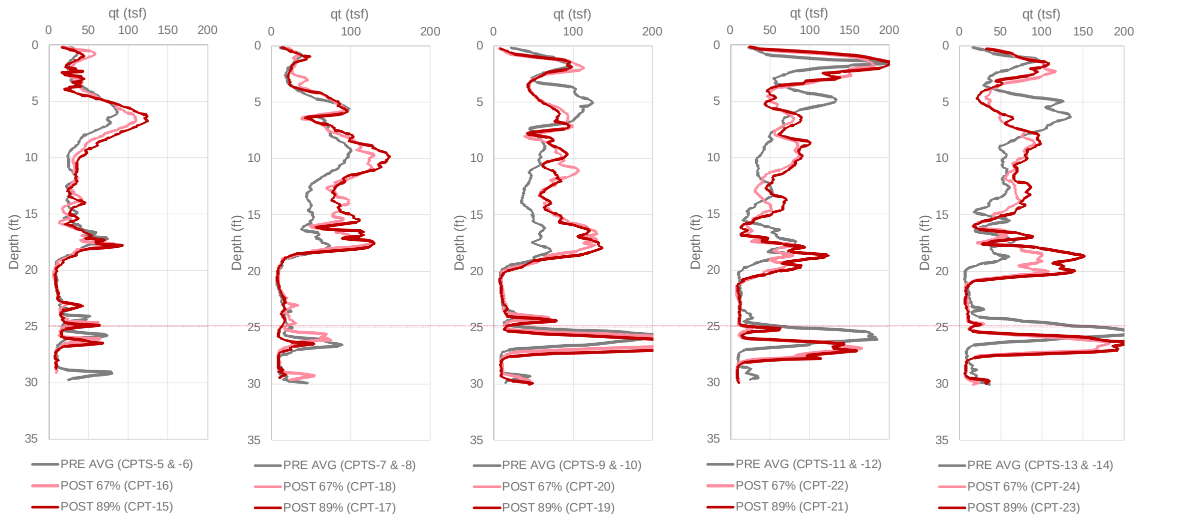


Case History: Camp Road Middle School



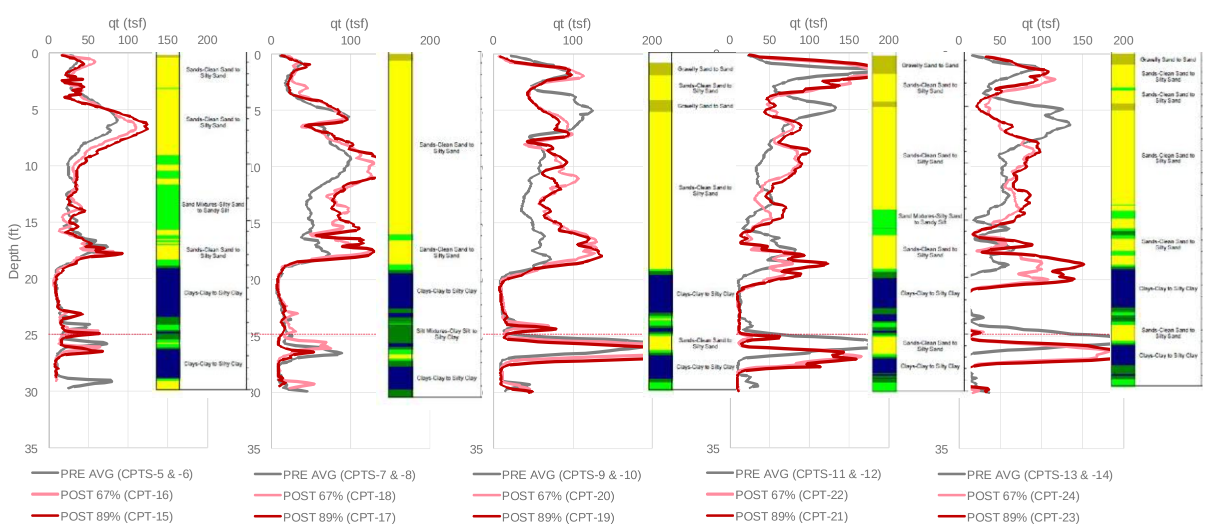
Case History: Camp Road Middle School

Average improvement Δqt ~ 1.5 to 1.9 MPa



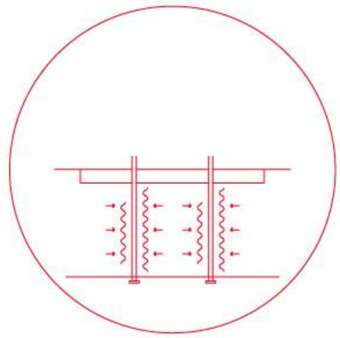
Case History: Camp Road Middle School

Average improvement Δqt ~ 1.5 to 1.9 MPa



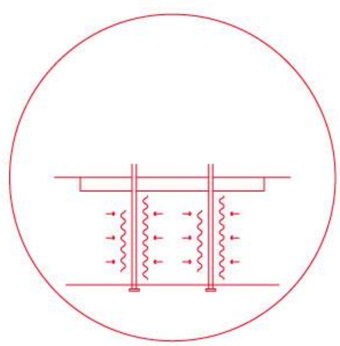
Earthquake Drains – Summary

- Primary action: Drainage
- Secondary action: Densification
- Used in liquefiable soils
- Alleviate pore water pressure during seismic events
- Greater flow capacity than stone columns
- Quick installation
- Cost-effective compared to other means of liquefaction mitigation



EQ Drains – Special Considerations

- Difficult to install through stiff/dense soils
- Consider contamination
- Artesian conditions



Emergency & Limited Access Geohazard Mitigation: Design-Build Solutions

Dylan Jones, P.E.[1] and Jose LuQuin, P.E.[2]

Abstract: Within the transportation industry, geohazards may include landslides, rockfalls, sinkholes, and slope failures. While many geohazards show early signs of failure, issues can often occur suddenly and in remote or rugged access locations, creating challenges and limitations for repair techniques. When critical infrastructure and the traveling public are put in harm's way by geohazards, this challenging access prevents quick response and prolongs road closures.

To overcome these access issues, engineers and contractors frequently utilize limited-access equipment and innovative practices to repair infrastructure in these situations quickly. Design-build solutions allow the owner/engineer/contractor to coalesce swiftly around the design and construction to begin promptly. This rapid response can significantly reduce infrastructure impacts and reduce overall repair costs by slowing or stopping progressive failures. During construction, drilling conditions can be monitored, allowing models to be calibrated and designs economized.

This presentation will present several projects where emergency response, quick design capability, and consistent site monitoring resulted in solutions that mitigated the geohazard and prevented further issues resulting from a slower response, including long delays and a more expensive final cost. The presenter will describe each project in detail, discuss the as-found conditions, itemize the different design options, and provide in-depth descriptions of the final solutions.

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[2] Regional Engineer, GeoStabilization International, Williamstown, KY. Email: jose.luquin@gsi.us

Emergency & Limited Access Geohazard Mitigation: Design – Build Solutions

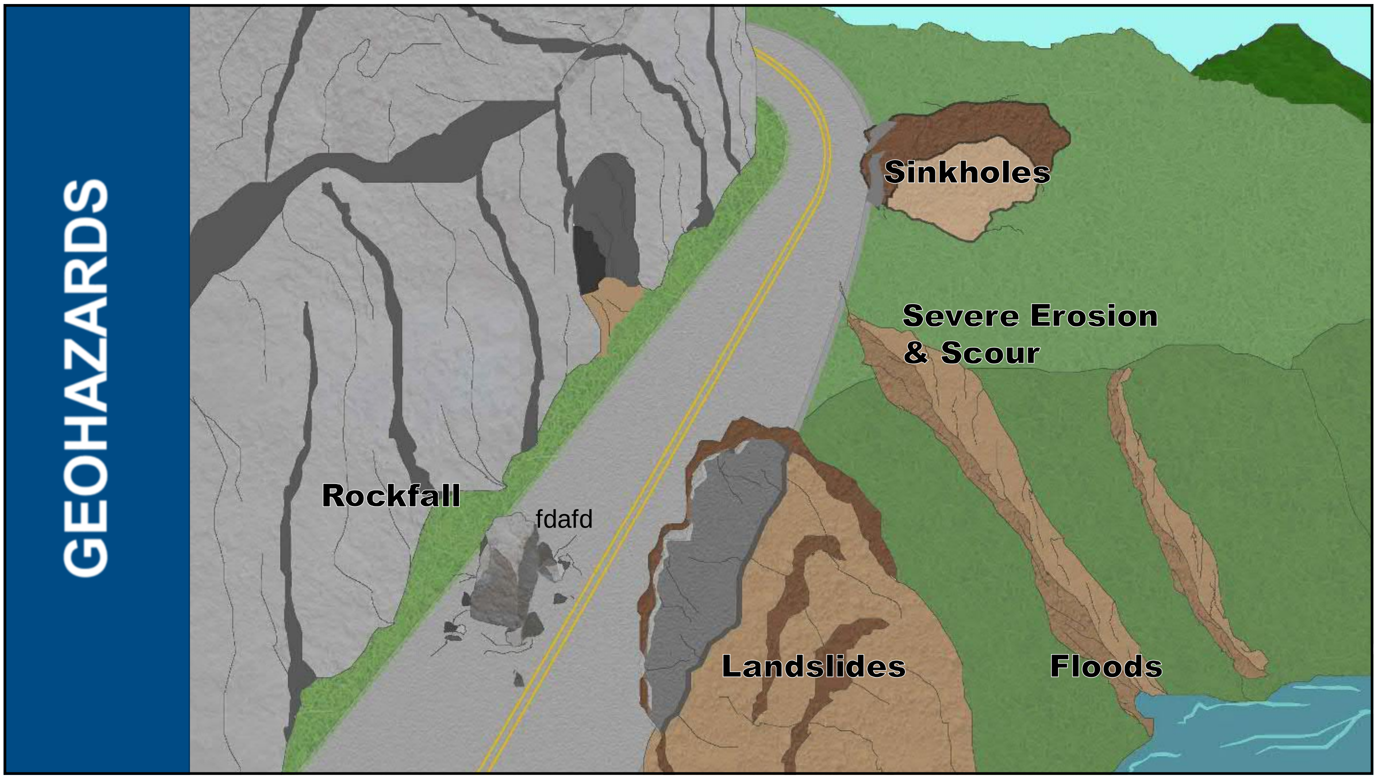
GSI
GeoStabilization International®

ETHISPHERE® WORLD'S MOST ETHICAL COMPANIES® 2019-2020

Topics to Cover

- Brief Description - Types of Geohazards
- Geohazards Affecting Ohio & Kentucky
- Design Build Process – Geohazard Mitigation
- Limited Access Equipment
- Case Studies

2

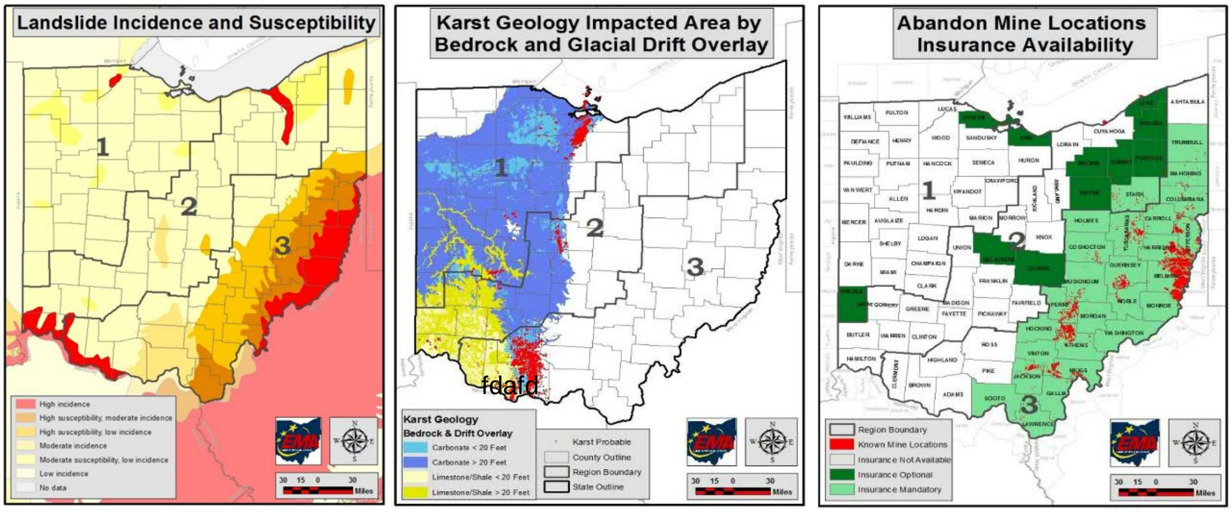


Geohazards in the Transportation Industry

- Landslides, rockfalls, ground subsidence, slope failures
- Can occur suddenly in rugged access locations
- Heavy rainfalls and flood events are often triggers

4

Geohazard Maps - Ohio



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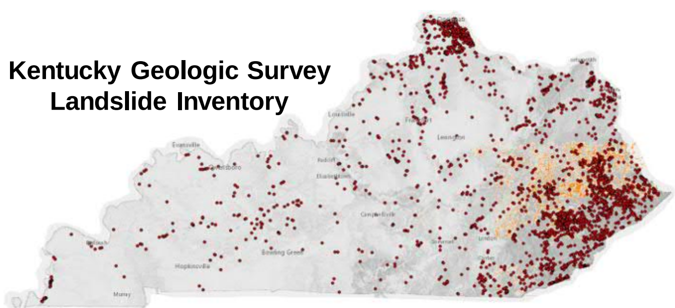
Source: www.ema.ohio.gov

Geohazard Maps - Kentucky

Kentucky Geologic Survey
Landslide Inventory



Kentucky Geologic Survey
Sinkhole Inventory

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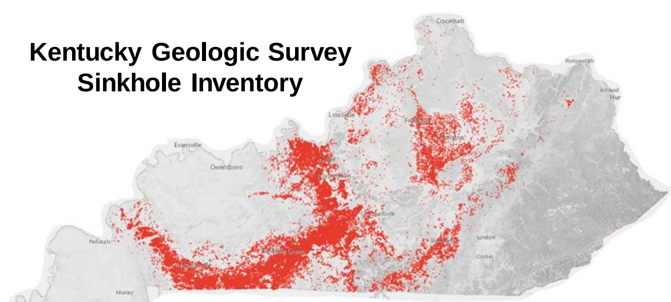


Geohazards in the Transportation Industry

- Geohazards can occur statewide and can create long detours and limit critical routes.
- When critical infrastructure and the public at risk, response must be quick to restore mobility
- Design-build solutions allow construction to begin quickly while designs are still being finalized

7

Design Build Solutions

- Design Build contracting methods are well suited for emergency and limited access geohazards
- Allows for innovative designs and construction methods
- Faster response and construction times
- Lower costs

8

Design Build Process - GSI

Investigate
Evaluate Site and Gather Data

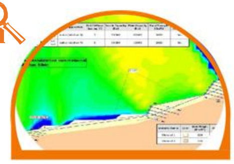
Analyze
Assess Mode of Failure

Design
Engineer Solution

Deliver
Implement Optimal Design

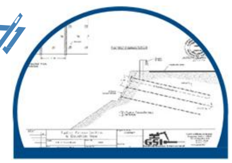
ü **No-cost**, no-obligation initial site assessment and data collection

ü Initial reconnaissance and cost proposals within **24 hours**



ü Dedicated engineers utilize **cutting-edge computer models** to analyze source and mechanism of failure

ü **Unmatched** technical know-how



ü Trained design engineers prepare robust, **cost-effective, and technically advanced** design solutions

ü Industry-leading **expertise and knowledge base**

ü Highly skilled crews can mobilize to sites within **24 hours**

ü **Unique multi-year warranty** ensures consistent customer satisfaction

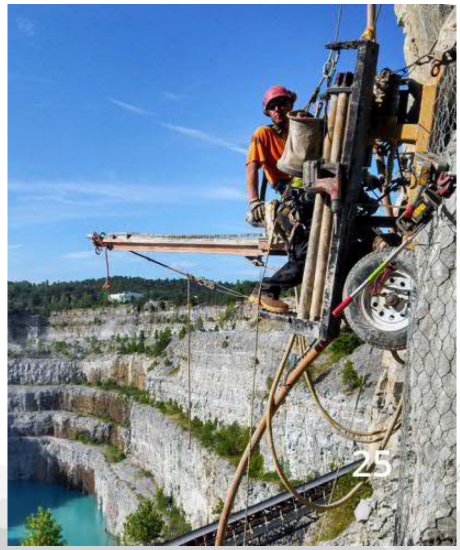
9

Limited Access Equipment

GeoStabilization International®

10

WAGON DRILL



PALLET DRILL

HIGH REACH DRILL

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SPIDER DRILL

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Case Studies

15

HAM 50-29.0 Retaining Wall No. 4

16

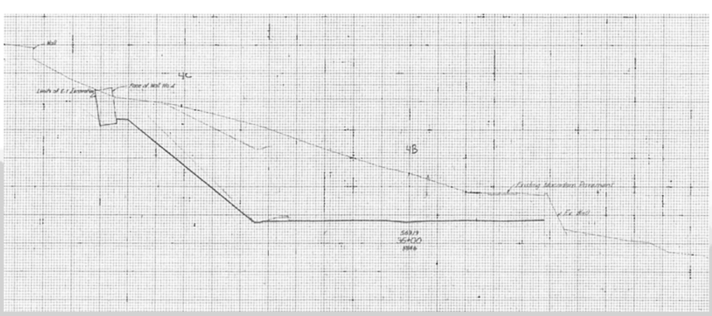
Project Background

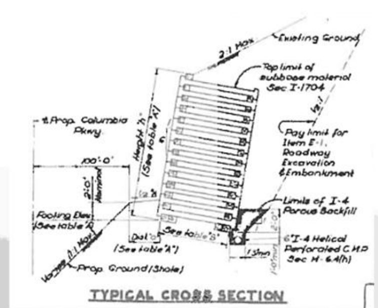
- Following heavy rains, movement began to occur on the slope below the wall causing toe to become undermined. This in combination of large amounts of water behind wall caused failure.
- Resulted in a progressive landslide that began to impact the properties behind Richwood Avenue
- Slide debris covered the parkway and closed the road

17

Project Setting

- Kope Formation – Weatherable Shale with interbedded limestone
- Existing retaining wall was built during construction of Columbia Parkway
- The 1:1 shale cut slope in front of wall was highly weathered





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Landslide as Seen from the Front and from Above

19

Landslide as Seen from the Front and from Above

20

View after Debris Removal

21

Design Goals & Constraints

Repair Goals

- Stabilize slope to prevent further upslope failures which could impact home
- Stabilize ends of retaining wall to prevent further unzipping
- Stabilize slope below existing wall to prevent future movement from undermining toe and promote vegetation

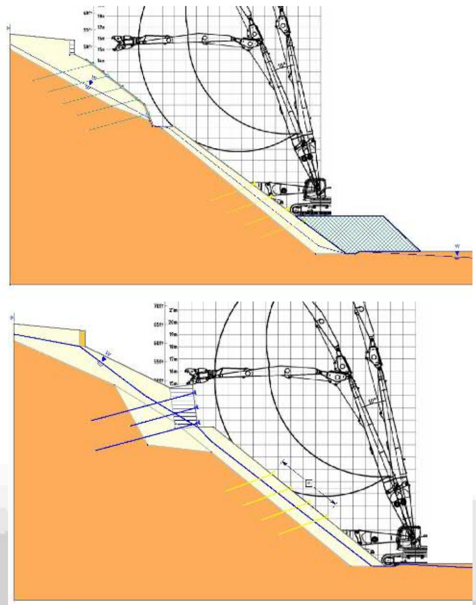
Key Constraints

- Limited and tight access from above
- Steep weathered and saturated downslope
- Allow one lane of traffic to be reopened
- Time: Slope was actively moving putting the home above in danger

22

Repair Plan

- Clear, excavate, and remove portions of failed retaining wall
- Reshape and excavate the center portion of the slide to create a soil nail and shotcrete wall that cut off failure surface
- Install horizontal drains & collection system to help control/collect groundwater and pipe to ditch.
- Install soil nails through the still standing retaining wall with a new reinforced shotcrete face to prevent further unzipping
- Install soil nails and high-tension mesh below the existing retaining wall elevation around mid slope to decrease unsupported length of slope, promote vegetation growth, and prevent future movement of the toe.



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Limited Access

- Working from above was ruled out because of the private residences along Richwood Avenue being too tightly packed together
- Constructed an elevated working platform at the toe of the slope so that a high-reach excavator could be utilized to reach the apex of the landslide
- High reach was utilized to excavate, drill and apply shotcrete

24

Design-Build Construction

- Once operations began, drilling and excavation showed some inconsistencies in the information we based our initial design upon regarding rock depths.
- The design-build nature of this project allowed us to continually confirm design and optimize nail lengths and wall heights to ensure the factors of safety were met.
- GSI site visited conducted, February 24, 2019
- Construction began on March 5, 2019
- Construction completed on May 6, 2019

25

Finished Product – Immediate & Two Years Later

May 2019

July 2021

26

US-50 Transmission Line Pole Foundations

GeoStabilization International®

27

Project Background

- Existing transmission line poles above Wooster Pike in Cincinnati were on an unstable slope and began to lean towards the roadway
- Two poles became a danger to the public due to worries of collapse

28

Project Setting

- Series of Transmission lines are located on the upslope portion of Wooster Pike, approximately 5 miles from downtown Fairfax, Ohio.
- This project also in an area of Kope formation consisting mostly of shale, in this north valley wall of the Little Miami River bedrock is sedimentary and consists of horizontally bedded shale and limestone but has sloping bedrock along the surface.
- Movement on the slope was a creep-type of failure, typical of this geology after shale has weathered, saturated, and gravity and erosion have taken control on these steep slopes producing colluvium.

29

Design Goals & Constraints

Repair Goals

- Stabilize slope to prevent further failures upslope and downslope of pole foundation
- Install two new micropile supported foundation base for new power pole

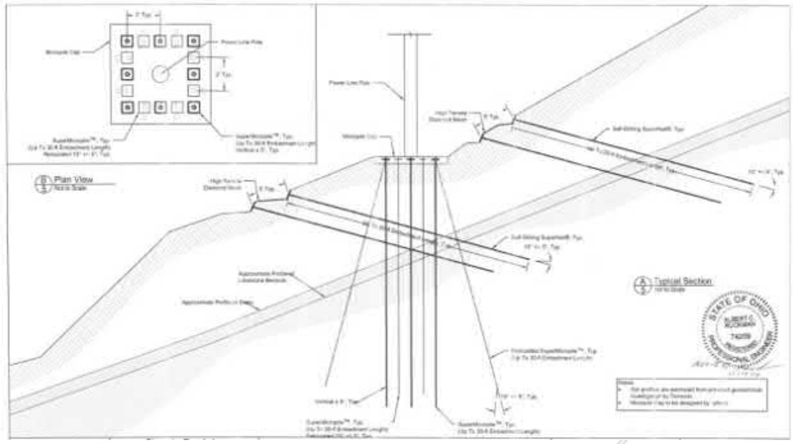
Key Constraints

- Overhead power to remain energized during construction limiting equipment and construction methods
- Existing retaining wall not to be disturbed
- Steep slope above retaining wall with limited ROW to allow benching and use of traditional equipment
- Allow bi-directional traffic flow and open all lanes during rush hours

30

Repair Plan

- Due to the steep slope beyond the in place retaining wall and overhead lines, a traditional drilled shaft foundation could not be constructed
- Bands of drilled soil nails and high-tension mesh were installed above and below the foundation to prevent movement
- An array of micropiles were installed to support a new reinforced concrete pedestal for a new steel power pole



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Repair Plan & Limited Access

- A drill mounted on a walking spider excavator that could be winched and maneuvered on the steep slope allowed all work to be performed on the slope.
- Spider drill was able to climb over the existing retaining wall to access the slope.
- Shorter drill mass allowed for drilling without power outages.

32

Photos After Construction Completed

33

Photos After Construction Completed

34

404 Baum St

GeoStabilization International®

35

Project Background

- From 398 to 406 Baum St. in Cincinnati a landslide impacted several properties from the north side of the slide bridging Oregon St. to Baum St.
- The slip initiated resulting in a failure of an existing retaining wall
- Created a head scarp that encroached on the uphill properties
- The toe bulge punched through a wall in two new townhouses at the base of the slide

36

Various views of Damage from the Landslide

37

Project Setting

- This portion of the Cincinnati area consists of a lot of urban land or fill described as clayey substratum over bedrock with a high percentage of sloping areas
- Without documented plans of the older retaining walls in place and based on the site assessment, it appeared that the existing retaining walls between the properties on Oregon St. and Baum St. were thought to be providing grade separation for shallow shale bedrock, but at the location of the head scarp it was apparent that colluvium was thicker, and the slope did not have the anticipated high shear strengths of the shale.

38

Design Goals & Constraints

Repair Goals

- Stabilize slope to prevent further failures from progressing up slope and impacting houses above
- Restore existing elevation of the ground at the top of the wall to regain loss back yard
- Stabilize toe and hill to protect the homes below

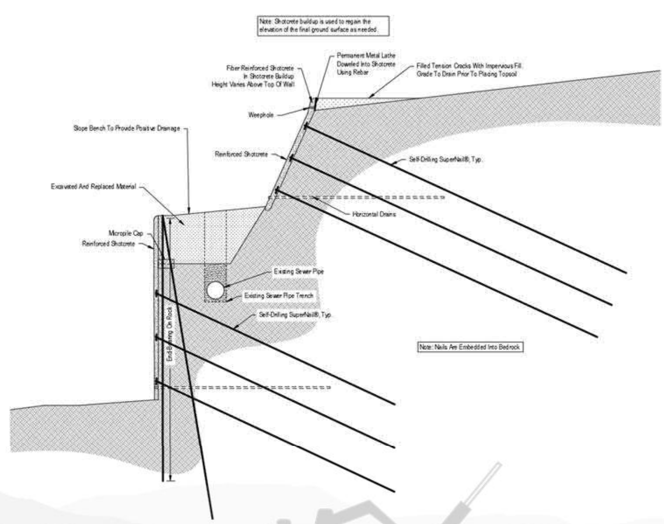
Key Constraints

- Equipment access above slope was not possible
- Tight working area at base of slope between homes and wall
- Time: Required a quick response to prevent slide from growing and impacting homes above
- Existing Sewer Line withing the slide mass

39

Repair Plan

- Construct an access ramp from bottom of slope to access and install a soil nail and shotcrete retaining wall at the top of the slope
- Drilled and Installed a Micropile Cap System below the existing sewer to allow for a stable working platform to access the sewer if needed in the future
- Excavated in front of bottom wall top down to install the lower soil nail and shotcrete wall.
- Backfilled cantilevered shotcrete wall
- Reconstruct existing retaining block wall to act as a buttress below the tiered wall system at the east end of the job where the blocks have not blown out



40

Finished Product – Immediate & One Year Later

41

KY 74, MP 6.12

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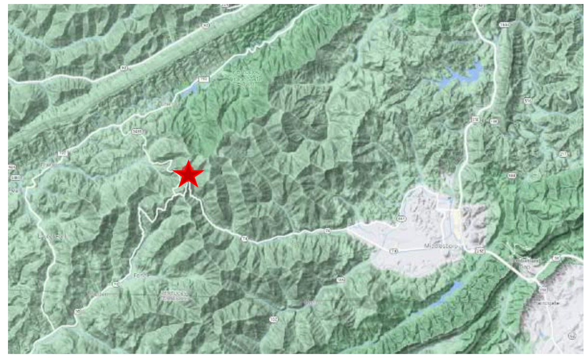
42

Project Background

- Following heavy rainfalls in February 2018 a slide occurred on KY 74 in the mountains of Eastern Kentucky.
- The slide occurred in a switchback taking out one lane of roadway covering the road below.



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Pictures of Failure

Design Goals & Constraints

Repair Goals

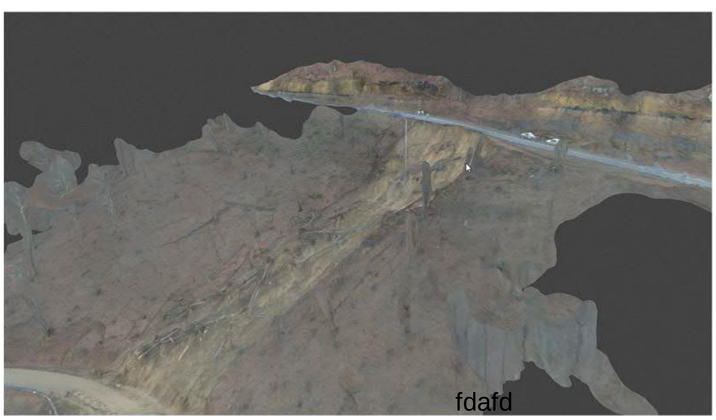
- Stabilize remaining roadway
- Restore loss pavement and shoulder width
- Install new culvert to promote better drainage of the area

Key Constraints

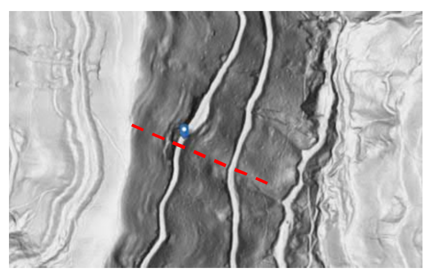
- Narrow and unstable working platform from top
- Detour was long and on narrow winding roads
- Road needed to be opened quickly to restore main route into Middlesboro

45

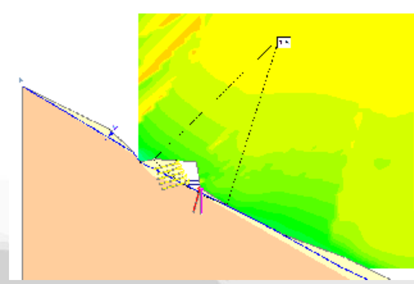
Design Approach



Pix 4d Model from Drone Flight



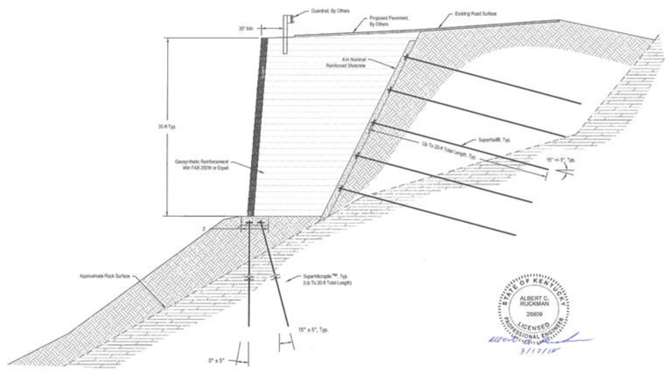
Lidar Imagery



Cross Section Used in Slide Model

Repair Plan

- Install soil nails and 20-ft tall shotcrete were to stabilize what was left of the roadway and create a stable working platform
- Install micropiles and a reinforced concrete beam to provide axial support for the GCS wall
- Install a 20-ft tall Geosynthetically Confined Soil (GCS) wall to restore the loss road platform and shoulder
- Install new culvert to promote better drainage of the area



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Construction Photos

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Construction Photos

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Completed Photos

- Failure occurred on February 14, 2018
- GSI was called and visited the site and conducted drone flight on February 24, 2018
- Design and cost proposal submitted on March 2, 2018
- Construction started on March 12, 2018
- Construction Completed on May 3, 2018

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Geohazards in the Transportation Industry

- Unexpected geohazards pose risks to infrastructure and public safety and can occur in locations requiring limited access equipment
- Being able to navigate these challenging problems quickly in a design-build fashion is important when critical infrastructure and human safety are at risk
- Fast response times can prevent geohazard sites from getting worse and creating larger problems
- Collaboration with these design-build solutions allow for stronger project performance and adaptability allowing for lower costs and quick scheduling

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Thank You

GeoStabilization International®

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HISTORY OF TILTING FLOODWALLS IN COVINGTON, KENTUCKY

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ABSTRACT

The Great Ohio River Flood of 1937 led to the passage of the Flood Control Act of 1938. In the subsequent decades the U.S. Army Corps of Engineers (USACE) constructed massive levee systems in nearly every city along the Ohio River, including Covington, Kentucky. The construction of that city's comprehensive levee system was completed between the years 1948 and 1955, including a series of large floodwalls along the Licking River. Soon after construction was completed, at the beginning of 1953, the local sponsors noted a small slide on the bank below where the monoliths were constructed. It was believed this slide was the result of the construction of a sanitary sewer in the riverbank. Within months significant movement of eight of these large concrete floodwall monoliths was first noted. The movement was not uniform and included both tilting and lateral translation. Movements continued and eventually grew to as much as three feet. Investigations led to efforts in 1956 to remediate what was deemed a slope stability issue, including the installation of a series of deep drainage pits. Additionally, attempts were undertaken in 1960 to salvage the expensive floodwalls by providing structural modifications to the walls themselves. Ultimately USACE was forced to completely demolish and replace six of the eight monoliths, and this work was completed in 1964. This paper explains the history and engineering reasons for the damage, and provides lessons learned that are still applicable today.

INTRODUCTION AND HISTORIC BACKGROUND

Covington is on the south shore of the Ohio River, directly across from Cincinnati, just downstream of where the Licking River flows into the Ohio. Covington's levee system was constructed in three Sections; Section C was constructed between July 1952 to December 1954. Section C consisted of 6,742 linear feet (lf) of earthen levee embankment and approximately 1,260 lf of floodwall, 465 lf of which are the flat-based concrete T-Wall monoliths that are the subject of this paper. Section C is located entirely along the Licking River (See Figure 1). The subject floodwalls were constructed in a tight bend alignment that allowed for the preservation of a number of fine homes near the intersection of Wallace and Glenway Avenues (See Figure 2). In the same neighborhood as many as 14 other homes were demolished to make way for earthen levee embankment. The original Definite Project Report from 1946 anticipated that the protection in the area of interest would be earthen levee embankment instead of floodwall. This different realization of the protection system would have resulted in the demolition of six additional homes that were ultimately spared.

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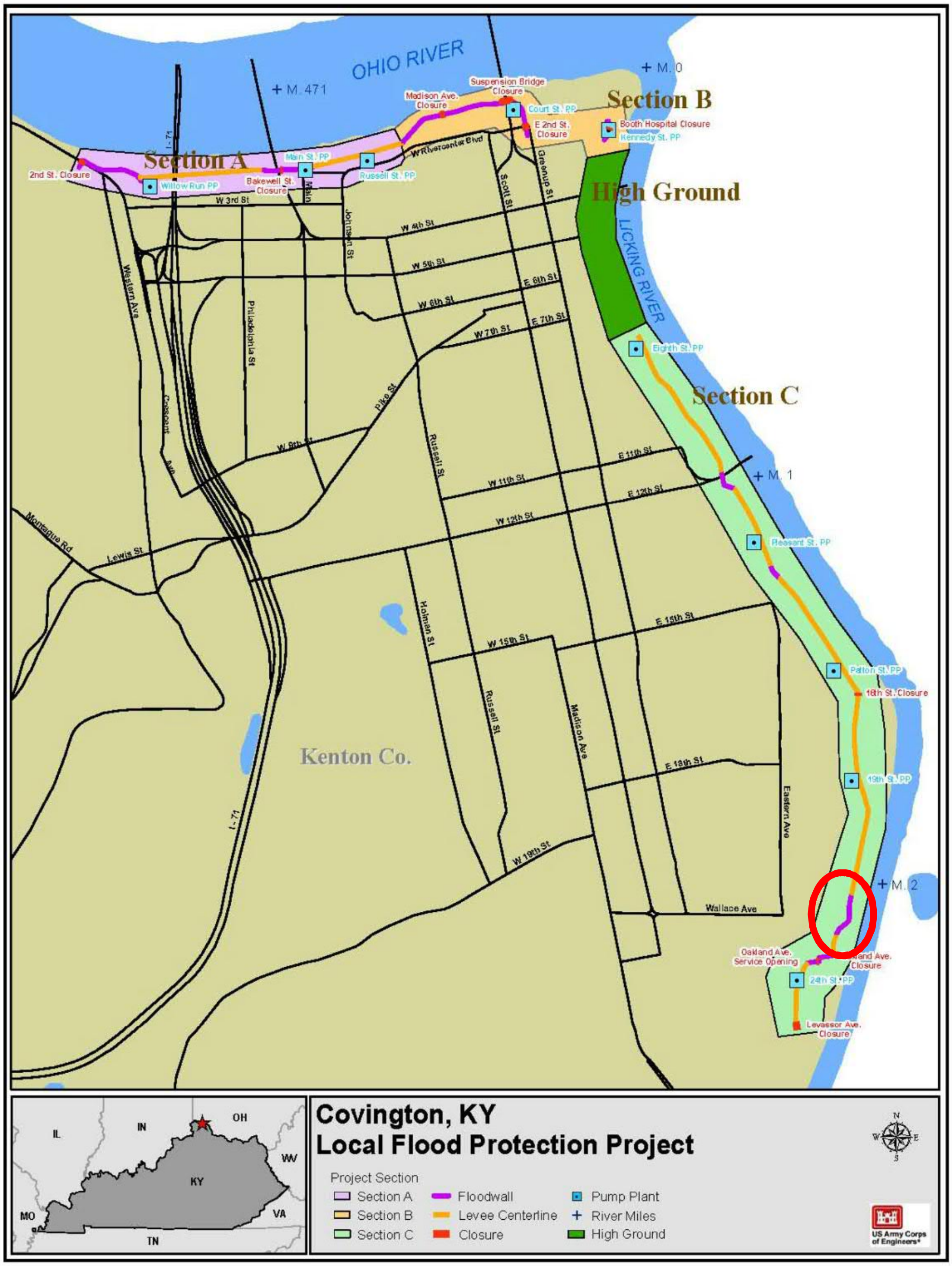

Figure 1: Covington, Kentucky LFPP Map with T-Walls highlighted.

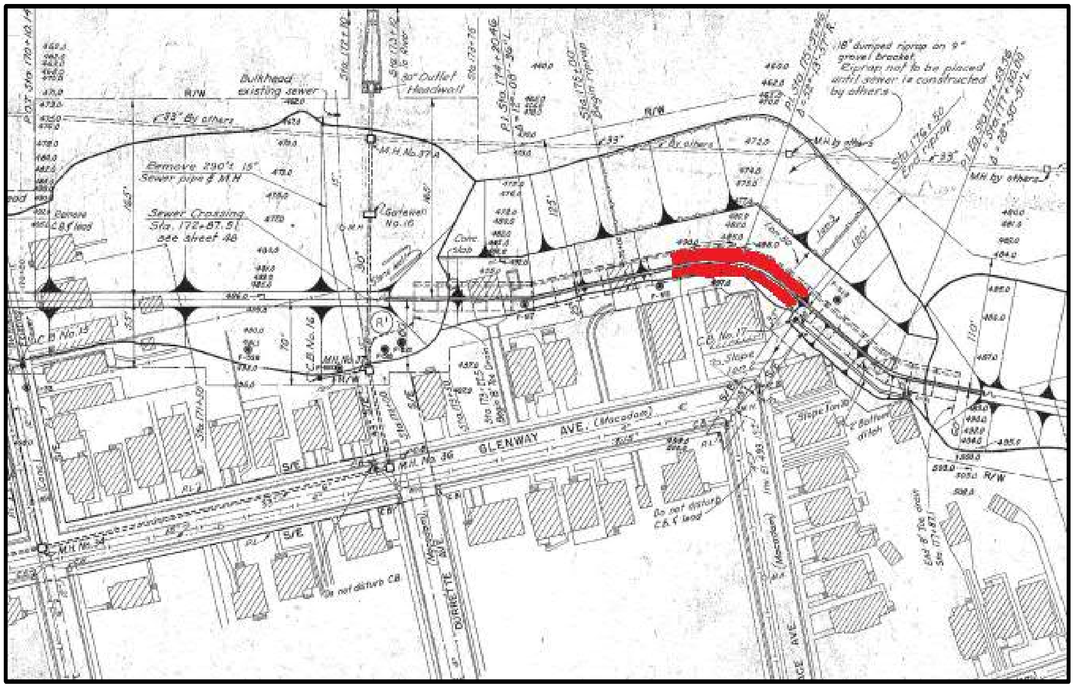

Figure 2: Covington, Kentucky LFPP Map with Damaged T-Walls highlighted.

DESCRIPTION OF THE ORIGINAL FLOODWALL CONSTRUCTION

The original T-Walls were flat-bottomed with deep shear keys, which was the conventional design for 1952, the year the project design was completed, and construction commenced (See Figure 3). These floodwalls were not founded on deep foundations, and no ground improvement was undertaken. The original design basis was memorialized in the Definite Project Report in 1946, which indicated earthen embankment levee was planned at the location where the failed floodwalls were ultimately constructed. This same report provided a statement that computations were completed indicating that the factor of safety against failure of all floodwall foundations would be greater than 1.5. A note in this report references an Exhibit 3 in Appendix B, but unfortunately this exhibit does not survive.

No design documentation was found that explained the decision to switch from earthen embankment to floodwalls at the critical location. Additionally, no slope stability calculations for the floodwall location could be located from the original files.

Critically, these floodwalls were constructed at the edge of the top of a steep slope, and the project plans indicate the top of the slope was widened significantly by the placement of fill towards the Licking River (see Figure 4), and extent and depth of this fill was increased during construction (see Figure 5). This fill was placed over "poorly consolidated foundation which was generally debris or unconsolidated clay." Another important feature of note is the 33-inch diameter sanitary sewer line shown on the plans near the bottom of the slope; the plans state that this line was to be installed by others. Because a portion of this paper's focus is on a slope stability problem that first emerged in 1953, it is important to note that vertical inclinometers were not introduced to the practice of geotechnical engineering until 1969.

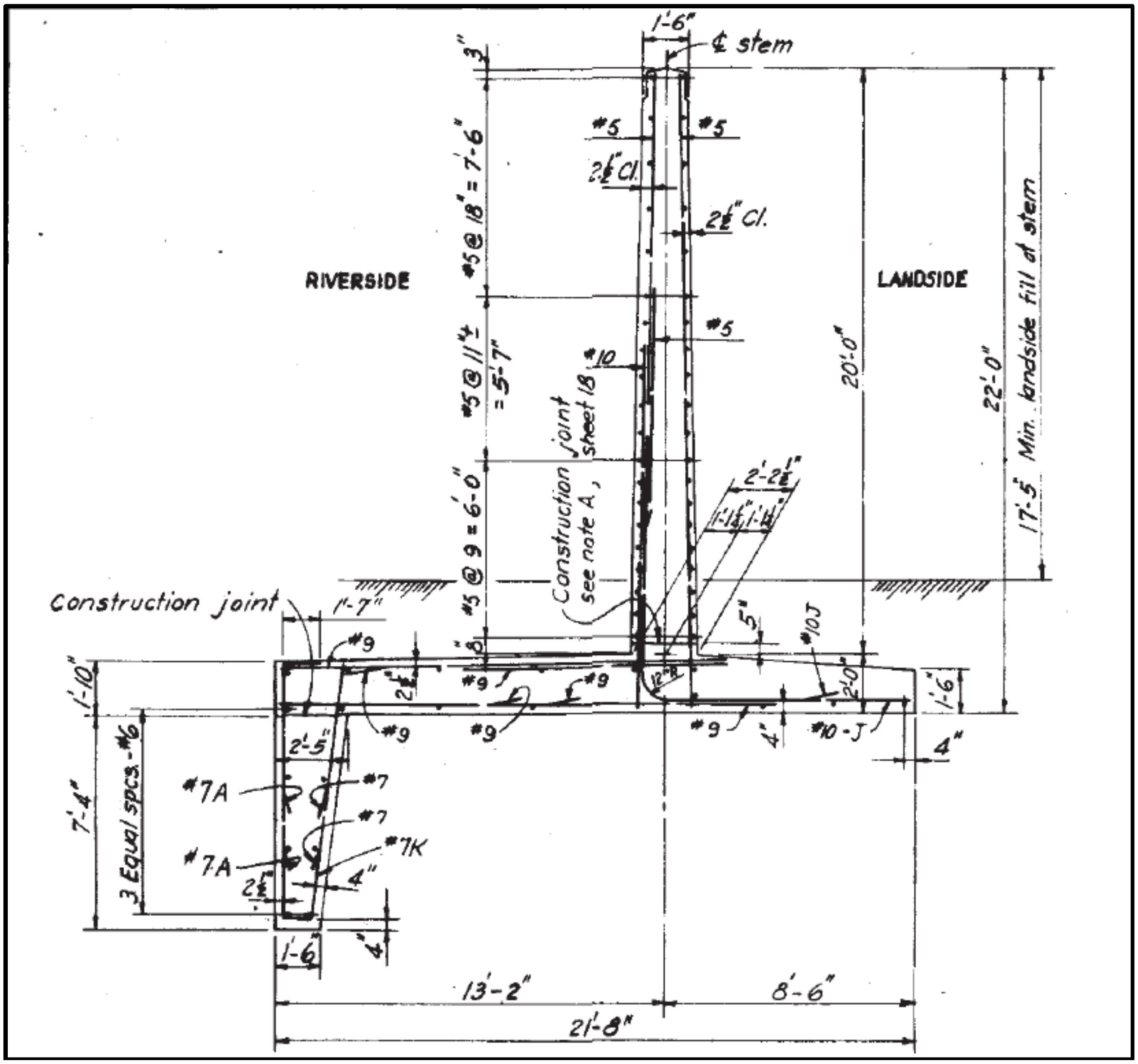

Figure 3. Original Flat-Bottom T-Wall Design

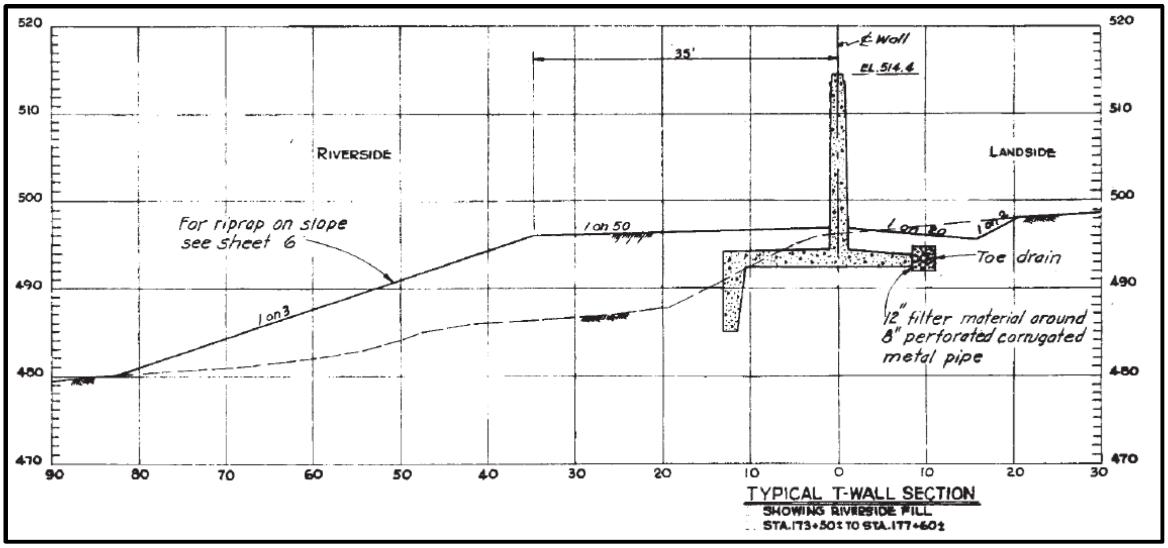

Figure 4. Planned Riverside fill placement at top of slope

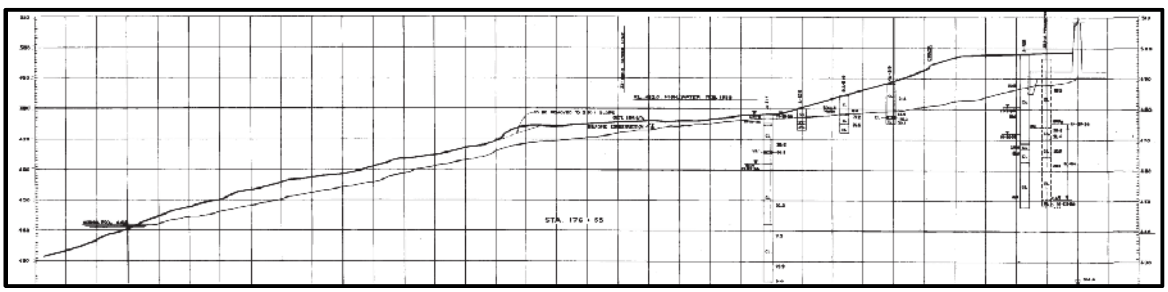

Figure 5. Actual Riverside fill placement across entire slope

1956 INVESTIGATION AND FIRST STAGE OF REMEDIAL WORK

The construction of the new flat-bottomed T-Walls in Section C was complete by April 1953. By early June 1953, the USACE Resident Engineer reported that a series of small slides had occurred that had necessitated rework and caused inconvenience to the construction of the Sanitation District's installation of a 33-inch diameter sanitary sewer near the base of the completed levee and floodwall fill slope. He further stated that "although certain foundation failures had occurred, none had encroached upon the flood protection works." It was believed this slide was the result of the construction of the sanitary sewer near the riverbank; in particular the sewer contractor had not shored the excavation. After the sanitary sewer contactor made "corrections" to his methods of construction, the slope and floodwall appeared to be stable until the high water in the Spring of 1956, when tension cracks appeared on the slope below the T-Wall. Additionally, *"slight differential settlement and lateral movement of T-Wall monoliths occurred, with movement from both ends toward station 175+97.46. The movement was noted as variable, with the end joints opening and those joints nearer to station*

175+97.46 closing and forcing joint material out." These together provided strong evidence that further riverward movement had occurred.

As recorded in the Louisville District "Section C Design Memorandum No. 1 Remedial Work", dated December 1956, USACE commenced with a geotechnical investigation of the issue by drilling 18 hand-auger borings and seven power-auger borings in October 1956. The results indicated a well-compacted embankment material of clay founded on a poorly consolidated foundation which was generally debris such as broken brick, mixed with clay. All borings upslope of the 33-inch sanitary sewer alignment encountered water, while borings drilled downslope of the sewer did not encounter water. Some borings upslope of the sewer encountered artesian water pressures. USACE concluded that *"the entire slope from the Licking River to the crest had moved riverward by erosion or slippage."* They also noted the nearly vertical bank at the toe of the slope where it met the river, and that this bank was unstable. USACE also concluded that the compacted clay backfill in the sanitary sewer trench stopped normal underseepage along the slope, and *"causes water to be perched landward from the sewer line with several feet of head available."* USACE further concluded that the *"1956 slide was aggravated by events which took place during the high water of February 1956. First, high water with correspondingly increased velocity removed some of the toe support and partially submerged the upper strata. Hydrostatic pressure from the water in the lower strata effectively reduced the normal force acting on the sliding surface. This surface has not been revealed by drilling; however, it is presumed that it is along the strata which bears the water. It is believed that the submergence and subsequent drawdown along with the conditions noted above caused the slide."*

The remedial plan centered on the installation of a series of seven drains evenly spaced along the slide area with 8-inch perforated metal pipes to be placed in conventional trenches surround by a filter-graded sand and gravel backfill. However, the remedial work that was actually designed and constructed looked very different than this (see Figures 6 and 7). Seven, braced sheetpile excavations served as collectors for underseepage. Each structure was 14'-6" by 14'-6" in plan, and filled with "free draining material", which was then covered with a 2-foot thick layer of filter material. The tip elevation of all sheetpiling in the seven pits was approximately 458 to 460 in order to penetrate below a pervious sand-gravel stratum. The sheetpile length varied considerably (from 19- to 46-feet) since some pits were constructed at the top of the slope and some were located near the bottom. The arrangement of the drainage pits in relation to the monoliths is indicated on Figure 8.

Research has not turned up any reporting on how productive the drainage pits were in intercepting and producing water. What is known is that they apparently did not have the positive effect on slope stability that the designers envisioned and intended.

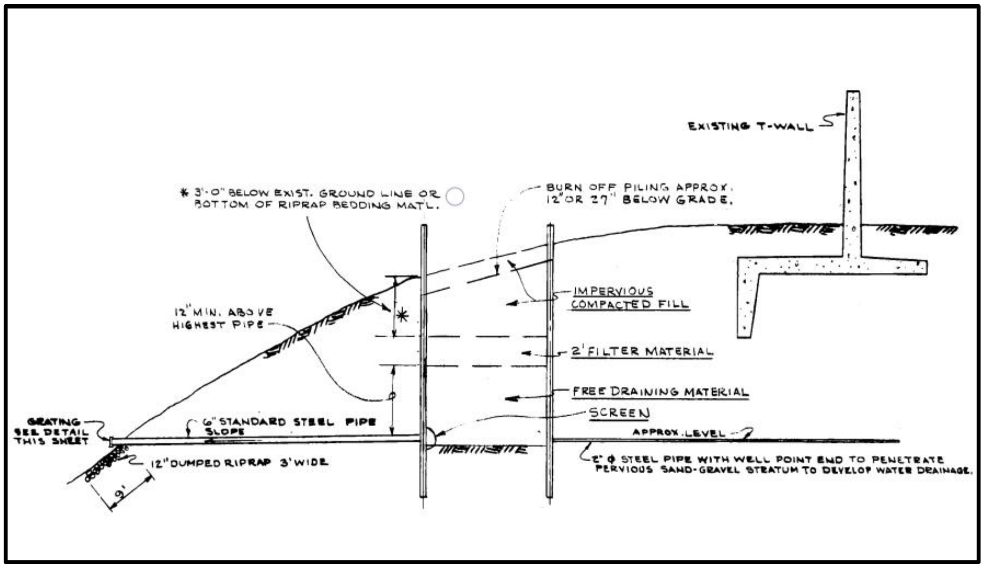

Figure 6: Typical Section of Drainage Pits

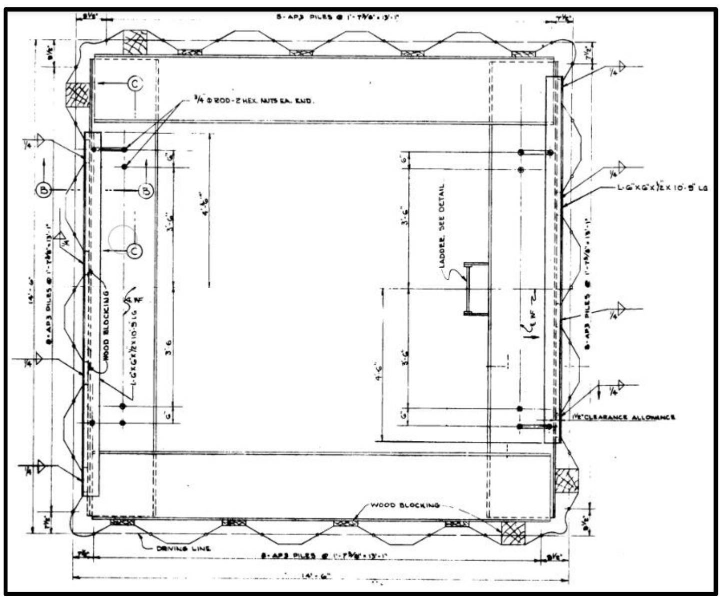

Figure 7: Plan of Braced Sheetpile Drainage Pit

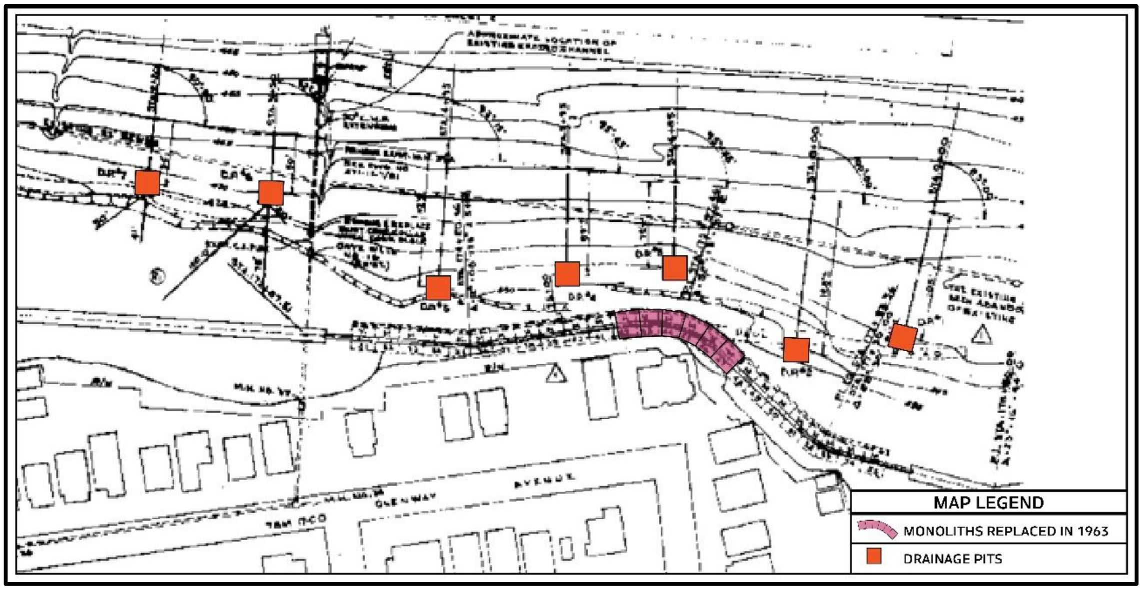

Figure 8: Arrangement of Drainage Pits Relative to Floodwall Monoliths

1958 STUDY OF ALTERNATIVES

The District undertook a study of alternatives for "Reconstruction of Wall" in 1958. No less than five alternatives were studied in detail. All of these alternatives would have replaced 25 existing monoliths and approximately 300 feet of earthen embankment with new T-Wall monoliths along various alignments with different setbacks from the edge of the slope. All would have required demolition of some existing homes.

A separate "Bank Stabilization" scheme was also developed. This would have required construction of 19 interlocked, gravel-filled sheet pile cells, each 46.15 feet in diameter, to be placed in the Licking River bank to buttress the slope. The sheet piling for the cells was to be driven to rock, which would have required sheets as long as 67 feet. This scheme would not have disturbed any of the existing houses nor the existing levee and floodwall alignment. Plans and specifications were fully developed in April 1960, but for reasons unknown, USACE did not issue the contract to bidders and the plans were ultimately shelved.

While USACE studied the problem, the slope and the floodwall continued to move. By July of 1960 significantly greater movement of the six monoliths 41 through 46 was noted. The movement was not uniform and included both tilting and lateral translation (see Figures 9, 10 and 11). Maximum movements eventually grew to as much as three feet.

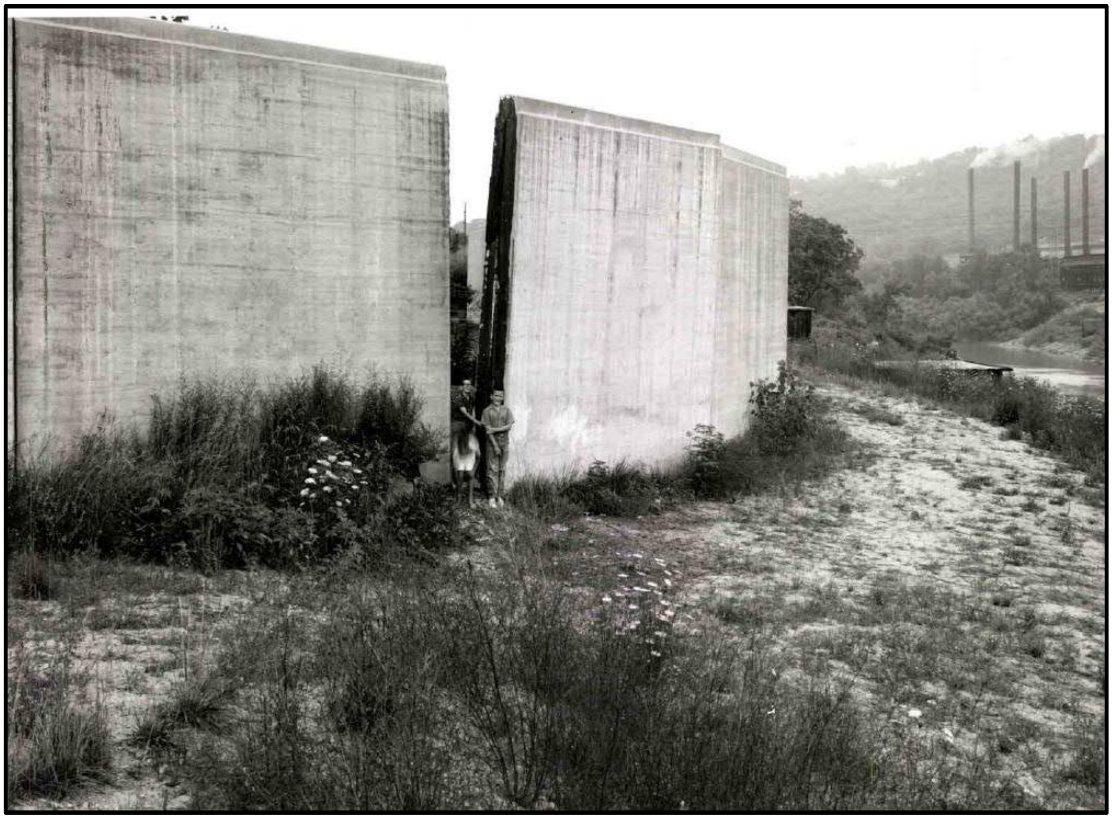

Figure 9: Displaced Floodwall Monoliths in July 1960 - Mono. 43 to right of boys

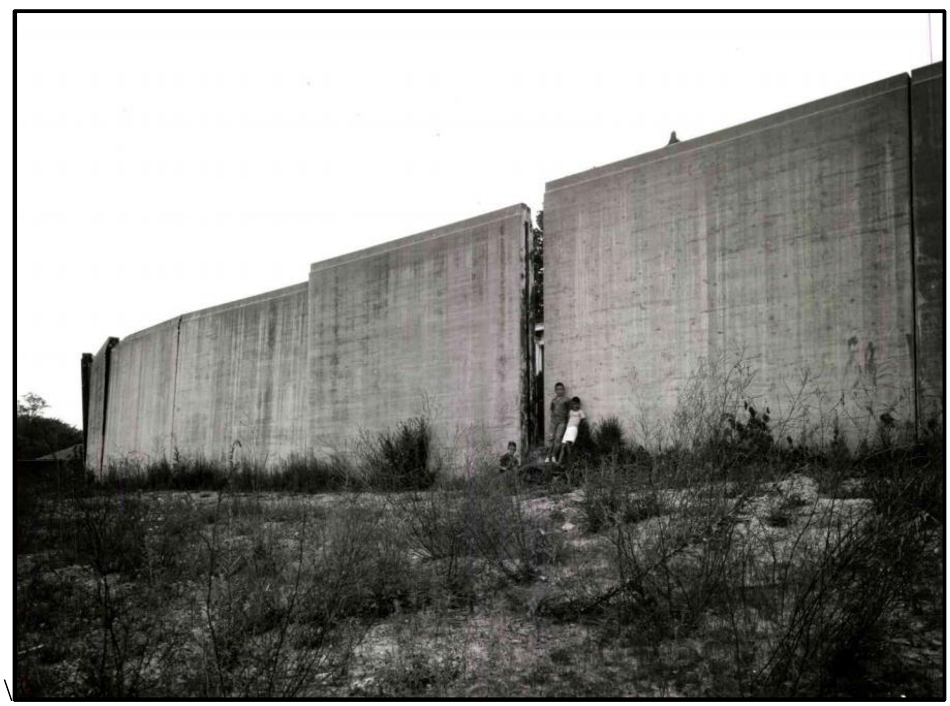

Figure 10: Floodwall Monoliths 43, 42, 41, 40, 39 and 38 (left to right)

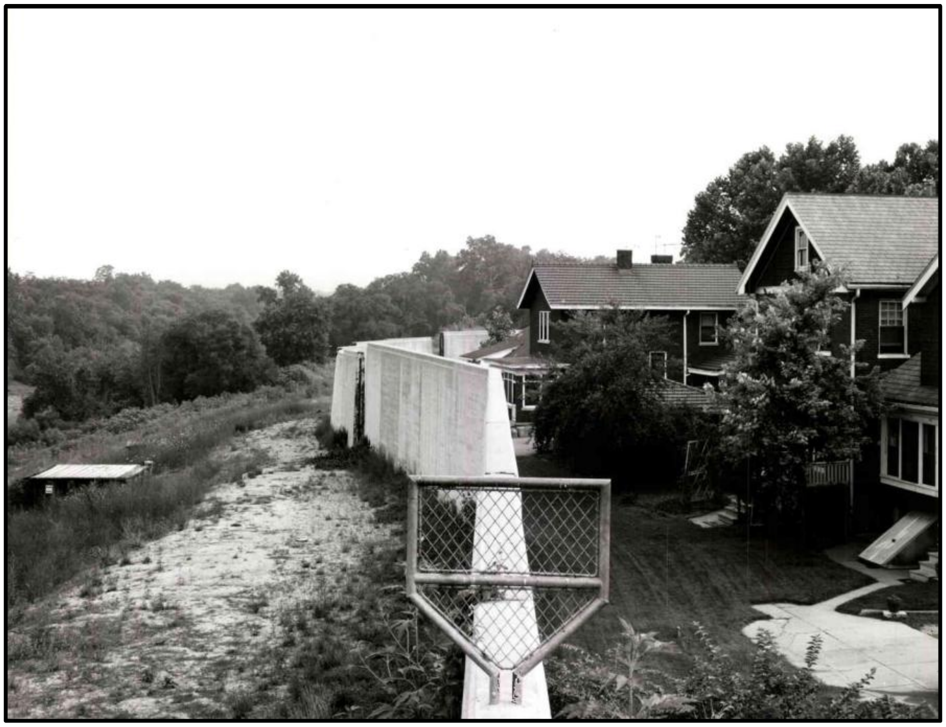

Figure 11: Looking south toward displaced monoliths (Drainage Pit No. 4 to the left)

1960 SECOND STAGE OF REMEDIAL WORK

The District issued a contract in September of 1960 to provide closures to the opened monolith joints with timber planking fixed in place to the concrete with anchor bolts (see Figures 12 and 13). No photographs could be located of the joint planking, possibly because it was only meant as an interim risk reduction measure. It is not known if continued movement of the monoliths destroyed the interim protection provided by the planking, but it is probably safe to assume it became heavily distressed.

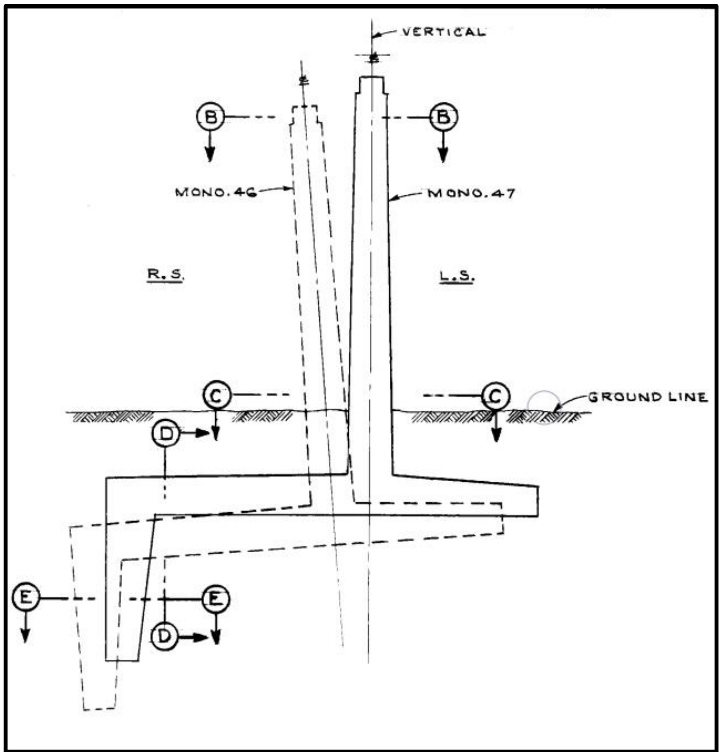

Figure 12: Typical Extent of Timber Planking to Close Joints

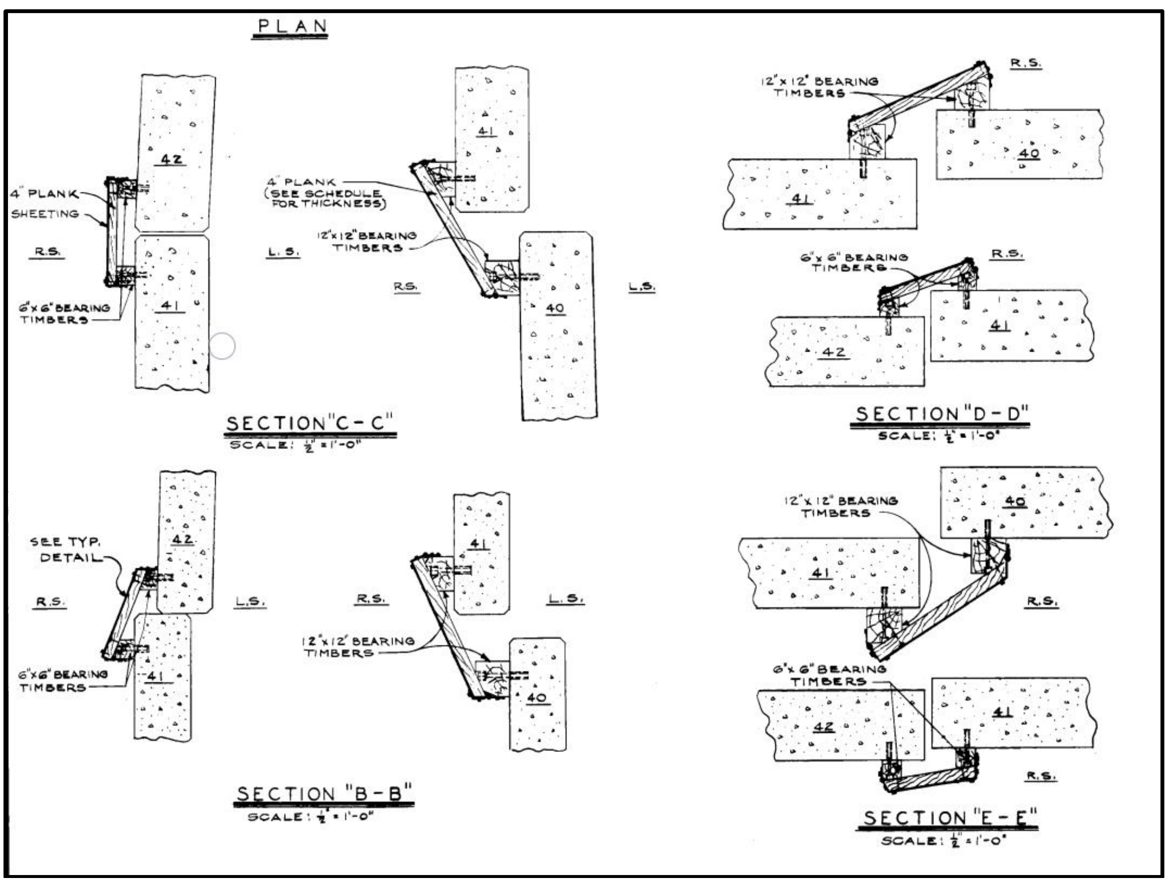

Figure 13: Details of Timber Planking to Close Joints

FINAL REMEDIAL WORK 1963-64

The final remedial work took place in 1963-64. None of the alternatives from the 1958 study of alternatives were selected. The scope included the complete demolition of six flat-bottomed T-Wall monoliths – monoliths 41-46 – and replacement with new sloping-based T-Wall monoliths (see Figures 14 and 15). The new monoliths had keys that extended to a depth of over 16 feet below finished grade, compared to the original walls' 12-foot-deep shear keys. Additionally, the bases of the new walls were 25'-8" wide, as compared to only 21'-8" for the original walls. Overexcavation of the foundation for each new monolith was required as well. The scope also included filling the previously constructed drainage pits with free draining material and filter material before topping each with impervious backfill. Finally, pressure grouting below the two adjacent flat-bottomed T-Wall monoliths 40 and 47 was included in the scope to ensure all voids had been filled. No construction documentation or photographs of the remedial work could be located.

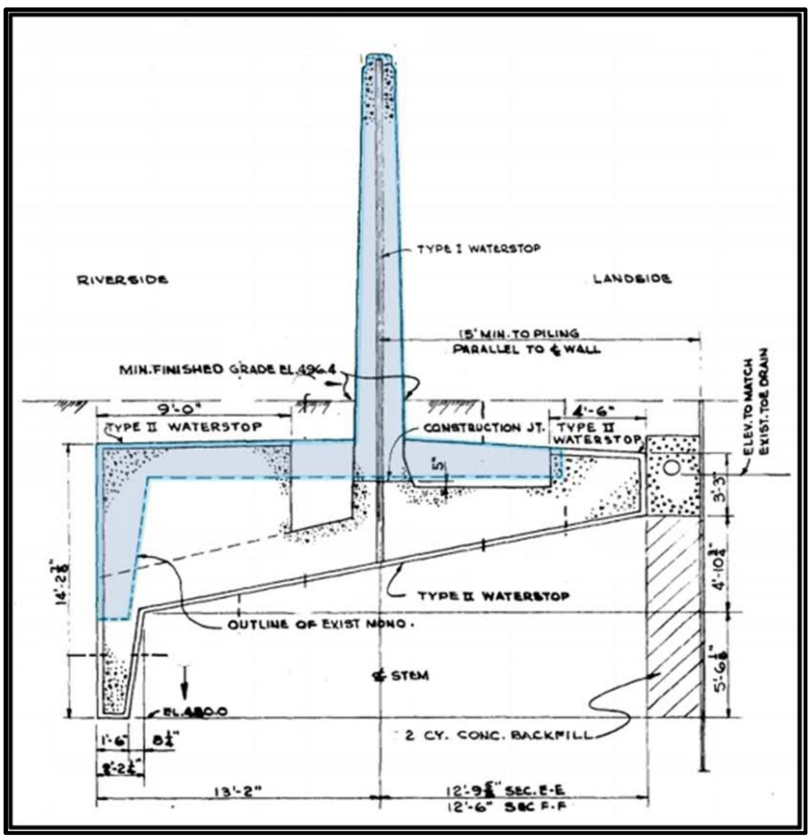

Figure 14. Final Remedial Sloping-Base T-Wall Design
(USACE Louisville District)

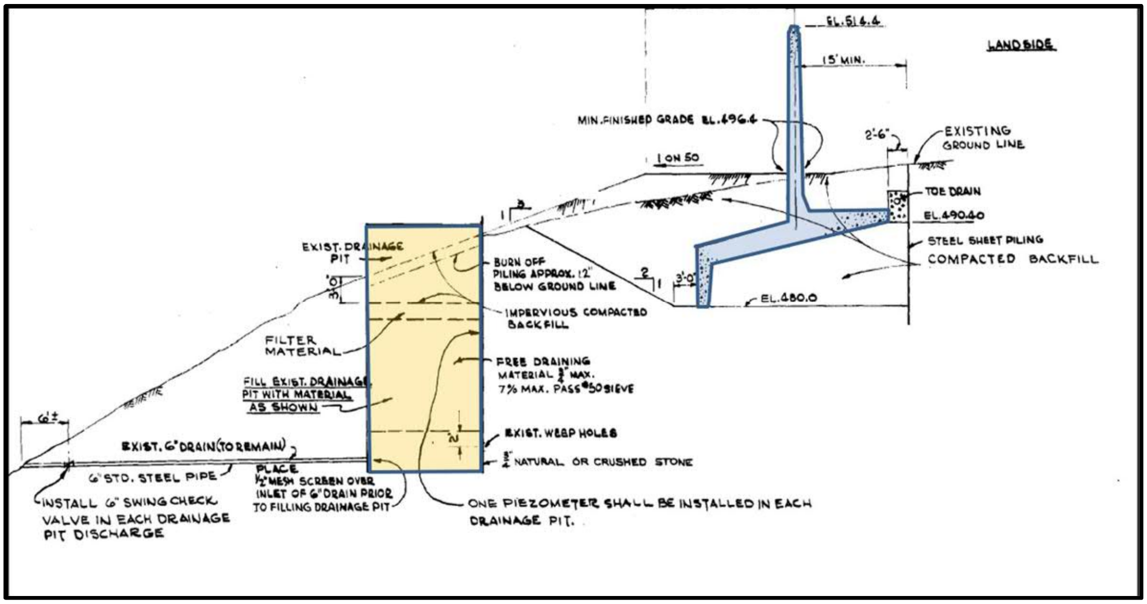

Figure 15. Section for Excavation & Backfill of T-Wall and Filling of Drainage Pits (USACE Louisville District)

PERFORMANCE OF THE COMPLETED REMEDIAL WORK

Despite the extensive investigations, multiple phases of remedial work and ultimate replacement of the failed monoliths, inspections of the floodwall since the 1980's have revealed that the new monoliths 41-46 constructed in 1964 have moved. The movements have been relative to each other, and relative to the adjacent original monoliths, and additionally, there is visual evidence of relative movement of all monoliths 31 to 54 (Figure 16). Although the current displacements of the floodwall monoliths are far lower in magnitude than the movements of the original monoliths witnessed in the pre-1964 time frame, it is important to note that at the time of this writing, the slope appears to be in a phase of active movement once again. A letter from USACE to the Covington Mayor's office dated 30 March 2020 stated in regard to the August 2019 inspection, "Slope instability was observed within 8 to 10 feet of the riverside face of the T-Wall near Station 175+00 *(author's note: this is the station for monolith 40)*. At the time of the inspection, the slope movement was approximately 4 inches down and away from the riverside face of the T-Wall and appeared to encroach over the base of the T-Wall foundation." About 110 feet away at Station 176+12, the scarp was approximately three feet high. Follow-up inspections in February 2020 and photos taken by an Unmanned Aerial Vehicle (UAV) in January 2021 (see Figure 17) indicated the slope has continued to move significantly more since the August 2019 inspection. Inclinometer casings have recently been installed near the bottom of the slope below the floodwall monoliths, and the entire area is now being monitored carefully with consideration for more remedial work in the near future. One press release from the City of Covington's website in July 2020 reported on the slow landslide movement and explained that one possible solution under consideration by the City was to install soil nails. The same article reported on the award of a contract for a geotechnical investigation of the site.

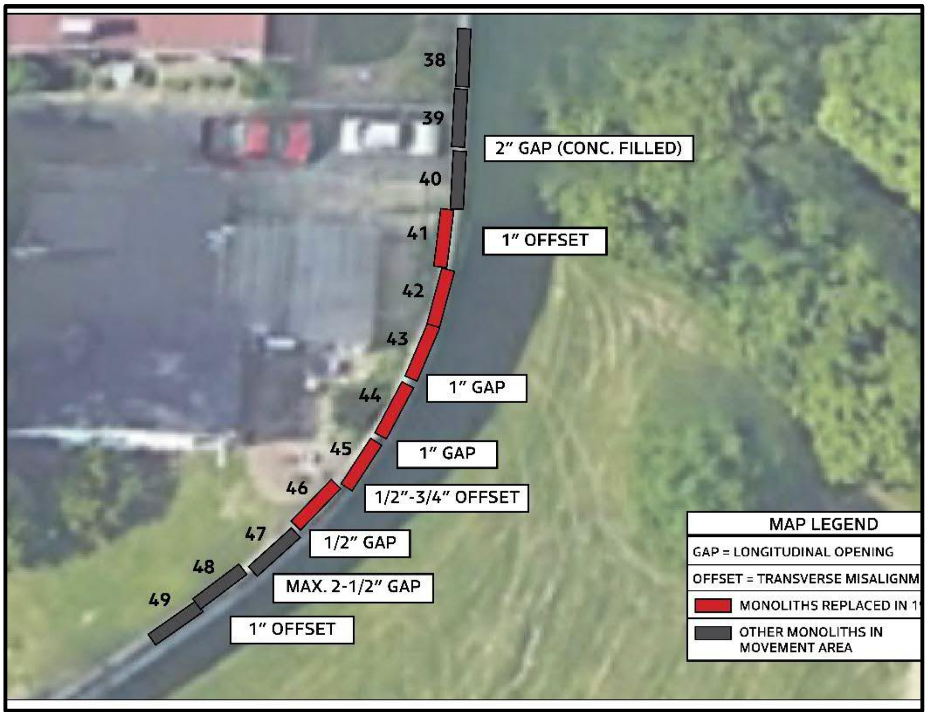

Figure 16: 2009 Relative Wall Movements

Figure 17: Slide scarp in January 2021 viewed from UAV over the riverside slope

CONCLUSIONS

This interesting case history offers us a glimpse into the past struggles of a government agency with a strong reputation of technical leadership and its attempt to understand a progressively worsening slope stability problem. Without the benefit of modern instrumentation such as down hole inclinometers, the engineers of the 1950's could only guess at the depth and extent of the slide mass that was producing such alarming damage to the levee and floodwall. It seems apparent that the original design engineers did not correctly assess the potential for slope instability, although the state of geotechnical practice at the time included rapid drawdown analysis. With the benefit of hindsight there are several important lessons that can be learned from this case history, including:

- More thorough site investigation and a detailed study of the site's geology might have foreshadowed the slope stability problems endemic to the site, and probably would have driven the designers to select a different plan for providing flood risk reduction in the neighborhood.
- Loading the top of a steep slope partially constructed of random fill can be expected to lead to slope movement over time.
- Construction sequencing is extremely important since the slide movement apparently was initiated by the unshored deep excavation for the sanitary sewer.
- A robust inclinometer program should have been initiated over 50 years ago, and possibly would have revealed the location, depth, and orientation of slide planes.
- Several significant efforts to mitigate the problem failed to fully arrest slope movement
- The current magnitudes of monolith movement do not indicate that any of the floodwall monoliths are highly likely to fail under flood loading. However, if movements continue, this conclusion may change.

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Drilled Displacement Elements for Liquefaction Mitigation

W. Morgan NeSmith, P.E., M. ASCE[1]

Abstract: Abstract The importance of seismic design considerations continues to increase in areas of the U.S. where, traditionally, they have not been considered. Liquefaction-induced settlement or structure movement due to lateral spread are two significant design challenges. In deep liquefiable sands (depths of 30 to 40 ft [9.1 to 12.2 m] and greater), traditional vibration or soil mixing techniques may prove to be financially and/or operationally inefficient. Drilled displacement (DD) systems that densify coarse-grained soils by mechanically displacing them laterally can be an efficient alternative in this scenario. This paper provides background on the development of DD tools in North America, the research and development of the ground improvement provided by DD tool installation, and the subsequent use of DD tools to install structural piles or ground improvement elements to mitigate potential liquefaction as a seismic hazard.

Introduction

The term "drilled displacement", for the purposes of this article, refers to the usage by the Deep Foundations Institute (DFI) Augered Cast-in-Place and Drilled Displacement (ACIP/DD) Pile Committee, which considers this a technique which results in a cast-in-place element or pile, installed by a single-pass, rotary drilling process. The term "pile" refers to structural deep foundations which are tied into the structure's foundation system and reinforced to resist the structure's compressive, tensile, and lateral loads. The term "elements" refers to non- or semi-structural elements which serve to improve the subsurface conditions to allow for the use of shallow foundation systems for support of the structure (and are not tied into the structure's shallow foundation system).

Several proprietary drilled displacement tools are available in North America (Figure 1) that use either pressure-grout placement or bottom-hole tremie concrete placement to form the pile once the tool has penetrated to the planned depth. The tool used in the examples of soil densification and at the example project presented in this article was an Augered Pressure Grouted Displacement (APGD) pile tool. A schematic of the pressure-grouted installed procedure for APGD piles and elements is shown in Figure 2.

The geotechnical benefits of these tools are most pronounced in coarse-grained soils where the mechanical (non-vibratory) displacement of these soils at or below the tool results in higher relatively densities of the soils around the tools than before installation. Most of the tools were developed in Europe and introduced to the North American market in the mid- to late-1990s to install higher capacity piles than non-displacement pile systems.

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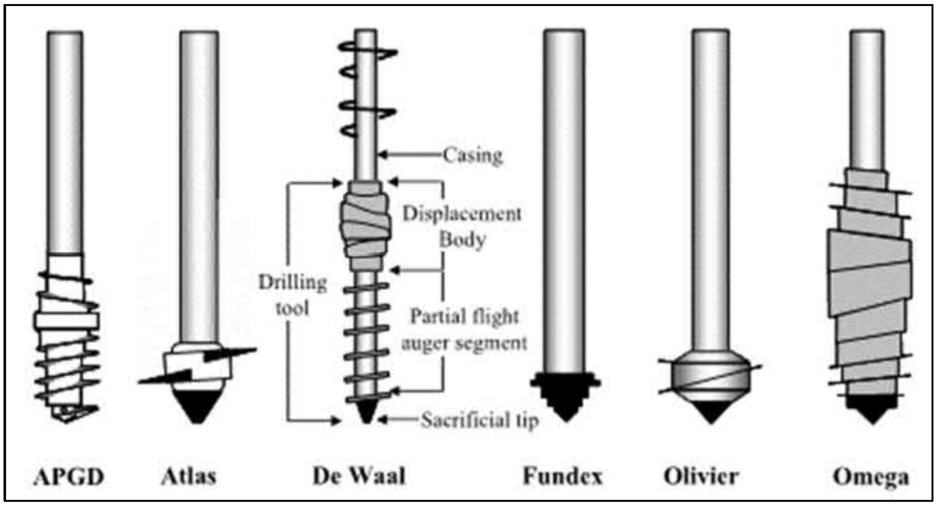

Fig. 1. Partial Example of DD Tools in North America (after Basu, et al, 2010)

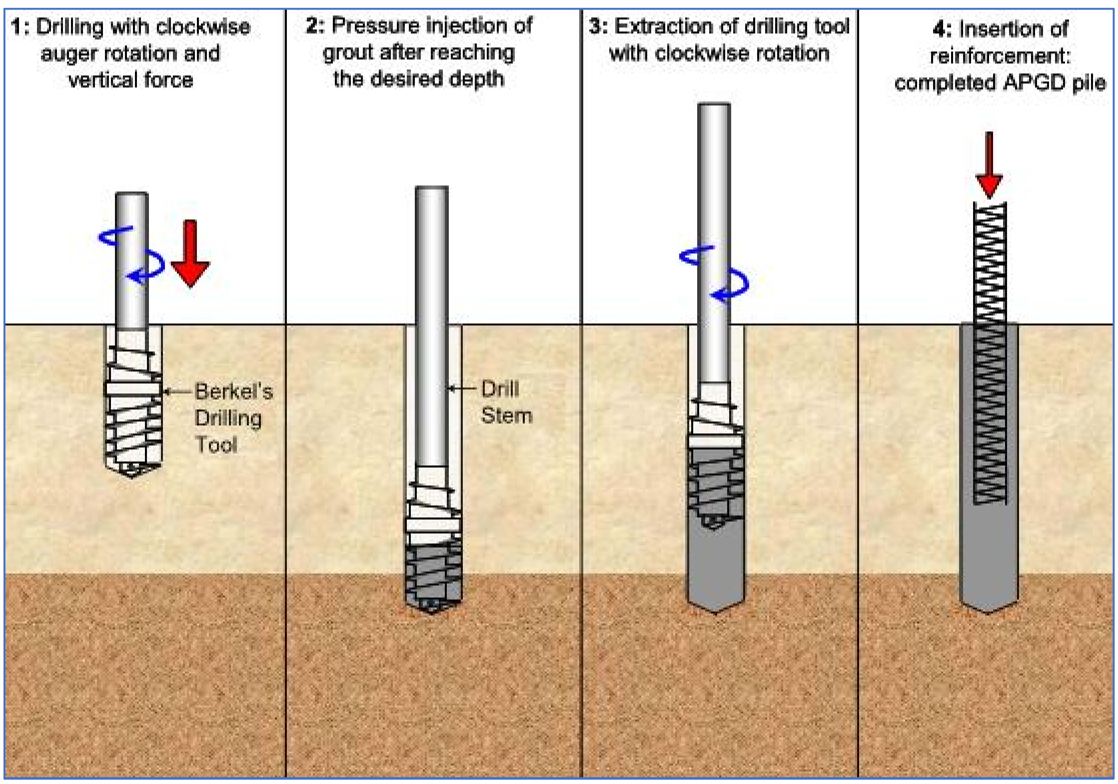

Fig. 2. Installation of DD Piles and CGEs (after Basu, et al, 2010)

Liquefaction Mitigation

DD piles/elements can mitigate the risk of liquefaction due to a seismic event by densifying coarse-grained subsurface soils at a project site. This is achieved due to the mechanical lateral displacement of the soils as described herein.

The geotechnical benefits of DD piles are most pronounced in coarse-grained soils where the displacement of these soils at or below the tool results in higher relatively densities of the soils around the tools than before installation.

An example of the amount of densification, as represented by the results of Cone Penetration Tests (CPTs) is presented in this section. Figure 3 is a schematic of the location of a set of CPTs that were performed near and then in-between a group of four 18-in diameter DD piles. Figure 4 shows the tip resistances measured by the CPTs performed between the DD piles and about 4.5-ft away from the group. The increase in CPT tip resistance after the installation of the four-element group is apparent in these plots.

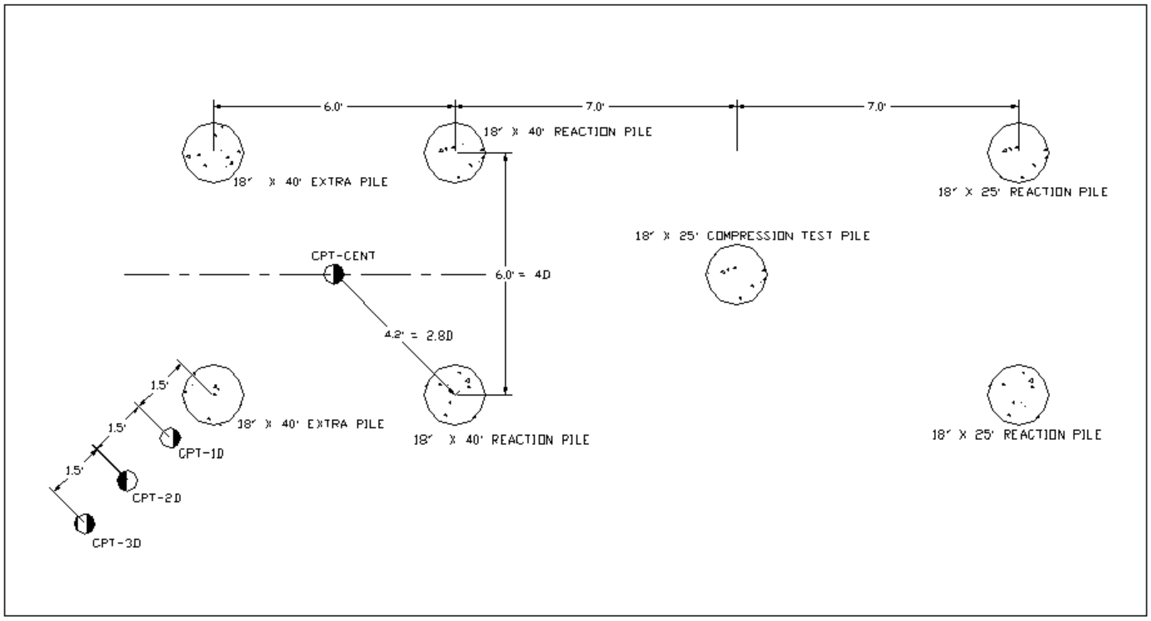

Fig. 3. Cone Penetration Tests Near/Between Installed DD Elements

Siegel, et al (2007a, 2007b and 2008) demonstrated how to develop databases of the level of increase in measured CPT tip resistance due to the installation of DD elements of various sizes and configurations by collecting pre- and post-installation CPT results. An example of the relationship between Area Replacement Ratio (the size and quantity of DD elements installed within a given area) and the expected increase in CPT tip resistance is shown in Figure 5.

Please note that this example is specific to results of CPTs performed after the installation of an APGD tool and may not accurately reflect the level of increase in CPT tip resistance for other displacement technologies (e.g. driven piles). Such a database can then be used to estimate the required size and spacing of DD elements to increase a soil's density, as indicated by CPT results, to the level necessary to resist liquefaction for a given design seismic event.

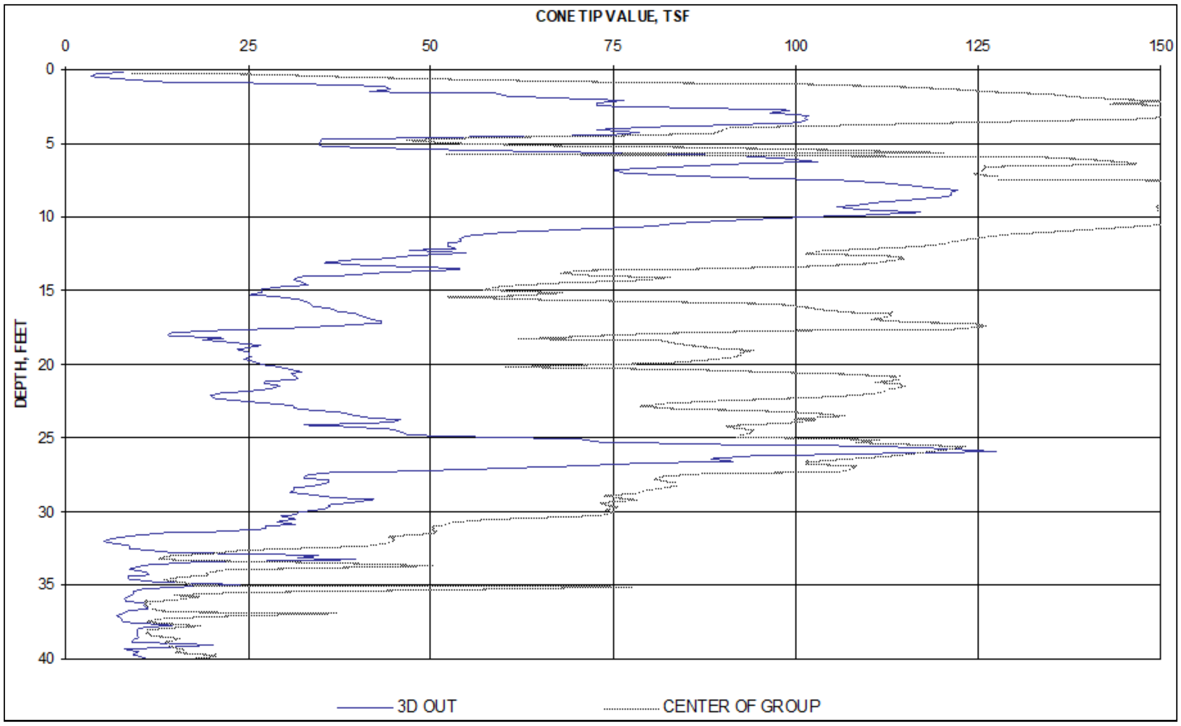

Fig. 4. CPT Results Outside and Inside of DD Element Group

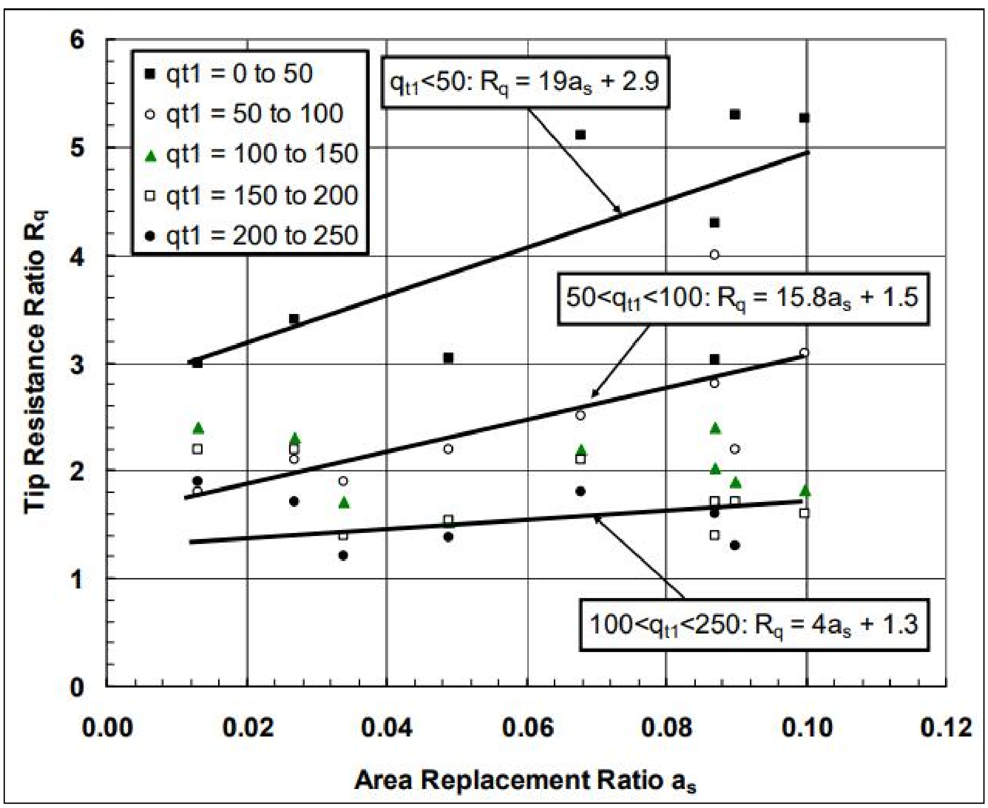

Fig. 5. CPT Tip Resistance Ratio vs DD Element Area Replacement Ratio

Installation Effort and Real Time Installation Data

The drilling platforms used to install DD piles/elements are typically configured with automated monitoring equipment (AME) to record, calculate and display various parameters during DD pile/element installation. During installation (advancing the tool into the ground), typical parameters recorded/calculated include time, depth, tool rotation rate and torque (as measured by the hydraulic fluid pressure driving the rotation of the turntables (NeSmith and NeSmith, 2006a). It is also possible to calculate additional parameters from those recorded, including an estimation of the energy expended by the drilling platform as the drilling too is advanced (aka Installation Effort (IE), NeSmith and NeSmith, 2006b).

Figure 6 shows an example plot of DD tool penetration rate, rotational fluid pressure (KDK pressure) and resulting calculated IE. These IE values are calculated at every 1-sec interval based on the KDK pressure and penetration rate recorded at that interval and provide a representation of soil stratigraphy, including density, like CPT tip resistance. This data can be displayed in the installation platform operator's cabin and transmitted wirelessly for monitoring by an inspector. The real-time display allows the inspector to observe soil stratigraphy during element installation and adjust the required DD element installation (i.e., densification) level as appropriate. indicated by CPT results, to the level necessary to resist liquefaction for a given design seismic event.

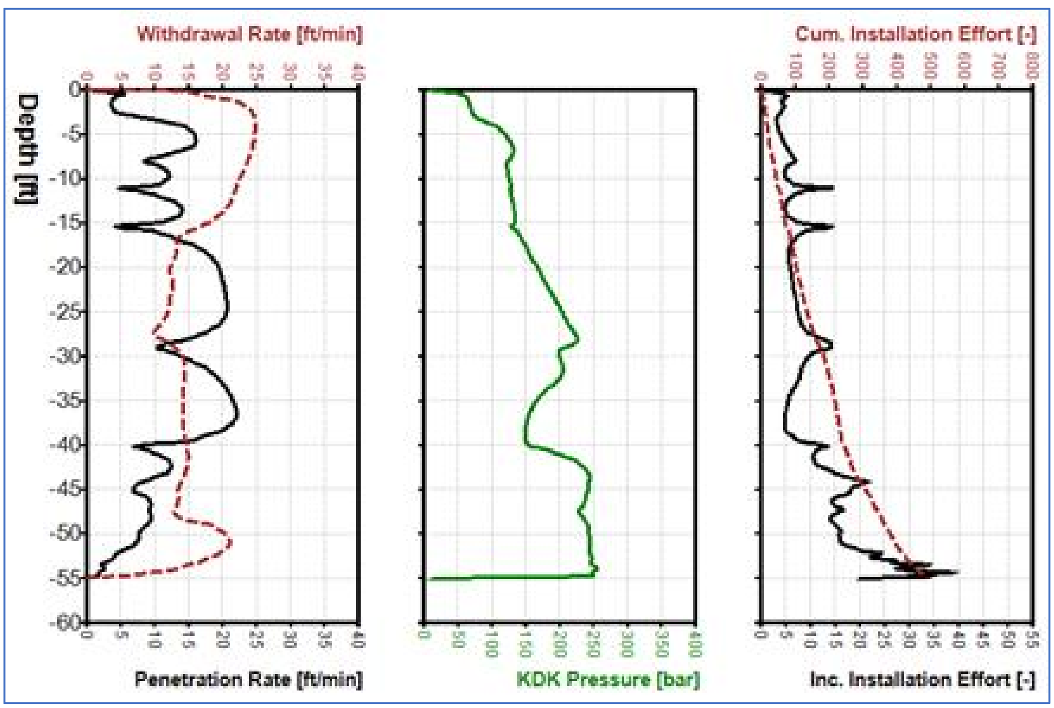

Fig. 6. Recorded and Calculated Parameters During DD Element Installation

TVA Power Facility – Memphis TN

The subject site was a new power generation facility in Memphis TN, near the New Madrid Seismic Zone. A separate liquefaction study for the site indicated that a magnitude 7.7 earthquake with a peak ground acceleration (PGA) of 0.55g should be considered in the final facility design. This PGA was obtained considering a 2% probability of exceedance in 50 years, considering the facility to be critical (i.e., must be operation post-seismic event).

Facilities included a large water-cooling facility and multiple stacks, generators, tanks, and ancillary facilities. Design bearing pressures ranged from 2500 to 4500 psf in the primary facilities and 1500 to 2000 psf in the ancillary facilities (Figure 7). In the stack and HRSG areas, there were also large lateral and uplift (overturning) loads that dictated structural pile support to resist these loads. The facilities were generally supported by mat foundations. Tanks were typically supported by ring footings with geogrid reinforced structural fill under the tank in the space between the footing.

An example preliminary CPT result is shown in Figure 8. Challenges to supporting the desired loads included settlement of the soft to firm clay in the upper 20-ft (along with small zones of similar soils from 20-ft to 50-ft depth) and settlement due to liquefaction (considering the design seismic event) of medium dense sands between 20-ft and 55-ft depth.

It was estimated that 14-in diameter DD elements could be installed as semi-structural elements on a 7-ft x 7-ft center-to-center triangular spacing under the majority of the foundations to (a) create a soil-grout block to transfer the design bearing load through the soft clay soils down to the lower sandy soils and (b) increase the density, as measured by post-installation CPTs, in any liquefiable sands to mitigate that risk. Under the stacks and HRSGs, it was estimated that 16-in diameter DD piles could be installed on a similar spacing to mitigate liquefaction but also to fully resist the design per-pile loads of up to 125 tons compression, 30 tons tension and 10 tons lateral.

During the early stages of CGE installation, a post-installation CPT program was conducted to verify an "improved" condition of the liquefiable sands using the 14-in elements as described above. A noted increase in the tip resistance can be seen in the post-installation CPT results (Figure 9). An analysis of the results, considering the seismic design parameters for the project, indicated that the liquefiable sands had been improved to a point where liquefaction was mitigated using this size element and spacing, resulting in CPT refusal levels of densification in the lower sands (early-stage elements were installed to a depth of about 55-ft below grade).

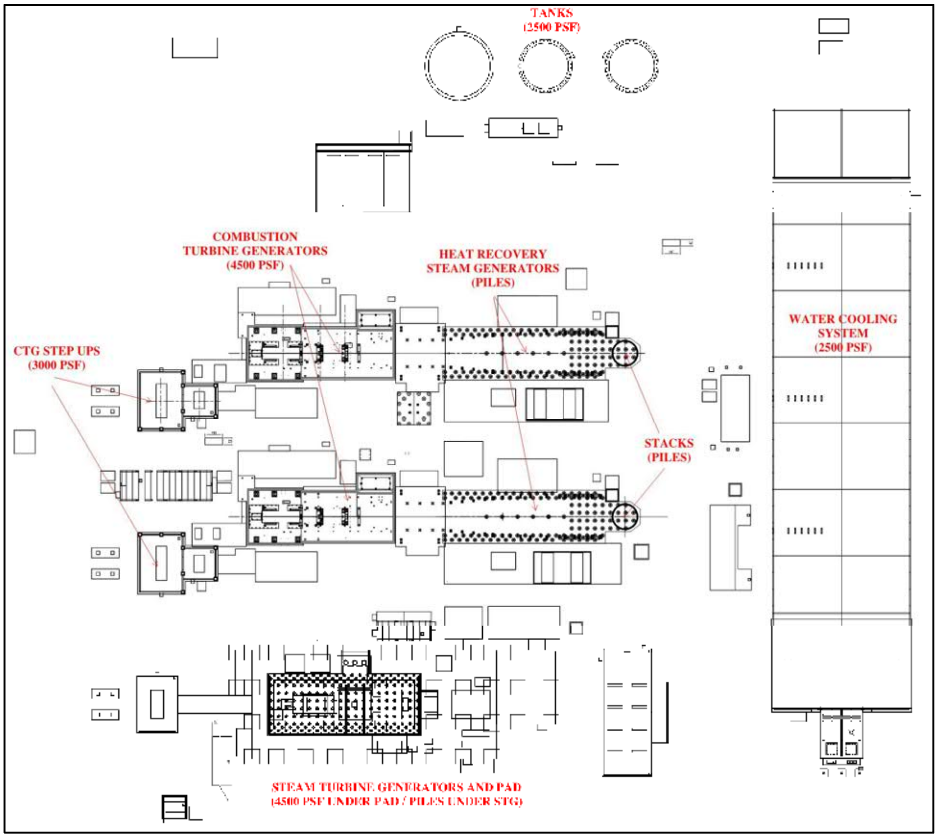

Fig. 7. General Facilities Layout with Bearing Pressures

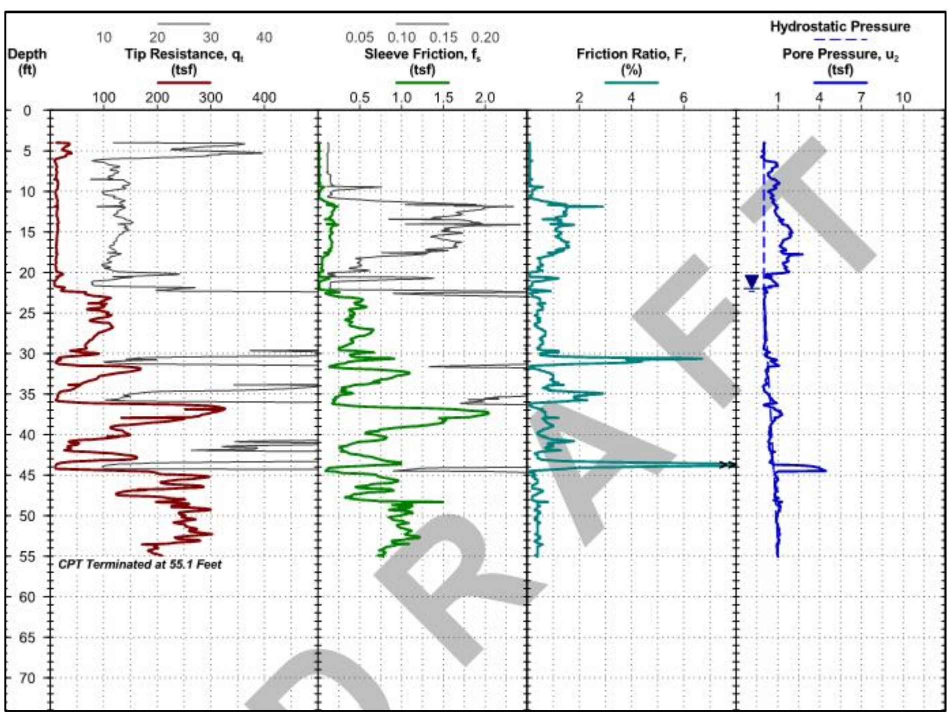

Fig. 8. Example CPT Result – Pre-installation Site Condition

Elements were typically installed to a minimum of 55-ft below grade under most structures. They were extended up to 65-ft when drilling resistances (as demonstrated by Installation Effort, IE) were encountered that indicated that the zone of medium dense, potentially liquefiable soils extended below 55-ft depth. Elements were typically cut-off 6-in below foundation level and covered with structural fill to the bottom of the mat level for each structure. However, the elements were reinforced with steel center-bars to increase ductility because of the lateral forces in the soil during the design seismic event. To obtain appropriate factors of safety for individual piles, the 16-in diameter DD piles were installed to depths of approximately 65-ft below grade in the HRSG and Stack areas and 70-ft below grade in the STG area, based on the results of the pile load test program for the project. These structural piles were reinforced to adequately resist the tension and uplift loads described above.

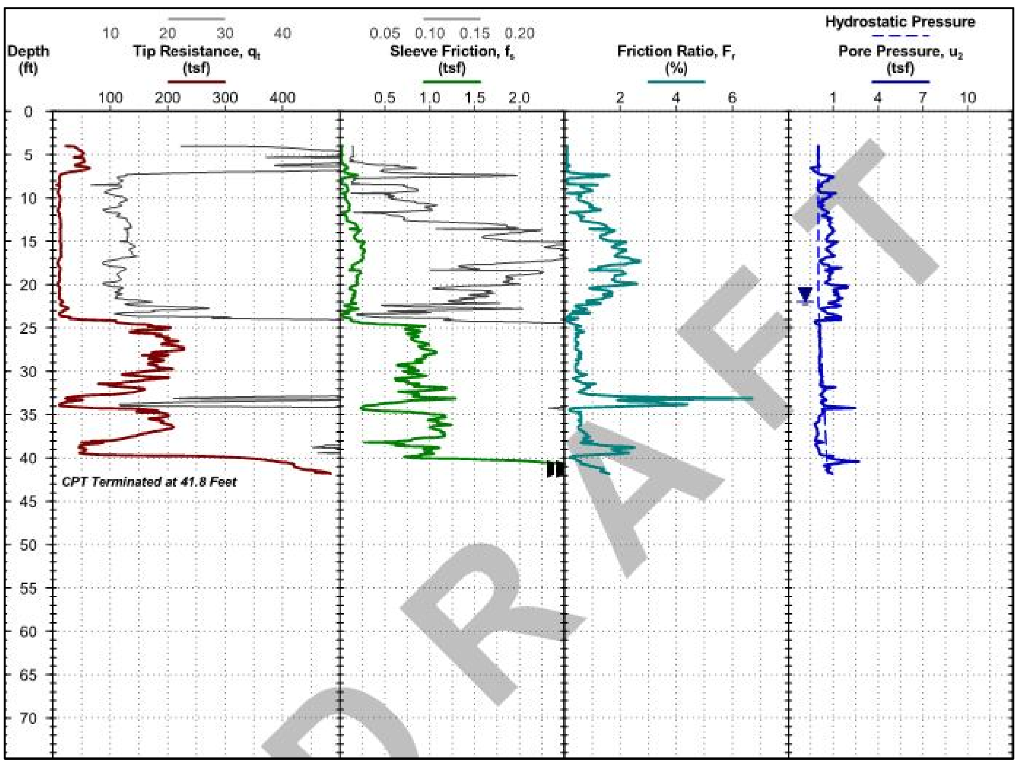

Fig. 9. Example CPT Result – Post-Improvement Site Condition

Conclusions and Moving Forward

The results of this project indicated that there is a measurable increase in the density of coarse-grained soils due to the installation of elements using drilled displacement tools and that this can be estimated by pre- and post-installation CPTs. It should be noted that post-installation testing is typically performed in the center of the element group, i.e., the point where improvement will be the lowest. There is some preliminary evidence that, over time, the density increase between elements becomes an average of this lowest measured density and the higher increases measured closer to the individual elements in the group.

As more information in this regard becomes available, designs should become more efficient, as lower target post-installation CPT results could be for immediate post-installation testing, with consideration for the averaging of soil density between elements over time. The required depth of installation of DD piles and elements to mitigate liquefaction can be varied, in real-time, across a project site, by monitoring the energy expended by the installation platform during element/pile installations.

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CASE STUDY FOR EXPLORATORY DRILLING AND GROUTING IN KARST AT ROUGH RIVER DAM

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Abstract: Rough River Dam is a high hazard embankment dam owned and operated by the US Army Corps of Engineers (USACE) on the Rough River in west-central Kentucky. The dam is operated in conjunction with 3 other dams in the Green River Basin to reduce flood impacts in South Central Kentucky and the Ohio River. Two high pool events in 2007 and 2011 triggered adverse changes in instrumentation trends related to the karstic foundation below the dam near the outlet conduit. A series of automated piezometers were installed in 2012 which confirmed changes in piezometric connectivity within the karst foundation. In response to these conditions, the project was approved for implementation of an exploratory grouting program intended to better define current foundation conditions, the extent and severity of karst, and to inform the design for a future cutoff wall. A construction contract to install two partial grout lines along the dam crest was awarded in 2015. The contract was modified to include completion of both grout lines after grout was found exiting into the stilling basin. The drilling and grouting program was completed in 2017 requiring the installation of 328 grout holes, 40,000 linear feet of drilling and a total of 212,763 gallons of grout. The drilling and grouting at Rough River Dam represented the culmination of several key drilling and grouting lessons learned from USACE and the grouting industry. The project required the use of down-hole instrumented packers for all water pressure tests and grout injections. Instrumentation readings for the dam were monitored in real time for over 100 instruments. The data collected during the drilling and grouting program and observations of subsequent instrumentation trends confirmed the need for a cut-off wall to permanently mitigate the risk posed from the karst foundation. This paper provides an overview of the Rough River Dam Safety Modification Project including key construction history, an overview of the Phase IB Exploratory Drilling and Grouting Project, and a summary of the upcoming Phase II Project.

Project Introduction

Rough River Dam is located in Falls of Rough, Kentucky on the Rough River approximately 60 miles southwest of Louisville, KY and approximately 120 miles north of Nashville, TN. The project is an earthen flood risk management dam working in conjunction with 3 other dams in the Green River Basin to reduce flood impacts locally and along the Ohio River. A summary of the project layout and terminology is provided in Figure 1. In 2012, a Dam Safety Modification Report (DSMR) was completed which identified four internal erosion related failure modes across the dam foundation with a fifth located along the outlet conduit as shown on Figure 2. The DSMR recommended implementation of an exploratory grouting program and installation of a future cutoff wall to reduce risks associated with the dam. The exploratory drilling and grouting project, referred to as Rough River Dam Safety Modification Phase IB, was completed in accordance with the DSMR recommendations between 2015 and 2017. This paper will focus on the planning and implementation of the foundation grouting at Rough River Dam.

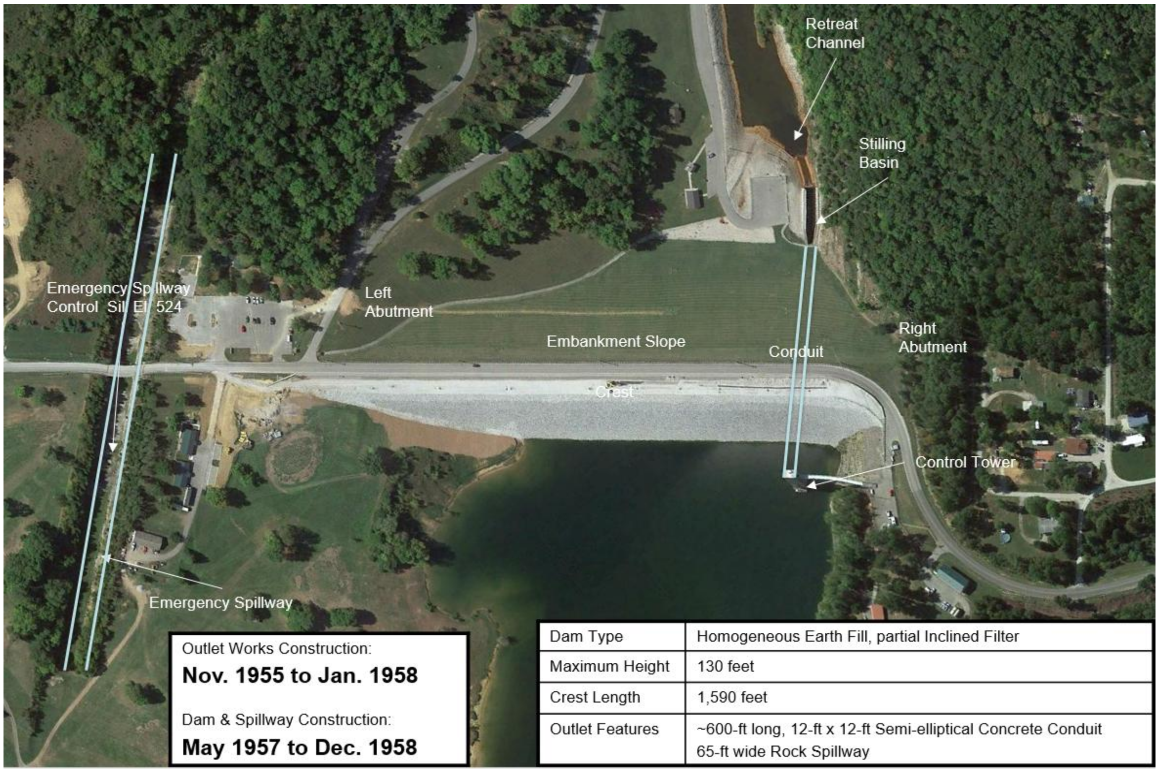

Fig. 1. Rough River Dam Project Layout

Project Geology

Rough River Dam is located on the northwestern portion of the Mississippian Plateau. The Rough River valley is a relatively narrow entrenchment incised into bedrock that generally dips to the southwest at a rate of about 85 feet per mile. Where exposed to weathering, both the Beech Creek Limestone (BCLS) and Haney Limestone are highly karstic with pinnacled upper rock contact elevations and continuous solution features along joints and bedding planes. Numerous interconnected karstic pathways exist in each formation. The Haney Limestone and BCLS are present on both sides of the incised valley but fully eroded at the valley center. The Haney Limestone ranges from 50-60 feet thick and is known to contain massive karst features over 50 feet in width. The BCLS is noted to be a ledge former and has been observed to range from 10 to 15 feet in thickness at the project. See Figure 2 for a profile of the dam embankment and dam foundation.

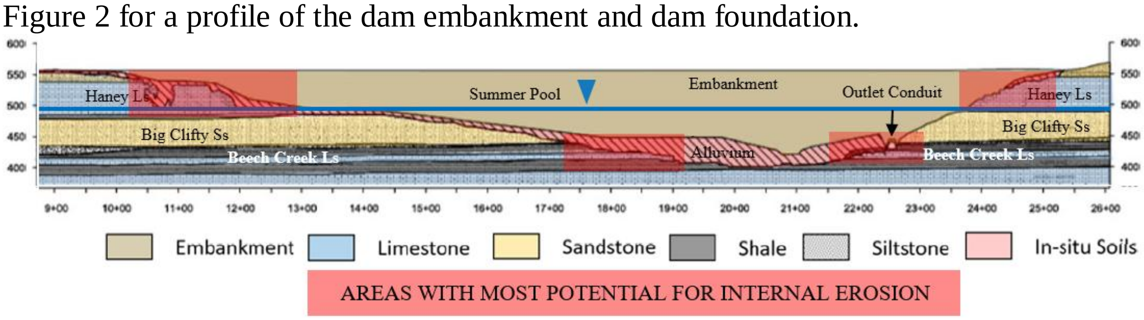

Fig. 2. Rough River Dam Stratigraphy Profile

Construction History

Construction of the dam began in November 1955 and was completed in January 1958. The dam was placed into service in 1960. The dam was constructed with no core trench, foundation grouting, or dental treatment across the historic river valley between the conduit and the lower left abutment with the exception of a narrow inspection trench excavated into the alluvium. A summary of foundation treatment is shown in Figure 3 below.

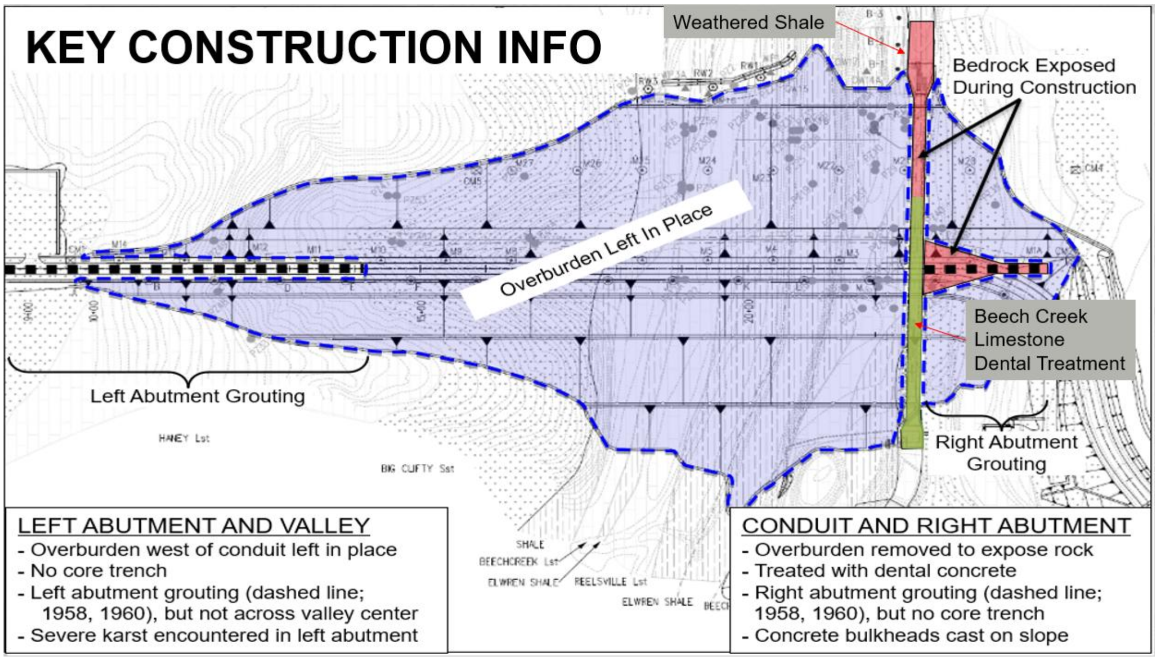

Fig. 3. Foundation Treatment Summary

Outlet Works Construction

The left side of the conduit trench was excavated at a 2H:1V slope into native alluvium soils (overburden); whereas 20 to 30 feet of rock excavation along the right side of the conduit trench was accomplished via blasting to achieve the 1H:4V rock slope. The conduit alignment generally followed the rock outcrop along the base of the right abutment. The tower and approximately two-thirds of the upstream (U/S) portion of the conduit were founded on the BCLS. The remaining downstream (D/S) portion of the conduit and stilling basin were founded on shale. During conduit foundation mapping in the BCLS, linear karst features were documented within the conduit footprint and dentally treated. These features were solutioned joints large enough a person could enter, ran sub-parallel to the alignment, and were often observed exiting the conduit excavation below the overburden on the valley side (Figure 4). Additional excavations were not performed to expose encountered karst features that exited the excavation or were otherwise covered by shale.

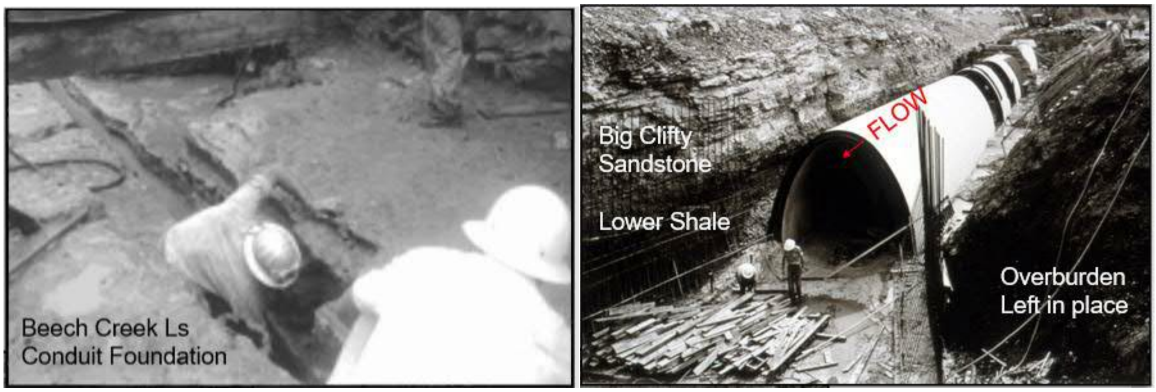

Fig. 4. Conduit Excavation in the BCLS and Encountered Karst

Embankment Construction

The embankment was placed on top of naturally deposited alluvial material across the historic valley that is referred to as "overburden" or "foundation soils" interchangeably in the construction documents. The dam foundation soils vary on each side of the historic river channel. Left of the historic river channel the dam was founded on 40 to 50 feet of lean clay, silts, and interfingered sands with sandy clays. To the right of the historic channel the foundation material is up to 30 feet thick and is generally composed of poorly draining cohesive material. The original dam designers were concerned with embankment stability associated with the poorly draining soils on the right side of the valley. The design incorporated a 2 feet thick filter blanket constructed above the poorly draining material extending to within 50 feet of the dam centerline (C/L) to address these concerns (Figure 5). Downstream of the dam C/L an inclined filter and 2 feet thick filter blanket was draped over the foundation soils across the D/S toe of the dam and was also extended to within 50 feet of the dam C/L. A series of 368 sand drains were advanced in both blankets between the historic channel and outlet conduit (141 D/S and 227 U/S). The sand drains are 12 inches in diameter and were advanced to the top of bedrock on 13-foot centers.

The sand drains and U/S blanket allow reservoir pressures to be transmitted to within 50 feet of the dam C/L which can be directly injected into the karstic BCLS. The dam cycles between high reservoir levels and high tailwater levels as part of routine operations. The vertical drains function as injection wells rapidly transmitting cyclical high pressure gradients into the BCLS from both U/S and D/S influences. Once inside the karst network these pressures can be transmitted directly to foundation soils in direct contact with karst features. Head loss across the dam foundation is controlled by the naturally deposited alluvium soils above the BCLS between the blanket drains that is in direct contact with these karst features.

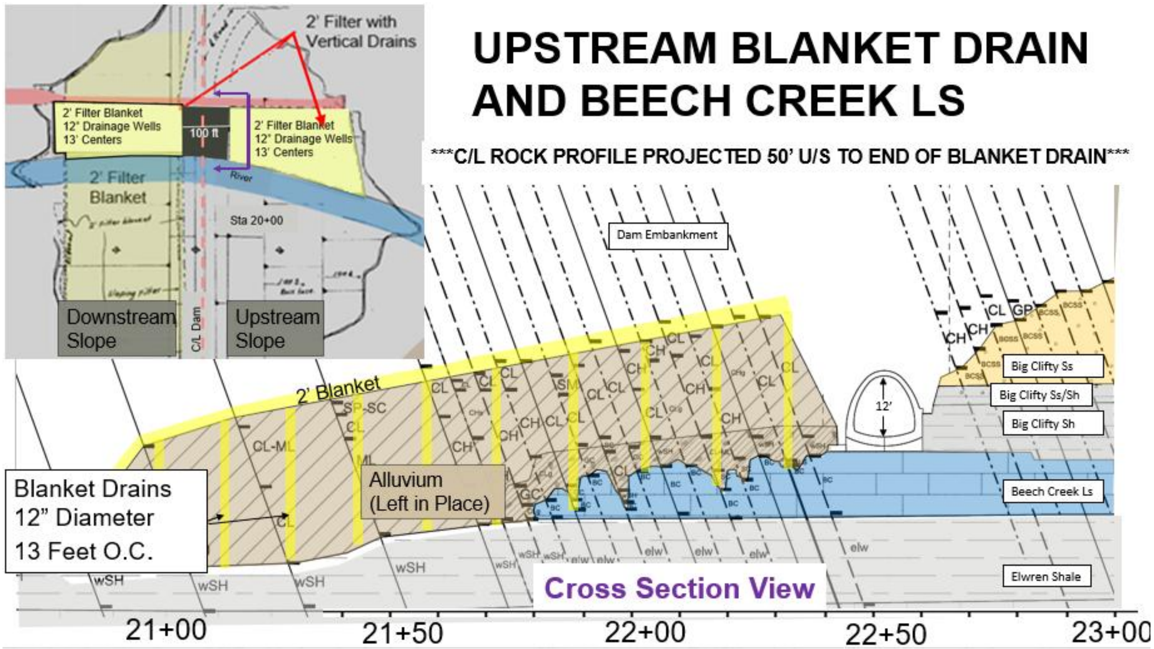

Fig. 5. Interaction of Upstream Drainage Blanket and Karst

Operational History

Several historic repairs and occurrences over the years have revealed key information about the dam foundation.

1. In 1984, The stilling basin floor slab was partially removed to repair undermining and scour of the shale below the stilling basin, at the time attributed to poor hydraulic performance. The excavation exposed karst features below the shale that were filled with concrete. (Figure 6 – Photo 1).
2. In 2003, a sinkhole formed on the D/S dam slope. The sinkhole was attributed to filter incapability between the dam embankment and the rock toe. In order to provide separation and prevent further migration of material, a D/S graded filter was installed across the rock toe in combination with a cement-bentonite slurry wall constructed across the deepest portion of river valley.
3. In 2007 during a stilling basin modification, the karstic BCLS was excavated below the protective shale layer. Upon inspection, open, clay filled, and partially clay filled karstic features were encountered with some saturated soils flowing from the exposures upon excavation. Solution features were observed to occur in multiple orientations along regional joint sets and valley stress relief fractures (Figure 6 – Photos 2 and 5). While the stilling basin was dewatered, heavy rains raised the reservoir by 22 feet over summer pool. This rainfall event resulted in gradients across the dam foundation never experienced during previous flood events, and is referred to as the "High Head Event". The exposed karst received dental treatment and construction was expedited so releases could be safely made to lower the reservoir.
4. In 2008, pin boils were noted outside of the stilling basin near the repaired area.

5. In May of 2011, the project reached a record pool exceeding summer pool by 32.4 feet. The left training wall experienced excessive movement during the high releases and nearly failed. In June 2011, the stilling basin was dewatered to assess damage. Artesian flow was observed exiting from a stilling basin weep hole (Figure 6 – Photos 4 and 6). This condition was not noted during previous dewatering events.
6. In November of 2012, during the stilling basin training wall replacement, the artesian condition was again noted flowing from the weep hole. Caissons were advanced into the BCLS to buttress the left training wall. While advancing the caissons, additional voids and karst conduits were encountered within the BCLS just left of the stilling basin. Recovered caisson cores exhibited severe solutioning and a down-hole camera video indicated high water flow entering the excavation from multiple orientations (Figure 6 – Photo 3).
7. In 2007 and 2012, a severe drought prevented the reservoir from obtaining summer pool which limited releases from the reservoir and created a low tailwater condition.

Fig. 6. Photo 1 – 1984 Repair, Photo 2 – 2007 Repair, Photo 3 – 2012 Caisson Excavation in BCLS, Photo 4 – 2012 Artesian Flow, Photo 5 – 2007 BCLS Karst, Typical, Photo 6 – Water Sample from Weep Hole with Freshwater Isopods.

Historic Instrumentation Observations

A cross section of the dam near Station 22+30 is shown in Figure 7 to clarify the relationship of key instruments and relevant features for the dam. PZ-50 and PZ-51 are installed in the upstream blanket to monitor reservoir influence acting on the upstream sand blanket and drains. PZ-24 and PZ-36 are located to the left of the conduit between the dam C/L and the U/S blanket drain, approximately 32 feet apart and tipped in the alluvial foundation. A comparison of the Elevation vs. Time History Plot for the 2007 and 2011 events for PZ-24 and PZ-36 is shown in Figure 8. After the 2007 high head event, PZ-24 entered a period of steady decline while PZ-36 began to increase. In 2011, the record pool event occurred which once again resulted in very high gradients acting on the dam foundation. After this event, both PZ-24 and PZ-36 had marked declines in piezometric level while PZ-50 and PZ-51 maintained historic instrumentation levels reading a few feet below the reservoir.

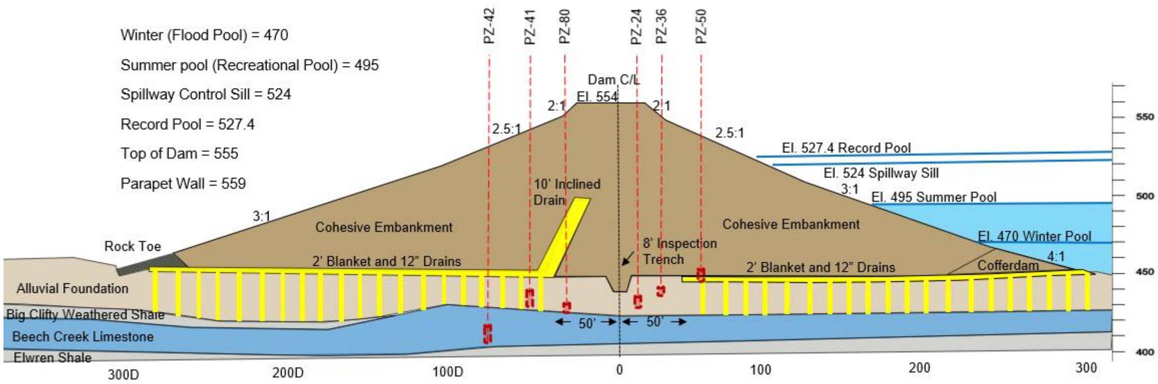

Fig. 7. Instrumentation Section for Rough River Dam near Station 22+30

Instruments located D/S of the dam C/L tipped in the BCLS also exhibited instrumentation trend changes after the 2011 record pool event. PZ-40 and PZ-42 are located 70 feet D/S on opposite sides of the conduit tipped in the BCLS (Figure 9). Prior to the record pool, piezometric potential indicated that water from the valley was seeking the abutment. After the record pool event this trend reversed and piezometric potential shifted from the abutment towards the valley. This was interpreted to indicate increased reservoir influence was acting on the dam foundation and right abutment as karstic connections in the foundation demonstrated increased permeability.

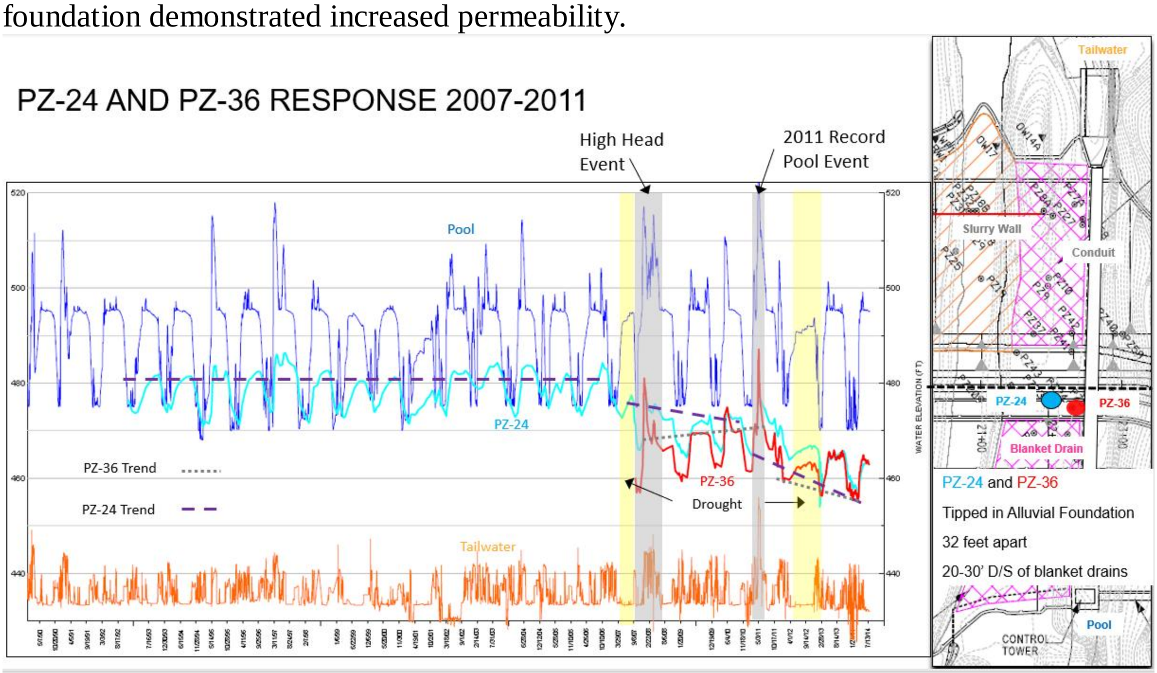

Fig. 8. Changes in PZ-24 and PZ-36 Between 2007 and 2011.

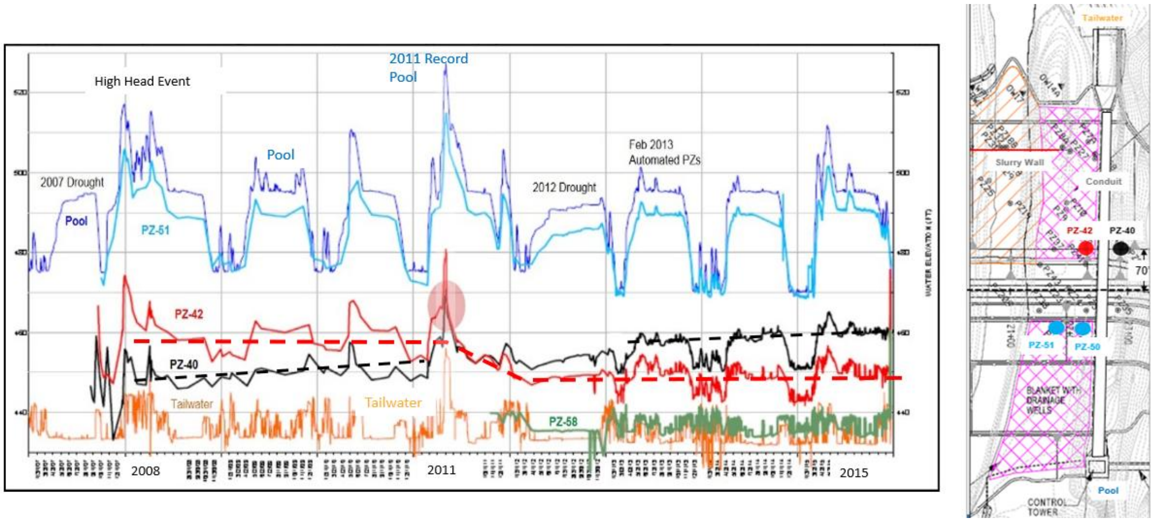

Fig. 9. Changes in PZ-40 and PZ-42 Instrumentation Trends between 2007 and 2015.

In 2012, an Automated Data Acquisition System (ADAS) was installed at Rough River Dam. During the 2012 stilling basin dewatering, significant and instantaneous drops in the PZ levels occurred along the entire length of the conduit alignment. Most notable are similar system responses between instrumentation tipped in the alluvial foundation and instruments tipped in the BCLS. It should also be noted that instruments tipped in the upstream sand blanket (PZ-50 and PZ-51) reacted to the dewatering. This event confirmed the suspected connection between the karst foundation and the upstream blanket drains. Between 2011 and 2015, the decline of U/S foundation instruments such as PZ-36 and PZ-24 became more rapid while D/S instruments in the BCLS began to climb and react more to reservoir fluctuations.

Exploratory Drilling and Grouting Scope

The DSMR recommended executing a series of contracts to relocate the existing state highway (Phase 1A), to perform exploratory foundation grouting (Phase IB), and for slurry control grouting and installation of a cutoff wall (Phase 2). The scope of a future Phase 2 project was contingent upon the results of the Phase IB grouting. The DSMR included plans for the future cutoff wall to be installed across the dam foundation and around the existing conduit with additional grouting from inside the conduit.

The goal of the Phase IB – Exploratory Drilling and Grouting Project was to explore and evaluate subsurface conditions, to determine if the cutoff wall was required, and to provide additional information to support cutoff wall design, if needed. Phase 1B was awarded to Advanced Construction Techniques, Inc. ("ACT") in April 2015. All on-site drilling conformed to USACE Engineering Regulation (ER) 1110-1-1807, Drilling in Earth Embankment Dams and Levees. Phase IB grouting also complied with the requirements of USACE Engineering Manual (EM) 1110-2-3506, *Grouting Technology* and was designed to effectively treat the weathered rock interface zone while minimizing the risk of inducing damage to the dam embankment and foundation .

The base contract required two partial grout lines totally 1,670 feet in length to be placed parallel and offset from the dam C/L by 7.5 feet U/S (U-Line) and 10 feet D/S (D-Line) respectfully. Holes were battered at 20 degrees toward each abutment crossing each other in the historic valley area. Primary holes were spaced at 20 feet on center. Critical areas where the embankment was in contact with the karstic limestone were split-spaced down to tertiary holes (5 feet on center). The Contractor was required to perform optical and acoustic televiewer (OATV) surveying for primary boreholes and verification boreholes. Each grout stage was water pressure tested and grouted using balanced, stable grout mixes. The project specifications required the use of instrumented packers for all water pressure testing and grout injection. Water pressure testing and grouted activities were monitored by an Automated Grouting Monitoring and Data Collection System to provide real-time data collection and reporting.

During construction of the base contract, it was determined that karstic foundation conditions warranted the completion of both partial grout lines (Figure 10). The contract was modified to shift focus from exploratory grouting to production grouting intended for slurry control in advance of the future cutoff wall to be constructed in Phase 2. In the drilling and grouting modification, all holes were rock drilled instead of cored and only limited O/ATV surveying was utilized.

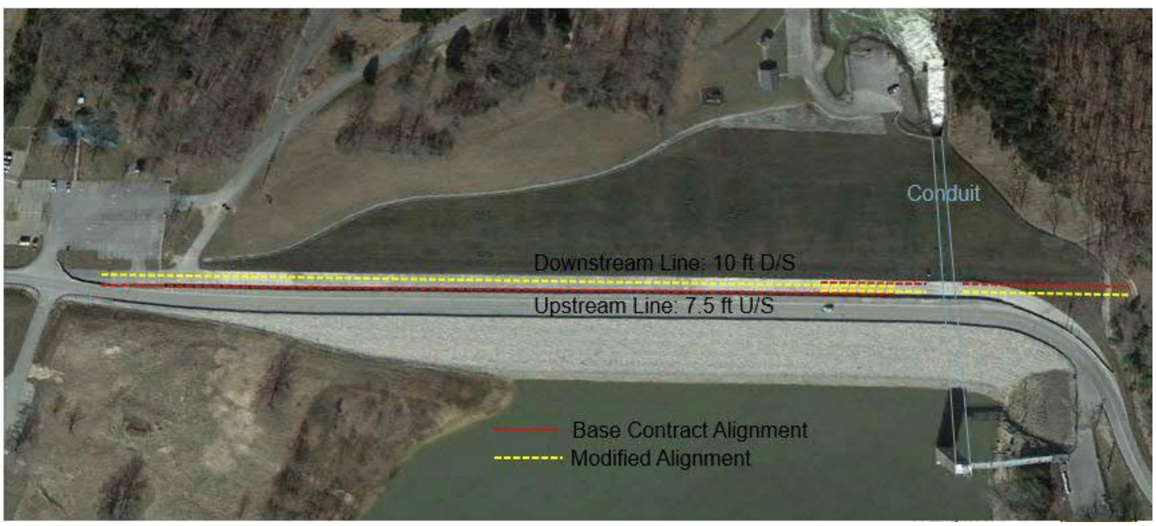

Fig. 10. Rough River Dam Base and Modified Grout Lines

Drilling and Grouting Technical Approach

The biggest challenge associated with drilling and grouting in karst below a high hazard dam is ensuring the bedrock foundation is isolated from the embankment. To accomplish this, the project was subdivided into 6 zones to isolate the embankment based on the primary rock formation in contact with embankment soils (Figure 11). The zones were broken up as follows: Zone 1 - left Haney Limestone, Zone 2 - left Big Clifty Sandstone and Shale, Zone 3- left BCLS, Zone 4 – historic river valley, Zone 5 – right BCLS and Conduit, Zone 6- Upper right abutment. The upper 15 feet of bedrock was required to be fully isolated from the embankment for all primary holes in a given zone before additional downhole drilling was permitted. Once the primary holes for a zone were completed, then the same procedure was applied to secondary holes and then to tertiary holes as work in a zone progressed until the entire zone was fully isolated from the embankment. Water pressure testing and grouting occurred in prescribed stages in bedrock ranging from 15-30 feet. Theoretical grouting pressures for each stage were calcluated at to the midpoint of the stage based on ½ psi/ft of soil and 1 psi/foot rock. Grouting utilized balanced, stable grouts which were thickened as appropriate to allow the target pressure to be achieved. The refusal criteria for each stage was defined as maintaining a flow of less than 1 gallon per minute held for 10-minutes at the maximum pressure specified for the grout stage. Refusal for gravity grout stages was defined as maintaining equivalent gravity grout pressure for the stage as measured from the surface elevation with 0 grout take for a period of one hour.

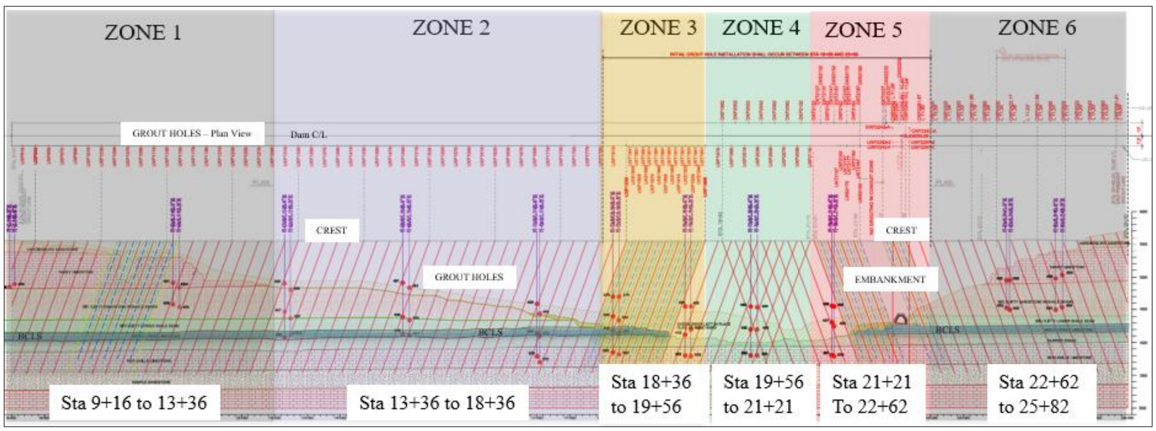

Fig. 11. Grouting Zones Across the Dam Profile (Base Contract Boreholes Shown).

Embankment Drilling

The dam embankment and underlying overburden were drilled using resonant sonic drills with a primary 6-inch steel casing advance first followed by an outer 7-inch override casing. The outer casing protected the embankment and foundation soils while the inner casing was removed and samples were extruded. This process was repeated until the top of bedrock was encountered. Both the inner and outer casing were advanced to form a minimum 4-foot socket into competent bedrock.

Installation of Casing and Rock Socket Treatment

The temporary steel casing installed from the roto-sonic drilling was required to remain in place until the permanent casing was grouted into the borehole. The permanent casing consisted of a Schedule 80 PVC multiple-port sleeve pipe (MPSP) that was inserted inside the sonic drill casing to the bottom of the borehole socket. An inflatable geotextile barrier bag was attached to the MPSP using double punch lock clamps at each end of the bag. The barrier bag was used to isolate the embankment at the rock socket. The MPSP was trimmed so that the barrier bag was centered at the top of rock elevation with sufficient additional ports below the bag, located at the bag, and above the bag to accommodate grouting the MPSP into place. The barrier bag was inflated and held to the lesser pressure of 35 psi or the calculated hydrofracture pressure for a period of 30 minutes to ensure an effective barrier was formed at the top of the rock socket. The annular space around the MPSP pipe was then backfilled above the barrier bag in stages to not exceed 90% of the theoretical hydrofracture pressure. After backfilling, the steel casing was pulled and the annular space around the MPSP was topped off with grout. After the MPSP pipe is backfilled, the rock socket interface below the barrier bag was grouted through open ports in the MPSP using gravity pressures.

15-Foot Gravity Grouting Zone

Once the MPSP and lower socket were treated and reached strength thresholds, rock drilling was able to commence. Grout hole drilling in rock was performed with water-actuated, down-the-hole hammers and standard rotary diamond coring drills inserted through the MPSP. The rock socket was then redrilled along with an additional 15 feet of rock below the MPSP. The hole was then washed, surveyed using O/ATV downhole equipment, and gravity grouted.

Rock Drilling and Pressure Grouting

The 15-foot gravity stage required mandatory downstage grouting for all grout holes in a specific zone before drilling and pressure grouting at deeper depths were allowed to proceed. The required downstage and the remainder of the grout hole was typically drilled all at once. The grout hole was then upstage grouted in prescribed stages ranging from 15-30 feet using a single packer or double packer assembly as required for water pressure testing and grouting. All water pressure testing and grouting for the program was performed with the use of the instrumented packer, real time monitoring of grouting parameters, and real time monitoring of geotechnical instrumentation. All primary holes in each zone were completed in this manner, followed by secondary holes, then tertiary holes.

Required Use of Instrumented Packers

Rough River was one of the first USACE projects to require the use of an instrumented packer based on lesson’s learned from Wolf Creek Dam. During the Exploratory Drilling and Grouting Program at Rough River Dam, proprietary instrumented packers (IntelliPacker) developed by the contractor, ACT, were used to monitor pressures at the point of injection during all injections including barrier bag inflation, casing annulus grouting, water pressure testing, and rock grouting. Advantages of using a properly calibrated instrumetned packer include:

1) Effective pressure caclulations are determed at the point of injection and not estimated from calculating dynamic line losses at the header gauge. This saves a considerable amount of time analyzing multiple grout placements at various distances from the grouting plant.
2) Results in more effective grouting. Effective grouting pressures as measured from the instrumented packer were slightly lower than calculations from the header gauge. This is attributed to conservatism in the dynamic line loss calculations. The instrumented packer reduces this uncertainty and allows for more effetive grouting.
3) Cost and time savings for gravity grouting and subsequent redrilling as equivalent gravity pressure could be applied using packers rather than filling the entire borehole with grout.

4) The instrumented packer is able to detect poor packer seals and grout blow-by in real-time. This translates into reduced risk to the dam embankment and foundation for hydrofracturing.

Real-Time Access to Data Records

During the performance of the exploratory grouting program, an integrated electronic records system was used to manage the flow of data as it was collected and interpreted and to manage and automate the submittal of completed records. Data from field drilling logs, real time water pressure testing, and grouting results were collected, organized, and made available to USACE via FTP site by ACT's proprietary data management system. All data obtained through the drilling and grouting program was made available via a real time geographic information system (GIS) interface.

Role of ADAS System During Drilling and Grouting

The previously installed ADAS system was critical for effective real time monitoring of the embankment, foundation soils, and bedrock during water pressure testing and grouting activities. Instrumentation on the dam includes 111 automated instruments including 58 nested vibrating wire transducers, 11 manual read PZ's, along with other movement monuments, carriage bolts, 3 seismic instruments, and 3 relief wells. Several critical piezometers were monitored during operations which resulted in halted drilling and grouting operations. Automated reports were developed detailing the instrumentation responses during production and were used to further the understanding of the overall karst connectivity across the site.

Phase IB Project Completion

The project was completed in April of 2017 having successfully installed a total of 308 production grout holes and 20 verification holes. These holes required 32,422 linear feet of overburden soil drilling, 7,477 linear feet of rock coring, 26,058 linear feet of percussion rock drilling, and a total of 212,763 gallons of grout. The grouting program was considered successful as a model state of the art grouting program. The results of Phase IB grouting were applied to the cutoff wall design and resulted in refinement of the cutoff wall extents based on grouting and water pressure test results.

Significant Karst Findings

BCLS - Unfiltered Exit Into the Stilling Basin

While grouting Borehole DXS2192 near the conduit, return fluid was lost while coring through the rock socket treatment zone. Per protocol, this triggered mandatory downstage grouting at gravity pressure to backfill the hole and to protect the dam embankment. During this grouting, PZ-42, located 72 feet downstream of the dam C/L and tipped in the BCLS, was grouted up. This area coincides with the area of historic tailwater connectivity previously referenced from the 2012 dewatering. It was theorized that a karstic connection linked to the stilling basin likely existed. Any grout exiting into the stilling basin would not typically be observable due to the depth and murkiness of the water. The stilling basin was dewatered before additional grouting was permitted in order to verify if there was a

D/S exit point. The borehole was cored again until drill fluid was lost, and gravity grouting resumed. Within 30 minutes of initiating the next gravity grouting stage, grout emerged 500 feet D/S in the stilling basin. This grouting event confirmed karst connectivity within the dam foundation. Instrumentation responses to the event are summarized in Figure 12. In late 2017, PZ-80 was installed between PZ-36 and PZ-41 tipped in the alluvial foundation soil 25 feet D/S of the dam C/L. The lower 4 feet of the borehole encountered grout above the bedrock contact. It is unlikely that hydrofracture occurred since grouting was conducted at gravity pressure and no deformation was detected at the dam surface. The grout encountered likely filled a pre-existing void at the soil-rock interface.

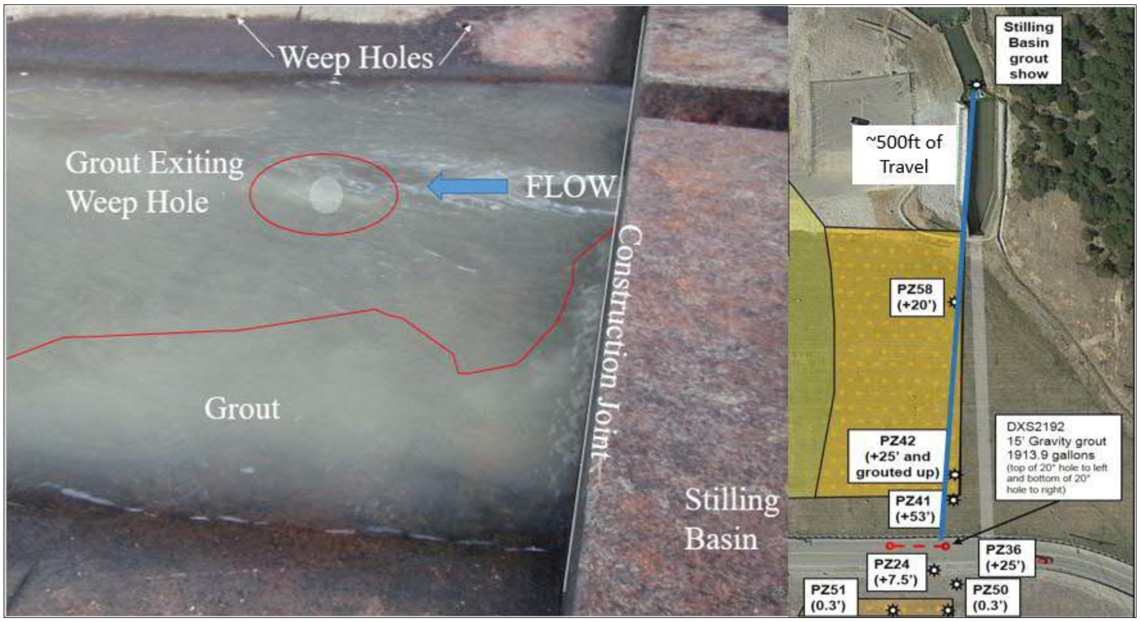

Fig. 12. BCLS Grout Show into the Tailwater

BCLS Karst Intensity

The Beech Creek Limestone was found to have high hydraulic conductivity and correspondingly high grout takes where the formation was unprotected by upper rock units, particularly in the vicinity of the exposed bedrock outcrops. Highly weathered, solutioned bedrock and karst with clay seams, voids, and interconnected pathways were encountered in most of the BCLS boreholes shown in Figure 13. In most instances, the infilled layers varied from a few inches to a few feet thick. The soil infill below the upper rock roof was composed of saturated, creamy, fine silt, and clay particles with weathered limestone clasts. The consistency of this material and the location within the rock mass is consistent with fine foundation and embankment material transported into the rock mass from low stress zones in contact with karstic limestone solution features. The largest voids encountered within the BCLS were located five feet left of the conduit in vertical holes DXP2242-A and UXP2242-2 (Figure 14). The feature was noted by no resistance in the advance of the roto-sonic tooling and varied from 4-6 feet thick in the two boreholes. The grout ribbons shown in the photo presumably originated from DXS2192, which was advanced prior to drilling DXP2242-A. The clay infill present demonstrates the limitations for grout to displace soil within karst and the high volume of infill material that can remain in place even with successful grouting efforts. It should also be noted that grout was encountered at the soil bedrock interface above the BCLS. The grout present at the interface is supporting evidence that the karst network is continuous and is in contact with the foundation soils in multiple locations. The left side Beech Creek Limestone is considered similar to the right-side Beech Creek Limestone although smaller void features were generally encountered.

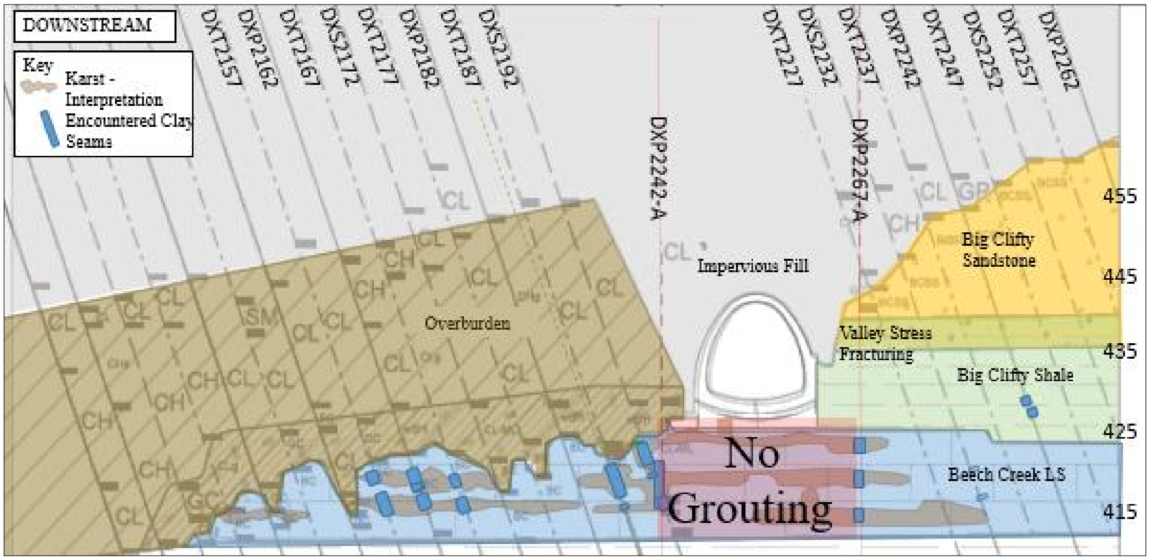

Fig. 13. Phase IB Grouting Concept Profile of the Beach Creek.

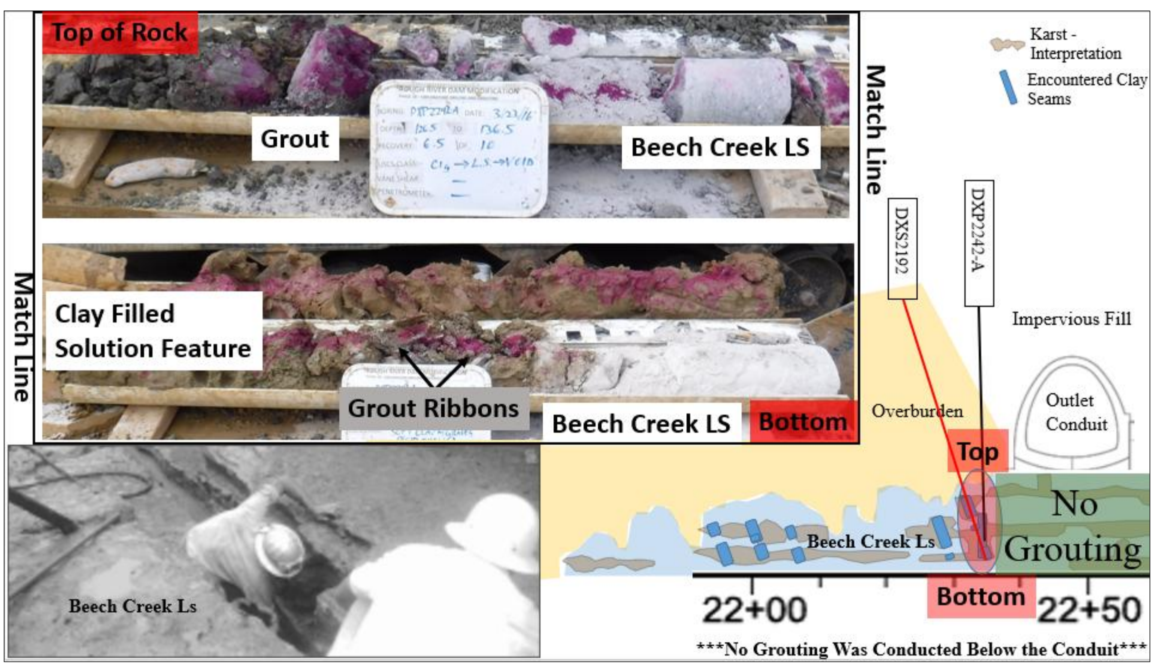

Fig. 14. BCLS Karst and Remaining Clay Infill

BCLS – Connectivity Below the Outlet Conduit

During grouting operations in borehole DXP2262, the instrumented packer detected core drilling operations in borehole DXP2162, located 100 feet away. The coring operation was halted so work could proceed safely. The hole to hole communication is evidence that fluid pressure from the abutment has a direct connection to the BCLS below the conduit (Figure 15). The untreated window below the conduit has the potential to allow circumvention of the grout curtain in addition to concentrating flow near the conduit. The cutoff wall design has since been modified to fully sever the conduit and extend fully across the foundation.

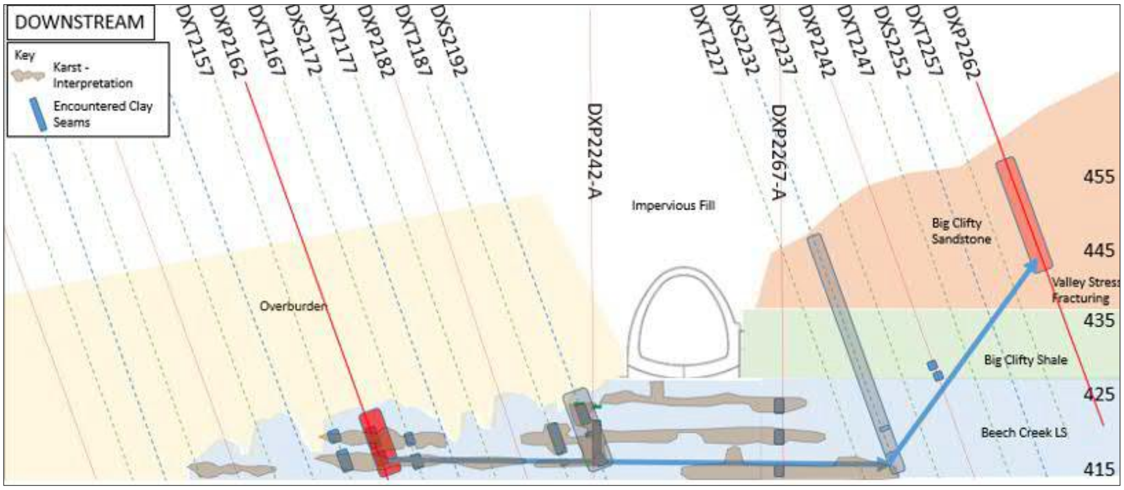

Fig. 15. Schematic of Cross-Hole Communication Below the Conduit.

Haney Limestone Karst Intensity

The Haney limestone (HLS) in the upper abutments on each side of the dam was determined to have a high concentration of karst features. The largest solution feature encountered in the Haney Limestone formation was clay filled between Station 10+45 to Station 10+90 and extended approximately 47 feet deep on the upstream side of the crest (Figure 16). A second solution feature was encountered on the downstream grout row between Stations 11+75 to Station 12+10 and is estimated to be approximately 35 feet deep. Both solution features were filled with soft sandy clay, gravel, and intermixed with grout from the historic upper abutment grouting along the perimeter. Other boreholes in the formation revealed that a large lateral network of open subdrains exist in the mid and lower parts of the formation with caves up to 10 feet in diameter. This network of subdrains is likely partially connected with the upper solution features and serve as conduits for moving water and the potential movement of soil. The underlying Big Clifty Shale creates a perched groundwater condition in the formation which acts independent of the reservoir loading. Grout holes in the left Haney Limestone eventually required mandatory downstage drilling and grouting to advance through the middle and lower portions of the formation. Several stages in the Haney on the right abutment exhibited high Lugeon values and grout takes associated with intercepting high angle fractures. The largest and most notable outbreak in the Haney Limestone occurred while grouting DXP2582 with grout exiting into the ravine downstream of the abutment at approximate elevation 535.

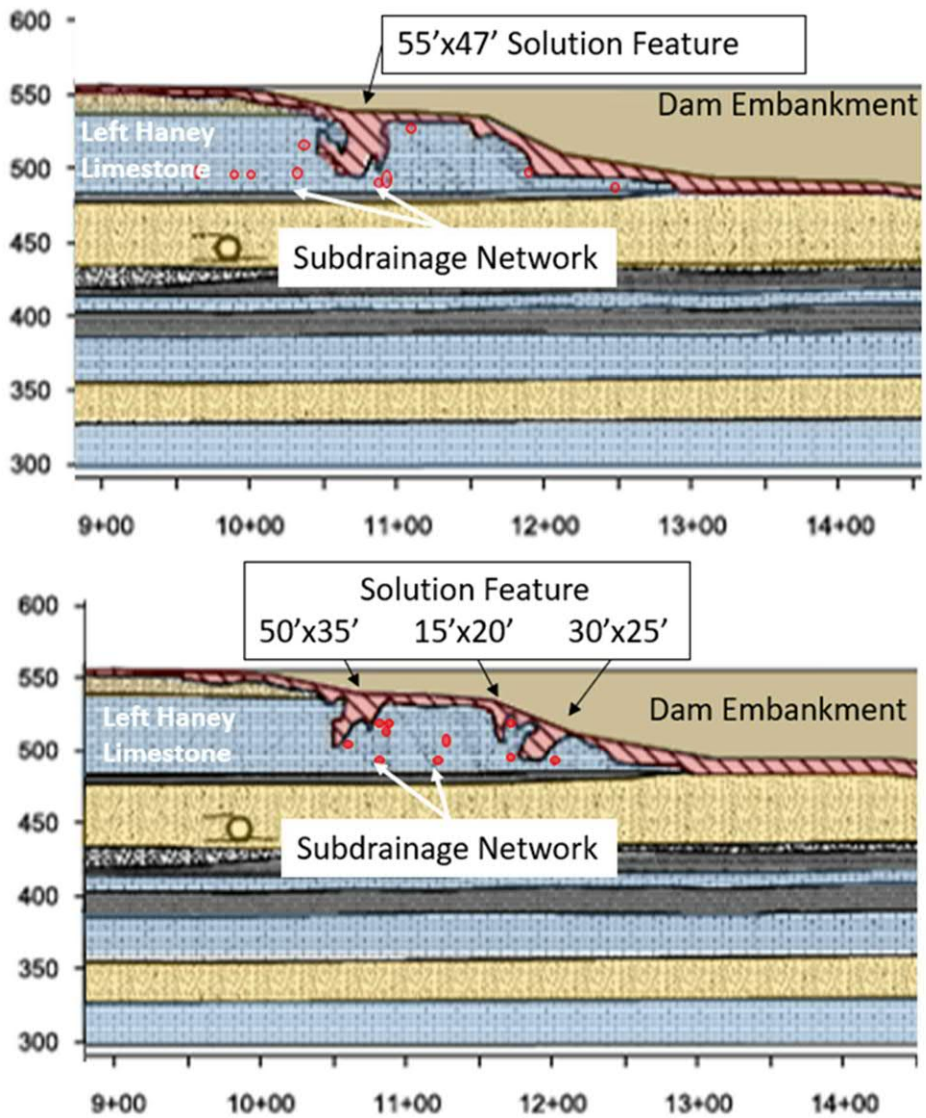

Fig. 16. Haney Limestone Left Upstream Karst (Left) and Left Downstream Karst

Grout Curtain Performance

No additional modifications have been completed at the project since completion of grouting in 2017 other than the addition of several instruments on the right abutment located along the conduit alignment. The instrumentation generally supports a conclusion that head loss has improved across the dam foundation post grouting. Instruments located in the deep valley foundation soils where the BCLS does not exist had no change in phreatic surface post grouting, as would be expected. Instrumentation along the U/S dam crest tipped in the blanket and foundation soils indicated an increased phreatic surface generally ranging from 1-4 feet in magnitude. Instrumentation tipped in the D/S BCLS generally decreased from 0-9 feet. Through seepage within the BCLS appears to have generally decreased at the dam C/L with the exception of a few locations where preferential seepage pathways remain within the karst, at the foundation contact, and within the zone of no grouting below the conduit; all of which will likely expedite future erosion. A strong tailwater influence is still prevalent to within 40 feet of the dam crest. The increased head loss below the dam has significantly increased gradients. For example, the post grouting change in theoretical gradient between the upstream sand blanket and the downstream BCLS has nearly doubled in some locations. High pool events have the potential to generate gradients which will accelerate the rate of erosion in the foundation. Several feet of clay infill in the BCLS and voids at the foundation contact exist which cannot be effectively treated via grouting and are now subject to these higher gradients. Previously installed grout present above the top of bedrock also provides an additional contact surface for erosion to occur. If a high pool event such as 2007 or 2011 were to occur again, it is anticipated that instrumentation declines would once again result; potentially at a faster rate.

Phase 2 Project Overview

In 2017, during design of the cutoff wall, the outlet conduit concrete structure was found to be unacceptably thin for the recommended approach to saddle the cutoff wall around the conduit. The 2012 DSMR recommendations were determined to be incomplete. A Supplement to the 2012 DSMR was completed between 2018 and 2020 to modify the recommended plan with suitable design measures to facilitate successful installation of the cutoff wall to reduce risks for the dam. The Phase II project now includes the construction of a new left abutment outlet works followed by installation of a full-length cutoff wall severing the existing outlet conduit and the karstic foundation bedrock below the dam. Completion of the new outlet works will require a new approach channel, control tower, service bridge, outlet conduit tunnel, stilling basin, concrete lined apron, and retreat channel (Figure 17). Completion of the cutoff wall (Figure 18) will require abandonment of the existing right abutment outlet works, construction of a cutoff wall across the dam foundation and through the existing outlet conduit, backfilling the existing retreat channel, and relocation of Highway 79 back to the dam crest. The Project is currently in the Corrected Final Phase.

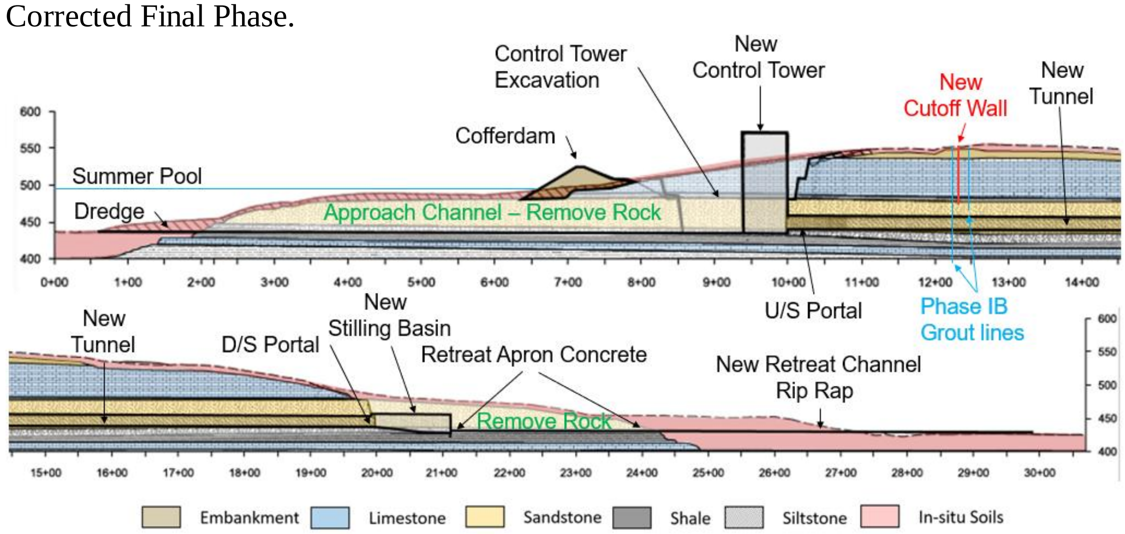

Fig. 17. New Outlet Works Construction Profile – Station 9+85 Shown at Dam C/L

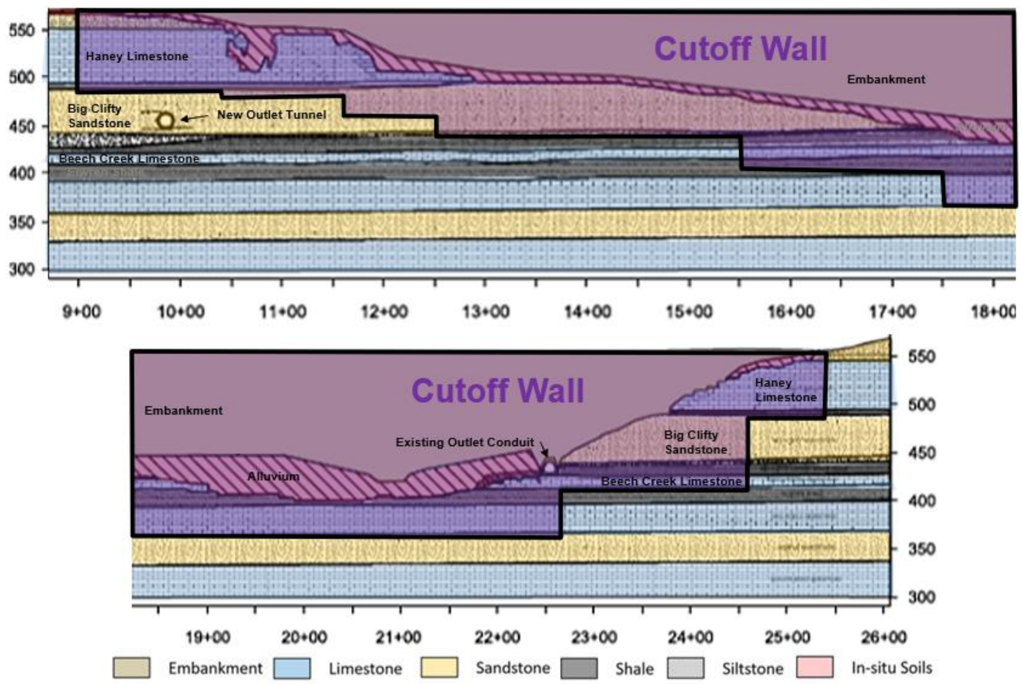

Fig. 18. Proposed Rough River Phase II Cutoff Wall

Conclusion

Effectively placed instrumentation indicated a continuous U/S to D/S connection from the pool to tailwater below the dam. The presence of extensive karst, progressing internal erosion, and the potential for an unfiltered exit was confirmed through foundation grouting observations. Massive karst was confirmed to exist in the upper left abutment. The modern grouting methods have been temporarily successful at filling open voids and increasing head loss across the dam centerline. The noted improvement in instrumentation response is considered temporary due to the extent of clay infilling and karstic voids documented in and above the karstic limestone formations. Post grouting evaluations indicate that preferential seepage pathways remain within the karst, at the foundation contact, and within the zone of no grouting below the conduit which will expedite future erosion. The project requires a full cutoff wall to achieve full risk reduction.

Columbia Parkway Landslide Stabilization

Joseph Hauber, P.E.[1] and Richard Pohana, P.E.[2]

Abstract: Seasonal hillside instability has been a recurring problem along Columbia Parkway for decades in the form of abrupt landslides and gradual downslope movement of the colluvial overburden soils. Heavy rains saturate the overburden soils resulting in landslides and mudslides that overtop the existing wall and cascade onto the roadway, blocking lanes and disrupting traffic.

Following a series of significant landslides in multiple locations along the uphill side of the parkway, the City of Cincinnati in 2019 sought a long-term stabilization plan, and selected the design-build team of Beaver Excavating Co. (Beaver) as the primary contractor and Geotechnology, Inc. (Geotechnology) as its geotechnical design consultant to address 9 landslide sites along a two-mile stretch of the parkway. Two types of stabilization mechanisms for the project were implemented after evaluating the geologic conditions of the areas: soldier pile and lagging (SPL) walls, and soil nails with high-strength steel mesh. The selection of the stabilization mechanism was based on the geometry of the ground surface and the subsurface conditions.

The presentation will discuss the complexities of the landslides and the subsurface exploration program, as well as the design and construction of the stabilizations.

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[2] Principal Geotechnical Engineer, City of Cincinnati, Ohio. Email: rich.pohana@cincinnati-oh.gov

Columbia Parkway
Landslide Stabilization

January 29, 2019 (Photograph Courtesy of City of Cincinnati)

ORVSS LI

November 2, 2021

Presented by:

Joseph D. Hauber, PE

Geotechnology, LLC

And

Rich Pohana, PE

CDOTE

GEOTECHNOLOGY
A Universal Engineering Sciences Company

city of CINCINNATI

BEAVER EXCAVATING COMPANY

Columbia Parkway

Columbia Parkway

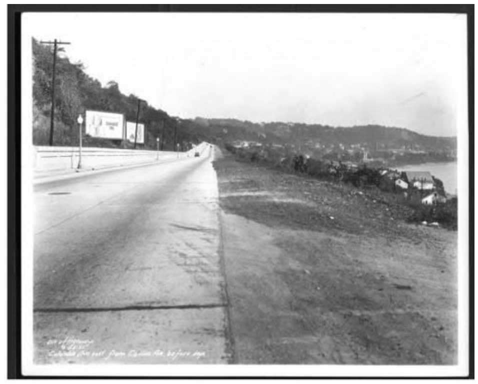
US Route 50 Limited Access Highway Landslide
Downtown Cincinnati to Cincinnati-Fairfax Corporation Line

3

Columbia Parkway: The 1930’s

4

Landslides on the Downhill Side



Landslides on the Uphill Side

1975 Landslide
(Photograph Courtesy of City of Cincinnati)

1975 Landslide
(Photograph Courtesy of City of Cincinnati)

1996 Landslide
(Photograph Courtesy of City of Cincinnati)

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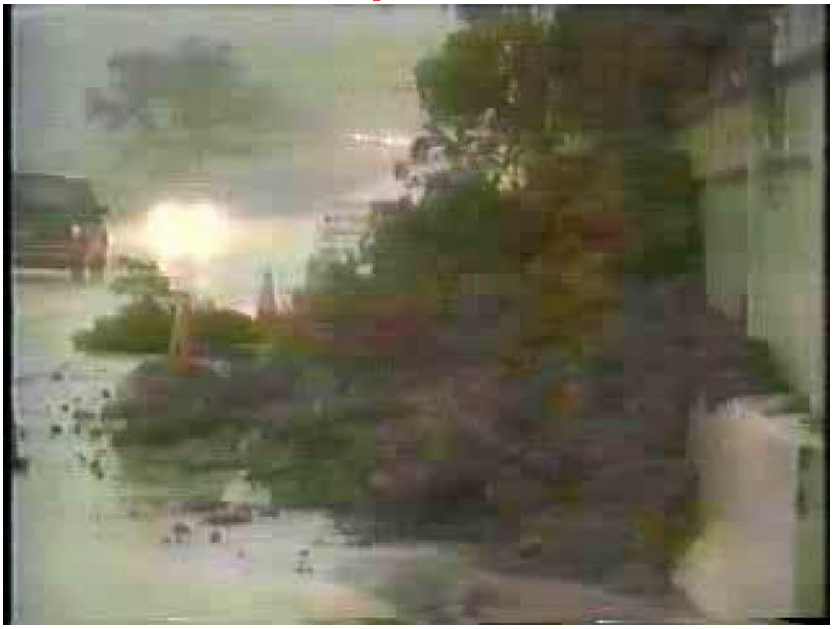
Columbia Parkway: 1996 Landslide



1996 Landslide
(Video Courtesy of City of Cincinnati)

7

Columbia Parkway: 2019 Landslide

January 29, 2019
(Photograph Courtesy of City of Cincinnati)

8

January 29, 2019
(Photograph Courtesy of City of Cincinnati)

9

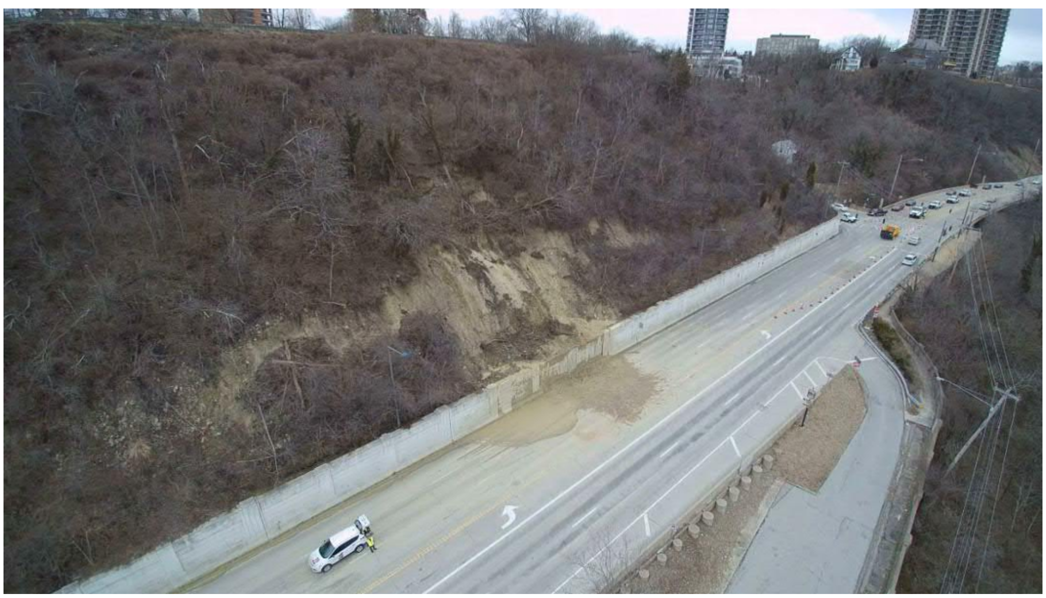
January 29, 2019
(Photograph Courtesy of City of Cincinnati)

10

February 12, 2019
(Photograph Courtesy of City of Cincinnati)

11

Columbia Parkway: 2019 Landslide



2019 Landslide
(2/14/2019 Video Courtesy of City of Cincinnati)

12

February 22, 2019
(Photograph Courtesy of City of Cincinnati)

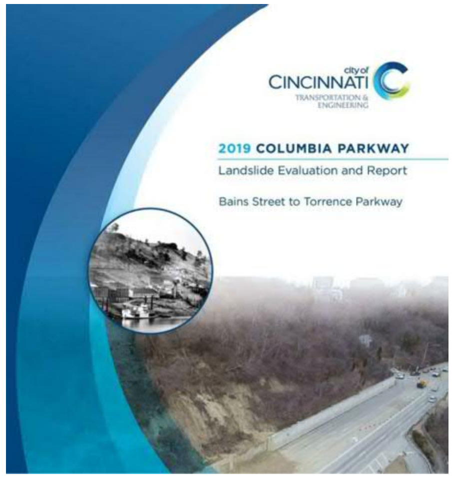
13

March 11, 2019
(Photograph Courtesy of City of Cincinnati)

14

Selection of Design Team

- CDOTE Request for Information (RFI)
  - Issued: March 28, 2019
  - Due: April 19, 2019



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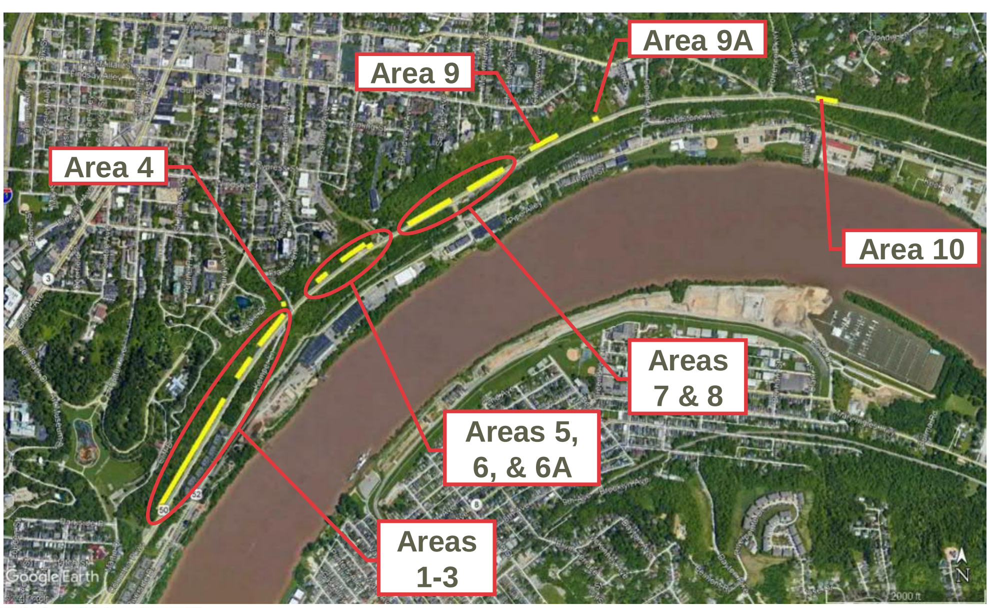
Stabilization Areas



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March 24, 2019
(Photograph Courtesy of City of Cincinnati)

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Selection of Design Team

- Seven submittals received
- Short-listed to 4 DB Contractors
- CDOTE Request for Proposal (RFP)
  - Issued: May 6, 2019
  - Due: May 28, 2019
- Selection Committee Members
  - CDOTE
  - HAM. Co.
  - ODOT
  - Cint. DEI

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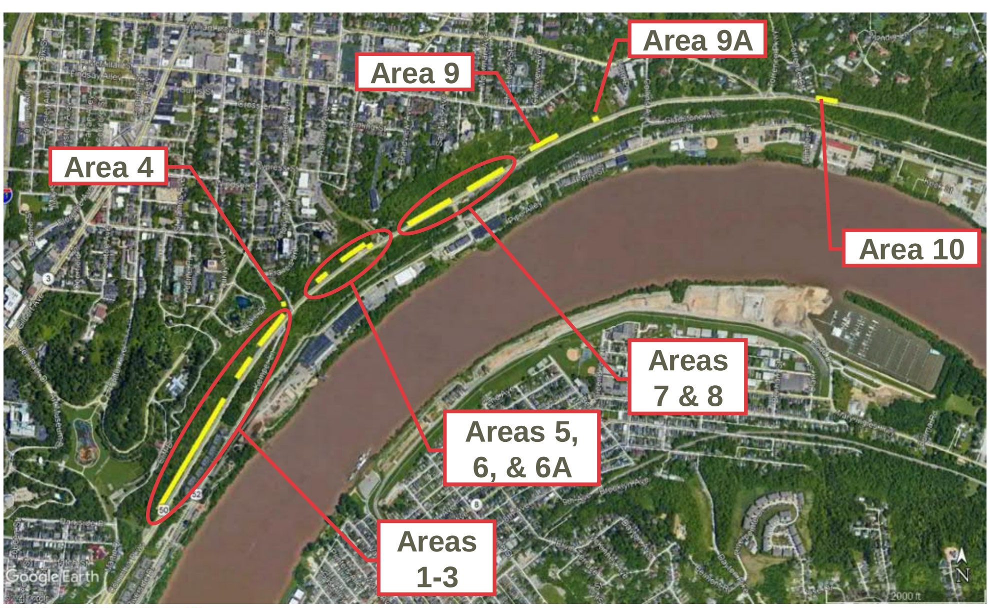
Selected Design Team

- General Contractor: Beaver Excavating Company
- Geotechnical Design Engineer: Geotechnology, Inc.

Stabilization Areas



June 2019
(Photograph Courtesy of City of Cincinnati)

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Stabilization Area Exploration & Design

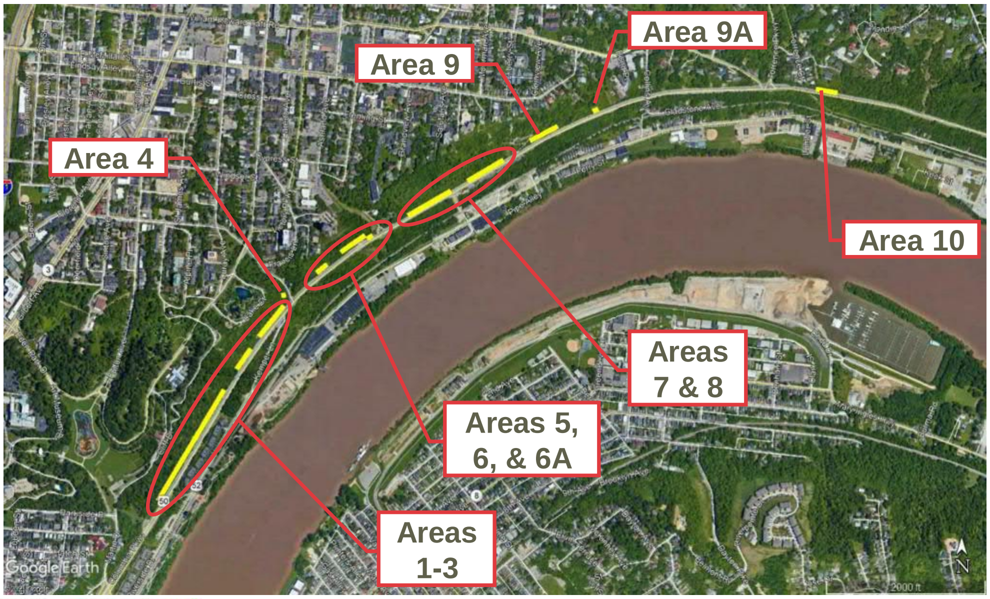
GEOTECHNOLOGY
A Universal Engineering Sciences Company

city of CINCINNATI

BEAVER EXCAVATING COMPANY

22

Stabilization Areas



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Area Dimensions

Area	RFI Length (ft.)	Design Length (ft.)
1 through 3	1,660 + 320 + 505 = 2,485	3,045
4	60	Not Stabilized
5, 6, & 6A	175 + 375 + 90 = 640	750
7 & 8	630 + 530 = 1,160	1,370
9	400	Stabilized by GSI
9A	80	105
10	280	190

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Exploration Phases

- Due to extensive area to be explored and the access paths necessary for drill rigs or track hoes to access boring and test pit locations, the exploration for the different areas was phased.
  - Concerns with prolonged timeframe between disturbance for exploration and construction of stabilization measures.
- Areas 7, 8, 9A, and 10 were explored first in 2019.
- Areas 1 through 3, 5, and 6 were explored in 2020 (90% design plans were completed prior to exploration).

Exploration Phases

Area	Exploration Dates	Exploration Means	Depth to Bedrock (ft.)
7 & 8	August 2019	8 Borings & 6 Test Pits	4.5 to 14
9A	August 2019	1 Boring & 1 Test Pit	5 to 10
10	August 2019	1 Boring & 3 Test Pits	3 to 10
5, 6, & 6A	June 2020	17 Test Pits	1.5 to 7
1 through 3	July & August 2020	31 Test Pits	2 to 13.5

- Total of 10 borings & 58 test pits

Shear Strength Parameters

- 8 CU triaxial tests were completed; however, 6 of them were completed on remolded samples due to poor recovery with limestone floaters in the overburden soils
- Triaxial test results were compared with fully softened and residual strength parameters based on liquid limit, clay-size fraction, and effective normal stress.
  - Gamez, J.A., and Stark, T.D. (2014). "Fully softened shear strength at low stresses for levee and embankment design." *J. Geotechnical and Geoenvironmental Engineering* 140(9), 06014010.
  - Stark, T. D., Choi, H., and McCone, S. (2005). "Drained shear strength parameters for analysis of landslides." *J. of Geotechnical and Geoenvironmental Engineering*, 131(5), 575-588.

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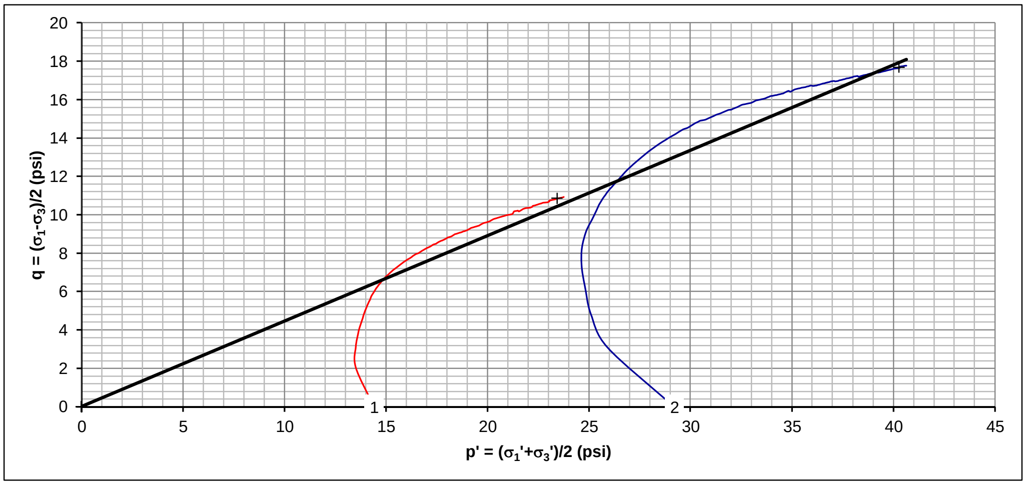
Shear Strength Parameters
Triaxial Test Results – Undisturbed Specimens

LL = 48, CF = 50%
γ = 131 pcf, WC = 16%
c' = 0 psf and ϕ' = 26°

LL = 46, CF = 46%
γ = 136 pcf, WC = 20%
c' = 0 psf and ϕ' = 26°



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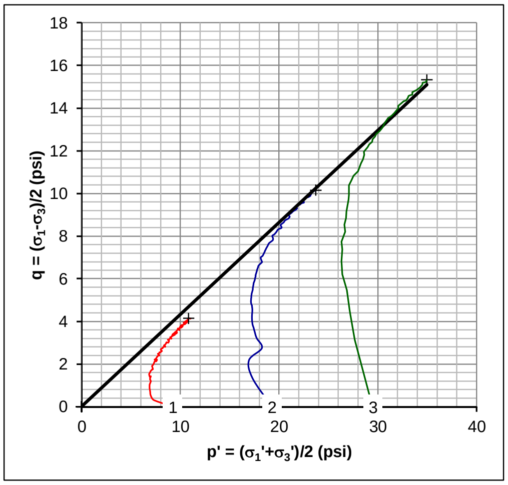
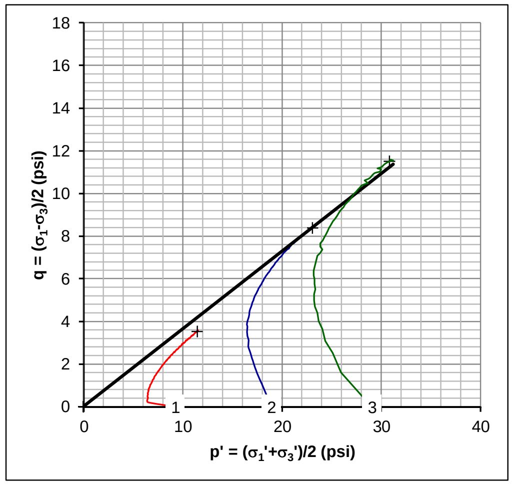
Shear Strength Parameters
Triaxial Test Results – Remolded Specimens

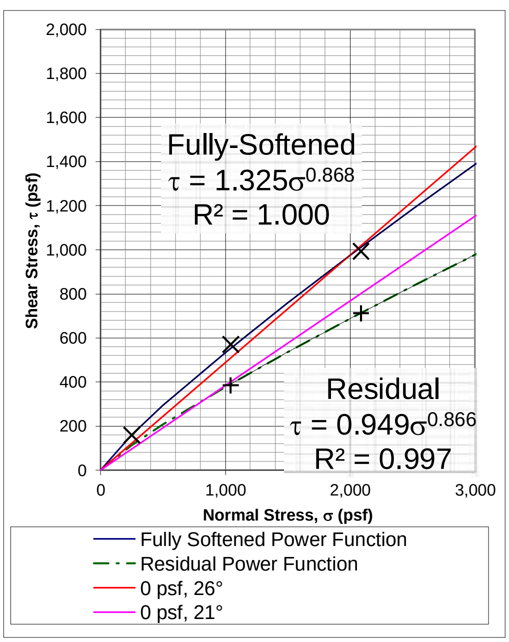
LL = 48, CF = 54%
γ = 121 pcf (remolded), WC = 15%
c' = 0 psf and ϕ' = 26°



LL = 45, CF = 52%
γ = 125 pcf (remolded), WC = 18%
c' = 0 psf and ϕ' = 21°

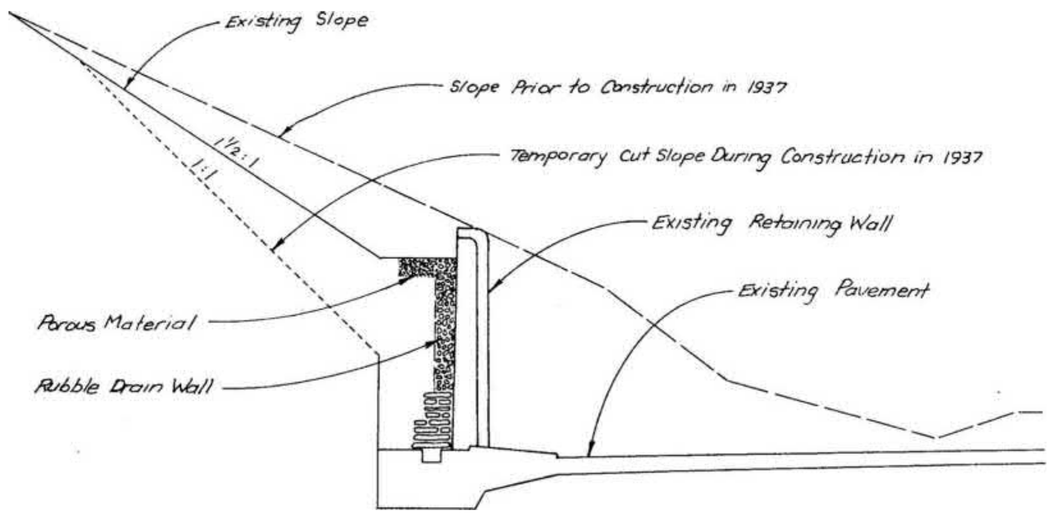


Shear Strength Parameters



- Fully softened and residual shear strength parameters from Stark et al. were based on average values of LL & CF + one standard deviation:
  - LL = 47 and CF = 57
- Residual parameters were used along soil/bedrock interface
- Fully softened parameters were used for the remainder of the overburden.

Generalized Geometry & Geology



(Sketch from Figure 1 of 2019 City of Cincinnati RFI)

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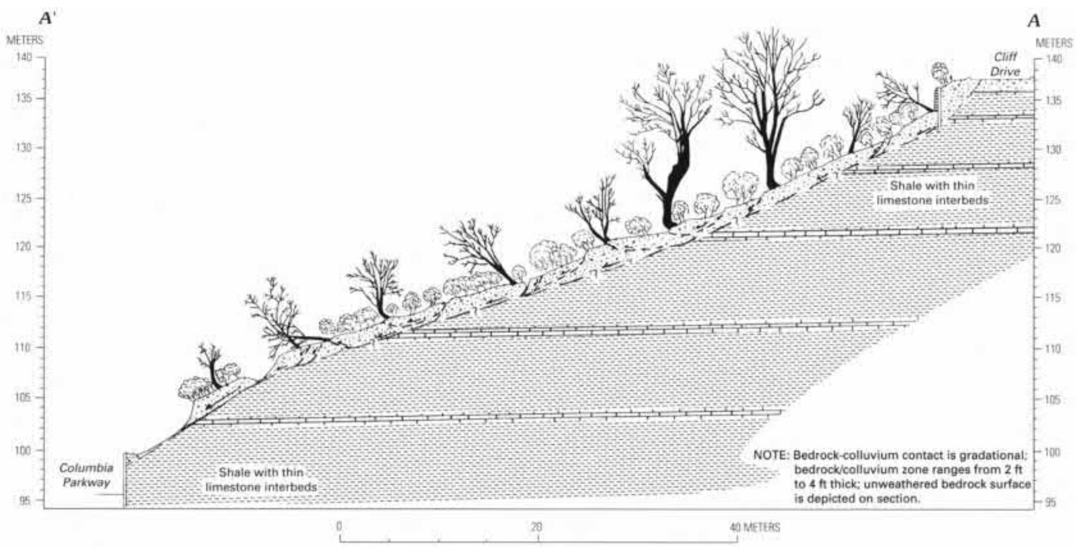
Generalized Geometry & Geology



(Figure 2 of 2019 City of Cincinnati RFI, Richards 1982)

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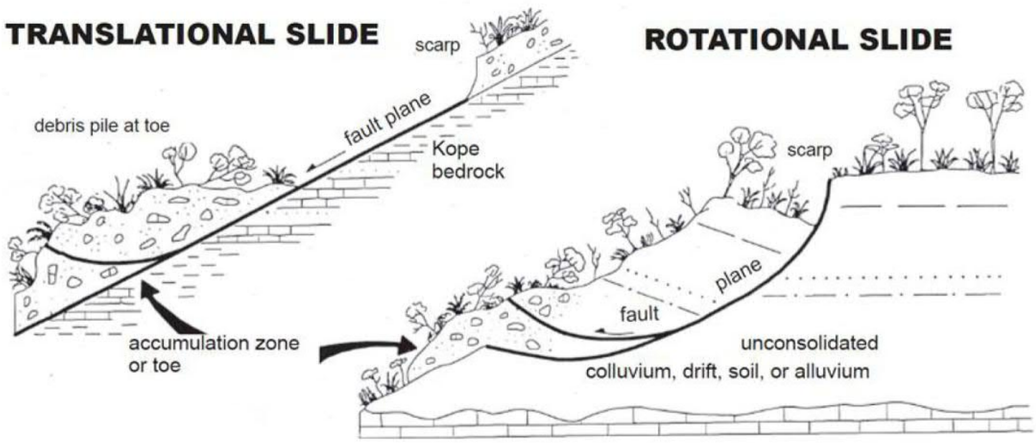
Generalized Slope Failure Mechanism



(Figure 61, Potter 2007)

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Stabilization Methods

- Soldier Pile & Lagging (SPL) Walls
  - Area 10: ~190 LF
  - Areas 7-8: ~1,370 LF
  - SPL Designs included:
    - A 20-ft.-wide catchment between new and existing walls
    - A few feet of "freeboard" above new wall
- Soil Nails (SN) with Steel Mesh
  - Areas 1-3, 5, 6, & 9A: ~3,900 LF parallel to Columbia Parkway
  - SN Designs included:
    - A 20-ft.- to 15.-ft.-wide catchment between stabilization and existing wall.

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Soldier Pile & Lagging Wall Sections – Areas 7, 8, & 10

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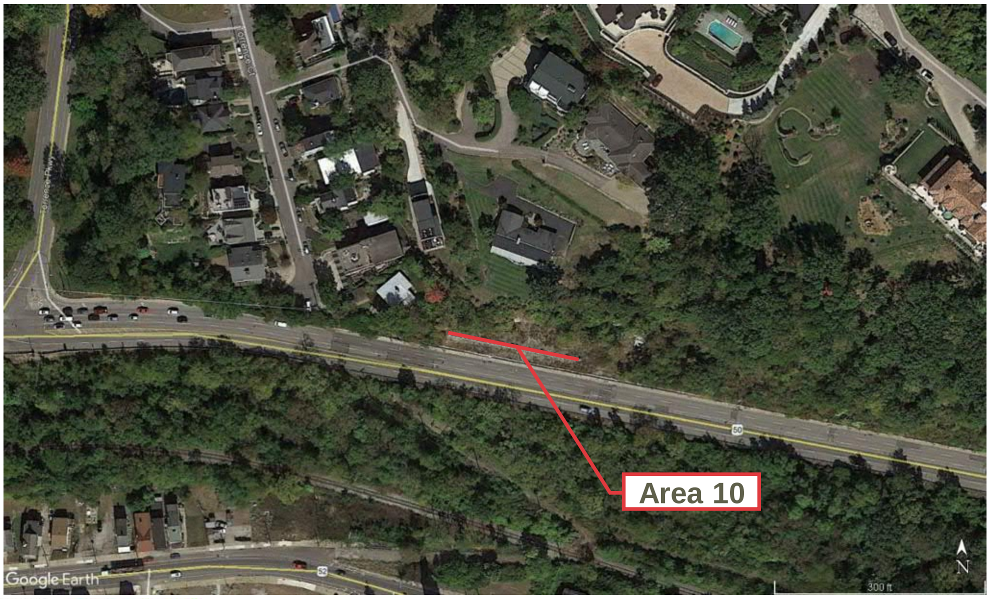
Columbia Parkway Landslide Stabilization: Area 10



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Columbia Parkway Landslide Stabilization: Area 10

(Photograph from 2019 Drone Video Courtesy of City of Cincinnati)

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Columbia Parkway Landslide Stabilization: Area 10

May 15, 2019

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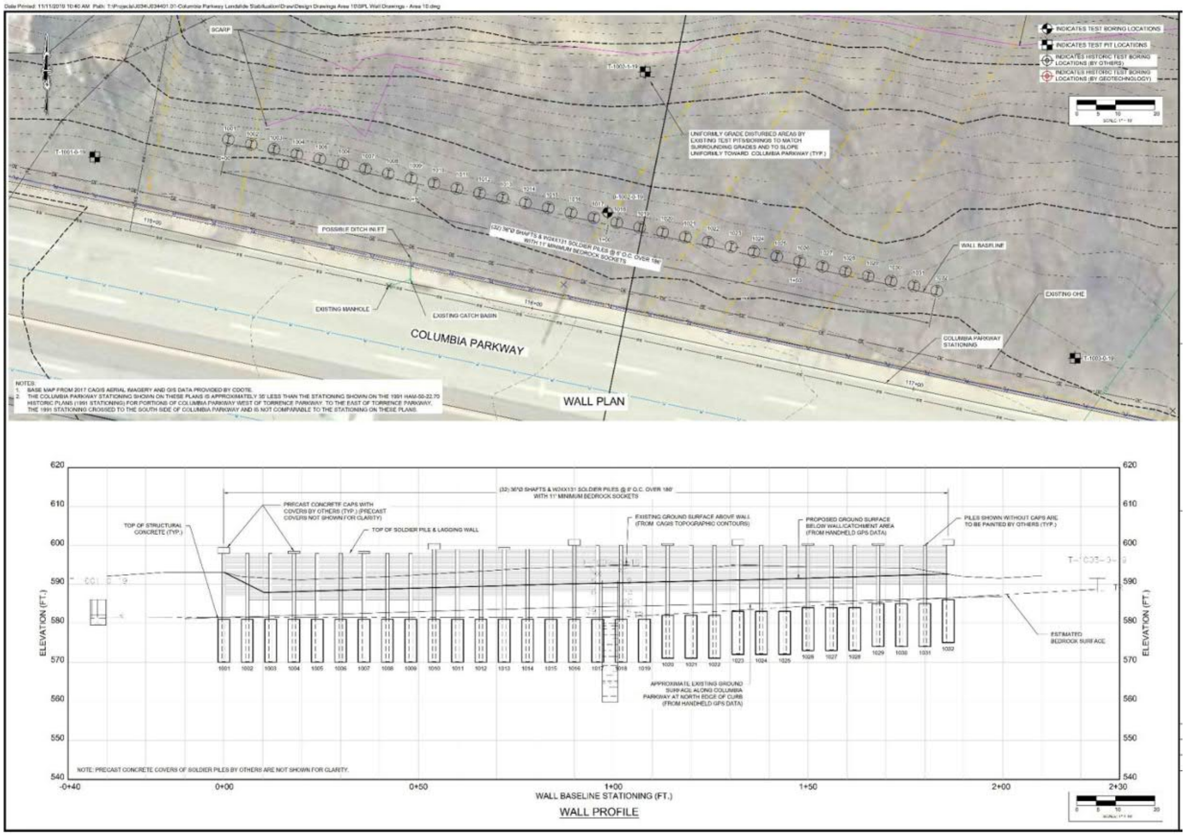
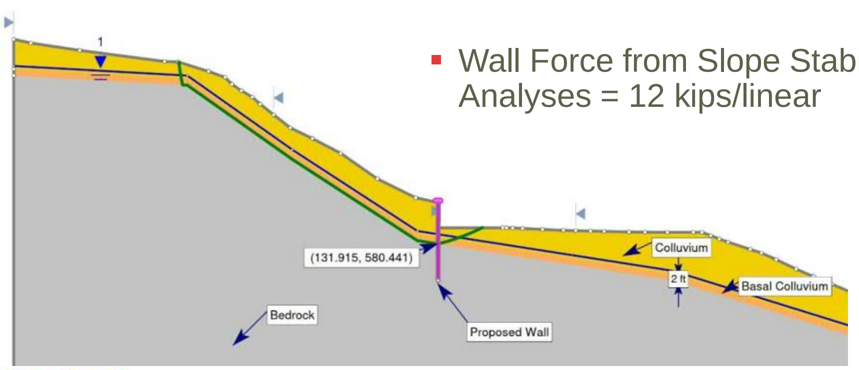


Columbia Parkway Landslide Stabilization: Area 10

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Direction
Support 4		Pile/Micro Pile	Passive (Method B)	1	Shear	12000	Perpendicular to pile

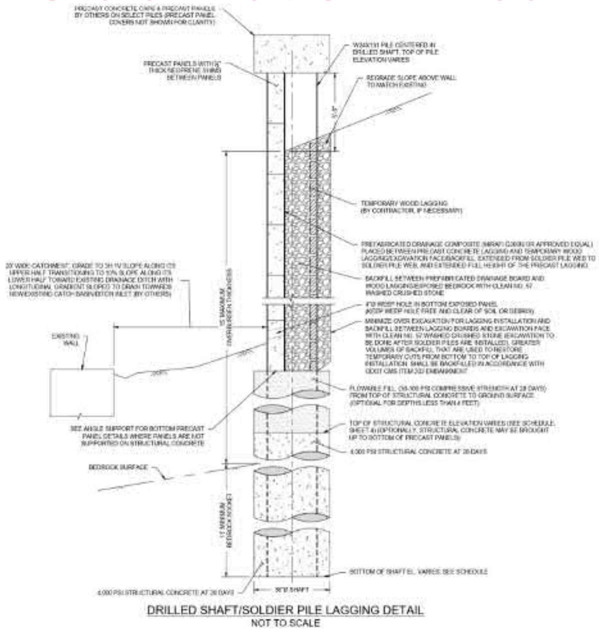
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Allow Sliding	PC a	PC b	Water Surface
Colluvium		125	Power Curve		1.325	0.868	Piezometric Line 1
Basal Colluvium		125	Power Curve		0.949	0.866	Piezometric Line 1
Bedrock		140	Infinite strength	Yes			Piezometric Line 1



- Wall Force from Slope Stability Analyses = 12 kips/linear

40

Columbia Parkway Landslide Stabilization: Area 10



- 186’ SPL wall
- (36) 25-30’ L W24x131 piles @ 6’ o.c.
- 11’ bedrock sockets
- 5’ freeboard
- Steel design considered:
  - Corrosion above concrete embedment
  - 5’ tall toe bulge above top of piles

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Columbia Parkway Landslide Stabilization: Area 10

February 4, 2020
(Photograph Courtesy of Beaver Excavating Company)

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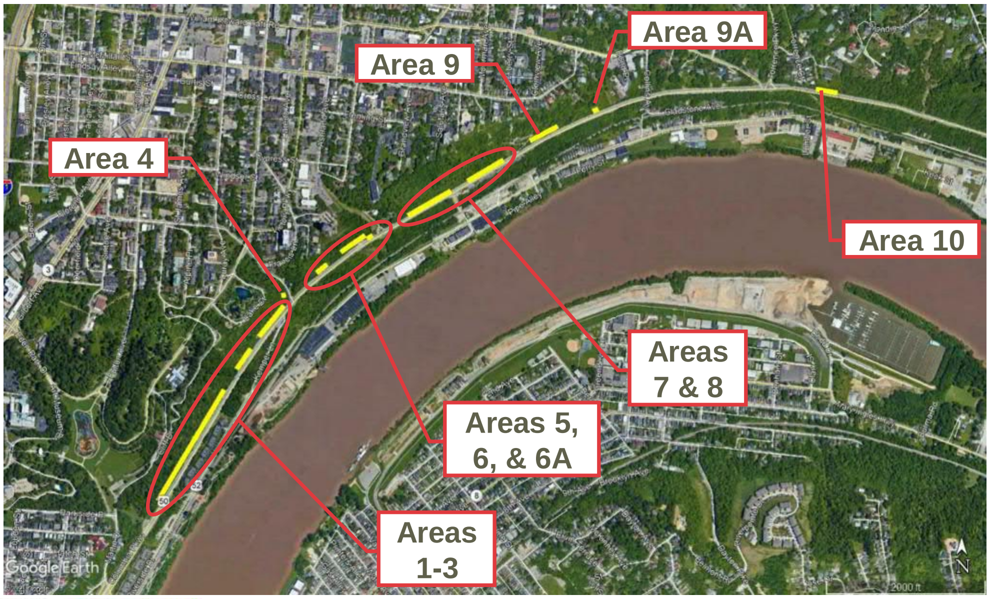
Columbia Parkway Landslide Stabilization: Areas 7 & 8



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Columbia Parkway Landslide Stabilization: Areas 7 & 8

January 6, 2020
(Photograph Courtesy of Beaver Excavating Company)

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Columbia Parkway Landslide Stabilization: Areas 7 & 8

(From Google Earth Street View)

May 15, 2019

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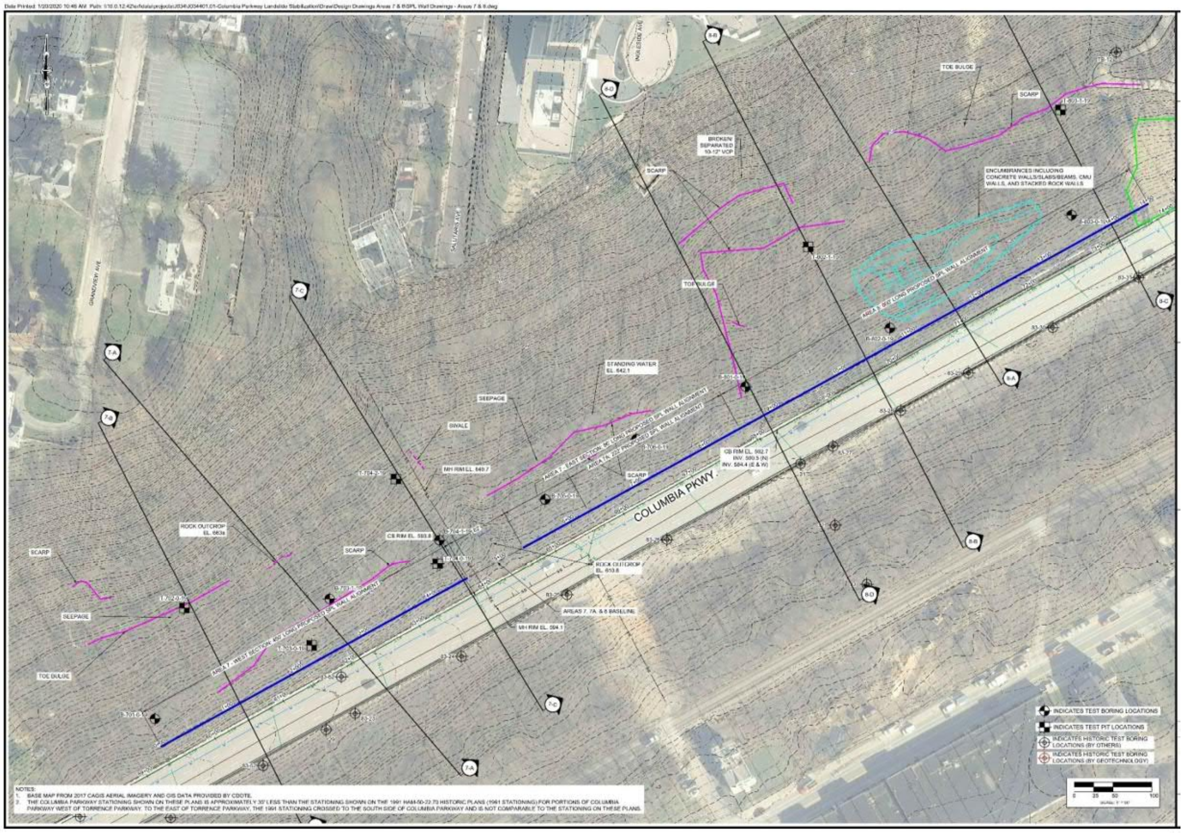
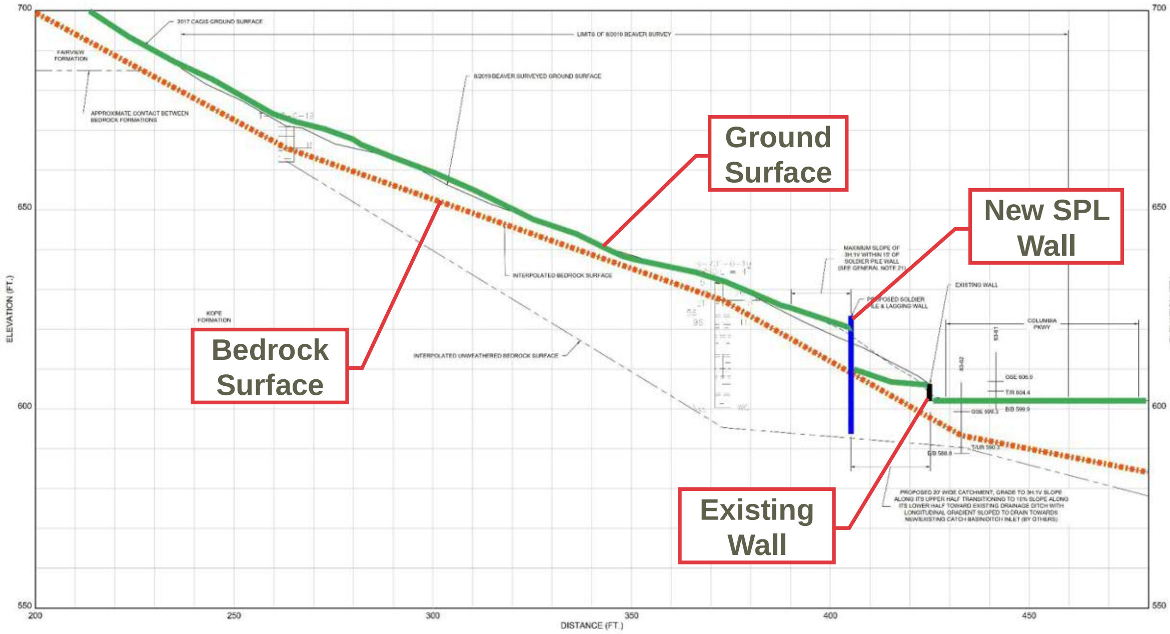


Columbia Parkway Landslide Stabilization: Areas 7 & 8



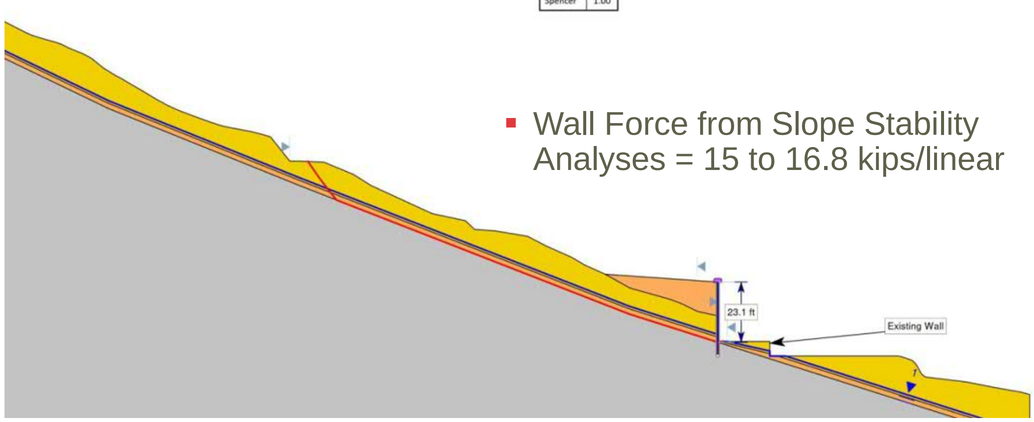
Columbia Parkway Landslide Stabilization: Areas 7 & 8

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Allow Sliding	PC a	PC b	Water Surface
Colluvium		125	Power Curve		1.325	0.868	Piezometric Line 1
Basal Colluvium		125	Power Curve		0.949	0.866	Piezometric Line 1
Bedrock		140	Infinite strength	Yes			Piezometric Line 1

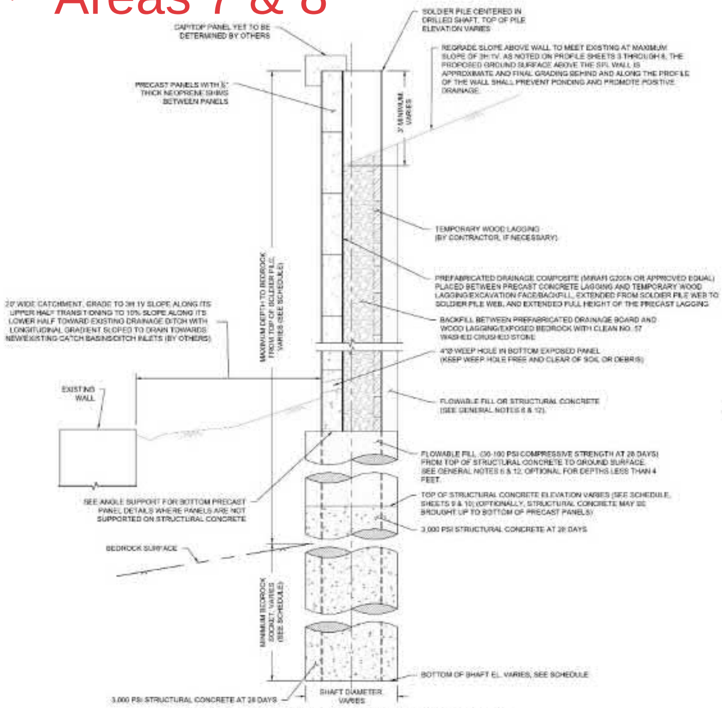
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Direction
Pile 2		Pile/Micro Pile	Passive (Method B)	1	Shear	15000	Perpendicular to pile

Method Name	Min FS
Spencer	1.00

- Wall Force from Slope Stability Analyses = 15 to 16.8 kips/linear



Columbia Parkway Landslide Stabilization: Areas 7 & 8



DRILLED SHAFT/SOLDIER PILE LAGGING DETAIL
NOT TO SCALE

- ~1,400' L SPL wall
- (230) 25-35' L W30 and W36 piles @ 6' o.c.
- 13-16' bedrock sockets
- 3-5' freeboard
- Steel design considered:
  - Corrosion above concrete embedment
  - 5' tall toe bulge above top of piles

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Columbia Parkway Landslide Stabilization: Areas 7 & 8

June 9, 2020

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Columbia Parkway Landslide Stabilization: Areas 7 & 8

October 22, 2020
(Photograph Courtesy of Beaver Excavating Company)

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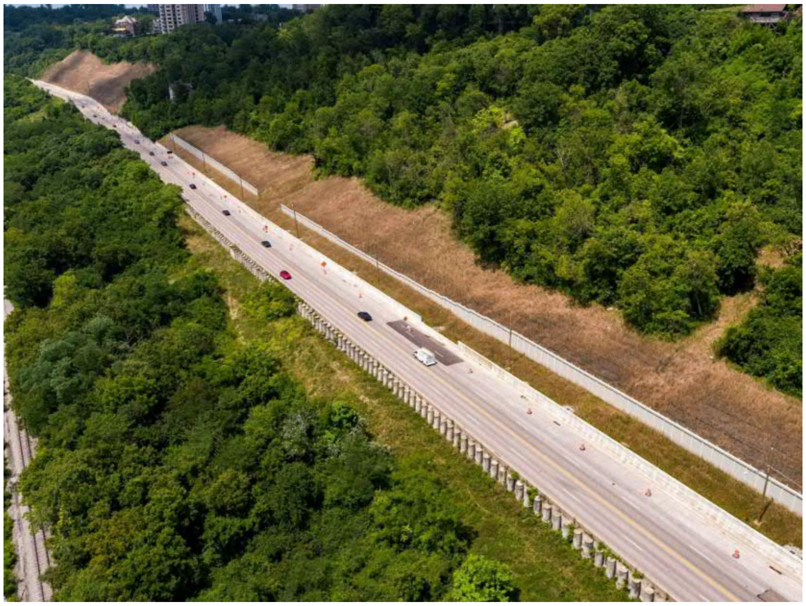
Columbia Parkway Landslide Stabilization: Areas 7 & 8



July 26, 2021
(Photograph Courtesy of City of Cincinnati)

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Soil Nail Stabilization Sections – Areas 1, 2, 3, 5, 6, & 9A

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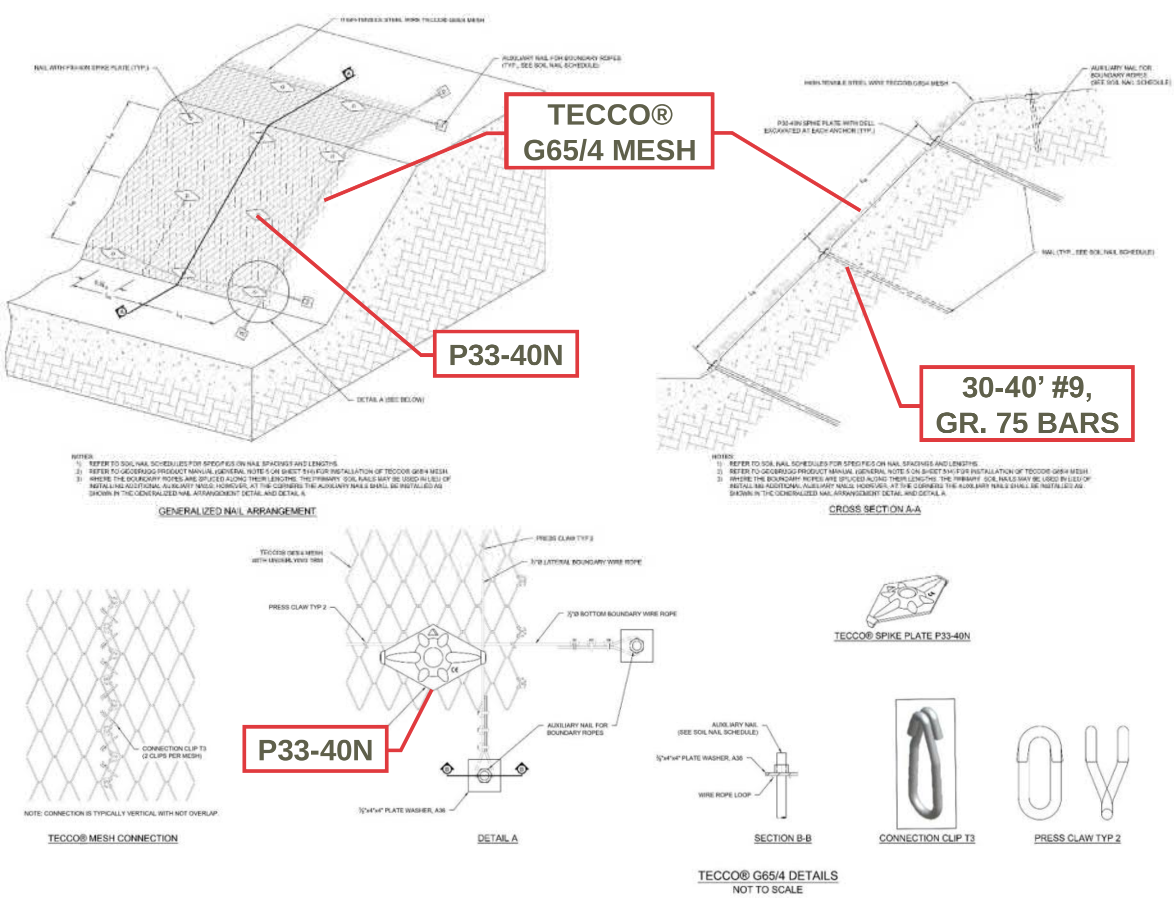

Soil Nail Stabilizations

- The following components were analyzed for the soil nail stabilization:
  - Global stability through the nails (included tensile and shear strength interaction)
  - Local stability between nails
  - Near-surface slope stability
  - Bearing resistance of the steel mesh against:
    - Puncturing
    - Shearing at the upslope edge of the spike plate (i.e., the bearing plate for each nail)
    - Slope-Parallel Tensile Stress
- With the exception of the global stability analyses, RUVOLUM® developed by Geobrugg was used for the analyses. Slide2 by Rocscience was used for the global stability analyses.

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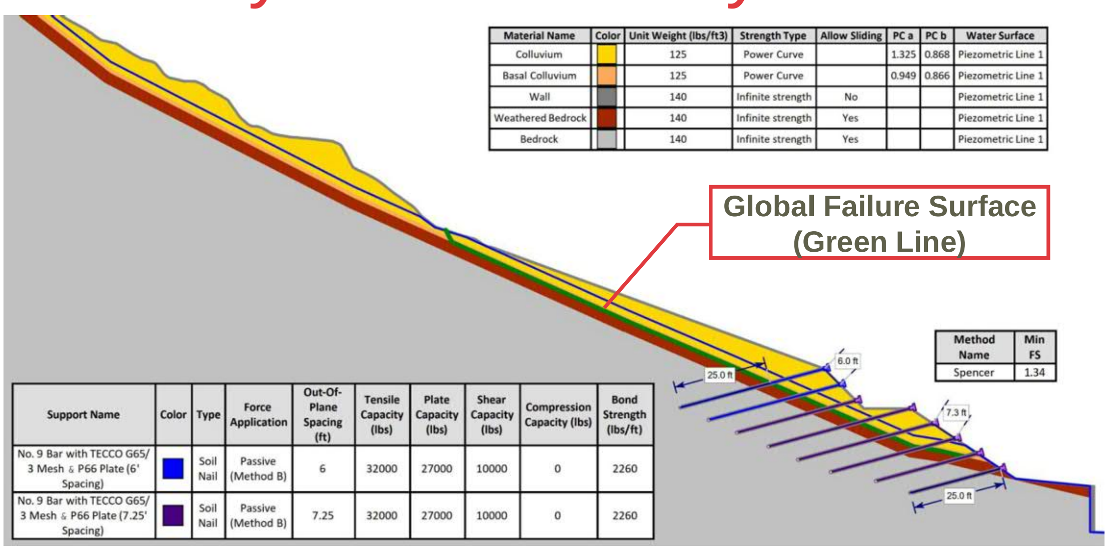
Soil Nail Stabilizations – Global Stability – Slide2 Analyses



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Allow Sliding	PC a	PC b	Water Surface
Colluvium		125	Power Curve		1.325	0.868	Piezometric Line 1
Basal Colluvium		125	Power Curve		0.949	0.866	Piezometric Line 1
Wall		140	Infinite strength	No			Piezometric Line 1
Weathered Bedrock		140	Infinite strength	Yes			Piezometric Line 1
Bedrock		140	Infinite strength	Yes			Piezometric Line 1

Method Name	Min FS
Spencer	1.34

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Shear Capacity (lbs)	Compression Capacity (lbs)	Bond Strength (lbs/ft)
No. 9 Bar with TECCO G65/3 Mesh & P66 Plate (6' Spacing)		Soil Nail	Passive (Method B)	6	32000	27000	10000	0	2260
No. 9 Bar with TECCO G65/3 Mesh & P66 Plate (7.25' Spacing)		Soil Nail	Passive (Method B)	7.25	32000	27000	10000	0	2260

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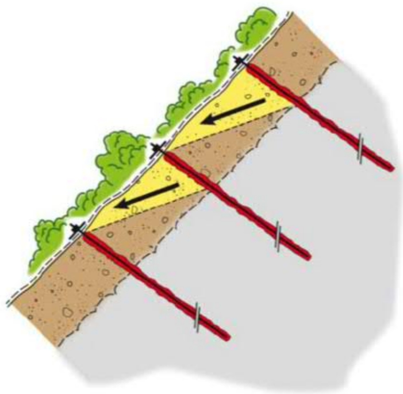
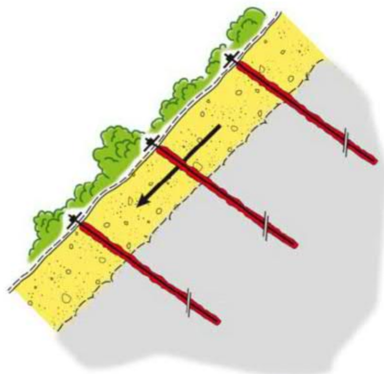
Soil Nail Stabilizations – Local Stability – RUVOLUM® Analyses



Investigation of local, simple, wedge-shaped as well as composite instabilities between the individual nails in the RUVOLUM® concept



Investigation of slope-parallel instabilities near the surface in the RUVOLUM® concept

(Illustration from Geobrugg AG 2018)

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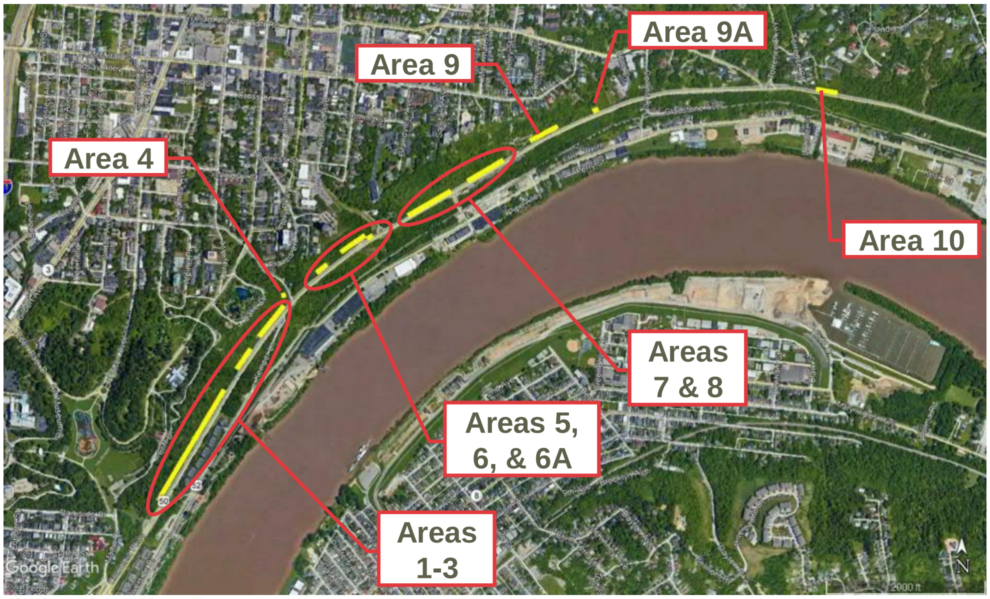
Columbia Parkway Landslide Stabilization: Areas 5 & 6



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Columbia Parkway Landslide Stabilization: Areas 5 & 6

January 6, 2020
(Photograph Courtesy of Beaver Excavating Company)

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Columbia Parkway Landslide Stabilization: Areas 5 & 6

May 15, 2019

(From Google Earth Street View)

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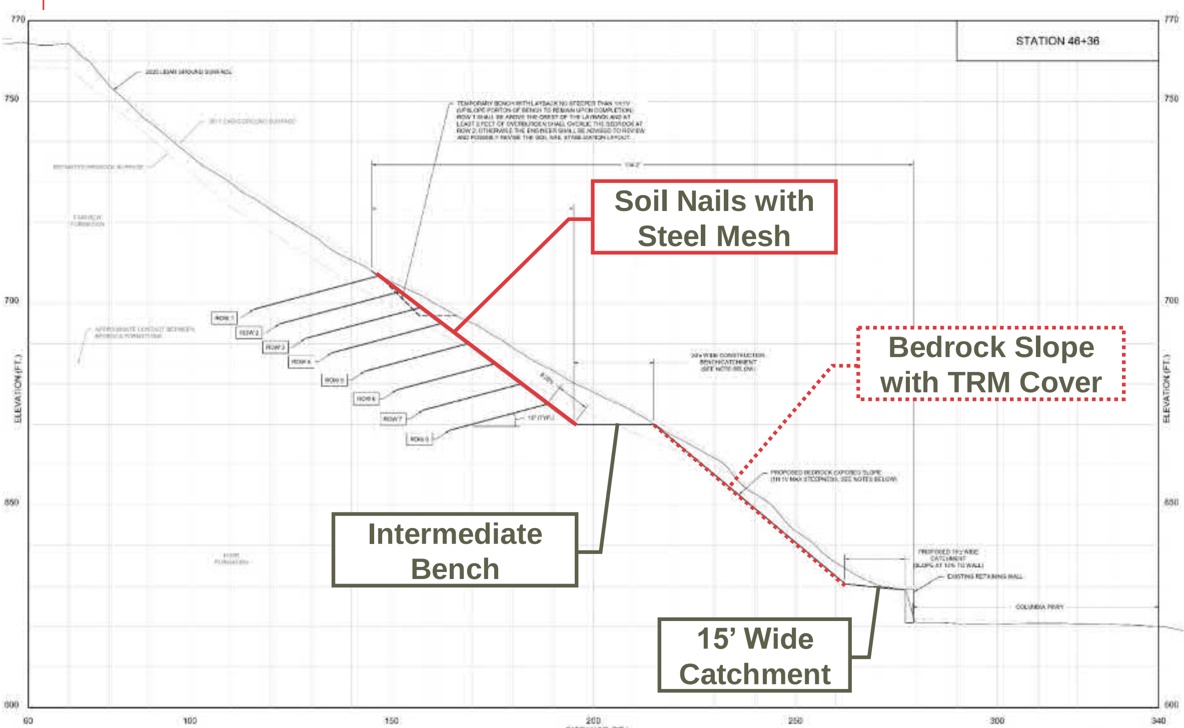
Columbia Parkway Landslide Stabilization: Areas 5 & 6

August 7, 2019

May 15, 2019

61

Columbia Parkway Landslide Stabilization: Areas 5 & 6



Columbia Parkway Landslide Stabilization: Areas 5 & 6

October 22, 2020
(Photograph Courtesy of Beaver Excavating Company)

63

Columbia Parkway Landslide Stabilization: Areas 5 & 6

December 2, 2020

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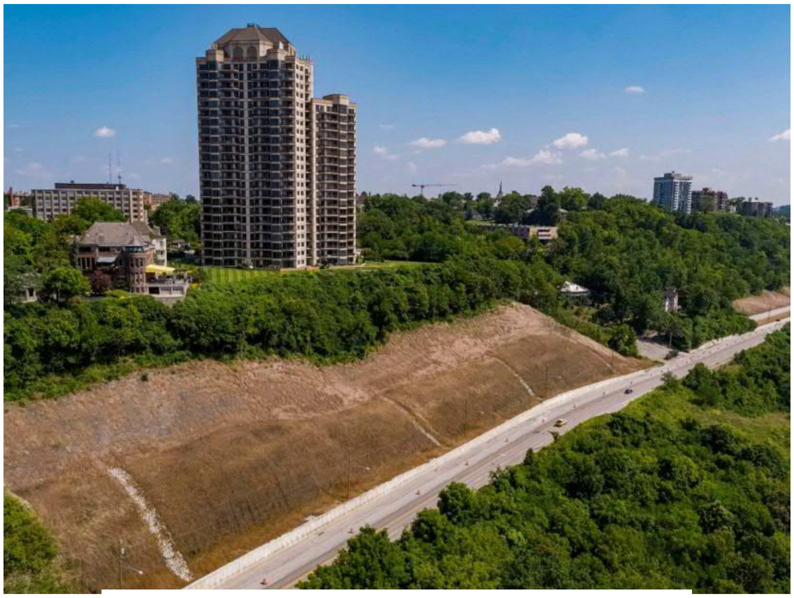
Columbia Parkway Landslide Stabilization: Areas 5 & 6



July 26, 2021
(Photograph Courtesy of City of Cincinnati)

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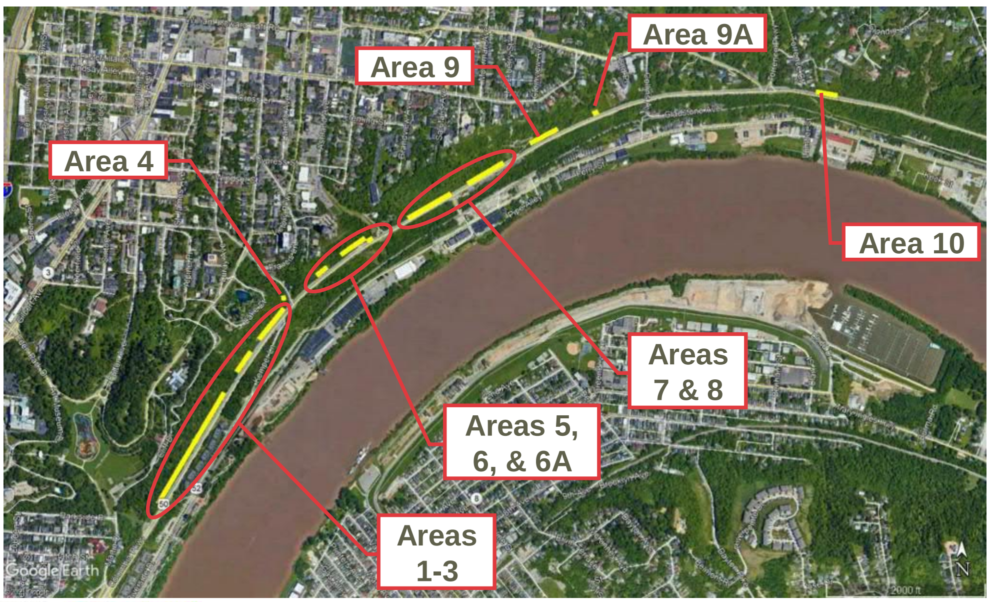
Columbia Parkway Landslide Stabilization: Areas 1-3



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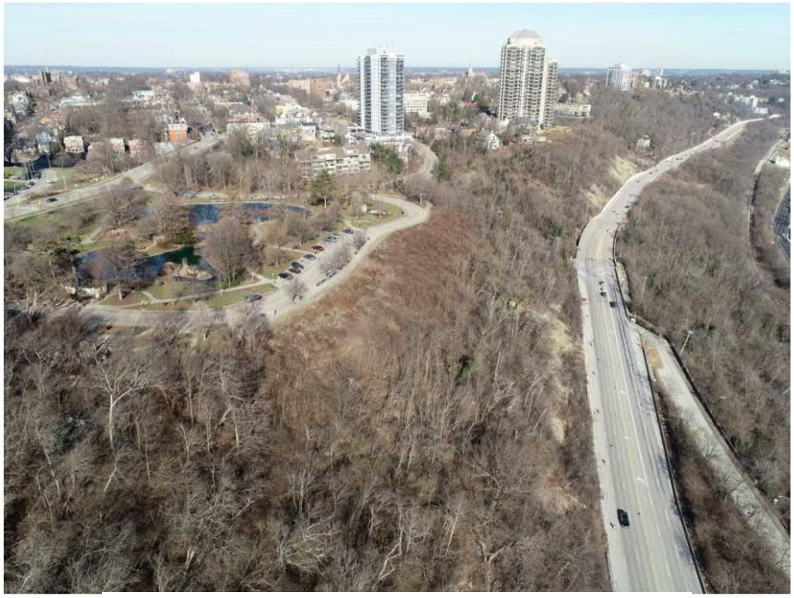
Columbia Parkway Landslide Stabilization: Areas 1-3



January 6, 2020
(Photograph Courtesy of Beaver Excavating Company)

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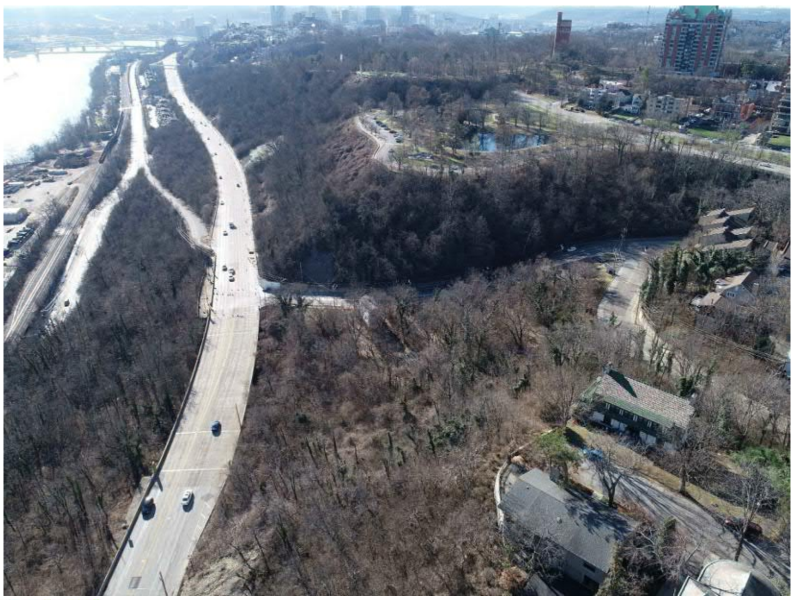
Columbia Parkway Landslide Stabilization: Areas 1-3



January 6, 2020
(Photograph Courtesy of Beaver Excavating Company)

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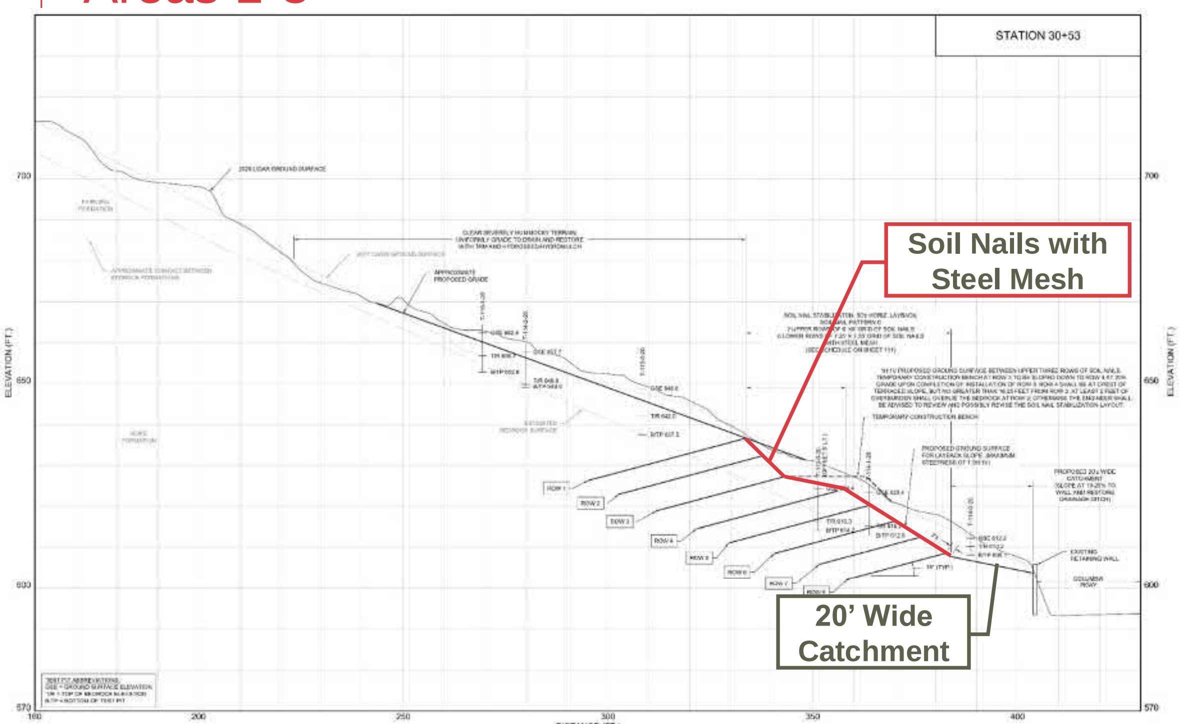
Columbia Parkway Landslide Stabilization: Areas 1-3

May 15, 2019

August 14, 2019

69

Columbia Parkway Landslide Stabilization: Areas 1-3



Columbia Parkway Landslide Stabilization: Areas 1-3

(Photograph Courtesy of Beaver Excavating Company)

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Columbia Parkway Landslide Stabilization: Areas 1-3

January 5, 2021

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Columbia Parkway Landslide Stabilization: Areas 1-3

January 5, 2021

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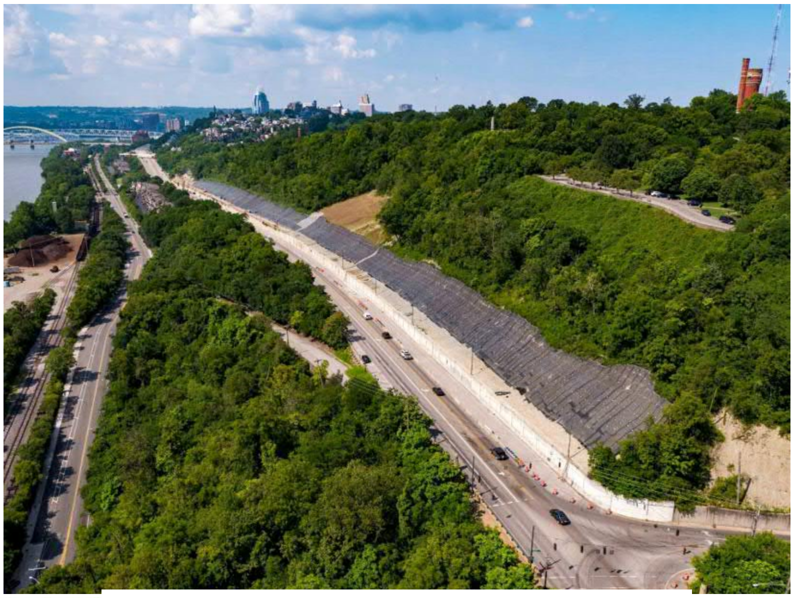
Columbia Parkway Landslide Stabilization: Areas 1-3



July 26, 2021
(Photograph Courtesy of City of Cincinnati)

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Verification Tests

- Verification tests were completed prior to soil nail construction to confirm design bond strength in bedrock.
- Design nominal grout-to-ground bond strength = 10 psi
- Verification tests were cyclically performed to evaluate 10, 15, and 20 psi bond strengths
- Verification test locations:
  - Three in Area 9A
  - Six in Areas 5 & 6
  - Five in Areas 1 through 3

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Verification Test Setup

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Verification Test Setup

(Photograph from FHWA GEC 007, Figure 9.1)

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Verification Test Results: Areas 1 through 3

Cycle No.	Nominal Bond Strength (psi)	Allowable Bond Strength (psi)
1	10	5
2	15	7.5
3	20	10

Test No.	Cycles Passed
VT-101	1, 2, & 3
VT-102	1
VT-103	1 & 2
VT-104	Damaged, Not Tested
VT-105	1 & 2
VT-106	1 & 2

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Verification Test Results: Areas 5 & 6

Cycle No.	Nominal Bond Strength (psi)	Allowable Bond Strength (psi)
1	10	5
2	15	7.5
3	20	10

Test No.	Cycles Passed
VT-5A	2 & 3*
VT-6A	2 & 3*
VT-6B	2 & 3*
VT-501	1, 2, & 3
VT-502	1 & 2
VT-503	1 & 2

* Cycle 1 was omitted because load tests were completed in gray shale at the toe of the slope in the catchment area, and did not encounter the weathered bedrock.

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Verification Test Results: Area 9A

Cycle No.	Nominal Bond Strength (psi)	Allowable Bond Strength (psi)
1	10	5
2	15	7.5
3	20	10

Test No.	Cycles Passed
VT-1	3*
VT-2	1 & 2
VT-3	1 & 2

* Cycles 1 & 2 were inadvertently omitted as the wrong ram was being used for the first load cycle, which actually exceed the load required for Cycle 3.

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Construction Issues

February 1, 2021 – Slump Near Station 29+00 of Areas 1-3

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Construction Issues

February 1, 2021 – Slump Near Station 29+00 of Areas 1-3

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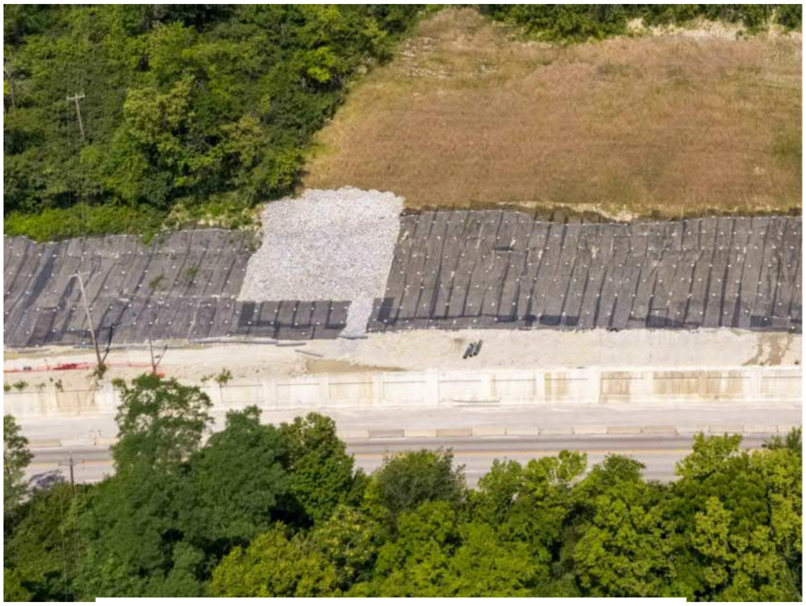
Construction Issues – Riprap Repair



July 26, 2021
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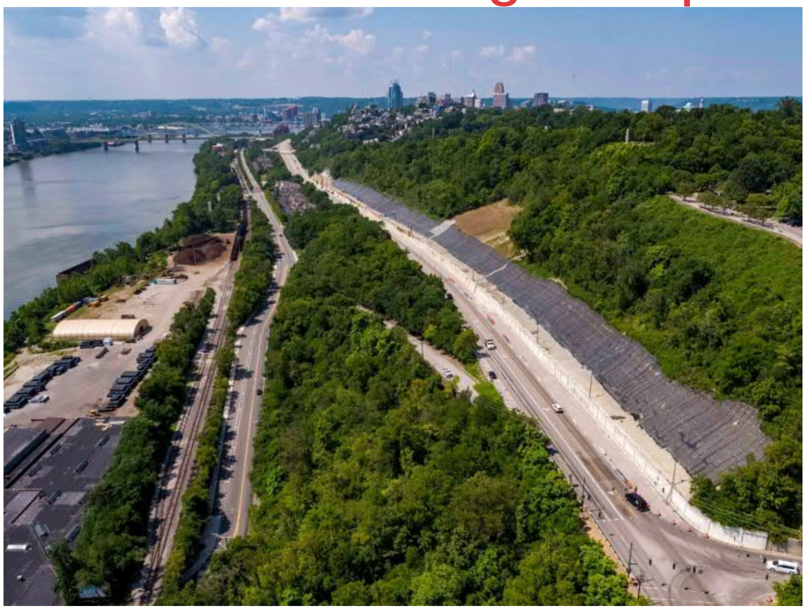
Columbia Parkway Landslide Stabilization: Nearing Completion



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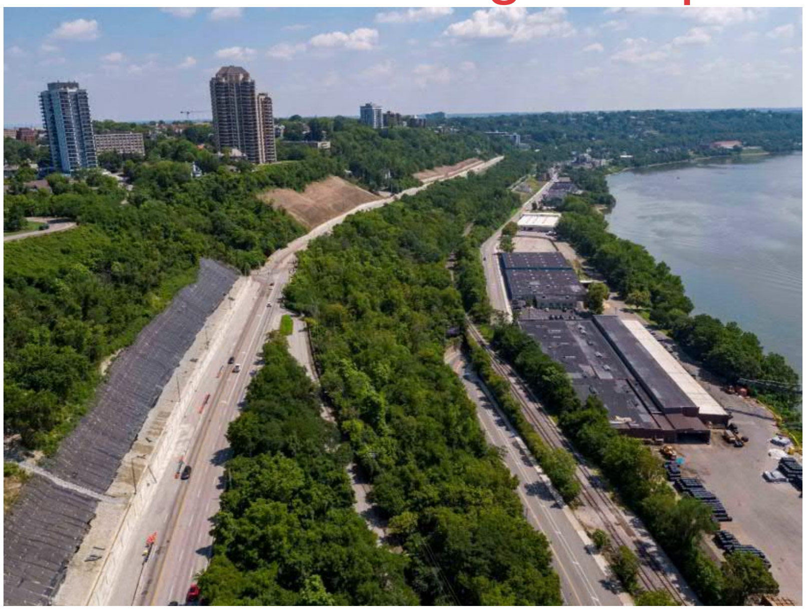
Columbia Parkway Landslide Stabilization: Nearing Completion



July 26, 2021
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Thank You

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Liquefaction, Lateral Spreading, and Pile Foundations in the Ohio Valley

Author: Enrique Farfan, PhD, PE, M ASCE

Abstract: The U.S. Geological Survey (USGS) Long-term National Seismic Hazard Map shows two main zones where the level of hazard for seismic events is considered high. One of the areas includes the States of California, Oregon, and Washington on the West Coast. And the other main area, includes the States of Illinois, Indiana, Kentucky, Tennessee, Missouri, and Arkansas, at the Ohio River Valley.

The New Madrid Seismic Zone (NMSZ) in the Ohio River Valley has the potential to develop earthquakes with magnitudes between 7 and 8 on the Richter's scale. Reports indicate that this seismic zone could develop peak ground accelerations (PGA) greater than 1.2g. Saturated and partially saturated soils could substantially lose strength and stiffness during an earthquake in the Ohio Valley, which represents an important geohazard for this zone. For river-spanning bridges, liquefaction-induced flow failure or lateral spreading on the bridge foundations need to be assessed as part of the seismic design efforts to ensure the performance of the structure during a significant earthquake event.

The analysis of lateral spreading is separated into two cases, in the first case, it is assumed that the foundation does not provide any resistance to lateral soil movement, and the second case assumes that the foundation limits the soil deformation during lateral spreading. This paper provides an overview of the method proposed by Caltrans and the latest recommendations provided by Deep Foundation Institute (DFI) to evaluate the forces and ground movement developed during the interaction of the soil and the structure during a lateral spreading. The US 60 Bridge Replacement over the Cumberland River in Livingston County is discussed as an example of lateral spreading analysis in the Ohio River Valley.

Introduction

Currently, the Ohio River Valley is referred to encompass parts of the States of Virginia, Pennsylvania, Ohio, Indiana, and Kentucky, which are associated with the Ohio River basin. In the Ohio River Valley, the area with a recurrent seismic activity is located at the west side of the basing, at the border between Missouri, Kentucky, Arkansas, and Tennessee. This area denominated New Madrid Seismic Zone (NMSZ) comprises a series of deep-seated faults covering an area of 45 miles wide and approximately 125 miles long (MDNR, 2021).

In the NMSZ a series of earthquakes were recorded with magnitudes greater than 7.0, with hundreds of earthquakes with magnitudes between 5.0 to 6.5 and thousands with magnitudes between 5.0 and 4.0. The area in general experiences about 200 earthquakes per year (SEMA, 2021).

The peak ground accelerations (PGA) in the NMSZ vary from 0.04g to 1.20g according to the national seismic hazard map (Petersen et al, 2011). A PGA greater than 0.1g could result in soil liquefaction wherever potentially liquefiable soils are present (NASEM, 2016) (see Figure 1). The softening of the soil due to the increase in excess pore water pressures during shaking could result in ground vertical and lateral displacement, triggering landslides, ground distortion, and lateral spreading compromising the safety of any engineering infrastructure.

The liquefaction potential areas in the NMSZ susceptible to significant earthquake activities are concentrated along the flood plains, where deposition of saturated alluvial material is concentrated as shown in the liquefaction hazard map that covers the MNSZ areas (Cramer et al, 2016).

Bridge abutments and bents could be at risk in these areas where lateral spreading could result from soil liquefaction.

Soil Liquefaction

Earthquake-induced soil liquefaction occurs when porewater pressures increase due to the cycling loading induced by a seismic event, resulting in the collapse of the soil structure, contraction, and densification of the material. The loss of strength of the soil due to liquefaction could result in large ground deformation and loss of the soil capacity to support overlying loads (NASEM, 2016).

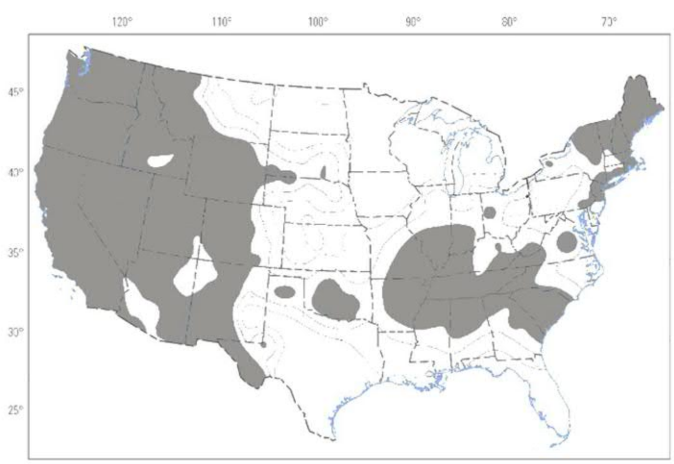

Figure 1. Potential areas for soil liquefaction (modified from Petersen et al., 2014)

The term soil liquefaction is used to refer to *flow liquefaction* and *cyclic softening*. Flow liquefaction occurs mainly on cohesionless granular material where the pore pressures increase to the point that the static equilibrium is destroyed, and a strain-softening occurs resulting in low residual strengths (Robertson, 2010). Due to the progressive mechanism of the load distribution, ground deformation tends to occur after the cyclic loading has ceased. Visual features such as sand boils, flow failure slopes, the buoyant rise of buried structures, ground settlement, and failure of retaining wall due to an increase in lateral loads could be observed in places when flow liquefaction has occurred.

There is a tendency in engineering practice to become distracted by the presence of flow liquefaction (classic liquefaction) and neglect the potential of cyclic softening on cohesive material (Seed et al, 2003). Cyclic softening generally occurs under cyclic undrained loading, where the deformations develop incrementally as a result of the static and dynamic stresses during a seismic event. The effective overburden stress can reach zero during cyclic loading leading to a reduction or loss of soil stiffness (Robertson and Wride, 1998). Cyclic softening is associated with fine-grained soils, and the consequences could be similar to flow liquefaction where the loss of soil strength leads to flow failure slopes and lateral spreading.

The terms sand-like and clay-like were adopted by Boulanger and Idris (2004) to identify the behavior of silts and clays under undrained cyclic loading. Fine-grain soils could exhibit a sand-like response, where the material experience large deformation due to the reduction in soil strength. For fine-grain soil, the increase in pore pressures is the result of the tendency of the material to contract. The clay-like response relates to a cyclic softening, where the material could exhibit a tendency to dilate and therefore a reduction in the soil strain due to the increase in pore pressure. Fine-grain soils with low-plasticity such as silts and clays that are near the transition between the general behaviors, sand-like or clay-like, evaluation could be more challenging (Boulanger and Idriss, 2004).

Evaluating Soil Liquefaction

Several methodologies to evaluate the potential for soil liquefaction were developed during the last 50 years. Given the complexity to determine if soil could be subject to liquefaction during a seismic event, simplified approaches have been developed since Witman (1971) and Seed and Idriss (1971). In 2001, The National Science Foundation and the National Center for Earthquake Engineering Research at the University of New York at Buffalo sponsored a workshop where a consensus on the methodology to evaluate soil liquefaction was achieved. As a result of the workshop, a technical paper by Youd and colleagues (2001) was issued to provide the framework for a standardized methodology to determine the triggering conditions for soil liquefaction. The set of techniques and recommendations in the technical paper focused on using data from the Standard Penetration Test (SPT), the Cone Penetration Test (CPT), and the Becker Penetration Test (BPT).

In subsequent years to Youd et al (2001), new methodologies to assess liquefaction were developed by Boulanger and Idriss (2004) and Cetin et al (2004). Different opinions also were expressed in the scientific community (Seed, 2010), generating some controversies. Specific methodologies for SPT and CPT were presented by Idriss and Boulanger (2010) and Boulanger and Idris (2014) respectively.

The contributions of Robert Peterson on the use of CPT to assess soil liquefaction have been very important. The use of the CPT SBT (soil behavior type) chart by Robertson (2010) to map liquefaction and cyclic softening potential, provides a means to visualize the spread and concentration of soil layers that are susceptible to liquefaction.

A recent publication in 2018 by The National Academies of Science, Engineering, and Medicine, provides the latest review on the art and practice in earthquake-induced soil liquefaction assessment. The report provides a review of the theoretical background, a discussion on different methodologies to assess liquefaction triggering, consequences of liquefaction, and a discussion on constitutive models.

Preliminary screenings for soil liquefaction are provided by different public agencies in their guidance documents. SECE (1999) recommends that liquefaction assessments are not required if the corrected SPT blow count $(N_1)_{60}$ is greater than 30. AASHTO (2014) suggests that a liquefaction assessment should be performed if the $(N_1)_{60}$ is less than 25, the corrected CPT tip resistance (q_{ciN}) is less than 150 or the normalized shear velocity (Vs1) is less than 660 fps (200m/s). FHWA (2011) indicates that significant soil liquefaction hazard in sands does not exist if the $(N_1)_{60}$ is greater than 30 or the corrected CPT tip resistance (q_{ciN}) is minimum equal to 160.

For cohesive soils, Boulanger and Idriss (2006) indicate that soils are not susceptible to liquefaction if plastic index (PI) ≥ 7. While Bray and Sancio (2006) assumed that the soil is susceptible to liquefaction if the PI < 12 and the ratio of water content to liquid limit (wc/LL) > 0.85.

Since most of the observed soil liquefaction ranged to depths less than 50 feet to 60 feet, the current simplified liquefaction methods are applicable to this depth range. According to AASHTO (2014) soils in the upper 75 feet could be susceptible to soil liquefaction. SECE (1999) indicates that soil liquefaction could occur on soil layers in the upper 40 feet for given conditions. Nevertheless, this depth limitation is in reference to the current methodologies. Soil liquefaction could occur beyond 75 feet, but more site-specific analyses are required to be performed such as soil laboratory testing.

Soil Liquefaction in the Ohio Valley

Several exploration borings were performed to characterize the underground conditions for the design of the new Smithland Bridge, located north of Smithland, Kentucky, approximately half a mile east of the confluence of the Cumberland and Ohio Rivers. The subsurface conditions are characteristic of Alluvium and Terrace deposits of the Pleistocene age underlain by Mississippian age bedrock, which typically consists of cyclic sequences of sandstones, siltstones, shale, and limestone beds (Stantec, 2020).

The liquefaction analyses were performed using the procedures outlined in Robertson (2008), Idriss and Boulanger (2008), and Boulanger and Idriss (2014). A PGA = 0.22g with an Mw = 7.7 was considered for the project site. The available laboratory and field classifications indicate that the Upper Silty Sand is sand-like and liquefiable.

CPT data was used to determine if each sample was clay-like (i.e., potentially susceptible to cyclic softening) or sand-like (potentially susceptible to classic liquefaction). Liquefaction and cyclic softening triggering analyses were performed at each CPT sounding and SPT boring location. Residual strengths were required for liquefied materials in the lateral spreading analyses, post-earthquake shear strengths were assigned using the empirical correlation from Idriss and Boulanger (2008).

The liquefaction analysis methodologies were implemented in a spreadsheet that provides the capability to plot the analysis results versus depth, which helps to visualize and correlate with the idealized soil profile. An example of the results of the soil liquefaction analysis of borings near the riverbank to the Cumberland River is shown in Figure 2. The first column represents the idealized soil profile; the second column shows the summary of the liquefaction analysis, which is the combination of the other adjacent column; the third column shows the factor of safety against liquefaction along with the soil profile per Idriss and Boulanger (2008) and Boulanger and Idriss (2014); the fourth column represents the soil behavior based on Robertson (2008) procedure. A PGA = 0.5 was considered for the project site; the fifth plot shows the liquefaction evaluation using SPT data.

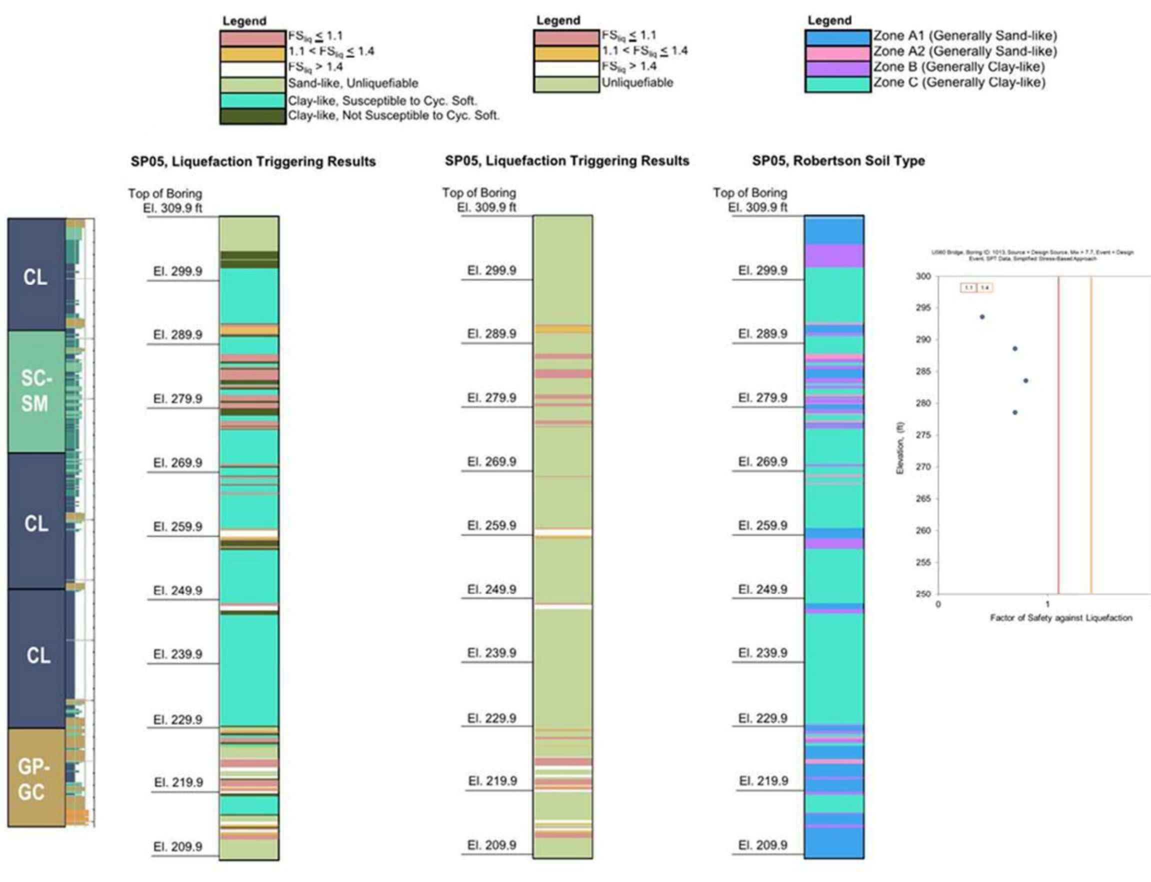

Figure 2. Soil liquefaction analysis results

A portion of the upper silty sand layer was judged to liquefy in the design seismic event and a residual strength ratio of $S_{liq}/\sigma'_v = 0.09$ was estimated for this material.

A cross-section at the riverbank with the idealized soil profile for the foundation design is illustrated in Figure 3.

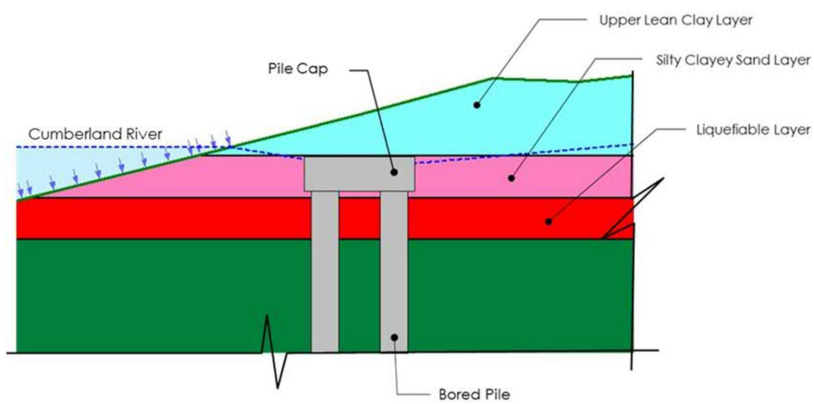

Figure 3. Cross-section and soil profile for Pier 4.

Lateral Spreading

Liquefaction-induced lateral spreading is defined as the finite, lateral displacement of (gently) sloping ground as a result of pore pressure build-up or liquefaction in a shallow underlying deposit during an earthquake (Rauch and Martin, 2000) (see Figure 4). Increments of the lateral displacement of the overlaying non-liquefiable soils sliding over the weaker liquefiable layer occur each time that the acceleration exceeds its yield acceleration. Case histories on lateral spreading also include slopes with gradients as flat as 0.5% (NASEM, 2016), therefore an assessment of lateral spreading potential cannot be only based on the slope gradient.

The lateral spreading could result in permanent ground deformations and damages to pile foundations. In the Ohio Valley, there is not a historical record in modern days of lateral spreading, nevertheless, the lack of a historical record does not guaranty that this phenomenon will not occur in the future. The lateral spreading analysis presented herein shows the potential for ground displacement and potential effects on bridge foundations.

The evaluation of lateral spreading, where the ground is mechanically restrained due to the complex seismic soil-foundation-structure interaction mechanism, has been proved to be difficult (Caltrans, 2020). The following methods are considered to estimate the lateral spreading displacement: (1) case-history-based empirical correlations; (2) integrations of the shear-strains in the liquefiable soil layer; (3) sliding block analysis; and (4) numerical simulations (Idris and Boulanger, 2008).

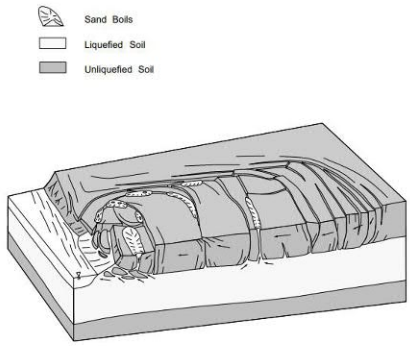

Figure 4. Schematic depiction of lateral spreading due to soil liquefaction (Rauch and Martin, 2000) [Reproduced with permission of the author].

The effects of a deep foundation on the ground displacements can be incorporated with the sliding block analysis method (Newmark and or numerical simulations. Numerical simulations require to evaluate the appropriate constitutive models for liquefying soils and validation of the earthquake ground motion through the soil profile.

The department of transportation from the states of California (Caltrans) and Washington (WSDOT) provide the same guidelines to assess the loads on a bridge pile foundation due to lateral spread during an earthquake. Both guidelines are based on using a limit equilibrium analysis to estimate the forces developed in the piles. The procedure is based on the guideline developed by the NCHRP (2002). The analysis comprises of the following steps:

1. Assess Liquefaction Potential
2. Estimate Residual Strength of Liquefied Soils
3. Develop Foundation Model
4. Displacement Analysis of Foundation Model
5. Slope Stability and Deformation Analysis Approach Embankment
6. Determine Force-Displacement Compatibility
7. Assess Foundation Performance

The steps are briefly described below:

Assess Liquefaction Potential and Estimate Residual Strength of Liquefied Soils

The assessment of soil liquefaction was discussed in previous sections. The evaluation of a boring located at the riverbank of the Cumberland River was presented to illustrate the results of the soil liquefaction analysis.

Develop Foundation Model

The development of the foundation model (see Figure 5) requires that the foundation geometry, pile diameters, pile spacings, pile configuration, and the geometry of the pile cap be defined. The interaction between the pile and the soil is modeled using the p-y curves; computer programs such as LPILE by Ensoft (2019) can be used to implement a numerical model to evaluate pile to lateral ground displacements. Group effects could be incorporated in the numerical model to represent the interaction in the pile group.

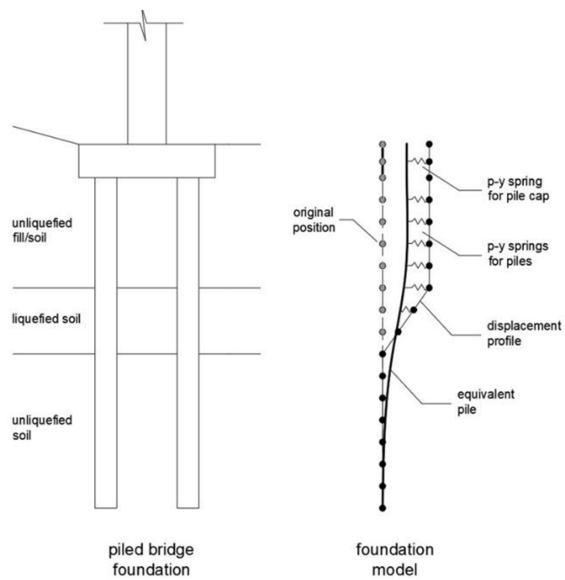

Figure 5. Representation of the physical bridge foundation by the foundation model showing the displacement profile for lateral spreading pushover analysis.

The stiffness of the pile cap can be determined using the formulations developed by Duncan and Mokwa (2001). The relationship between force and displacement is represented with a hyperbolic curve as shown in Figure 6. It is assumed that the maximum force experienced by the pile cap is equal to the passive force that can be developed by the soil.

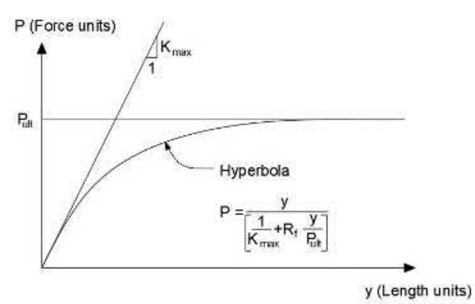

Figure 6. Hyperbola load-deflection curve (Duncan and Mokwa, 2001)

Displacement Analysis of Foundation Model

Once the foundation model is developed, a series of pushover analyses are performed to develop a relationship between force and displacement at the top of the pile.

Slope stability and Deformation Analysis Approach Embankment

The contribution of the pile foundation to retrain the movement of the embankment is performed by calculating the yield acceleration for different shear forces developed in the piles that contribute as stabilization forces. The relationship between yield acceleration and slope displacement is performed using a Newmark rigid sliding block analysis (Newmark, 1964).

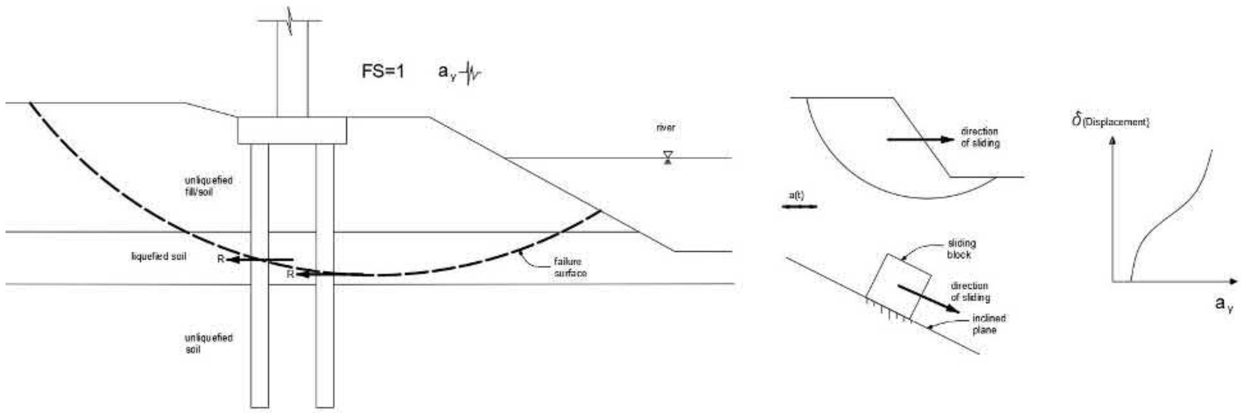

Figure 7. Schematic of slope stability analysis and Newmark Block analysis

As a result of these two analyses, a curve can be graphed that represents the relationship between slope displacements and shear forces developed in the piles.

Determine Force-Displacement Compatibility

The compatibility force-displacement that contributes to the stability of the slope and shear forces developed in the pile due to the slope deformation is determined by superposing the results from the pushover and the slope stability/deformation analysis in a single plot. Since the shear force calculated in the slope stability analysis represent a force per unit width, this force needs to be scaled to the effective width of the group of piles.

The intersection of the two plots represents the compatibility point where the force and displacement represent the design shear force on the pile.

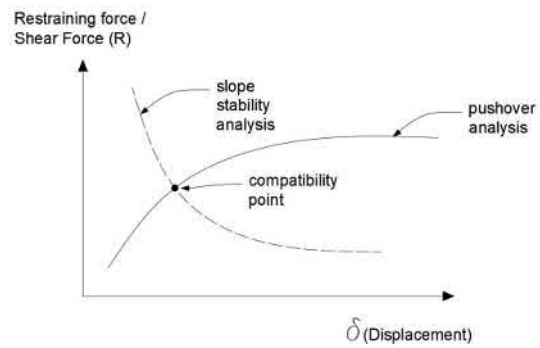

Figure 8. Determination of compatible force-displacement state

Assess Foundation Performance

The performance of the foundation is evaluated by combining the lateral loads from the kinematic and inertial loads from the structure. The combination of inertial and kinematic demands per different design guidelines are summarized in Table 1.

Table 1. Design guidelines on the combination of inertial and kinematic demands on piles

Design code	Recommendation
ASCE 61-14 Port of Long Beach	Resultant moments are spaced do not need to superimpose.
Port of Anchorage Modernization Program Seismic Design Manual (POA 2017)	100% kinematic demand (no less than 25% if peer-review is performed)
AASHTO (2014)	100% kinematic + 100% inertial if M > 8
MCEER/ATC (2003)	Independent effects
PEER (2011)	100% kinematic + (65% to 85%) inertia (multiply by .35 to 1.4 to account for the effects of liquefaction on peak inertial load)
Caltrans (2012) and ODOT (2014)	100% kinematic + 50% inertia
WSDOT (2015)	100% kinematic + 50% inertia

Souri et al (2019, 2021) after reviewing large-scale centrifuge tests in pile supported wharves, developed the following recommendations to access the foundation performance (Table 2):

Table 2. Proposed load combinations for design piles subject to combined inertial load and kinematic load from lateral ground deformations. Souri et al (2019, 2021)

Case	Load combination	Portion of permanent soil displacements applied at end nodes of p-y springs	Portion of peak deck inertial force applied at deck	Applicability
A	Inertia only	NA	100%	Moment at pile head
B1 [deep-sated liquefaction underlying significant nonliquefiable crust]	Kinematic + inertia	100%	0.3 to 0.6	Bending moment below grade down to depth 10D
B2 [small kinematic demands/loads]	Kinematic + inertia	100%	0.9 to 1.0	Bending moment below grade down to depth 10D
C	Kinematic only	100%	NA	Bending moment deeper than 10D

Application of Lateral spreading Analysis

The new bridge over the Cumberland River is a steel truss bridge with a total length of 1,900 ft (10 spans). The main span is 700 ft, completely spanning across the river between piers 3 and 4, and a maximum vertical clearance of 86.5 feet. The piers supporting the main span, comprised of eight 8'-drilled shafts with an average pile length of 96 ft, socketed into the limestone approximately 19 ft. The pile spacing is 3 times the pile diameters and the piles are connected with a 40ft x 88ft rectangular pile cap.

There is potential for permanent ground displacement toward the Cumberland River as a result of the seismic-induced movement at Pier 4 (north riverside) under a strong earthquake shaking. Ground movements at Pier 4 could impact the foundation of one of the main bridge span supports and required evaluation to assess the effects of the potential lateral ground movements on the piles.

Slope stability analyses were performed for the slope of the riverbank at Pier 4 toward the Cumberland River to evaluate the static and seismic stability of the slope and whether the slope instability would affect the foundation of the new bridge. Seismic stability and deformation were assessed based on pseudo-static stability analysis and the Newmark displacement method, respectively.

Post-earthquake soil strengths included liquefied strengths were assigned to idealized sand-like layers susceptible to liquefaction, and softened strengths were assigned to idealized clay-like layers susceptible to cyclic softening.

Factor of safety (FOS), yield acceleration, and Newmark displacement were determined using a two-dimensional limit equilibrium slope stability analysis program Slope/W (GeoStudio 2019), using Spencer's (1967) method of analysis. Newmark displacements were calculated using the earthquake time histories series developed by Wang et al (2008) that are available at the Kentucky Transportation Center (KTC) website.

The analysis section through Pier 4 foundation is along the alignment of the new bridge. Assessment of potential foundation movement was undertaken using the concepts presented in the National Cooperative Highway Research Program (NCHRP) Report 475 (2002) and Caltrans MTD 20-15 (2017) to determine the expected displacement demand on the shafts. The pinning forces from the shafts were incorporated when assessing the failure surfaces extending through Pier 4 shafts. The computer program LPile (Endsoft, 2019) was used to evaluate the typical shaft response due to lateral soil movement. Stability analysis incorporating shaft pinning forces and the resulting design plot indicating compatible ground movement and shear forces mobilized. The pinning forces from the shafts were incorporated in the stability analysis as vertical elements with an equivalent shear resistance equal to the shear resistance by each shaft divided by the shaft spacing out-of-plane of the model.

The evaluation of the pinning effect procedure is summarized below:

1. Perform Slope/W analysis, incorporate shaft representative shear forces and determine ky for equivalent pile shear force.
2. Perform Newmark sliding block analysis to calculate soil displacements for the values of ky defined in Step 1.

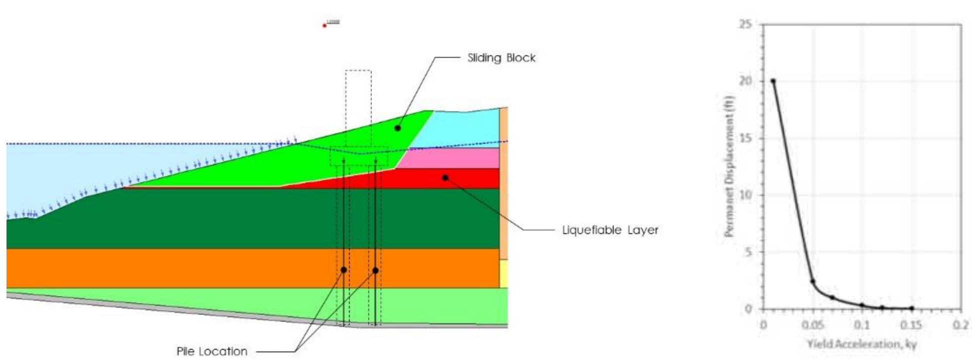

Figure 9. Slope Stability and Newmark displacement analysis

3. Perform LPile analysis to determine shaft shear forces for a range of soil displacements.
4. Plot results from steps 2 and 3. The intersection of these two curves represents the expected displacement and demand shear on the piles at Pier 4.

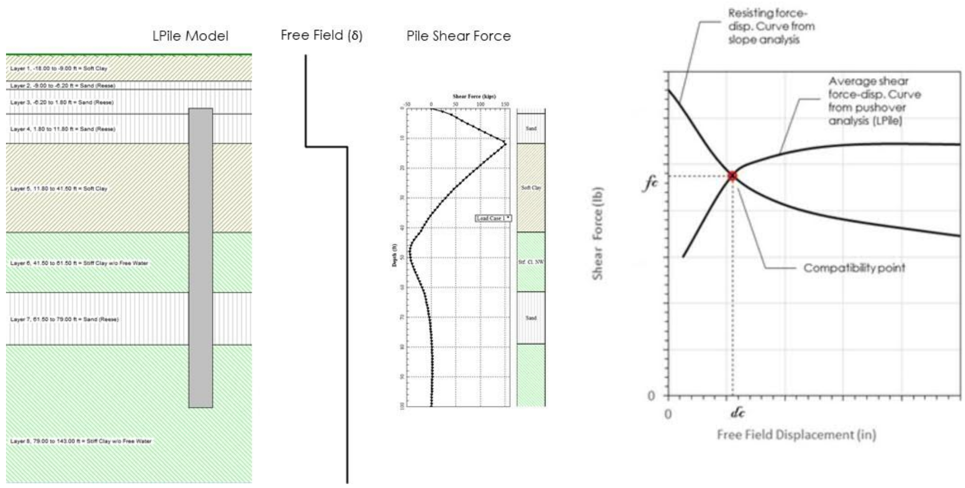

Figure 10. Pile lateral analysis and compatibility point (displacement vs shear forces)

The resulting shaft forces due to the pinning effect were provided to the Structural Engineer for its consideration in the structural design of the drilled shafts. The load combination selected for the design was based on the recommendation by Caltrans and Souri et al (2019, 2021).

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CHRONOLOGY OF OHIO RIVER VALLEY SOIL SEMINARS

ORVSS I	BUILDING FOUNDATION DESIGN AND CONSTRUCTION, October 16, 1970, Lexington, KY
ORVSS II	EARTHWORK ENGINEERING, START TO FINISH, October 15, 1971, Louisville, KY
ORVSS III	LATERAL EARTH PRESSURES, October 27, 1972, Fort Mitchell, KY
ORVSS IV	GEOTECHNICS IN TRANSPORTATION ENGINEERING, October 5, 1973, Lexington, KY
ORVSS V	ROCK ENGINEERING, October 18, 1974, Clarksville, IN
ORVSS VI	SLOPE STABILITY AND LANDSLIDES, October 17, 1975, Fort Mitchell, KY
ORVSS VII	SHALE AND MINE WASTES: GEOTECHNICAL PROPERTIES, DESIGN, AND CONSTRUCTION, October 8, 1976, Lexington, KY
ORVSS VIII	EARTH DAMS AND EMBANKMENTS: DESIGN, CONSTRUCTION AND PERFORMANCE, October 14, 1977, Louisville, KY
ORVSS IX	DEEP FOUNDATIONS, October 27, 1978, Fort Mitchell, KY
ORVSS X	GEOTECHNICS OF MINING, October 5, 1979, Lexington, KY
ORVSS XI	EARTH PRESSURE AND RETAINING STRUCTURES, October 10, 1980, Clarksville, IN
ORVSS XII	GROUNDWATER: MONITORING, EVALUATION AND CONTROL, October 9, 1981, Fort Mitchell, KY
ORVSS XIII	RECENT ADVANCES IN GEOTECHNICAL ENGINEERING PRACTICE, October 8, 1982, Lexington, KY
ORVSS XIV	FOUNDATION INSTRUMENTATION AND GEOPHYSICAL EXPLORATION, October 14, 1983, Clarksville, IN
ORVSS XV	PRACTICAL APPLICATION OF DRAINAGE IN GEOTECHNICAL ENGINEERING, November 2, 1984, Fort Mitchell, KY
ORVSS XVI	APPLIED SOIL DYNAMICS, October 11, 1985, Lexington, KY
ORVSS XVII	NATURAL SLOPE STABILITY AND INSTRUMENTATION, October 17, 1986, Clarksville, IN
ORVSS XVIII	LIABILITY ISSUES IN GEOTECHNICAL ENGINEERING AND CONSTRUCTION, November 6, 1987, Fort Mitchell, KY
ORVSS XIX	CHEMICAL AND MECHANICAL STABILIZATION OF SOIL SUBGRADES, October 21, 1988, Lexington, KY
ORVSS XX	CONSTRUCTION IN AND ON ROCK, October 27, 1989, Louisville, KY
ORVSS XXI	ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING, October 26, 1990, Fort Mitchell, KY
ORVSS XXII	DESIGN AND CONSTRUCTION WITH SYNTHETICS, October 18, 1991, Lexington, KY
ORVSS XXIII	IN-SITU SOIL MODIFICATION, October 16, 1992, Louisville, KY
ORVSS XXIV	GEOTECHNICAL ASPECTS OF INFRASTRUCTURE RECONSTRUCTION, October 15, 1993, Fort Mitchell, KY
ORVSS XXV	RECENT ADVANCES IN DEEP FOUNDATIONS, October 21, 1994, Lexington, KY
ORVSS XXVI	SITE INVESTIGATIONS: GEOTECHNICAL AND ENVIRONMENTAL, October 20, 1995, Clarksville, IN
ORVSS XXVII	FORENSIC STUDIES IN GEOTECHNICAL ENGINEERING, October 11, 1996, Cincinnati, OH

CHRONOLOGY OF OHIO RIVER VALLEY SOIL SEMINARS (CONTINUED)

ORVSS XXVIII	UNCONVENTIONAL FILLS: DESIGN, CONSTRUCTION AND PERFORMANCE, October 10, 1997, Lexington, KY
ORVSS XXIX	PROBLEMATIC GEOTECHNICAL MATERIALS, October 16, 1998, Louisville, KY
ORVSS XXX	VALUE ENGINEERING IN GEOTECHNICAL CONSULTING AND CONSTRUCTION, October 1, 1999, Cincinnati, OH
ORVSS XXXI	INSTRUMENTATION, September 15, 2000, Lexington, KY
ORVSS XXXII	REGIONAL SEISMICITY AND GROUND VIBRATIONS, October 24, 2001, Louisville, KY
ORVSS XXXIII	GROUND STABILIZATION AND MODIFICATION, October 18, 2002, Covington, KY
ORVSS XXXIV	APPLICATIONS OF EARTH RETAINING SYSTEMS AND GEOSYNTHETIC MATERIALS, September 19, 2003, Lexington, KY
ORVSS XXXV	ROCK ENGINEERING AND TUNNELING, October 20, 2004, Louisville, KY
ORVSS XXXVI	GEOTECHNICAL INNOVATIONS IN TRANSPORTATION ENGINEERING, October 14, 2005, Covington, KY
ORVSS XXXVII	INNOVATIONS IN EXPLORATION OF SUBSURFACE VOIDS, October 27, 2006, Lexington, KY
ORVSS XXXVIII	CIVIL INFRASTRUCTURE AND THE ROLE OF GEOTECHNICAL ENGINEERING, November 14, 2007, Louisville, KY
ORVSS XXXIX	URBAN CONSTRUCTION, October 17, 2008, Covington, KY
ORVSS XL	GEOTECHNICAL ENGINEERING AND ENERGY INFRASTRUCTURE, November 13, 2009, Lexington, KY
ORVSS XLI	NATIONAL INFRASTRUCTURE: DAM AND LEVEE SAFETY, October 20, 2011, Louisville, KY
ORVSS XLII	LESSONS LEARNED: FAILURES AND FORENSICS, October 21, 2011, Cincinnati, OH
ORVSS XLIII	WALLS: ABOVE AND BELOW GRADE, November 19, 2012, Lexington, KY
ORVSS XLIV	THE APPLICATION OF GEOLOGY TO GEOTECHNICAL ENGINEERING PRACTICE, November 15, 2013, Louisville, KY
ORVSS XLV	GEOTECHNICAL ASPECTS OF WATERFRONT DEVELOPMENT, October 17, 2014, Cincinnati, OH
ORVSS XLVI	GROUTING SOLUTIONS TO GEOTECHNICAL PROBLEMS, December 16, 2015, Lexington, KY
ORVSS XLVII	GEOTECHNICAL ASPECTS OF THE LOUISVILLE-SOUTHERN INDIANA OHIO RIVER BRIDGES PROJECT, November 16, 2016, Louisville, KY
ORVSS XLVIII	INFRASTRUCTURE INNOVATION IN GEOTECHNICAL DESIGN, November 17, 2017, Cincinnati, OH
ORVSS XLIX	TOOLS FOR ASSESSING GEOTECHNICAL SITE CONDITIONS, November 28, 2018, Lexington, KY
ORVSS L	50 YEARS OF GEO-PROGRESS, November 13, 2019, Louisville, KY
ORVSS LI	GEOHAZARDS – CHALLENGES TO GEOTECHNICAL ENGINEERING, November 2, 2021, Cincinnati, OH