Ohio River Valley Soils Seminar XLV

Geotechnical Aspects of Waterfront Development

October 17, 2014

Horseshoe Casino
1000 Broadway Street
Cincinnati, Ohio

Planned Agenda – Friday, October 17, 2014

6:30-7:30 am	Exhibitor Registration and Setup
7:30-8:15 am	**Registration**
8:15-8:30 am	Welcome Remarks – Jon Huff, P.E., Richard Goettle, Inc.
8:30-9:05 am	"Ohio River and Tributary Local Protection Projects: Adjacent Bank Erosion and Failure-Related Endangerment after Action Report" – Brian W. Ball, P.E., LRH, U.S. Army Corps of Engineers – Huntington District
9:05-9:40 am	"SRT System Stabilizes Levee at Power Plant" – Miriam Smith, Ph.D., P.E., Geopier Foundation Company, Inc.
9:40-10:15 am	"Slope Stability Analysis under Rapid Drawdown Conditions" – Mark T. Bowers, Ph.D., P.E., University of Cincinnati
10:15-10:35 am	**Break**
10:35-11:10 am	"Use of Small Diameter Micropiles to Mitigate Scour around Bridge Embankments" – Nathan Beard, P.E., GeoStabilization International
11:10-11:45 am	"Scioto Greenways – Geotechnical Challenges of Development within an Existing Riverfront" – Rich Williams, Ph.D., P.E., Stantec Consulting Services Inc.
11:45-12:35 pm	**Lunch**
12:35-12:45 pm	Keynote Introduction – Jacqueline Harmon, P.E., Stantec Consulting Services Inc.
12:45-1:40 pm	**Keynote:** "The Evolution of Levee Safety within the U.S." – Tammy L. Conforti, P.E., U.S. Army Corps of Engineers
1:40-2:15 pm	"Several Relief Well Design Considerations for Dams and Levees" – Halis Ider, P.E., Thelen Associates, Inc.
2:15-2:45 pm	**Break**
2:45-3:20 pm	"Major Roadway Landslide Repair along the Rolling Fork River in West-Central Kentucky" – Adam Wynn Lewis, E.I.T., GeoStabilization International
3:20-3:55 pm	"Design and Construction of Underseepage Pressure Relief Wells along a Federal Levee" –Justin Anderson, P.E., HDR Engineering, Inc.
3:55-4:30 pm	"Relief Well Assessment and Design, Saving Taxpayer Dollars – Southwestern Illinois Levees" – Mary Knopf, P.E., AMEC
4:30-4:45 pm	**Closing Remarks**

Proceedings of the 45th Annual Ohio River Valley Soils Seminar

Geotechnical Aspects of Waterfront Development

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Sponsored by:

ASCE Cincinnati Section Geotechnical Group

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Joseph D. Hauber, P.E.
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Akshat Saxena, E.I.T.
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Donald B. Thelen, P.E.
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Ohio River and Tributary Local Protection Projects: Adjacent Bank Erosion and Failure-Related Endangerment after Action Report

Brian W. Ball, P.E.[1], Ashley M. Matheny, E.I., S.M. ASCE[2], Philip R. Hatfield, E.I.[3], Andrew M. Keffer, E.I.[4], and Michael F. Spoor, E.G.[5]

Abstract: After ten years of work and observations through the Ohio River erosion program and the inspection and monitoring process of the levee and flood wall systems, there is a clear opportunity for improved management of this critical infrastructure. The certified levees and flood wall systems on the Ohio River were constructed between the 1930's and the 1960's. These projects have prevented billions of dollars in flood damage and are still relevant and critical to the protection to the people, lands, and structures within their bounds.

At the time of construction of these levees and flood walls, the designers believed that there was sufficient stable land between the lines of protection and the adjacent river. Over time, these "buffer lands" have been eroded by natural processes. The prescribed operation and maintenance plans do not give enough guidance to provide timely and cost effective repairs to Local Protection Projects (LPPs) threatened by adjacent riverbanks erosion. In most cases, emergency riverbank repairs have been constructed after conditions have progressed from threatened to immediately endangered. This report provides guidance for making timely, cost effective decisions that protect critical infrastructure and save overstressed tax dollars.

Key words: Local Protection Project, Ohio River bank terraces, and stone slope protection.

Background

Regarding the requirement to conduct after action reports (AAR) upon project completion, the District determined monitoring, maintenance, and immediate endangerment-related emergency actions were necessary to determine timely applications by experienced staff during site specific evaluations and emergency responses. The Point Pleasant 84-99 Repair Project, the Massillon Emergency Levee Protection Project (LPP), and the Ceredo-Kenova Levee Flood Damage Recovery Protection Project all demonstrated engineering management capacities to make system wide applications of design and to prioritize response actions at these projects. Background, for each of these projects, includes erosion damage by flooding and related river bank failure, which

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threatened the stability of the LPPs. During the review of each of these projects, it was apparent that Charles Dickens' observations regarding bank erosion along the Ohio River are as relevant today as they were in 1842[1]: "The River has washed away its banks, and stately trees have fallen down into the stream. Some have toppled over, and having earth yet about their roots, are bathing their green heads in the river." Therefore, significant linear foot of treatment cost savings could have been achieved if river bank protections had been constructed before local protection projects become endangered by recent storm, high flow, and flood events.

The Problem

After evaluating bank topography adjacent to several of the Huntington District local protection projects, this report was prepared to identify reaches of threatened river bank that, with timely protection, would result in significant reductions in cost to construct. Referring to Point Pleasant as an example, if the river bank had been protected from initial erosion impacts, the cost of the 2011 upper bank protection project would have been reduced. These lower bank erosion features were located channelward of the project defined 15 foot maintenance and monitoring areas. Additional information, which was included during the development of these after action reports, established that the local entity responsible for the maintenance and operation of the LPP had not been obligated to maintain nor inspect beyond the 15 feet channelward area adjacent to the riverward toe or foundation of the structure or embankment (see Fig. 1). This monitoring limitation is significant since flood-related processes continue to erode adjacent river bank areas and thereby affect the stability of the local protection project embankment and structure foundations.

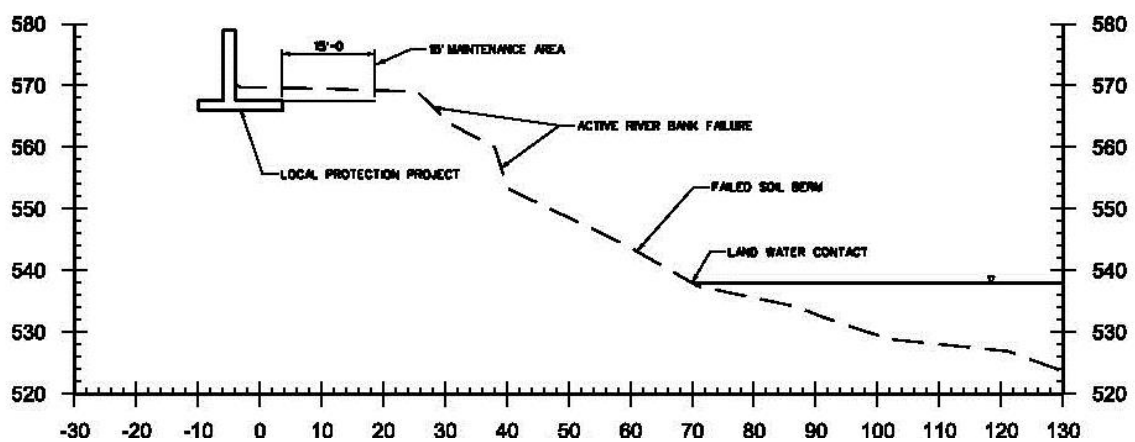

Fig. 1. Typical cross section displaying the LPP, foundation, 15-foot maintenance easement area, and riverward bank failure conditions

[1] Charles Dickens American Notes

Therefore, comprehensive monitoring requires that the adjacent river banks must be included in the levee inspection program, and evaluations and consequent designs would be necessary for construction of river bank protections. These projects must be prioritized and treatments constructed to reduce risk of subsequent LPP failures. Should treatments be deferred, the extents of treatment required per foot of stabilization would increase. The increase in stabilization cost per linear foot would be nonlinear and directly related to flood initiated extents of bank, terrace, and slope erosion (represented in Fig. 2 as treatment extents required). This paper includes feasibility level evaluations of partial treatment requirements and treatment alternatives for Ohio River drainage area LPPs.

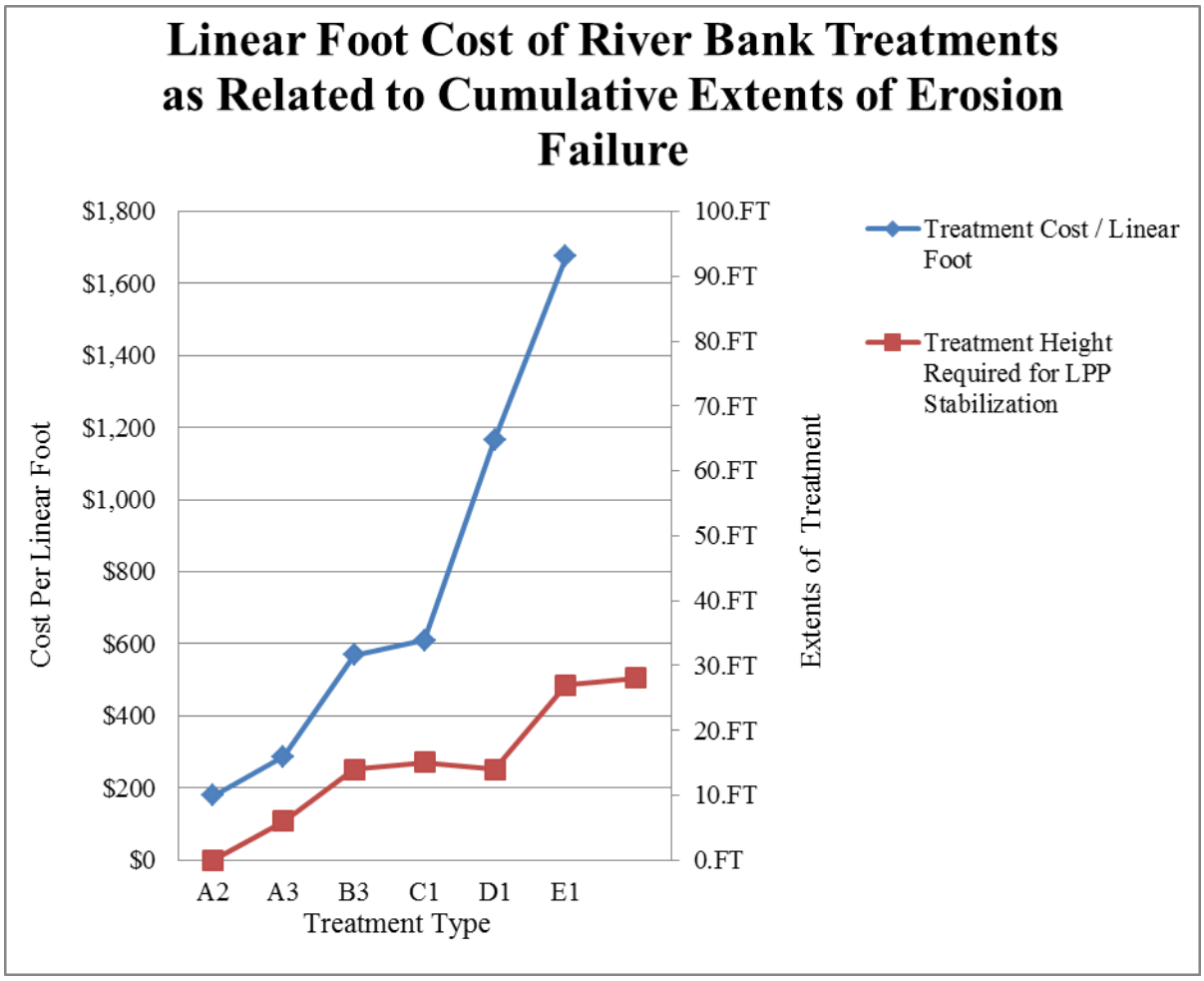

Fig. 2. As extents of bank erosion and failure increase, the cost and height of treatment required for stabilization would increase as a nonlinear component.

Overview

Recently completed emergency stabilization projects adjacent to the Point Pleasant, West Virginia and Massillon, Ohio LPPs were re-evaluated. Flood flow tractive and shear force-related sub-aerial and sub-aqueous bank erosional oversteepening and piping initiated terrace and slope failures, resulting in embankment, berm, and wall foundation

instability. Unstable wall foundations, drain outfalls, embankment fill tension cracking, and subsidence conditions resulted in threatened endangerment conditions at these LPPs. Project specific emergency responses required the excavation of failed bank soils and the placement of stone slope protection to stable geometries together with up and down channel transitions to preclude upper bank treatment outflanking.

LPP stabilization, after recent high river stage and flood events, when cumulative bank erosion and failure conditions had extended into maintenance areas and embankment and wall foundations, required extensive construction projects to prevent LPP breaching. Additional Mainstem Ohio River drainage area LPP evaluations, inclusive of bank erosion and failure processes, are ongoing and have been utilized to define incremental and cumulative landward extents of instability and increased heights of scarp features. These sub-aqueous and sub-aerial terrace, berm, and scarp features were often located channelward from LPP easement areas. As Ohio and tributary river bank erosion and failure features extended landward, more immediate stabilization of somewhat remote scarps was determined to be an appropriate, cost effective alternative to the previously referenced emergency actions. Timely stabilization of limited terrace scarp features could include placement of dikes, together with bank excavation to stable geometries and construction of adjacent stone slope protections. These LPP evaluations, determinations of bank erosion and failure extents, and bank retreat-related impacts would be defined when determining relevant and cost effective limited stabilization treatments for each site.

Discussions of the emergency stabilization measures at the Point Pleasant, West Virginia and Massillon, Ohio LPPs are included in this report. Site reconnaissance data and evaluations of bank erosion and failure conditions at other Mainstem Ohio River LPPs are also included. This paper provides evaluations of a series of conceptual treatment alternatives that have been designed to stabilize site-specific bank erosion and failure conditions. Several previously completed projects were evaluated and are cited as examples of alternative treatments for stabilization of eroding and failing terraces adjacent to LPPs. Treatment costs, extents of terrace and LPP stabilization, subsequent monitoring, and operations and maintenance requirements were evaluated. Adaptive engineering methodology and phased construction of these stabilization projects is discussed. Extents of adjacent terrace erosion- and failure-related instability are referenced when describing site specific conditions of LPP endangerment, construction priorities, and proposed project scheduling. Site-specific monitoring methodology and evaluation reports, submittals, and additional maintenance extents and requirements are discussed. Recommended actions include evaluations of adjacent site-specific river and terrace changes which would, without construction requirements of stabilized structures, be subject to high stage- and flood-related erosion and failures and subsequent LPP instability.

Causes, Effects, and Engineering Management-Related Lessons Learned

Studies regarding bank erosion and failure along the Ohio River and other underfit rivers and tributaries with relic terrace areas have been conducted by the Corps of Engineers

since 1824. These erosion and failure processes are interrelated and episodic. After reviews of studies and recent river reconnaissance, the District determined that mechanisms of bank failure along the Ohio River and tributaries include flood flow-related shear and tractive forces, erosion, wave swash, seepage, and piping, which result in internal erosion of terrace area alluvial soils and subsequent flood event toppling and block failures. Piped sands and soil blocks accumulate at the base of scarp features and within adjacent terrace areas where these failed soils are reworked and transported by overland flows, secondary currents, and high stage and flood events.

Other significant bank and terrace erosion and failure conditions, which were encountered along the Ohio and Tributary Rivers, are as follows (Fig. 3):

1) Flood-related erosional undercutting and oversteepening of river banks and terraces resulting in site- and reach-specific sliding and slumping of alluvial terrace soils.

2) Site-specific bank and terrace failures, including rotational failure defined displacement and slumping of saturated soils, during floods and high river stage recession and as defined by groundwater discharges, which, after these events, result from the Ohio and Tributary Rivers falling more rapidly than saturated upper bank soils can drain.

3) Bank, terrace, and slope area groundwater seepage-related internal erosion and overlying alluvial soil displacement resulting in block and cusp shaped failures. Bank, terrace, and slope area seepages are more persistent within areas of infiltration from adjacent wetlands, ponds, and drains.

4) Failed soil block displacement and additional slabbing, as a result flow-related plucking and surface and groundwater infiltration and discharge within bank, terrace, and slope areas, result in tension crack formation and uplift and cleft pressures.

5) Wet/dry and freeze/thaw cycles initiate slaking and ice lensing and form frozen soil area blockages of groundwater discharges from terraces and slopes. These conditions result in increased groundwater hydraulic gradients and outflanking flows adjacent to frozen soil areas together with reworking and erosion of soil spalls.

6) Unsuitable fills, cut and cast soil excavation, and limited stone placement within terrace areas, results in the blockage of internal drainage and causes slope failures, which, together with localized turbulent flows and secondary currents during floods, results in additional reworking and channelward erosion.

7) Precipitation, inundation, and related back of bank overland flows, together with groundwater discharges, cause gullying and riverward secondary erosion and transport of terrace and slope area soils.

8) Motor vessel transiting-related impacts, flow induced turbulence, and return flows cause resuspension and transport of failed soils, soil spalls, and recently deposited sediments. These soil and sediment deposits within navigable and non-navigable channel reaches are extensively eroded during subsequent high water and flood events.

9) Fetch and transiting waves result in rework and transport of failed soil and recently deposited sediments.

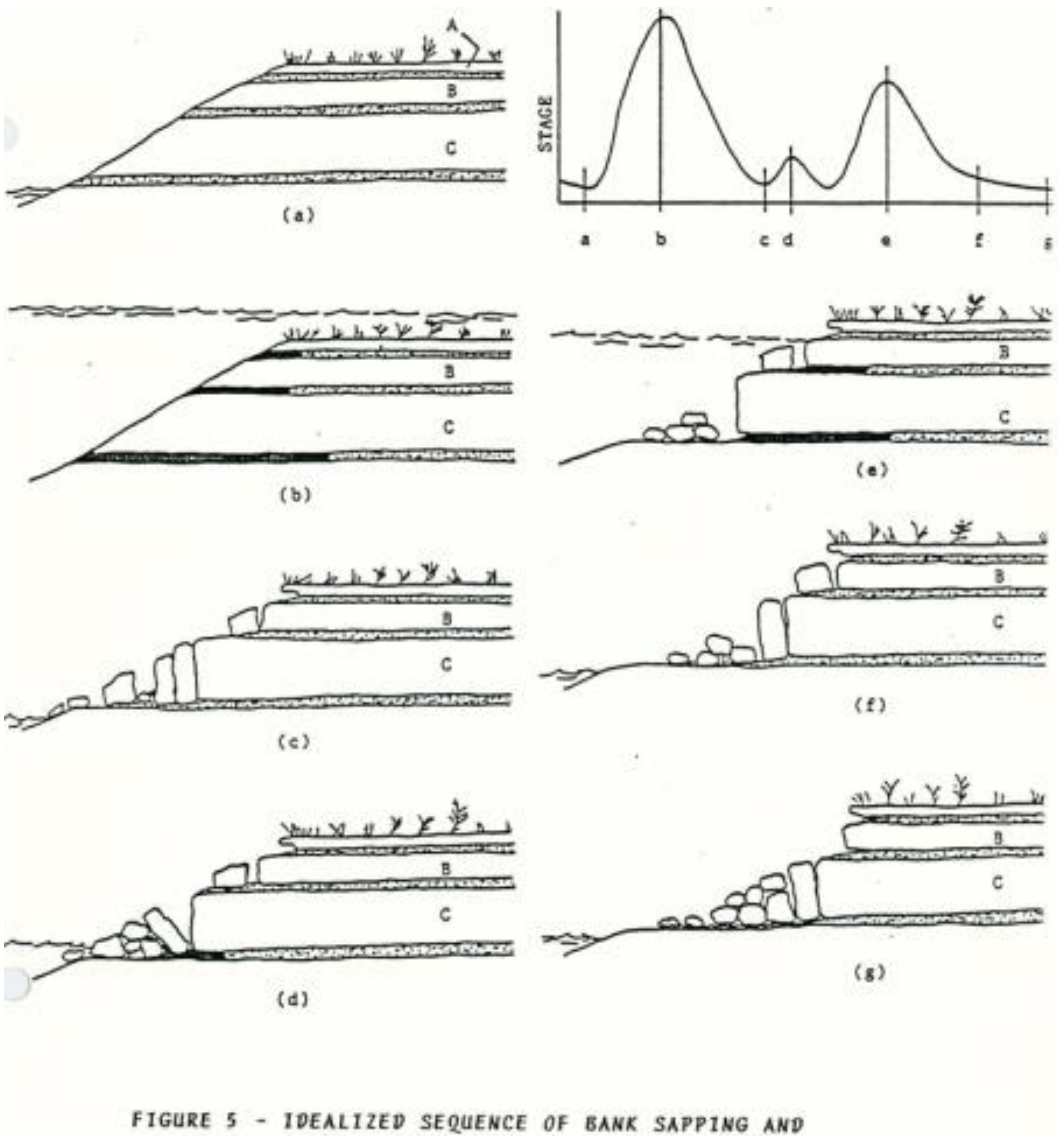

Fig. 3. Idealized Sequence of Bank Sapping and Erosion from Clough & Duncan, 1986, page 40

Geotechnical stability analyses have established that retained navigation pools could somewhat limit the severity of bank, terrace, and slope recessional failures since the

extents of drawdown after flood events would be less than that which would occur along a river without navigation pools. Additionally, groundwater discharge gradients, following flood events, could be somewhat reduced as a result of pool retention. Based on studies of several rivers with retained navigation pools, it has been determined that discontinuous benches, with variable widths, may form within limited reaches because the sub-aerial extents of recessional bank failures and erosion had been limited by navigation pool retention. However, since navigation pool retention does not alter flood stages or result in reduced current velocities and erosional severity of these events then bank, terrace, and slope area erosion and failure conditions are not changed.

Recent LPP Stabilization Projects

Point Pleasant, West Virginia

The Point Pleasant LPP was reconnoitered and evaluated in 2012. Kanawha and Ohio River terrace area erosion and failure conditions were observed during the Pittsburgh, Pennsylvania to Cairo, Illinois reach of river bank erosion studies and during on-site monitoring, design, and construction of stone dike structures from 1976 to present. From 1976 to 2010, erosion and failure conditions were generally referenced as to extents and height of lower bank, terrace, and slope scarp features. These evaluations did not, however, reference thalweg, bank, or upper terrace features or adjacent LPP structures, embankments, and maintenance areas. This mapping and general descriptions of site and reach topography could not, because of tree and brush cover and debris mantling together with river fluctuations, fully define bank, terrace, and slope areas or adjacent embankment geometries. These high stage and flood flow initiated bank, terrace, and slope area scarp features were also not defined as to erosion extents or bank failures adjacent to or within LPP embankments or wall foundations. Therefore, during the monitoring period from 2004 through 2012, high stage and flood event bank, terrace, and slope erosion and failure conditions along the Kanawha and Ohio Rivers were not fully monitored nor evaluated with regards to interrelated hydraulic and geotechnical processes. However, flood-related conditions continued to affect embankment slope and wall foundation stabilities and threatened and/or endangered the LPP.

Point Pleasant LPP Ohio River Reach as an Example

If riverbank protection had been constructed at this project years earlier, the construction costs would have been reduced by nearly half. Where toe of terrace erosion has occurred with progressive upslope collapse as a result of piping, significant riverbank materials had been lost. This loss of material had endangered the LPP. The completed project used a stone buttress treatment type E1 (described later in the Treatment Alternatives section) to protect and reconstruct the LPP foundation, at a cost of $1,538 per linear foot. If the project would have been constructed years earlier, a stone blanket (treatment type D2) (described later in the Treatment Alternatives section) would likely have been sufficient at a cost of approximately $750 per linear foot.

The Ohio River reach of riverbank at the Point Pleasant LPP had progressed from Fig. 3c to Fig. 3g during the period of observation. The continued process of high water and bank groundwater recharge and discharge with erosion of sandy bank materials by piping, then

the successive upslope collapse of the more cohesive soil layers, and the rework and transport of the failed soils re-exposing sand layers and removed the riverbank materials. However, years ago, the bank erosion features were located channelward of the project defined 15-foot maintenance and monitoring areas and were, therefore, not eligible for repairs (Fig. 4).

Fig. 4. Point Pleasant pre-emergency construction depicting the 15-foot maintenance zone, looking down river (Humphreys, 2010, 1)

Fig. 5. Point Pleasant, West Virginia LPP from toe of bank pre-construction

Fig. 6. Point Pleasant, West Virginia LPP from toe of bank during construction

Massillon, Ohio 2011-2012

The Massillon, Ohio LPP was reconnoitered and evaluated in 2011. Construction of the project included channel excavation and backfilling and stabilization of a relic meander and cutoff features. Previous evaluations of the straightened and relocated channel had referenced significant adjacent bank erosion. However, the extents of terrace, slope, and embankment erosion were not defined. During the 2011 evaluations, additional surveys were conducted, and this data was compared to as-constructed project channel, bank, and embankment geometries. Comparisons utilizing survey data showed areas of: (1) sediment accretion at the toe of the LPP embankment together with near vertical channelward scarp features, (2) launched stone slope protection, and (3) levee toe erosion and upslope terrace and embankment failures. The areas of levee toe erosion and continuing upslope failures were further evaluated and stabilization treatments were prioritized based on embankment failure risks. High risk areas were treated and lower risk area treatments were included in engineering management defined out-year work plans and proposed budgets.

Monitoring, Reconnaissance, and Initial Findings for LPP

Site reconnaissance at the Ceredo-Kenova and Point Pleasant, West Virginia and Massillon, Ohio projects defined bank and terrace erosion and failure conditions that threatened and endangered reaches of embankment and adjacent structure. These instability conditions resulted from recent flood events and could have been limited by stabilization treatments. Additional site evaluations established that incremental erosion and failure conditions within adjacent bank, terrace, and slope reaches, although somewhat distant from the LPP structures, embankments, and stone slope protections, had recently encompassed adjacent maintenance areas.

As these bank, terrace, and slope features became erosionally truncated, scarp features formed, merged, and encompassed areas adjacent to the LPP. These flood flow-related conditions were observed to threaten LPP stability. The District therefore determined that additional monitoring and evaluations of these erosion conditions between the maintenance easement, bank and terrace land-water contact, and river thalweg would provide data for timely assessments of long-term treatment requirements, and to address the probability of subsequent flood event-related LPP endangerment and/or failure.

The District determined that local sponsors of these Ohio River and tributary LPPs are not fully responsible for maintenance or inspection of bank features channelward from 15 foot easements which had been established for these LPPs[1]. These monitoring and inspection limitations are most significant because flood erosion processes continue to fail banks, terrace, and slope areas and then destabilize LPP foundations.

Based on this information and data obtained during the Ceredo-Kenova, Point Pleasant, and Massillon emergency LPP protection projects, the District determined that river bank conditions, as generally referenced by the attached site maps, should be monitored and evaluated along all reaches adjacent to these LPP. Results of a limited reconnaissance of

[1] 33 CFR 208.10

these LPPs are subsequently presented. Treatment cross-sections within other eroded and failed LPP bank reaches, and approximate costs to construct are provided in Table 2. Since these reconnaissances are continuing, additional evaluations of bank, terrace, slope erosion and failure conditions, and stabilization treatments adjacent to the LPPs have been conducted during 2014.

Maysville, Kentucky

As requested by the City of Maysville, Huntington District Staff conducted a reconnaissance of the LPP and adjacent areas. This site visit was requested by the City Engineer to better define project operation and maintenance requirements. The reconnaissance extended from the public launch ramp down channel to a reach of railroad fill below the Simon Kenton Bridge. Bank, terrace, and upslope scarp features were evaluated to better determine interrelated Ohio River erosion and failure processes which could threaten the LPP. Within terrace reaches, with extensive stone slope protection, the adjacent banks remained stable. Some of this stone had weathered and these materials should be further evaluated by District Staff during periodic inspections to better ensure long term LPP protection.

A reach of bank, terrace, and adjacent slopes without stone or rubble placement, included scarp and piping features and vegetation with exposed roots. These bank and terrace features were encountered from the placement area of stone slope protection along a down channel reach of 1,400 feet. These bank, terrace, and scarp features were near vertical and varied from 4 to 8 feet in height and from 20 to 60 feet in width. These bank, terrace, slope, and thalweg area erosion and failure features result from high water and flood events, which, during the next twenty years, could extend landward and upslope with resulting embankment oversteepening and subsequent LPP failures.

This bank reach terrace feature included recessional scarp and piping defined failures and trees with exposed roots. Additional bank and terrace erosion and failure features extended an additional 2,380 feet down channel from the previously referenced LPP reach. Upper bank and terrace scarp features were near vertical and were 8 to 10 feet high along the toe of the LPP embankment. These terrace erosion and failure features, during the next 5 to 10 years, as a result of high water and flood events, would extend further landward and upslope, which could result in LPP embankment oversteepening and failure.

During this reconnaissance, bank, terrace, and slope erosion and failure conditions, within the floodwall reach of the LPP, were not determined to be significant. However, these reaches of the project should continue to be monitored together with the previously referenced bank, terrace, and slope area erosion and failure features. The District has determined that the bank, lower terrace, and upper terrace erosion and failure conditions could, during the next 25 years, form scarp features with cumulative heights of greater than 20 feet. Presently, the upper terrace scarps include areas of tree removal and exposure of alluvial soils. Slope excavations within these upper terrace scarp features, adjacent to the LPP embankment, were extended to form stable slope geometries. Since these scarps included piping features and exposed tree root systems, additional excavation was observed to have been required to expose in-place soil and form a

suitable surface for placement of granular bedding and stone slope protection. During scheduled inspections and after high water and flood events, District personnel and the local sponsor should continue to evaluate, by motor vessel and area surveys, all river reaches adjacent to the LPP to better determine extents and severity of bank failure and erosion conditions. The data obtained during these seasonal and periodic inspections, together with side scan sonar, far infrared overflights, and lidar imagery derived surveys will enable the District to submit timely baseline recommendations for cost effective LPP maintenance and/or stabilization treatments. District personnel determined that the LPP was not presently endangered by these adjacent bank and terrace erosion and failure conditions.

Portsmouth and New Boston, Ohio

Limited reconnaissance of the Portsmouth and New Boston, Ohio LPP and adjacent banks and terraces have been conducted. Since these reconnaissances were not complete, additional evaluations will be conducted during 2014. Near Portsmouth, Ohio, bank and terrace reaches adjacent to, and up-channel from, the Ohio River-Scioto River confluence have been affected by erosion conditions and failure-related exposure and misalignment of debris fills and relic LPP wall structures. Interior drainage and ponding areas, pump station outfalls, and adjacent slope reaches included erosion and failure features. At and up-channel along the Ohio River, bank and terrace reaches of rubble and stone stabilization treatments were observed. Adjacent bank reaches up-river from the City Park and Boat Landing included relatively stable banks, terraces, and slopes. Monitoring and evaluations of relatively unstable bank and terrace reaches are on-going. Up-river from the bridge, marina, and campsite areas, the Section 32 project protection has provided limited stabilization along a 1,600-foot segment of bank which includes slag and concrete rubble debris disposal areas and a boat launch ramp. Up-river from the launch ramp, bank and lower and upper terrace erosion and failure scarp features threaten and, for limited reaches, endanger adjacent embankments and walls within the Portsmouth and New Boston LPP. Within the New Boston reach of the LPP, slag had been dumped on bank, slope, and terrace areas during the operation of adjacent steel mills. This slag continues to function as incidental protection. Additional monitoring and evaluations of unstable bank, terrace and slope area reaches within New Boston and Portsmouth, Ohio LPP are on-going. Tributary drainage features, ponding areas, pump station outfalls, and Section 32 project bank treatments are also being monitored.

Russell, Kentucky

The Russell, Kentucky LPP has been inspected; bank failure and erosion conditions, stone dike, vegetated and stone slope, and stone buttress treatments were re-evaluated. These bank stabilization projects were fully functional and, with the exception of a City Park access way and an adjacent sewage lift station foundation area, have not required maintenance. Interior drainage features, ponding areas, and pump stations, adjacent to Ohio River banks and terraces had been affected by recent high stage- and flood flow-related erosion and slope failures. These limited bank and terrace reaches and an approximate 50-foot drainway confluence area will continue to be monitored, and as necessary, these drainage channels and structures would be stabilized, repaired, replaced, or abandoned.

Ironton, Ohio

The Ironton, Ohio LPP was reconnoitered and evaluated. This reconnaissance defined extensive bank and terrace area slag placement from adjacent steel mills and iron casting facilities. Recently, Ohio River bank and terrace reaches up and down-river from the Ironton Russell Bridge have been stabilized by the City. At the water works facility, stone slope protection and soil nailing were used to stabilize banks and terraces adjacent to raw water intakes and treatment plant foundations. Down-river from the bridge, terrace and slope failure scarps were stabilized by the construction of a concrete rubble dike. Areas down-river from the boat launch ramp, park, and parking areas, have also been stabilized by placement of bank and terrace area stone slope protection. These areas and down-river reaches of the LPP include other limited bank and terrace area stone slope protections. Within this bank reach, which extends to an interior drainage feature and pump station area, several bank and terrace and slope erosion and failure features were encountered. These features included bank failure scarps 4 to 6 feet in height. Upper terrace and floodplain areas 60 to 80 feet wide were encountered adjacent to the LPP. These areas, inclusive of bank, terrace, and slope erosion and failure features and the previously referenced treatment reaches, have been evaluated annually and after high stage and flood events. No bank, terrace, or slope erosion or failure conditions, which would affect LPP stability, have been defined during these evaluations.

Ashland, Kentucky

The Ashland, Kentucky LPP has been recently reconnoitered and evaluations are continuing. Several LPP reaches with extensive adjacent bank, terrace, slope, and embankment scarp features were encountered. CSX mainline track ballast, embankment fill, and inert debris, including steel slag disposal areas, were defined. The City Park launch ramp, and adjacent stone slope placement areas, which extend up-river from the Ashland/Ironton bridge abutments, included limited bank, terrace, and slope area erosion and failure features. Bank, terrace, and slope reaches, within an adjacent, approximately 6,000-linear-foot reach, included erosion and failure scarp features. However, these features are somewhat riverward and up-channel from the Ashland LPP. Previously unstable bank and terrace reaches had been stabilized by CSX mainline fill placement-related maintenance. Without this incidental stabilization, these conditions could have threatened or endangered the LPP. Reconnaissance and evaluations of the bank and terrace areas and the LPP have been conducted after Ohio River high stages and flood events.

Catlettsburg, Kentucky

The Catlettsburg, Kentucky LPP evaluations included Ohio and Big Sandy River bank, terrace, and slope area erosion and failure. These features were adjacent to LPP embankment and wall structures. These Ohio and Big Sandy River reaches included limited stone slope treatments and concrete launch ramps. Terrace areas adjacent to the LPP were narrow, and bank, terrace, and scarp area features, with significant cumulative heights, could affect adjacent embankment and wall stabilities. Bank monitoring and evaluations will continue to be conducted seasonally and after river high stages and flood events. These erosion and failure conditions and the merging of bank and lower and upper scarp features could, within the 8,400-foot reach, result in LPP embankment and

wall instability. Therefore, the District has determined that the LPP is presently threatened within these bank, terrace, and slope reaches.

Ceredo-Kenova, West Virginia

The Ceredo-Kenova, West Virginia LPP was recently re-evaluated. Bank, terrace, and slope areas within the LPP reaches adjacent to the Big Sandy River were relatively stable. Mowing-related rutting and limited interior drainage outfall and ditch erosion were observed. Two 400-foot reaches and one 2,000-foot reach of Ohio River bank, terrace, and slope erosion and failure, which had exposed the adjacent LPP embankment stability berm were stabilized during 2004 to 2005. Construction included excavation of in-place soil to stable slopes and the placement of stone dikes, granular fill, geotextile filters, and tied-back structures together with up and down channel keys and transitions. These treatments remain functional and have stabilized adjacent LPP berms and embankments. Additional project maintenance and repairs have not been required. Limited up-river bank reaches contain bank, terrace, and slope scarp features, which are adjacent to the LPP. Barge mooring areas within these reaches may limit moderate and low river stage near bank erosion and failures and increase extents of lower terrace sediment mantling. These conditions have been monitored and evaluated after Ohio and Big Sandy River high stage and flood events to better determine cumulative erosion and failure extents within this 6,000-foot bank reach adjacent to the LPP. Reaches of this LPP are presently threatened (Fig. 7).

Fig. 7. Ceredo-Konova, West Virginia LPP Emergency Flood Recovery project pre-construction 2005

Fig. 8. Ceredo-Kenova, WV LPP Emergency Flood Recovery project post-construction

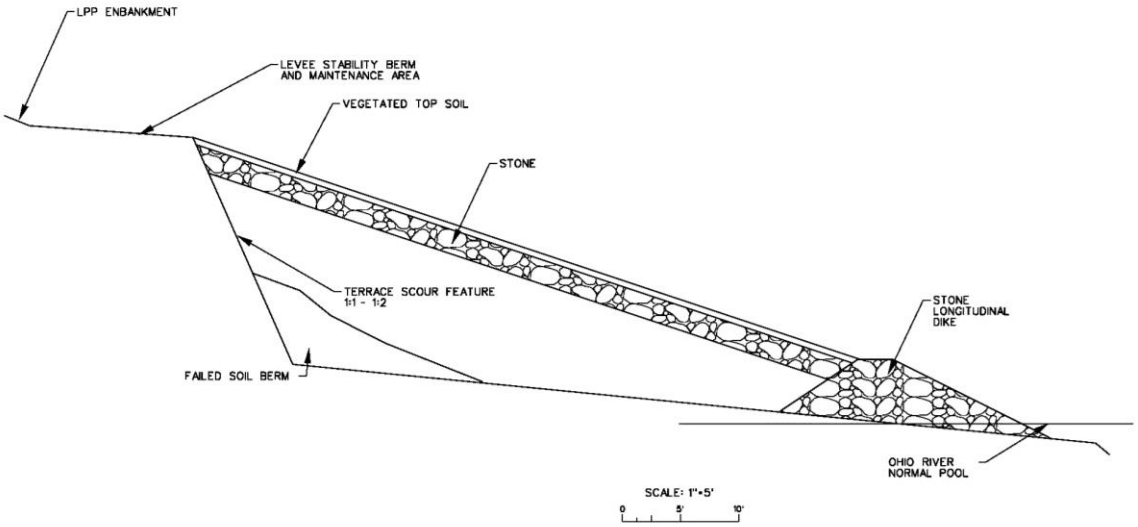

Fig. 9. Ceredo-Kenova, West Virginia LPP Emergency Flood Recovery project design cross-section

Huntington and Guyandotte, West Virginia

The Huntington and Guyandotte, West Virginia LPP were recently reconnoitered and re-evaluated. Extents and severity of bank and terrace failure and erosion were defined along reaches of the Ohio and Guyandotte Rivers. Approximately a 14,000-foot reach of bank, terrace, and slope erosion and failures could, during the next several years, threaten these LPPs. Within the LPP, three reaches have been partially protected by the construction of 2,000 feet of stone dikes adjacent to, and channelward from, bank, terrace, and scarp erosion and failure features. Along the Guyandotte LPP wall reaches, grouting and restoration of toe drain and adjacent combined storm and sanitary sewer integrity were required to stabilize foundations, adjacent subsidence areas, and to better assure flood stage underseepage interception and control.

The Ohio River LPP reach and 10,300 feet of right descending bank and 4,000 linear feet of left descending bank LPP along the Guyandotte River are significantly affected by bank, terrace, and slope failure and erosion. These bank, terrace, and slope areas will continue to be monitored and evaluated to better define erosion and failure conditions and to determine extents of related instability along adjacent LPP embankment and wall structures. Within this and other recently evaluated LPP projects, bank, terrace, and slope areas were observed to contain exposed tree roots and/or root burial, and stem misalignment, which has also been evaluated to better define extents of erosion, failure, and sediment mantling.

As previously referenced, two bank and terrace reaches adjacent to the Huntington LPP have been partially stabilized by completion of Section 14 Projects, which were constructed to protect channelward sewers. These limited treatments precluded the more extensive placement of stone slope protections, which would have been required had bank, terrace, and slope erosion and failure continued. Additionally, combined sewer systems were grout stabilized along a 3,000-foot, eroding, and failing bank, terrace, and slope defined reach of the Guyandotte River located up-channel from the Pats Branch Pump Station. These treatments limited flood stage-related underseepage, internal erosion, fill and bank alluvium failures and assured long-term LPP functionality (Figs. 10 and 11).

As previously referenced, several LPP reaches are presently threatened by Ohio and Guyandotte River flood flow-related erosion and failure of bank, terrace, and slope areas.

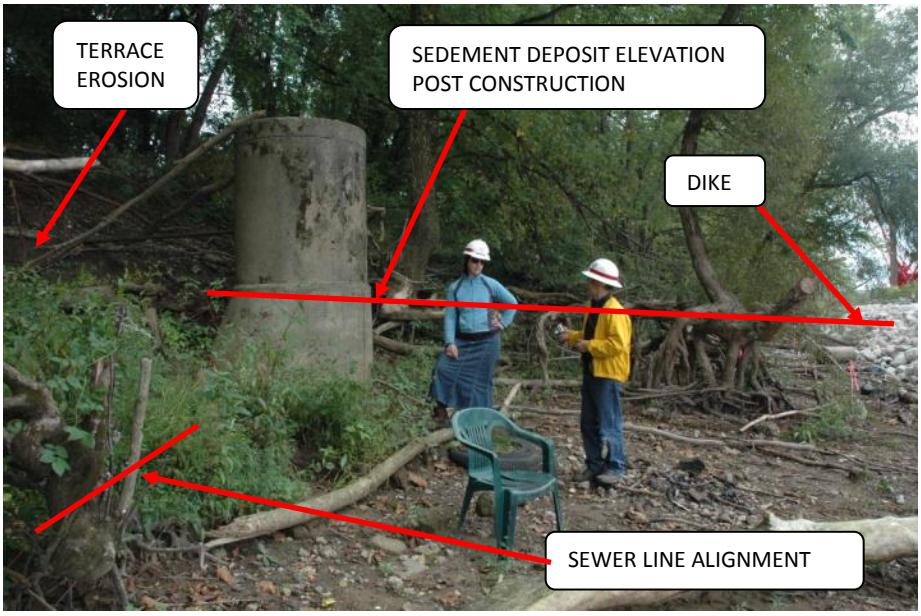

Fig. 10. 7th Street West Emergency Sewer Line Stabilization during construction (Huntington, West Virginia LPP is on left past the tree line)

Fig. 11. 7th Street West Emergency Sewer Line Stabilization sediment deposits post construction (Huntington, West Virginia LPP is on left past the tree line)

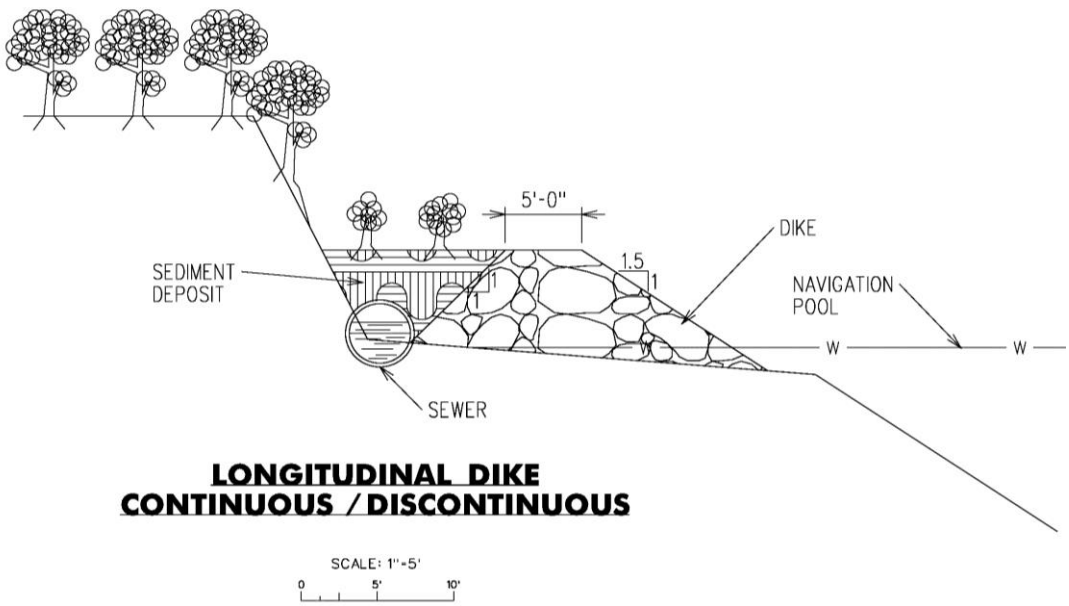

Fig. 12. Cross section of treatment alternative constructed at 7th Street West Emergency Sewer Stabilization project

Point Pleasant, West Virginia

Point Pleasant, West Virginia LPP stabilization actions required detailed surveys and soundings to define extents of bank, terrace, and slope erosion and failure, and consequential embankment displacement, wall misalignment, and foundation exposure-related endangerment. Bathymetric surveys, inclusive of side scan sonar data, were evaluated when determining relatively stable sub-aqueous bank topography. Along both the Ohio and Kanawha Rivers, bank areas had, from 1950 to 1988, been relatively stable. Historically, bank failure and erosion features varied from 6 to 12 feet in width along the Kanawha River. The Ohio River bank features were from 22 to 60 feet in width. Upslope terrace erosion and failure scarp features were near vertical (2V:1H to 4V:1H) and varied in height from 12 to 14 feet. Similar erosion and failure features were frequently encountered along both the Kanawha and Ohio River banks.

Recently, 2004-2010 flood-related erosion failure, bank, terrace, and slope area scarp features along the Kanawha River exposed LPP wall foundations and extended an additional 2 to 6 feet upslope within LPP embankment reaches. Stabilization treatments required excavation of failed soils and placement of geotextile filters and stone slope protection including up- and down-river transitions. Within areas landward from the stone slope protection, top of slope stone keys were required. Indurated clay fill was then placed and compacted to restore wall foundation cover and to reduce the potential for excessive underseepage. During this construction, 2004-2005, a longitudinal dike was

placed along an adjacent Ohio River reach to limit extents of flood flow-related bank and terrace erosion and failure-related LPP wall endangerment.

During high water and flood events in 2010, an extensive reach of the LPP, along the Ohio River, became endangered as a result of additional bank, terrace, and slope erosion and failure together with merging and landward progression of slope area scarp features. These conditions resulted in displacement of rubble fills, discontinuous slope protections, relic head walls and drains, and exposed extensive debris fills. Drift and tree clearing, excavation of failed soil, debris, and in-place alluvial soils, and the placement of filter fabric and stone buttress treatments together with transitions and an up-channel trench revetment were required for limited LPP stabilization. The previously constructed longitudinal dike was included within this stone buttress treatment. Down-channel reaches included angle dikes, which were constructed along the toe of the stone buttress, adjacent to the Point Pleasant Riverfront Park, to divert drift and debris. Access roads were restored and disturbed areas were re-vegetated.

These limited LPP flood endangerment-related stabilization treatments were costly to construct. A retaining wall structure would have provided for more extensive LPP stabilization at a linear foot cost of three times that of the stone buttress treatment. If bank, terrace, and slope scarp treatments had been completed prior to the coalescence of these flood flow erosion and failure features, then landward foundation failures would not have occurred, and LPP endangerment and instability would have necessitated the construction of longitudinal dike and stone slope blanket stabilization features. These treatments would not have necessitated extensive bank excavation, filter fabric, and stone placement to affect slope treatments and could have been constructed with limited stone requirements and lower cost.

Parkersburg, West Virginia

The Parkersburg, West Virginia LPP was recently reconnoitered and re-evaluated. Bank erosion and failure along the Little Kanawha and Ohio Rivers had resulted in development of extensive scarp features, which threatened adjacent LPP walls and embankments. The recently constructed Riverfront Park Project also included limited stabilization of bank and terrace areas and adjacent LPP wall foundations. Additional reconnaissance and evaluations will be conducted to define extents of bank, terrace, and scarp erosion and failure, and related impacts to the adjacent LPP. These conditions along a 7,000-foot reach of the Little Kanawha River right descending bank, from Worthington Creek to the Ohio River confluence, excluding CSX mainline track stabilization, have adversely affected LPP embankment and wall foundation stability.

Massillon, Ohio

Along the Massillon, Ohio LPP, terrace areas and the riverward toe of the levee had been erosionally oversteepened. As defined by field evaluations and determinations of LPP instability, over 4,000 linear feet of terrace and embankment in-place and failed soils were excavated together with placement of stone slope protection. During subsequent land- and river-based surveys, the extents and severity of terrace and toe of levee erosion features were defined. These surveys were compared to the as-built condition of the LPP

by referring both constructed slope geometries and recent State of Ohio mapping. Based on comparative cross-sections and field investigations and evaluations, 4,110 linear feet of significant terrace and toe of embankment erosion were defined within the project and up and down river levee transitions. This LPP reach was determined to be most critical because during high flow events, the levee would be extensively eroded and, together with recessional failures, would continue to fail within up- and down-channel reaches.

Additionally, limited terrace and LPP slope failures had occurred as a result of piping-related internal erosion of bank and terrace alluvium and embankment fill materials. These erosion features had extended upslope and, if not repaired, would have resulted in additional embankment instability and breaching during and after additional high stage and flood events. These embankment reaches required immediate emergency repairs to assure LPP functionality. In addition, the District identified 1,700 feet of terrace erosion along the west descending bank terraces and levees. This bank erosion included up- and down-river LPP transitions. Adjacent levee reaches and banks were eroding and failing during high flows and will require stabilization. These LPP conditions have been evaluated and preliminary designs and cost estimates have been completed.

The construction of LPP stone slope protection treatments down-river of Cherry Road required approximately 7 tons of stone per linear foot of protection. These treatments included stone tonnages, which would launch and armor near bank and terrace and adjacent channel scour features. These features, which most often formed during both high flows and as a result of moderate and low flow secondary currents, included both erosional features and depositional planforms. Embankment stabilization within the reach up-river of Cherry Road was constructed using approximately 4 tons of stone per linear foot of treatment. These treatments also included a launchable stone section. The treatments up-river of Cherry Road also included stabilization of a waterline crossing.

Within a downchannel project reach, adjacent to Wetmore Creek, a pressure conduit outfall discharge had formed both scour features and channelward depositional features. These fine sediments became mantled, during subsequent high stage events, with gravel and cobble size deposits. The excavation of these sediments and placement of a stone ring dike were required to control conduit discharges and high-stage in-channel turbulence.

Approximately $1,780,000 was expended to evaluate, design, and construct this emergency repair project.

Formulating Alternatives

As a result of emergency stabilization actions at Point Pleasant, West Virginia, Ceredo-Kenova, West Virginia, and Massillon, Ohio and evaluations of similar flood flow-related bank, terrace, and slope instability conditions adjacent to other Mainstem and Tributary LPPs, the District has developed treatment alternatives to address extents and severity of bank, terrace, and slope erosion- and failure-related LPP instability. These limited treatments have been designed to stabilize bank erosion and failure conditions prior to endangerment or failure of the LPP. Ongoing monitoring and evaluations have established that timely bank, terrace, and slope protection measures would provide

reductions in LPP failure risks and are significantly less costly than emergency actions necessitated when LPP wall foundations and embankments become unstable. These proposed treatment options are presented at a feasibility level and must, therefore, be further defined during site-specific evaluations of the LPP bank, terrace, and slope erosion and failure extents.

Local Protection Project Conditions

Designated Local, State, and District personnel should continue to conduct site evaluations after high water and flood events wherever bank, terrace, and slope erosion and failure conditions endanger an LPP. The established policies and project certification, as required to affect LPP evaluations, which were implemented more than 70 years ago, are not adequate to determine timely maintenance and repairs for interior drainage systems, pumping facilities, closures, and the stabilization of walls and embankment. Additionally, routine operational system repairs and equipment replacement must also be affected to assure LPP functionality.

During the period from 2004 to 2014, data from reconnaissance at the Point Pleasant, West Virginia, Ceredo-Kenova, West Virginia, and Massillon, Ohio LPPs were evaluated to better define the extents and severity of up- and down-river and in-channel bank, terrace, and slope erosion and failures. These reconnaissance and evaluations were conducted at District LPP sites that are located within bank reaches with terrace and channel alluvial deposits and reworked fluvial glacial outwash, which most often overlay Pre-Illinoian bedrock surfaces. These deposits and recent alluvium have been deposited within the Post Pleistocene Ohio River. Post Pleistocene bank erosion and failure conditions have formed several discontinuous bank features including terraces, slopes, transitions, and floodplain levees and swales together with exposure of bedrock features. During colonial occupation and more recent periods, these areas have become populated, and in the 1900's, urban areas required flood damage reduction measures resulting in the construction of LPPs on and off the mainstem Ohio River. These LPPs, which were constructed to optimize protection of these urban areas, were located within these bank, terrace, and floodplain topographic features. These LPP embankments, walls, and pump stations were often constructed adjacent to eroding and failing river banks and terraces, unstable slope features, floodplain drainways, and incised secondary streams with meandering channels.

During the construction of these LPPs, adjacent river bank and terrace areas were graded to stable slopes, re-vegetated, and included limited areas of stone placement. Within steep bank, terrace, and slope areas, construction included placement of stability berms, adjacent to embankments and wall foundations and transitions. These bank, terrace, and slope protections and channel stabilization features have been recently re-evaluated, and although the placed stone was somewhat weathered and showed limited surficial spalling, these treatments have remained in place and are relatively effective. However, adjacent unprotected reaches of these LPPs include numerous bank, terrace, and slope erosion and failure features. Since these features are often somewhat remote from LPP easement and maintenance areas, these features had not been referenced as potentially problematic.

During both construction phases and completed project transfers to local interest, additional LPP stabilization treatments have not been required.

With regards to previously referenced projects, after flood events that occurred during the 2004-2009 period, the District was notified that bank, terrace, slope, scarp, and subsidence features had formed adjacent to, and within, the Point Pleasant LPP embankment and wall reaches. Further evaluations confirmed that embankments and wall areas included extensive bank, terrace, and slope area flood flow erosion and failures and that the project was immediately endangered. Adjacent bank, terrace, and slope areas included extensive erosion and failure features, which were located adjacent to the project, but did not pose immediate endangerment to the LPP. However, these less critical reaches included numerous erosion failure and scarp features and discontinuous bank, terrace, and slope areas. These conditions required additional evaluations of river bank reconnaissance data and site-specific conditions. Based on these evaluations, the District determined that more extensive erosion and failure features had, after 2010 flood events, resulted in bank, terrace, and slope truncation and scarp feature coalescence, which extended landward such that these conditions endangered the LPP. These continuing bank, terrace, and slope erosion and failure conditions resulted in increased scarp heights. These 2010 evaluations enabled the District to define bank, terrace, slope, and scarp areas and floodplain high stage- and flood flow-related LPP endangerment.

Other emergency response programs have been used to construct repairs to adjacent utilities. These actions have included construction to maintain and protect endangered utilities. Additionally, other emergency projects adjacent to channelward utilities have provided incidental riverbank protection for adjacent levees and flood walls. Completed USACE projects adjacent to LPPs are presented in Table 1.

Table 1. USACE construction projects since 2003 adjacent to LPPs

Local Protection Projects included in the study	Number of Locations	Type of USACE Construction Project since 2003
Ceredo-Kenova, West Virginia	3	Emergency Flood Recovery
Huntington, West Virginia	2	Emergency River Bank Protection
Ironton, Ohio	1	Emergency River Bank Protection
Parkersburg, West Virginia	3	River Fount Park Constitution
Point Pleasant, West Virginia	2	Project Rehabilitation
Massillon, Ohio	7	Project Maintenance Package

Scope of investigation

Alternative LPP stabilization treatments (Table 2) were established by defining extents and severity of adjacent bank, terrace, slope, and floodplain erosion and failure features. These stabilization alternatives were defined on the basis of high stage and flood flow failure and erosion extents. These design efforts included determinations of constructability, costs to maintain, and long-term utility stabilization project function.

Table 2. Cost to make riverbank repairs at selected reaches in Fiscal Year 2014 before the LPP foundations are immediately endangered

Ohio River Cities with LPPs	Sum of Current Cost/Reach	Sum of Reach LFT
Catlettsburg, Kentucky	$1,435,556	5,000
Ceredo-Kenova, West Virginia	$2,009,778	7,000
Huntington, West Virginia	$10,262,111	36,100
Ironton, Ohio	$574,222	2,000
Maysville, Kentucky	$1,785,472	4,100
Parkersburg, West Virginia	$1,842,296	10,500
Point Pleasant, West Virginia	$162,696	1,360
Portsmouth - New Boston, Ohio	$1,433,163	8,200
Russell, Kentucky	$57,422	200
Grand Total:	**$19,562,717**	**74,460**

Treatment Alternatives

The treatment methods as described are presented at a feasibility level and should be referenced when conducting evaluations and designing stabilization treatments for site specific LPP erosion- and failure-related endangerment. Additional maintenance requirements and treatment alternatives and costs are also referenced. Costs are based on current 2014 pricing and are subject to revisions as a result of changes in material prices and equipment, operator, and labor rates as well as bank, terrace, and increases in slope scarp heights. Should these LPP treatments be deferred, the amount of stone, piling, and other materials, required per foot of protection would also increase. Additional material requirements would be incurred in nonlinear increments (Fig. 2).

Treatment Categories A and B within Bench Scarp Areas

Treatment alternatives presented in categories A and B are designed to stabilize oversteepened banks and eroding and failing slope areas together with upper slopes and terraces adjacent to LPP. Excavation, stone placement, and dikes, as required for buttress treatments adjacent to LPP, would provide for site-specific stabilization of LPP endangerment, resulting from high river stages, flood flows, recessional slumping, and piping-related collapse. Long-term functionality of these treatments, constructed along river banks that are somewhat remote from the LPP, would preclude additional bank, terrace, and slope erosion and failures.

Maintenance requirements for these treatments should be minimal. However, without adequate maintenance and timely repairs, these buttress, dike, and stone terrace and slope protections could be outflanked by adjacent erosion and failures. Post-construction terrace area native species re-vegetation would be permitted, except at locations where stone treatments include geosynthetic filter components. Limited, less than 3-inch diameter at breast height (DBH), woody vegetation would affect increased flow roughness and could result in additional sediment deposition.

Cost to construct these treatments would be site-specific. Treatment costs per linear foot of bank stabilization would be determined by excavation extents, filter, and wall or stone structure geometries and upslope stabilization requirements. Within scarp features, along failing and eroding banks, and lower terrace areas that are 6 to 8 feet in height, the cost to construct would vary from $120 to $500 per linear foot of treatment.

Treatment Category C for Stabilization of Bank and Lower Terrace Scarp Features

Bank and lower terrace scarp treatments would include stone buttress and stone blanket placement together with geosynthetic filter components. These treatments would be sufficient to address flood flow bank, terrace, and slope erosion and recessional and piping failure conditions. These treatments could be subject to high stage- and flood-related outflanking as a result of adjacent bank, terrace, slope, and scarp area erosion and failures. Additional channel thalweg erosion and migration could also cause launching of these stone structures. Maintenance requirements would be limited to regrading and replacement of launched stone and buttress reconstruction. Terrace and slope native species re-vegetation would be permitted, except at those locations where treatments include geosynthetic filter components. Limited, less than 3-inch DBH, native woody vegetation would be allowed. Linear foot costs for these treatments would be determined by extents of excavation, filter, and stone placement. Along reaches with scarp features 8 to 12 feet in height, treatment costs would vary from $360 to $750 per linear foot.

Treatment Category D

More extensive bank, terrace, and slope erosion and failure features would be treated by excavation to within in-place soils, and installation of geosynthetic filter, stone buttress, and stone slope protection. Construction of stone buttress features, keys, and trench revetments would stabilize adjacent bank, terrace, and slope areas. Maintenance requirements would include reconfiguration, replacement of launched stone, and buttress reconstruction. Lower bank native re-vegetation would be permitted. Limited upper slope and terrace area re-vegetation with woody shrubs would be allowed. Woody vegetation would not be allowed within the stone slope protections, which include geosynthetic filters and/or previously established maintenance and easement areas. Costs to stabilize these 12 to 16 feet high scarp features would be $580 to $680 per linear foot.

Treatment Category E

These treatments would be constructed within bank, terrace, and slope reaches with extensive high stage and flood flow erosion and failure features, which are adjacent to LPP walls and embankments. This treatment would require excavation to in-place soils and the placement of stone buttress and slope protection within bank, terrace, and adjacent areas required for height of scarp stabilization. Stone keys and transitions would be necessary to address outflanking. Extents of treatments at and upslope from bank, terrace, and scarp features would require placement of geotextile filters. Flood flow erosional oversteepening and failure-related treatment outflanking would be addressed by reconstruction of these stone buttress features together with transitions and the placement of launchable stone within toe of treatment areas. Stone launching and terrace and scarp sediment mantling may somewhat limit maintenance requirements. Costs for treating

scarp features extending 22 to 32 feet above the channel thalweg and/or high flow and/or flood stages would be $1,200 to $1,600 per linear foot. Stone buttress treatments, which extend upslope from bank and terrace areas, together with channelward thalweg stone placement to river depths of 16 to 28 feet, would cost $2,200 to $2,800 per linear foot.

Treatment Category F

At locations where river high stage- and flood-related erosion have resulted in bank, terrace, and slope erosion and failure features within embankments and wall foundations would require construction of sheet pile, H pile and lagging, and/or concrete pile wall systems. The construction methods for these structures would be limited by equipment access, installation requirements, and the effects of placement as related to vibration initiated consolidation of adjacent LPP foundation soils. Proximate failure scarps and LPP foundation instability would most often preclude the construction of cantilevered or anchored wall systems. At locations with 22 to 32 feet high bank and terrace scarp features, these wall systems would extend to in-place soil depths of 60 to 80 feet. Costs for wall construction, including stone placement at the channelward toe of the wall and within up- and down-river transitions, as required to stabilize bank, terrace, and slope erosion and failure scarps 22 to 32 feet in height, would be $3,300 to $4,500 per linear foot of treatment.

Conclusions and Recommendations

Local sponsors have, since the transfer of completed LPP, been obligated to effect timely LPP operations and maintenance responsibilities. These responsibilities included embankment, wall, and gate closure monitoring, evaluations, and reconstruction and related pump station tests and repairs to assure LPP functionality and reliability. Mowing, tree clearing, and evaluations of utilities and commercial cable conduits and pipeline transitions were also required. Pre- and post-flood event LPP foundation and easement area conditions are reported to the District. Requirements for repairs and/or remedial actions are then prioritized, coordinated, and affected, as mutually agreed to, and as scheduled. However, the adverse effects of high stage and flood flow bank erosion and failure often occur within areas somewhat remote from these maintenance easements (channelward of the LPP easement area). Additionally, these erosion and failure defined bank, terrace, slope, and scarp features may occur within densely wooded areas. These bank, terrace, and slope scarp features, which could extend from the river channel to the maintenance easements and the LPP, should be monitored after flood events and during periodic inspections. Therefore, in-channel and near bank reconnaissance should be conducted at 5-year intervals or as required by flood events.

Overall, the cost of protecting the 25 reaches of LPPs in the current state of terrace erosion is $19,570,000 (see Table 2). The cost of stabilizing these same 25 reaches of LPP foundation protection within reaches when the maintenance zone is affected and the foundation of the LPP becomes critically endangered will be significantly more costly. All costs are presented at the fiscal year 2014 prices.

Additional Responsibilities

Within LPP maintenance and adjacent bank areas, local and District evaluations of bank, terrace, and slope erosion and failure scarps, and subsidence features should be referenced and described, as to date, location, and extents. Embankment slopes, wall misalignments, and/or foundation exposure should be delineated. Bank erosion and failure features and subsidence areas are often defined by recently exposed soil and fill and areas with irregularly spaced herbaceous species, grass cover, and misaligned trees with exposed root systems. These features and conditions may also occur within mowed areas. The previously referenced evaluations of bank, terrace, and slope areas adjacent to, and channelward from, the LPP would require additional inspections, evaluations, and timely remediation. These observations, measurements, and analysis would permit timely determinations of critically necessary cost effective LPP treatments to stabilize bank, terrace, and slope erosion and failure features, which threaten and/or endanger adjacent LPP.

Additional Evaluations of Ohio River Bank Erosion- and Failure-Related Local Protection Project Endangerment Treatment Alternatives and Extents of Structures and Embankment Stabilization

During the period from 1967 to present Consultants, University Staff, and Ohio River Division and ERDC personnel have conducted Pittsburgh to Cairo mainstem reconnaissance, including evaluations of extensive bank erosion and failure conditions.

The data and conclusions from these efforts and reports were submitted as professional papers and, subsequent to Department Justice review and approval, were utilized as trial exhibits. Summation expert witnesses referenced these exhibits during both direct and rebuttal testimony before the United States Claims Court, District Courts, and Commission Hearings.

The District is utilizing these reports and site-specific investigations to design and construct several projects, related to flood erosion and bank failure endangerment of essential public facilities and LPP, and to complete bank stabilization work for others including the USFWS and State and Local Government entities.

These evaluations of terrace and slope topography, along reaches of the Ohio River and tributary stream banks, included measurements of channel incisement, thalweg migration, and erosion- and failure-related instability. Exposed intermittent bank and shallow water bench features exposed recently deposited sediments, failed soils, and Pleistocene reworked outwash and alluvial deposits.

Flood flow erosion has truncated these relic topographic features and formed oversteepened subaqueous and sub-aerial banks, terraces, and slopes. These erosion and failure sites were most often stabilized by placement of stone dikes, stone buttress, and stone slope protection treatments.

At locations where subaqueous bank topography was determined to be a relatively stable upper bank, treatments were constructed within terrace and slope areas above normal navigation pool. At locations where limited lower bank, terrace, and slope failure and erosion conditions were encountered, cost effective longitudinal dike treatments were constructed, which functioned to retain sediments and failed soils and to limit piping-related internal erosion and extents of recession-initiated bank collapse.

Timely monitoring and evaluations of these interrelated hydraulic and geotechnical processes was determined to be a most critical design consideration as required to affect construction of relevant low cost bank stabilization treatments.

However, more comprehensive design-related evaluations of these Local Protection Project stabilization treatments is required to better define flood event impacts, which most often result in extensive bank erosion and failure.

This data is also required to determine the long-term functionality of treatments at locations where flood event high velocity shear, tractive, and helical flows most probably result in bank and shallow water bench truncation and related displacement of longitudinal dikes and stone slope and buttress protections. Additional project maintenance and stabilization would be required within projects with limited stabilization components.

Additional Site Reconnaissance References

This presentation is based on reconnaissance, evaluations, analysis, design, and construction of limited bank treatments along navigable reaches and tributaries within the Ohio River drainage system.

Navigation pool retention has been affected by wicket roller and tainter gate dams, which are operated to provide for operation of commercial tows. These navigation system dam gate openings require adjustments to assure navigable depths during low and moderate flows. These gates are raised to required openings to permit relatively unimpeded passage of high flows and during flood events.

With the exception of localized swell head conditions, approximately one foot or less at, and adjacent to, the navigation dam, natural open river conditions are unaffected during flood events. During recession from high river stages to navigable pool stages, additional gate adjustments are again required to provide for commercial tow transiting.

These operations effect in-channel, bank, and terrace area changes in erosion and failure conditions as a result of reduced extents of river drawdown and extended periods for suspended and mixed load sediment deposition.

Since these interrelated processes are affected by navigation pool retention, applicability of the site evaluations, design, and construction methodology, as submitted in this presentation, must be limited by Ohio River navigation system conditions. Recently acquired bathymetric data has defined in-channel and near bank high stage and flood

flow erosion-related instability. These reach- and site-specific conditions could result in the launching and failure of longitudinal dikes and stone blanket and buttress treatments.

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SRT System Stabilizes Levee at Power Plant

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Abstract: The New Madrid Power Plant site in Missouri has a 23-acre raw water pond that stores water for use in plant operations. The pond is retained by a clay-fill levee that has experienced several shallow slope failures. A series of three slope failures occurred in the levees over several years as a result of rapid drawdown. The slopes were repeatedly repaired by traditional earthwork operations. These operations were continuously hampered by groundwater from the adjacent pond. After the third failure, it was determined that the Geopier SRT™ system would be an economical and long-term solution to the habitual failures. The SRT system utilizes patented Plate Pile steel reinforcing elements to rapidly and economically stabilize shallow slope failures. This paper presents the design and the construction related to the levee repair at the New Madrid Power Plant project.

Introduction

The occurrence of shallow failures on levees is very common. Levee embankments are often constructed of poor soils that have been excavated from the adjacent waterway. Levees also have differential slope conditions, with one side under water and saturated while the opposite side remains in the dry. Levees are often at risk of slope failure due to rapid drawdown conditions and/or poor foundation soils. Repair or reinforcement can be costly because the existing embankment must be maintained functional during the construction operations.

This article presents the design and the construction related to the levee repair at the New Madrid Power Plant project in New Madrid, Missouri.

Project Description

The coal-fired New Madrid Power Plant in New Madrid, Missouri has a 23-acre raw water pond to store water for use in plant operations. A rectangular-shaped levee surrounding the pond retains the water, which is collected from the nearby Mississippi River. A series of three slope failures occurred over several years along a 1,000-foot long stretch of the levee's embankment (see Fig. 1).

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Fig. 1. Slope failures in New Madrid Power Plant levee

The levee was constructed of silty clay fill. Borings indicated that the upper 20 feet consists of medium stiff silty clay fill, underlain by native soft to very stiff silty clay to depths of about 35 feet. Beneath the silty clay, medium dense to dense sand extends to the maximum depth explored of about 75 feet. Groundwater was encountered at the time of drilling at a depth of 30 feet, but the stabilized groundwater level is assumed to correspond to the elevation of the water in the pond.

The levee is approximately 20 feet high and has a slope inclination of 3 (horizontal) to 1 (vertical). The past slope failures in the levees were likely a result of rapid draw-down of the pond. Based on site observations and stability analyses performed by the project geotechnical engineer and the SRT (Slope Reinforcement Technology) engineers, the depth of the slope failures varied from 5 to 7.5 feet. The failures were repaired three times using earthwork re-grading operations, but these fixes were limited by the ground water associated with the adjacent pond. After the fourth episode of slope failure, a more robust and permanent stabilization method was sought by the client.

Initially, the geotechnical engineer proposed stabilizing the levees by installing a sheet pile wall at the toe of the levee in conjunction with excavation and recompaction. After several discussions between the geotechnical engineer, the owner, and the engineers at SRT, the patented SRT Plate Pile method was selected to provide a long-term repair. The SRT method requires minimal earthwork and provided an overall cost savings of about $1,000,000 for this project.

Geopier SRT System

The Geopier SRT™ slope stabilization method consists of driving steel reinforcing elements called Plate Piles™ into and through a slide mass or a potentially unstable soil layer. Plate Pile elements are installed in marginally stable slopes to increase the factor of safety against a shallow slope failure. They are also used in active or dormant landslides to restore the slope configuration and raise the factor of safety to accepted levels.

Plate Piles consist of steel sections to which rectangular plates are welded. The welds securing the plate to the shaft are only used to keep the plate in-place during driving. Once in the ground, the plate is compressed against the shaft by the soil pressures. Therefore, the welds are not stressed once the driving is completed. The pile shaft typically consists of a steel angle or S-shape section. The plate is typically 12 inches wide, with varying length based on depth to failure surface. Plate Piles may be galvanized but are more typically black steel. For Plate Piles in corrosive soil with a design life of 50 years, the pile section and plate thickness are increased by 1/8 inch (0.125 inch) to account for cross-sectional loss due to corrosion over its design life (FHWA 1990, FHWA 2001, CalTrans 2008).

Plate Piles are typically installed using small, tracked excavators with a hydraulic hammer attachment. Installation is a fast, clean, dry process that can occur even in bad weather. Following installation of the Plate Pile reinforcement, a vegetative erosion protection blanket may be placed over the reinforced slope area.

The Plate Piles are driven through the unstable layer to penetrate the underlying stable materials, as shown on Fig. 2a. Plate Piles are installed in a staggered grid pattern, as shown on Fig. 2b. Plate Piles mobilize the strength of the soil through arching and transmit slide forces to the underlying stiffer soil. The downslope force on each Plate Pile is resisted by the shear and bending strength of the Plate Pile shaft in combination with the passive resistance of the soil behind the plate.

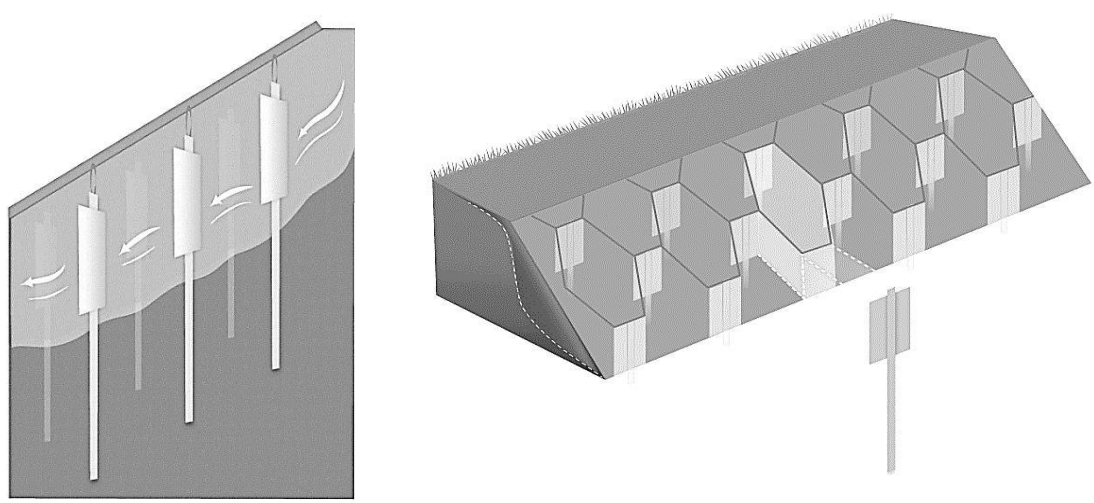

Fig. 2. Illustration of (a) Plate Piles in section view; (b) Plate Piles in plan view

SRT Design Approach

SRT worked closely with the owner's geotechnical engineering firm to create a design that would repair the levee and raise the stability factor of safety to 1.5. The pertinent design parameters include Plate Pile spacing and dimensions (e.g. steel section, length of pile shaft, length of plate). The shear and bending capacity of the Plate Piles are

dependent on the pile dimensions and the subsurface soil profile. The SRT design approach is discussed here.

Stability of Unreinforced Slope

The stability of the existing (unreinforced) levee was evaluated using the 2D limit equilibrium software *Slide* by Rocscience (2013). Since the slopes had already failed, the drained material property values of the subsurface soils were back-calculated based on a factor of safety of 1.0 (see Table 1). The maximum depth to the back-calculated failure plane was approximately 8 feet (see Fig. 3), which corresponded to the conditions observed in the field.

Table 1. Back-calculated drained material property values

Soil Layer	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)
Levee Fill	120	25	17
Silty Clay	120	50	27
Medium Dense Sand	120	0	35

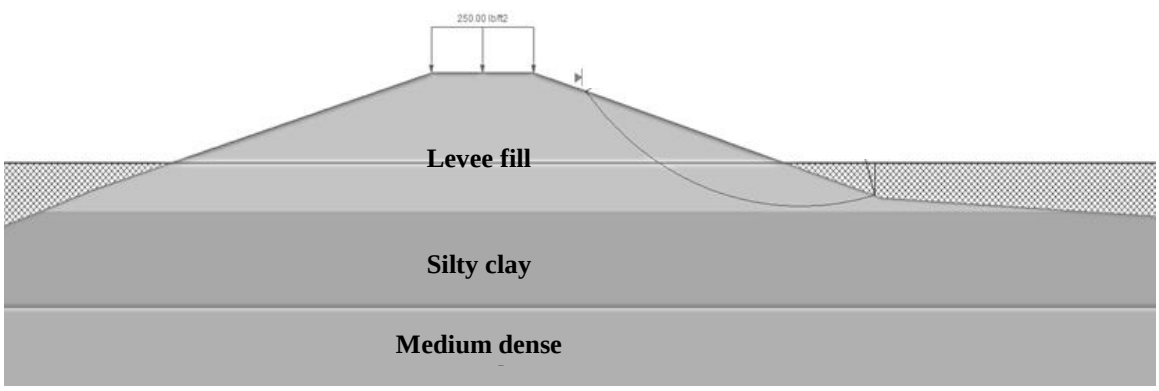

Fig. 3. Back-calculated failure surface

Plate Pile Spacing

Plate Piles are always installed 4 feet on center in the horizontal direction (i.e. parallel to the slope) in order to mobilize arching between the Plate Piles. The vertical (i.e. up-slope) spacing of the Plate Piles is dependent upon soil conditions, the predicted or actual depth of sliding, and the slope inclination. Initial estimates of the Plate Pile spacings are evaluated based on Geopier SRT feasibility charts. These design charts were developed using proprietary chart solutions that were validated through full-scale slope failure tests in partnership with researchers from the University of California, Berkeley and 3-dimensional numerical models (Short et al. 2012).

Based on the Geopier SRT proprietary chart solutions (Short et al. 2012), an initial Plate Pile spacing of 6 feet on-center in the up-slope direction was chosen. Plate Pile lengths of 10, 12 and 14 feet were chosen, with pile lengths increasing in the up-slope direction.

Shear and Bending Capacity of the Plate Piles

The shear and bending capacity of the piles were evaluated using the finite difference software program LPILE by ENSOFT, Inc. (2012). A 3 x 3 x ⅜ in. steel angle was chosen as the Plate Pile section. From AISC (2011), a 3 x 3 x ⅜ steel angle has an area (A) equal to 2.11 in.^{2}, moment of inertia (I) equal to 1.75 in.^{4}, and section modulus (S) in the x-direction equal to 0.825 in.^{3}; the elastic modulus value (E) is 29,000 ksi and yield strength (F_y) is 50 ksi.

LPILE computes deflection, shear, bending moment, and soil response with depth of laterally loaded piles in nonlinear soils. Soil behavior is modeled with internally-built p-y curves. The built-in p-y curves for cohesive soils in LPILE use undrained parameters. Though drained parameters were used in the stability analyses, it is reasonable to use undrained parameters to evaluate the immediate response of Plate Piles to lateral soil movement. The material property values used in the LPILE analyses are presented in Table 2. The p-y parameters, k and ε_{50}, were chosen based on recommendations provided in ENSOFT, Inc. (2012).

Table 2. p-y input values used in LPILE analyses

Soil Layer	Depth (ft)	p-y Model	Unit Weight (pcf)	Cohesion (psf)	k (pci)	ε_{50}
Levee Fill	0 - 8	Soft Clay	120	400	30	0.02
Silty Clay	8 - 35	Med. Stiff Clay	57.6	1000	150	0.01

The Plate Piles are loaded in LPILE by lateral soil movements over the depth of the slide plane until a limiting state is reached. The displacement value is transitioned to zero over a distance of 12 inches at the approximate location of the failure surface (Loehr & Brown 2007), as illustrated in Fig. 4. This approach is empirical but has been verified by instrumented case histories and numerical analyses, and is supported by Kourkoulis et al. (2012), Loehr & Brown (2007), and White et al. (2008). The limit state of the Plate Pile may be equal to the ultimate bending moment or the allowable lateral soil movement. Using the ultimate bending moment as a limit state is supported by FHWA (2005) and Loehr & Brown (2007). The shear force at the sliding depth when the first limit state is reached is considered to be the mobilized resistance for that sliding depth (Loehr & Brown 2007, FHWA 2005).

For this project, the pile length and depth to sliding plane varied as a function of pile location. (Pile lengths increased moving up the slope.) The results are presented in Table 3 for the case in which the pile length is equal to 14 feet and the depth to sliding is 8 feet.

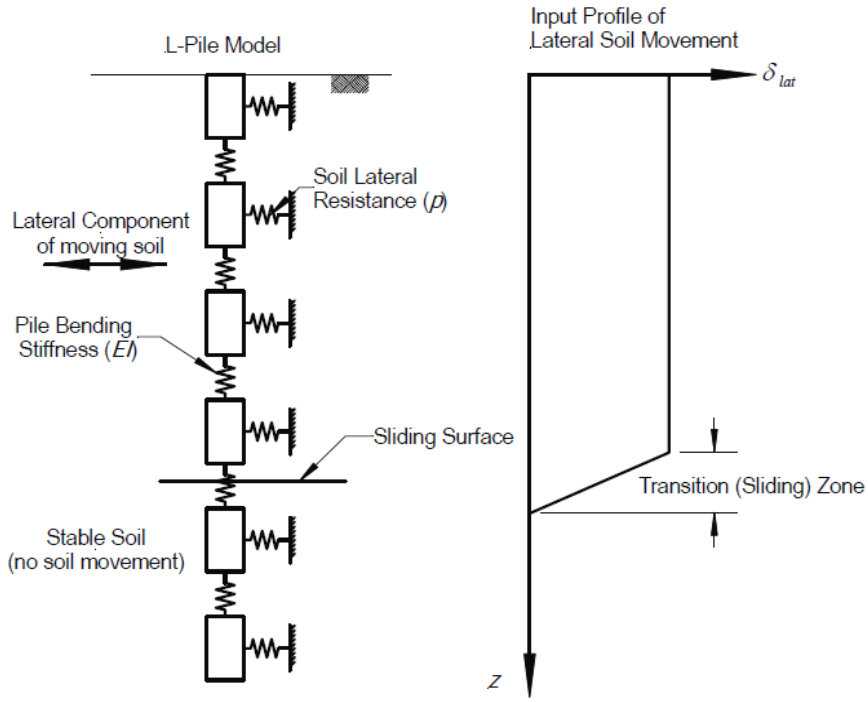

Fig. 4. Conceptual illustration of LPILE model used to compute lateral response of piles subjected to lateral soil movements (from Loehr & Brown 2007)

Table 3. Results of LPILE analyses

Input/Limiting Deflection (in.)	Max. Bending Moment (in-kips)	Yield Bending Moment[1] (in-kips)	Shear Forces at Slide Plane (lbs.)	Yield Shear[2] (kips)
1.0	26.2	41.25	1,590	105.5

[1] Yield Bending Moment: $M_A = F_y \cdot S$

[2] Yield Shear: $V_A = F_y \cdot A$

Stability of Reinforced Slope

Using the same slope geometry, subsurface profile, and loading condition as used in the analysis of the unreinforced levee, the factor of safety of the reinforced levee was evaluated. Plate Piles are modeled in *Slide* as "Micro Piles;" input parameters include up-slope spacing and the mobilized resistance, which is set equal to the shear force at the slide plane. The analysis of the reinforced slope resulted in a factor of safety approximately 1.7.

Plate Pile Installation

The final design for the levee stabilization included six rows Plate Piles driven into the failed slope. The Plate Piles were fabricated from 3 x 3 x ⅜ steel angles with 12 x 48 x ¼ inch steel plates; lengths of 10, 12, and 14 feet were used.

Fig. 5. Installation of Plate Piles at the New Madrid Power Plant

Prior to Plate Pile installation, the levee was track-rolled to form the final slope contours; no major grading was required. The Plate Piles were installed with a 3-man crew and two pieces of equipment (Fig. 5). Plate Pile locations were flagged prior to installation. A hammer attached to an excavator was used to install the Plate Piles. A chain attached to the hammer was wrapped around the individual Plate Piles just below the plate to lift and set the Plate Pile into position. Plate Piles are tilted into the slope at about a 5-degree angle from vertical to provide a slight batter against slope movement. Once in position, the chain was unhooked and the Plate Pile was driven to about one foot below the ground surface. Because of the small, mobile equipment, installation could occur close to the water's edge.

At the New Madrid levee project, the installation of 1,500 Plate Piles was completed in 12 days between June 24 and July 10, 2012. No problems have been observed in the levee to date.

Closing

The New Madrid Power Plant experienced chronic slope failures along a levee surrounding a raw water storage pond. The levee embankment was constructed of clayey fill that had failed three times in the recent past. The slope failures were previously repaired using earthwork re-grading operations, which were limited by the groundwater

associated with the adjacent pond. The levee was permanently fixed using the Geopier SRT™ method, providing the New Madrid Power Plant with a long-term slope repair option and an overall cost savings of $1,000,000. The solution involved minimal re-grading, and six rows of Plate Piles, ranging from 10 to 14 feet in length. Production rates varied from 90 to 150 Plate Piles per day; a total of 1,500 Plate Piles were installed in 12 days. This project demonstrated that the SRT system can successfully be installed in levees, while decreasing construction time and earthwork operations.

The Geopier SRT Plate Pile technology is best suited for real or predicted slope failures 10 to 15 feet deep. It may be used on slopes with inclinations up to 45 degrees (1H:1V) and in all soil types (with the exception of very loose to loose sand) overlying an underlying competent layer into which the Plate Piles penetrate. Plate Piles may be installed into soft rock (e.g. siltstone, claystone, mudstone, weathered shale, etc.). The SRT system is not suited to stabilize deep-seated (i.e. greater than 15 feet) failures

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An Annotated Bibliography (1954-2014) Concerning Slope Stability Analysis under Rapid Drawdown Conditions

Mark T. Bowers, Ph.D., P.E., M. ASCE[1]

Abstract: This annotated bibliography concerning slope stability analysis under rapid drawdown conditions arbitrarily begins in 1954 and continues to the present (2014). Earlier works by Taylor from 1937 and 1948 presented the first slope stability charts. Chowdury et al. (2010), summarized later in this bibliography, discuss Taylor's pioneering work and relevance. A sixty-year retrospective seemed appropriate as it would capture the major works that have been published during the lifetimes and careers of most of today's conference attendees. Most of these works are readily available through university Engineering and Geology libraries, on-line journal archives, and on-line (Google) searches. Some of these works are found in earlier proceedings of this annual Ohio River Valley Soils Seminar. This paper is not meant to be exhaustive. This editor is certain that there are many important works that could have been included that would have added to the conversation. The objective of this editor was to provide a thorough listing that would help practicing Geotechnical Engineers to better understand the issues surrounding rapid drawdown impacts on slope stability and to provide, in one source, a concise summary of the major contributions made over the last six decades to our approaches to solutions to the rapid drawdown case.

Chronologic Listing of Articles

Skempton, A.W. (1954), "The Pore Pressure Coefficients A and B," *Geotechnique,* Vol. 4, pp. 143-147.

Synopsis: In a number of problems involving the undrained shear strength of soils (especially in the design of earth dams) the change in pore pressure Δu occurring under changes in total stresses must be known. The equation:

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$$

is derived, and some typical values of the experimentally determined pore-pressure coefficients A and B are given.

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Bishop, A.W. (1954), "The Use of Pore Pressure Coefficients in Practice," *Geotechnique*, Vol. 4, pp. 148-152.

Synopsis: The pore-pressure coefficients defined by Skempton (1954) have been applied to the problems of determining the effective stresses in an earth dam during construction and during rapid drawdown. Reference is made also to several other practical applications of these coefficients.

Lowe, J. and L. Karafiath (1960), "Stability of Earth Dams Upon Drawdown," *Proceedings of the First Panamerican Conference On Soil Mechanics and Foundation Engineering*, Vol. 2, pp. 537-552.

Abstract: A new method of stability analysis is presented in this paper. The method has been developed primarily for the analysis of sloping core type earth and rock fill dams under the condition of rapid drawdown. For the purpose of determining the available shear strength upon drawdown, the equilibrium conditions at high reservoir level are analyzed. The principal stress ratios for the condition of anisotropic consolidation under high reservoir level are determined along the assumed failure surface. The shear strength used in the drawdown analysis is that obtained for undrained shearing from the anisotropic consolidation condition described above. The slices method of stability computation is used. Earth forces as well as water forces on the sides of the slices are considered. As a practical example the stability analysis for the Hirfanli Dam, Turkey, is given and comparisons are made with other conventional methods of analysis.

Sherard, J., R. Woodward, S. Gizienski, and W. Clevenger (1963), *Earth and Earth-Rock Dams*, p. 283:

The two main problems of nonsteady-state seepage which interest the designer are the rate of change in pore pressures during construction and the magnitude and rate of dissipation of residual pore pressures in the upstream portion of the embankment after reservoir drawdown. Neither problem is amenable to simple analytical treatment.

The prediction of "drawdown" pore pressures in the upstream zones of dams made with fine-grained soil is another of the major problems for which we have no reliable theoretical solutions. The most common method of estimating these pressures is to assume that the Laplace equation is applicable and that the reservoir level will be lowered rapidly enough to prevent substantial drainage inside the embankment. A flow net can then be constructed for the transitory seepage condition which would exist directly after "instantaneous drawdown." Assuming an incompressible embankment and no capillary forces, this is the most conservative prediction possible.

[Flow nets are discussed on pp. 284-286.]

On pages 370-377, Sherard et al. provide an outline as follows:

- Stability Condition Following Reservoir Drawdown

- Analytical Procedures
  - Total Stress Method
  - Estimating Pore Pressures for the Effective Stress Analysis

- Pore Pressures Due to Gravity Seepage after Instantaneous Drawdown

- Influence of Reservoir Level

- Influence of Drainage and Rate of Reservoir Lowering on Drawdown Pore Pressures
  - Transient Flow Nets
  - Casagrande Approximate Theory
  - Influence of Capillary Action on Rates of Drainage

Morgenstern, N. (1963), "Stability Charts for Earth Slopes During Rapid Drawdown," *Geotechnique*, pp. 121-131.

Chowdhury et al. (2010) summarized this paper stating:

> Morgenstern prepared stability charts for rapid drawdown conditions. Pore pressures after rapid drawdown are calculated in terms of the pore pressure parameters A and B.... Stability can be checked for complete or partial drawdown corresponding to the actual drop in reservoir water levels. For complete drawdown the critical slip circle is tangent to the top of the assumed rigid base but for partial drawdown it may be located within the slope. Therefore the factor of safety in the latter case is not found directly but by trial and error using the relevant charts.

Cedergren, H. (1967), *Seepage, Drainage and Flow Nets*, John Wiley and Sons, Inc., New York.

On page 148 Cedergren discusses a method of designing free-draining upstream shells of earthen embankments. The idea is to increase the stability of the upstream slope during rapid drawdown. The US Bureau of Reclamation (1987) Design of Small Dams draws from Cedergren noting:

> Where only fine material of low permeability is available, such as that predominating in clays, it is necessary to provide a flat slope if rapid drawdown is a design requirement. Conversely, if free-draining sand and gravel are available to provide a superimposed weight for holding down the fine material of low permeability, a steeper slope may be used. The same result may be secured by using sound and durable rock from required excavations. In the latter case, a layer of sand and gravel or quarry fines must be placed between the superimposed rock and the surface of the impervious embankment to prevent damage and displacement from saturation and wave action.

Hoek, E. (1970), "Estimating the Stability of Excavated Slopes in Open Cast Mines," *Transactions of the Institution of Mining and Metallurgy*, Vol. 79, pp. A109-A132.

Hoek writes in his introduction:

> On the basis of theoretical concepts which have proved successful in the analysis of slope failure, the author has compiled a series of design charts which are intended to provide an estimate of the stability of a slope under a given set of conditions. In order to use these charts, a certain amount of information on the slope geometry, material properties and groundwater conditions is necessary, and the paper contains suggestions for obtaining this information. Worked examples [6] are also given to illustrate the application of these charts to various slope problems.
>
> It should be emphasized that the design charts presented in this paper are not intended to replace the more sophisticated and more accurate methods of slope design which should be applied when the stability of a slope is critical and when the accuracy of the input information justifies the application of these methods. In preparing these charts, some of the accuracy of the more refined methods of slope stability analysis has been sacrificed in order to achieve the primary aim of simplicity and speed of application. This aim is motivated by the belief that an understanding of an engineering problem can most easily be acquired by working through a number of examples of that problem. Consequently, if the analysis of slope stability can be reduced to a set of easily understood charts, the engineer will be able to try out a number of solutions to a particular problem and so gain an appreciation of the sensitivity of that problem to changes in slope height and slope angle, material properties and groundwater conditions. Critical slopes may well require more detailed and accurate analysis; but, having proceeded as far as these charts will allow, the problem will have been defined with sufficient clarity to permit an informed decision to be taken on how these more detailed studies can best be carried out.

Desai, C. (1972), "Seepage Analysis of Earth Banks Under Drawdown," *Journal of the Soil Mechanics and Foundations Division*, ASCE, pp. 1143-1162:

Abstract: The problem of transient unconfined seepage under drawdown in riverbanks and dams is solved by using a finite element procedure. An iterative procedure is employed to compute movements of the free surface caused by time wise fluctuations in the external water levels. The finite element solutions are compared with laboratory experiments on a parallel-plate viscous flow model and field observations at a section along the Mississippi River. Correlation between the numerical solutions and observations is found to be good. Consideration is given to such special requirements of numerical techniques as discretization of infinite media and various possible flow situations at discretized end boundaries. An effort is made to obtain the numerical formulations and the computer codes that yield acceptable accuracy with economy. Some projections for use of the method for design analysis are presented.

Hoek, E. and J.W. Bray (1974), *Rock Slope Engineering*, Institution of Mining and Metallurgy, London, 310 pages.

Hoek, E. and J.W. Bray (1977), *Rock Slope Engineering*, revised second edition, Institution of Mining and Metallurgy, London, 402 pages.

Hoek, E. and J.W. Bray (1981), *Rock Slope Engineering*, revised third edition, Institution of Mining and Metallurgy, London, 358 pages.

These authors note that "the presence of groundwater in the rock mass surrounding an open pit has a detrimental effect upon the mining program for the following reasons":

- Water pressure reduces the stability of the slopes by reducing the shear strength of potential failure surfaces.... Water pressure in tension cracks or similar near vertical fissures reduces stability by increasing the forces tending to induce sliding.

- High moisture content results in an increased unit weight of the rock and hence gives rise to increased transport costs. Changes in moisture content of some rocks, particularly shales, can cause accelerated weathering with a resulting decrease in stability.

- By far the most important effect of the presence of groundwater in a rock mass is the reduction in stability resulting from water pressures within the discontinuities in the rock. Methods for including these water pressures into stability calculations are dealt with ... [in] this book.

Desai, C. (1977), "Drawdown Analysis of Slopes by Numerical Method," *Journal of the Geotechnical Engineering Division*, ASCE, pp. 667-676:

His introduction is as follows:

> The purpose of this paper is to illustrate the application of a numerical finite element scheme for development of guidelines and charts for analysis and design of slopes subjected to transient drawdown conditions. The physical problem considered concerns analysis of slopes at certain sections of the banks of the Mississippi River.
>
> The numerical procedure combines a finite element scheme for determination of the time dependent locations of free surface, and the circular arc-modified Swedish method of computing the factors of safety. Thus, the method yields variation of factors of safety during and after drawdown.
>
> As a part of the investigation, the factor of safety at the end of drawdown computed from the numerical procedure is compared with the factor of safety from conventional sudden drawdown analysis. A series of charts are obtained for

various combinations of problem parameters, and comments and suggestions concerning the potential of the procedure for analysis of slopes are presented.

Cousins, B. (1978), "Stability Charts for Simple Earth Slopes," ***Journal of the Geotechnical Engineering Division*****, ASCE, pp. 267-279.**

Cousins' conclusions are as follows:

- Stability charts have been presented which cover a wider range of slope angles and conditions than either Bishop and Morgenstern's, Spencer's or Janbu's charts.

- The format of the charts using the parameter, λ_c, is such that the charts contain all the advantages of Bishop and Morgenstern's charts and Spencer's charts without any of the disadvantages. For example, no trial-and-error approach is required in the selection of a suitable slope angle given the other variables or to determine the safety factor for a given slope.

- The limit equilibrium procedure used to construct the charts gives results very similar to those obtained from methods used by Bishop and Morgenstern, Spencer, and Taylor. However, the charts give consistently higher values for the safety factor than Janbu's charts for the case of pore pressure ratio $r_u = 0$.

- Janbu's method for allowing for the pore pressure ratio by incorporating it in the parameter $\lambda'_c = (1 - r_u)\lambda_c$ is reasonable for low values of the slope angle, α, the pore pressure ratio, r_u, and the parameter, λ_c. However, for large values of these variables the error is significant and is on the nonconservative side. Janbu's charts for intermediate values of the slope angle, α (approximately 20°), and the pore pressure ratio, r_u (approximately 0.25), give safety factors in close agreement with the charts given here because of compensating errors.

Duncan, J.M. and S.G. Wright (1980), "The Accuracy of Equilibrium Methods of Slope Stability Analysis," ***Engineering Geology*****, Vol. 16, pp. 5-17.**

These authors "provide guidance for selecting methods of analysis of slope stability, to achieve the best combination of accuracy and ease of analysis." They provide five principal conclusions:

1) The Ordinary Method of Slices is an accurate method for analysis for total stresses.

2) The Ordinary Method of Slices is very inaccurate when applied to effective stress analysis of flat slopes with high pore pressures.

3) Bishop's Modified Method is an accurate method of analysis for all conditions. Its limitations are that, like the Ordinary Method of Slices, it is only applicable to circular slip surfaces, and it is sometimes subject to numerical convergence problems.

4) Force equilibrium procedures (methods that do not consider moment equilibrium, such as wedge methods, the Corps of Engineers Modified Swedish Method, and Lowe and Karafiath's Method) can be used to analyze either circular or non-circular slip surfaces. Their drawback is that the calculated factors of safety are affected by the assumed inclinations of the side forces between slices.

5) Methods that satisfy all conditions of equilibrium (such as Morgenstern and Price's Method, Janbu's Method, and Spencer's Method) can be used for circular and non-circular slip surfaces. Their drawbacks are that they sometimes do not converge, and they may take an appreciable amount of computation time, depending on the speed of the computer on which they are run.

They continue:

> ...methods which satisfy all conditions of equilibrium give accurate results for all practical conditions. Regardless of the assumptions they employ, these methods (Janbu's, Spencer's, and Morgenstern and Price's methods) give values of F [safety factor] which differ by no more than +/- 5% from the correct answer. Bishop's modified method is also equally accurate, even though it does not satisfy all conditions of equilibrium.
>
> Based on these findings, equilibrium methods of stability analysis can be selected which can be relied on to produce results which involve no more than +/- 5% inaccuracy as a consequence of the approximations made in treating the mechanics of the problem. When this is the case, the engineer performing the analysis is justified in considering the factors of safety he calculates to be "correct" in terms of the mechanics of the problem, and he can devote his attention and concern to accurate evaluation of the properties of the soil.

Duncan, J.M. (1986), "Methods of Analyzing the Stability of Natural Slopes," *Proceedings of the 17th Ohio River Valley Soils Seminar*, October 17, 1986, Clarksville, Indiana.

Duncan notes the following:

> For many conditions stability charts provide factors of safety that are as accurate as those that can be calculated by detailed computer analyses, and they can be performed in a fraction of the time required for computer analyses. The main requirement for achieving good accuracy with slope stability charts is use of systematic procedures for averaging soil strengths and unit weights.... In many cases the accuracy with which factors of safety can be calculated using charts is equal to or better than the accuracy with which soil conditions and strength values are known. In the writer's opinion, slope stability charts are under-appreciated and under-used.

U.S. Bureau of Reclamation (1987), *Design of Small Dams*, Third Edition, United States Government Printing Office, Denver.

On page 242 the effect of rapid drawdown of a reservoir on the embankment slope is noted:

> The most critical operating condition so far as the stability of the upstream slope is concerned is a rapid drawdown after a long period of high reservoir level.

An example of Alcova Dam, Wyoming is given in which the reservoir water surface was lowered 120 feet in 40 days. Figures are presented that demonstrate that appreciable pore water pressures remain in an embankment after drawdown. They continue:

> If a dam is subject to rapid drawdown after long-term storage at high reservoir levels, special provisions for drainage should be made in the design. The upstream slope of an embankment with an appreciable upstream pervious zone usually is not critical for the rapid drawdown condition. Rapid drawdown may require a flatter slope of a homogeneous embankment than would otherwise be needed for stability.

On page 245 it is noted that "a storage dam subject to rapid drawdown of the reservoir should have an upstream zone with permeability sufficient to dissipate pore water pressure exerted outwardly in the upstream part of the dam. The rate of reservoir drawdown is important to the stability of the upstream part of the dam. For a method of designing free-draining upstream shells, refer to Cedergren (1967)."

On page 245-246 they note the impact of rapid drawdown of a reservoir on embankment stability.

Pariseau, W.G., S.C. Schmelter, and A.K. Sheik (1997), "Mine Slope Stability Analysis by Coupled Finite Element Modeling," *International Journal of Rock Mechanics and Mining Sciences*, Vol. 34, Issues 3-4, Proceedings of the 36th U.S. Rock Mechanics Symposium, Paper No. 242.

Abstract: The potentially destabilizing effect of water pressure on rock slope stability is examined assuming coupled poroelastic/plastic behavior and compared with previous poroelastic results. A coupled finite element code that accounts for the simultaneous effects of rock mass deformation and transient fluid flow is used for this purpose. Rock mass behavior is based on the concept of effective stress, Hooke's law, Darcy's law, associated plasticity and a parabolic yield condition appropriate to rock masses. The main effect of plasticity, which limits the range of purely elastic behavior by rock mass strength, is greater displacements and persistent yielding. Yielding anticipated in poroelastic analyses, where the ratio of strength to stress is less than one, is initially somewhat more extensive than in the poroelastic/plastic case, but diminishes considerably with time. In the poroelastic/plastic case, yielding that occurs during a slope cut persists in time and space despite depressurization. Applicability of poroelastic/plastic finite element analysis to actual open pit mine slopes is demonstrated.

Duncan, J.M. (1999), "The Use of Back Analysis to Reduce Slope Failure Risk," *Civil Engineering Practice*, Journal of the Boston Society of Civil Engineers Section, ASCE, Vol. 14, No. 1, Spring/Summer, pp. 75-91.

His conclusions include the following points:

> Back analysis can be used to develop highly reliable computational models of slopes. It can be used where slopes have failed, such as at the San Luis Dam and Olmsted Locks and Dam, and where slopes have not failed, such as at La Esperanza Dam.
>
> When the results of back analysis are used for the design of slope stabilization measures, they should be used consistently with the assumptions made in the back analysis. While assumptions are inevitably required to develop computational models, adjusting the elements of the model through back analysis results in a model that is not significantly affected by the assumptions, provided that they are reasonable and fit all the known facts.

Marr, W.A. (1999), "Effective Uses of Finite Element Analysis in Geotechnical Engineering," *Civil Engineering Practice*, Journal of the Boston Society of Civil Engineers Section, ASCE, Vol. 14, No. 2, Fall/Winter, pp. 89-98.

Among his conclusions, Marr notes the following:

> The use of finite element analysis in day-to-day geotechnical practice will increase considerably over the next few years. This greater use is due to the presence of tremendous computing power on most engineers' desks, the availability of reliable finite element software that most engineers can learn to use and the increasing computer literacy of young geotechnical engineers.
>
> This widespread capability does cause some concerns. Analysts with inadequate geotechnical knowledge should not use finite element programs to solve complex geotechnical problems. A strong understanding of effective stress principles and of soil behavior is essential to anyone doing finite element analysis of geotechnical problems for design.
>
> There is also the problem of inexperienced persons consuming project resources trying to do finite element analyses without coming to a useful answer. These analytical failures give finite element analysis a bad name. While it is possible to obtain an answer with finite element analysis in a few hours, some geotechnical problems can become quite complex. Getting an appropriate model can become quite involved. Evaluating and interpreting the output can be intellectually demanding and time consuming. Any team working on a complex problem and using finite element analysis should have at least one person on the team who is well versed and experienced in the finite element tools being proposed for the project.

There is also the trend for people to be impressed with nice-looking graphics even though the information presented in those graphics may not make sense or address the key issues of the project. Impressive graphics can be prepared from meaningless information. Engineers will become ever more professionally challenged trying to figure out which of these impressive graphics make sense and help advance a project.

As finite element tools become more sophisticated and easier to use, the emphasis is decreasing on how to do the analysis and focusing more on obtaining meaningful input information.

Pauls, G.J., E. Karl Sauer, E.A. Christiansen, and R.A. Wigder (1999), "A Transient Analysis of Slope Stability Following Drawdown After Flooding of Highly Plastic Clay," *Canadian Geotechnical Journal*, Vol. 36, pp. 1151-1171.

Abstract: The stability of slopes at bridge abutments across the Carrot River in east-central Saskatchewan was not influenced significantly by drawdown after flooding in the spring of 1995. Traditional methods of analysis for rapid drawdown predicted the factor of safety of slopes on highly plastic clays of proglacial Lake Agassiz would drop to 0.65 from an initial value of 1.0. Deformation along a well-defined slip plane has persisted at a more or less constant, slow rate since the bridge was constructed in 1975. The river rose approximately 10 m during a flood in the spring of 1995, yet there was only minimal response in piezometers and no measurable increase in the rate of deformation recorded by inclinometers. Pore-water pressures from a steady state seepage model, which was calibrated from piezometer measurements, were integrated into a stability analysis. Changes in pore-water pressures caused by flooding and subsequent drawdown were characterized from a transient seepage model using the flood hydrograph as a flux boundary. The stability analysis integrated with the transient seepage model estimated the factor of safety would drop from 1.0 to 0.91 after drawdown. Field measurements indicated the reduction in factor of safety was even less.

Whittle, A.J. (1999), "The Role of Finite Element Methods in Geotechnical Engineering," *Civil Engineering Practice*, Journal of the Boston Society of Civil Engineers Section, ASCE, Vol. 14, No. 2, Fall/Winter, pp. 81-88.

Whittle states in his introduction that "successful application of finite element analyses in geotechnical engineering requires a clear understanding of the design problem/parameters to be solved (based on sound knowledge of the geological context, soil/rock mechanics, groundwater hydrology, etc.) and an understanding of the capabilities and limitations of finite element modeling techniques." An outline of the paper is as follows:

- Current Program Capabilities
- Some Limitations
    - Complexity of ground conditions or inadequate site characterization

- Complexity of geomaterial behavior or inadequate data for appropriate constitutive models
- Complexity of construction processes that are difficult to represent in finite element models

- Future Needs

Whittle concludes that long-term development of finite element analyses will require progress in three main directions:

1) Improvements in the measurement, interpretation and representation of material properties in finite element analyses.

2) A need to overcome conceptual difficulties associated with non-deterministic methods of analysis and also the need to demonstrate practical benefits in design.

3) Substantial and sustained educational efforts are needed to train geotechnical engineers in the most effective use of finite element analyses. This process is already occurring in the form of professional short courses related to the recent spread of commercial geotechnical finite element programs. However, there is also need for adequate training in finite element methods in most geotechnical graduate degree programs (in the United States). This training requires providing both background (theoretical basis and techniques of numerical analyses) and application knowledge (modeling nonlinear soil behavior, parameter selection, etc.) This situation will certainly change now that high-quality (user-friendly and robust) software is available.

Hoek, E., J. Read, A. Karzulovic, and Z.Y. Chen (2000), "Rock Slopes in Civil and Mining Engineering," published in *Proceedings of the International Conference on Geotechnical and Geological Engineering, GeoEng 2000*, November 19-24, 2000, Melbourne, Australia.

Abstract: Rock slopes with heights approaching 1000 m are now being designed and excavated in various open pit mining and civil engineering projects around the world. The economic impact of excessively conservative designs or of failures in these slopes can be very large and every effort has to be made to optimize their design. Their size means that they will almost always contain a number of significant structural features and a variety of geological materials.

The input required for these models includes a comprehensive geological data base that contains both structural geology and lithological information, a hydrogeological model that permits water pressures to be estimated throughout the slope, estimates of rock mass and discontinuity strength and deformation properties and an understanding of external forces, such as those due to earthquakes, that may be imposed on the slope. Site investigation techniques that can be used to obtain this information and the methods, that

can be used to estimate rock discontinuity and mass properties, are reviewed in this paper.

Developments in limit equilibrium and numerical modeling techniques are reviewed and their applicability to the design of these large slopes is presented. The importance of blasting control in the excavation of the slope is discussed. The use of drainage to improve the stability of the slopes is also considered. Monitoring the behavior of the rock mass and the subsurface groundwater during construction and subsequent operation of the slope is an important component of rock slope engineering and these techniques are reviewed. Practical examples from large projects around the world are presented.

Lane, P.A. and D.V. Griffiths (2000), "Assessment of Stability of Slopes Under Drawdown Conditions," ***Journal of Geotechnical and Geoenvironmental Engineering,*** **ASCE, pp. 443-450.**

Abstract: The traditional approach for estimating the stability of slopes under different submergence conditions is the charts of Morgenstern and, more recently, proprietary computer programs, both utilizing limit-state analyses. The chart approach is limited by geometry and material property considerations and the limit-state approach by assumptions about analysis method and failure mechanism. The finite-element method offers a powerful method for analyzing complex geometries and properties of slope stability problems, but may be unattractive for routine use by supervisory staff. By comparison a chart based approach is useful, particularly when setting operating conditions on, for example, drawdown rates for dams and reservoirs. This paper seeks to explore the use of the finite-element method to produce operating charts for such circumstances that should be applicable to real structures.

Pariseau, W.G. (2000), "Coupled Geomechanic-Hydrologic Approach to Slope Stability Based on Finite Elements," Chapter 11 in ***Slope Stability in Surface Mining*** **edited by Hustrulid, McCarter and Van Zyl, Society for Mining, Metallurgy, and Exploration, Inc., Littleton, Colorado.**

His conclusions are as follows:

> The detrimental effect of water on the stability of soil and rock slopes is well known. This effect is quantified in coupled stability analysis where rock-mass deformation and water flow interact instantaneously with a mining cut and subsequently in time as a new equilibrium position is approached. A coupled finite element computer program allows for site-specific analyses that include consideration of pre-excavation stresses and fluid pressures, given rock-mass elastic moduli, strengths, fluid compressibility, hydraulic conductivities, and coupling constants. Different rock types, anisotropic rock, and rock joints may be taken into account. Generally, fluid compressibility needs to be included; the assumption of incompressibility is not usually justified in rock mechanics, unlike the situation in soil mechanics. Computed data for a specified mining step are changes induced by the advance and include changes in displacement, strain,

stress, pressure, fluid velocity, and fluid content. These data provide guidance to stability evaluation, especially in the form of local safety factor contours in the vicinity of the slope walls. A "local" safety factor is simply the ratio of strength to stress at a considered point in the region of analysis.

In a coupled analysis, the most dangerous time is immediately after a blast when the fluid pressure is momentarily elevated and effective stresses are reduced. In time, depressurization occurs and greater stability is achieved. There seems to be little difference in rock slope stability based on depressurization in comparison with desaturation, which would require a much greater drainage effort. As a matter of computational economy, two program runs may suffice, one using a time step of zero and a second using a very large time step.

Uncoupled analysis that uses a conventional "dry" computer program for stress analysis of a mining cut and then imports a fluid pressure distribution attributed to steady-flow through the cut slope geometry was shown to be generally erroneous and an approximation of unknown reliability. An analysis of this all-too-common approach demonstrated the nature of the problem and led to a proposed modification that alleviates the error to some extent. Of course, the technically sound coupled approach is preferable.

Lorig, L. and P. Varona (2000), "Practical Slope Stability Analysis Using Finite Difference Codes," in Chapter 12 in *Slope Stability in Surface Mining* edited by Hustrulid, McCarter and Van Zyl, Society for Mining, Metallurgy, and Exploration, Inc., Littleton, Colorado.

They introduce their paper as follows:

> Numerical models tend to be general purpose in nature, i.e., they are capable of solving a variety of problems. While it is often desirable to have a general-purpose tool available, each problem must be constructed individually. The elements must be arranged by the user to fit the limits of the geomechanical units and/or the slope geometry. Hence, numerical models often require more time to set up and run than special-purpose tools (such as limit-equilibrium methods).
>
> Numerical models are used for slope stability studies for a variety of reasons, including:
>
> - Empirical methods cannot confidently be extrapolated outside their databases.
> - Other methods (e.g., analytic, physical, limit equilibrium) are not available or tend to oversimplify the problem, possibly leading to overly conservative solutions.

- Key geologic features, groundwater, etc. can be incorporated into numerical models, providing more realistic approximations of real slope behavior.

- Observed physical behavior can be explained.

- Multiple possibilities (e.g., hypotheses, design options) can be evaluated.

They particularly address the finite difference programs FLAC and UDEC. They note that "FLAC is formulated to study continuum problems, although a limited number of discontinuities (in the form of interfaces) can be included. UDEC is formulated to study discontinuum problems involving large numbers (hundreds) of explicit discontinuities that divide the rock mass into blocks."

Fell, R., P. MacGregor, D. Stapledon and G. Bell (2005), *Geotechnical Engineering of Dams*, A. A. Balkema Publishers, Taylor and Francis Group, London.

On pages 466-470 they present the following outline:

- Drawdown Pore Pressures and the Analysis of Stability Under Drawdown Conditions

    - Some general issues

    - Estimation of drawdown pore pressures, excluding the effects of shear-induced pore pressures

    - Methods for assessment of the stability under drawdown conditions

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Duncan, J.M. and S.G. Wright (2005), *Soil Strength and Slope Stability*, John Wiley and Sons, Inc., Hoboken, New Jersey.

On page 32 they note the following:

> Rapid or sudden drawdown is caused by a lowering of the water level adjacent to a slope, at a rate so fast that the soil does not have sufficient time to drain significantly. Undrained shear strengths are assumed to apply for all but the coarsest free-draining materials ($k > 10^{-3}$ cm/sec). If drawdown occurs during or immediately after construction, the undrained shear strength used in the drawdown analysis is the same as the undrained shear strength that applies to the end-of-construction condition. If drawdown occurs after steady-state seepage conditions have developed, the undrained strengths used in the drawdown analysis are different from those used in the end-of-construction analyses. For soils that expand when wetted, the undrained shear strength will be lower if drawdown occurs some time after construction than if it occurs immediately after construction.

On page 33 they note:

> Sudden drawdown removes the stabilizing effect of external water pressures and subjects the slope to increased shear stress. Either drained or undrained strengths are used, depending on the permeability of the soil.

Chapter 9, pages 151 through 160, cover analyses for rapid drawdown. They present the following outline in this chapter:

- Drawdown During and At the End of Construction
- Drawdown for Long-Term Conditions
  - Effective Stress Methods
  - Total Stress Methods
    - First-stage computations
    - Second-stage computations
    - Third-stage computations
    - Example
- Partial Drainage

Kerkes, D.J. and J.B. Fassett (2006), "Rapid Drawdown in Drainage Channels with Earthen Side Slopes," ***Proceedings of the ASCE Texas Section Spring Meeting*****, Beaumont, Texas, April 2006, 9 pages.**

Abstract: The rapid drawdown case is one of the most severe loading conditions that an earthen slope can experience and is quite common in storm water drainage channels along the Texas Gulf Coast. Flooding in adjacent rivers can leave water levels high in these drainage channels, which can drop rapidly once floodwaters recede. While the development of deep seated failure surfaces is possible, the effect on earthen side slopes is most commonly seen in the form of relatively shallow slope failures, which if left unattended lead to gradual deterioration of the channel slopes. Though the information presented in this paper is generally applicable to any earthen side slopes, of particular interest are slopes consisting of overconsolidated clays where it is not uncommon for slope failures to occur long after a channel has been constructed. This paper discusses a number of the factors associated with this loading condition, as well as some of the design issues, and presents the results of a series of finite element seepage analyses that were performed to investigate the effect of some of the variables on the advance of the zone of saturation into the channel side slope.

Viratjandr, C. and R.L. Michalowski (2006), "Limit Analysis of Submerged Slopes Subjected to Water Drawdown," ***Canadian Geotechnical Journal*****, pp. 802-814:**

Abstract: A rapid draw of water from a reservoir can cause a temporary increase in the hydraulic gradient that may not be tolerated by the slope of an earth dam. The increased seepage forces may lead to slope instability, causing the collapse of the structure. The

kinematic approach of limit analysis is used to examine stability of slopes subjected to a rapid or slow drawdown. Combinations of slope inclination, soil properties, and hydraulic conditions are found for which the slope becomes unstable. The results are presented in the form of charts for convenient practical use, and the safety factors can be obtained from the charts without the need for iteration. For granular slopes, particularly if shallow, subjected to drawdown, a simple translational mechanism with a shallow failure surface is not the most adverse mechanism for all water-draw regimes.

Berilgen, M.M. (2007), "Investigation of Stability of Slopes Under Drawdown Conditions," *Computers and Geotechnics*, pp. 81-91.

Abstract: Because of the rapid drawdown there will be a decrease in the slope stability which might lead to instability in slopes that do not have sufficient level of safety against failure. This paper presents an investigation of slope stability during drawdown depending on the soil permeability, drawdown rate and drawdown ratio, considering the nonlinear material and loading conditions. For this purpose, a coupled transient seepage and deformation analysis (including consolidation), together with the stability analysis, were performed using FEM for submerged slopes. Nonlinear elasto-plastic behavior of the slope soil is taken into account while analysis of the generation and dissipation of pore pressure is carried out.

Pinyol, N.M., E.A. Alonso, and S. Olivella, (2008), "Rapid Drawdown in Slopes and Embankments," *Water Resources Research*, 22 pages.

Abstract: The rapid drawdown condition arises when submerged slopes experience a rapid reduction of the external water level. Classical procedures developed to determine the flow regime within the slope and the resulting stability conditions are reviewed in the paper. They are grouped in two classes: the "stress-based" undrained approach, recommended for impervious materials, and the flow approach, which is specified for rigid pervious materials (typically a granular soil). Field conditions often depart significantly from these simplified cases and involve materials of different permeability and compressibility arranged in a complex geometry. The drawdown problem is presented in the paper as a fully coupled flow-deformation problem for saturated/unsaturated conditions. Some fundamental concepts are first discussed in a qualitative manner and, later, explored in more detail in synthetic examples, solved under different hypotheses, including the classical approaches. Some design rules, which include a few fundamental parameters for the drawdown problem, have also been solved in a rigorous manner to illustrate the limitations of simplified procedures. A significant portion of the paper is devoted to the discussion of a comprehensive case history. In Shirs, earth dam pore pressures were recorded at different points inside the embankment during a controlled drawdown. Predictions of four calculation procedures (instantaneous drawdown, pure flow, coupled flow-elastic, and coupled flow-elastoplastic, all of them for saturated/unsaturated conditions) are compared with measured pressure records. Only the coupled analysis provides a consistent and reasonable solution. The role of the different soil properties in explaining the phenomena taking place during drawdown is finally discussed.

Michalowski, R.L. (2009), "Critical Pool Level and Stability of Slopes in Granular Soils," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, pp. 444-448:

Abstract: The influence of pore-water pressure and the pool water pressure on stability of submerged slopes was investigated using the kinematic approach of limit analysis. For soils with some cohesive component of strength, the critical pool level is slightly below half of the slope height, whereas for slopes built of purely granular soils the critical pool level is not well defined. The most critical mechanism of failure for submerged granular slopes was found to have the failure surface intersecting the face of the slope, with one intersection point above, and the other one below the pool level. The solution to the stability problem was found to be independent of the length scale (slope height), and equally critical mechanisms of failure can be triggered "locally" with any water level in the pool. The safety factor associated with these mechanisms is lower than the well-known factor defined by a planar failure surface approaching the slope face.

Chowdhury, R., P. Flentje, and G. Bhattacharya (2010), *Geotechnical Slope Analysis*, CRC Press, Taylor and Francis Group, London.

Pages 174-175 discuss rapid drawdown of water level in a slope of cohesionless soil.

Pages 178-186 discuss rapid drawdown of water level in a slope of cohesive soil using the following outline:

- Effective stress analysis: proposed new approach
- Total stress analysis
- Notes of finite slope with partial submergence
- Examples using both the effective stress approach and the total stress approach

Page 245 discusses effective stress charts and average pore pressure ratio

Pages 649-675 is an Appendix entitled "Slope Stability Charts and Their Use for Different Conditions Including Rapid Drawdown." The contents of this appendix include the following:

1) Chart for parameter m_a in Bishop simplified method (also Janbu's method).
2) Introduction to slope stability charts.
3) Taylor's charts and their use.
4) Cousin's (1977) charts–studies in terms of effective stress.
5) Example concerning use of Cousin's charts.

6) Charts by Hoek (1970) and Hoek and Bray (1974, 1977).

7) Rapid drawdown-effective stress approach (after Bishop (1954) and Skempton (1954)).

8) Construction pore pressures in impervious fill of earth dam (after Bishop (1954)).

Leroueil, S. and L. Picarelli (2012), "Assessment of Slope Stability," *Geotechnical Engineering State of the Art and Practice*, Geotechnical Special Publication No. 226, ASCE, pp. 122-156.

Their conclusions are presented as follows:

> Assessing the stability of slopes is a very difficult task that requires an experienced engineer and a rigorous approach. For that purpose, the geotechnical characterization of slopes that classifies the information on slopes is a very useful tool.
>
> The paper evidences the tremendous progresses that have been made in the domain of slopes: there are new technologies, with possibilities to take automatically readings and send them via wireless links to an interpretation center; there are also interesting progresses in our understanding of some aspects of soil and slope behaviors; there has been development of very powerful coupled hydro-mechanical numerical models that can consider saturated and unsaturated soils, progressive failure, etc., and allow the analysis of complex problems. All this constitutes a real revolution in our way to monitor and analyze slopes.
>
> A methodology, not new as such, is proposed for the assessment of slope stability. Its main characteristics are as follows:
>
>> It includes a qualitative stability assessment based on past performance and on comparison with slopes in the vicinity that have similar geomorphology.
>>
>> It recognizes the shortcomings of limit equilibrium analyses but considers that they have been extremely useful to the profession and will remain in use in practice.
>>
>> It is thought that the required factor of safety should not have a fixed value, but a value that integrates the concepts of risk, increasing with the consequences of a potential failure, decreasing when the uncertainty decreases and when the understanding of the processes involved progresses; also, the value could be reduced if associated with an adequate monitoring and an observational approach. The authors however recognize the difficulties for establishing guidelines for this approach.

Because the processes involved are completely different from those at the stages of failure and reactivation, and because its consequences may be major, it is considered that the post-failure stage should be examined systematically and separately.

The coupled hydro-mechanical numerical models overcome most of the shortcomings of the limit equilibrium methods and give the possibility to assess slopes in terms of their behavior (mostly deformations) rather than in terms of factor of safety. There is no doubt that they will be more and more often used in practice in the future.

VandenBerge, D.R., J.M. Duncan, and T.L. Brandon (2013), "Rapid Drawdown Analysis Using Strength Reduction," ***Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering,*** **Paris.**

Abstract: The undrained shear strength during rapid drawdown is controlled by the properties of the embankment fill material and the consolidation stresses prior to drawdown. Current design methods use limit equilibrium analyses to evaluate both the consolidation stresses and the stability of the slope after drawdown. The method described in this paper uses the finite element method to calculate the consolidation stresses throughout the slope during steady state seepage before drawdown. Undrained shear strengths are calculated for all nodes in the model based on the major principal effective consolidation stresses and the results of ICU triaxial tests. The undrained strength of each element in the model is determined by interpolation from the strengths at the surrounding nodes. Using these strengths and an elastic-plastic constitutive model, the stability of the slope is evaluated by the strength reduction method. Back analysis of rapid drawdown failures suggests that undrained strengths from ICU tests should be reduced by 30% for the rapid drawdown conditions.

These authors present two examples of rapid drawdown failures in a comparison between their proposed method and the limit equilibrium method.

They compare the widely accepted limit equilibrium procedure for rapid drawdown stability analysis to their proposed finite element method noting the following strengths and advantages of their approach:

It follows the conventional approach for analysis of rapid drawdown and other short-term loading problems by using total stress stability analysis. They later note that a total stress representation is appropriate for undrained problems because the very great difficulty in predicting pore pressures during undrained loading makes it infeasible to use effective stress analyses for these cases.

The use of the finite element method to determine the consolidation stress state is an improvement over the use of limit equilibrium methods for this purpose. In 1960, when Lowe and Karafiath developed their groundbreaking method, using limit equilibrium to calculate consolidation stresses was the only choice. Today,

however, with finite element analyses becoming widely available, it is logical to use the finite element method for calculation of consolidation stresses.

Lopez-Acosta, N.P., Fuente de la H.A., and G. Auvinet (2013), "Safety of a Protection Levee Under Rapid Drawdown Conditions. Coupled Analysis of Transient Seepage and Stability," ***Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering,*** **Paris.**

Abstract: The rapid drawdown condition arises when submerged slopes of protection levees experience a rapid decrease of the external water level. In this paper the safety of a protection levee under rapid drawdown conditions is studied by numerically modeling this phenomenon as a coupled problem of transient seepage-deformation in a saturated/unsaturated medium. Analyses are performed based on finite element method by using the PLAXFLOW program for transient seepage analysis and the PLAXIS program for deformation, consolidation and stability analyses. The details of the proposed methodology are presented in this work. Also, recommendations for definition of material type (drained or undrained), type of soil constitutive model (Hardening Soil and Mohr Coulomb), boundary conditions and mesh generation of finite elements are provided. In the main part of the paper, the effects of multiple parameters such as position of phreatic surface, water drawdown ratio, drawdown rate and hydraulic conductivity are evaluated by a 2D model of stress-strain. Special emphasis is given to the study of the safety factor variation as a function of time obtained when assessing the stability of these earth structures. Finally, concluding comments about the results are exposed.

Conclusions

This editor has presented abstracts, synopses, summaries, and conclusions from 36 publications which appeared during the period 1954-2014 on the subject of slope stability analysis under rapid drawdown conditions. It is hoped that this annotated bibliography will serve a useful purpose to bring into the hands of practicing Geotechnical Engineers the background, theories, and approaches to the solution of rapid drawdown problems. The works cited cover a broad range, from the development of the pore pressure parameters A and B, into slope stability charts, effective stress approaches, total stress approaches, limit equilibrium methods, and recently into coupled hydro-mechanical numerical methods. Overall, it seems fitting to repeat the conclusion drawn by Duncan (1986):

> Analysis of stability of natural slopes requires careful consideration of the method of analysis, and the slope geometry, groundwater conditions, and soil strength. Consideration of the accuracy, advantages, and limitations of the various methods of slope stability analysis provides a rational basis for selecting a suitable method of stability analysis. For many conditions slope stability charts provide an accurate and efficient method of analysis.

Use of Small Diameter Micropiles to Mitigate Scour around Bridge Embankments

Nathan Beard, P.E., M. ASCE[1]

Abstract: The Minisceongo Creek is a tributary river to the Hudson River located in Rockland County, New York. In recent years a significant amount of scour has occurred along a nearly 1,000-foot stretch of the stream situated in glacial till; coincidently this project is located in the area where subsurface utilities cross the stream. In September of 2011 a large single event caused by Hurricane Irene occurred in which a substantial amount of erosion of the cutbank occurred. The erosion was mainly found in the sandy gravel embankment that overlies the large gravel boulder till of the riverbed. Results of the single event included a cut back of approximately 60 feet into the hillside that had dramatically over-steepened the slope. The remaining exposed slopes are now nearly 90-foot tall and at grades of up to 2V:1H. To address the protection of the subsurface utilities a large concrete buttress in combination with grouted rip-rap was utilized. However, the long-term stability of the embankments and associated lands above the river was still required. GSI designed and implemented a combination of soil nail walls to protect the steep embankments, while maintaining adjacent property lines. Scour protection was accomplished using a combination of scour micropiles, Geosynthetically Confined Soil (GCS)® Wall, and riprap.

This design is often used to avoid scour of river and waterfront structures where slope instability, abutment integrity, or limited access are a concern. This presentation will highlight the design, review and construction of the scour remediation efforts for this project. The system was verified through an independent Scour Evaluation Report.

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Scioto Greenways – Geotechnical Challenges of Development within an Existing Riverfront

Rich Williams, Ph.D., P.E., M. ASCE[1], Travis White, P.E., M. ASCE[2], and Eric Reeves, P.E., M. ASCE[3]

Abstract: The Scioto Greenways project in downtown Columbus, Ohio is transforming a portion of the over-widened Scioto River into a vibrant, more natural water course, which will improve the ecological systems and river habitat while creating a new 33-acre greenway/park environment with new recreational options within the existing river limits. Integral to this project is the removal of a low head dam in downtown, along with the narrowing of the predominantly active river flow channel formed from the placement of 400,000 cu. yd. of embankment material within the existing river footprint (see Fig. 1). The new embankments support the park environment, which includes walking/biking paths, greenspace and special event venue areas. Many of these park features are supported, in part, by retaining structures, since the new river banks connect the existing city streetscape to the normal river water level, some 35 feet difference in elevation.

Fig. 1. Scioto Greenways Project Location, Columbus, OH

Two typical geotechnical challenges routinely must be addressed when developing such projects in an existing urban environment: 1) the soil-structure interaction associated with

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new retaining structure designs; and 2) the soil-structure interaction associated with new earthwork impacts on the existing structures which remain in place. This document focuses on both of these issues by addressing the geotechnical/structural approach to design of the 30-ft. high by 650-ft. long Lower Promenade Wall (adjacent to the east river edge) and the soil-structure response of the over 80-year old Olentangy Scioto Interceptor Sewer (OSIS) to the placement of up to 35 ft. of new embankment against this critical existing structure. Due to the height and length of the Lower Promenade Wall, the concepts for retention of the new embankment were honed through a combination of multiple design refinements coupled with contractor-suggested alternatives associated with cost containment. Due to the age and unknown condition of the OSIS structure, the sewer structure condition required assessment, and the structure response to the new river embankment required modeling/quantifying with the aid of finite-difference software (FLAC) analyses.

Project Background and Purpose

Over the past century and a half, the Scioto River in Columbus, Ohio has experienced dramatic development along the river banks within the area which now constitutes much of the western portion of the downtown area. The development was mostly industrial-based but included some residential development. As with many inland US rivers, occasional flooding has occurred within the Scioto River basin passing through the downtown Columbus area. The most notable flood occurred in 1913 (see Fig. 2), with subsequent floods of lesser proportions in later years. As a result of the destruction imparted to the development along the river, a system of reservoirs and low-head dams was constructed along the Scioto and Olentangy Rivers, since the Olentangy and Scioto River merge near the approximate western edges of downtown Columbus. In 1918, the Main Street low-head dam was constructed at a location immediately downstream of the Main Street bridge, creating a constant shallow pool of water within the river behind the dam throughout the downtown area, which promoted a reduction of river flow velocity and an increase in storage capacity, especially under elevated water events. The dam was raised to a height of 13.5 feet (an additional 1.5 feet) in 1938.

However, these low-head dams had some other effects. One dangerous side effect was that swift (submerged hydraulic jump) and turbulent flow was promoted in the immediate below-dam areas of the river, often claiming lives of people trying to cross the dams due to the apparent shallow water depth at the dam crest. A second side effect was the stagnation of water flow, especially during periods of minimal precipitation, leading to fish kills, low oxygen content and unpleasant odors.

One remedy for this situation involves full dam removal and subsequent stream restoration, wherein the impediments to flow (i.e., the low-head dams) are removed in order to increase flow velocity. The dam removal is then enhanced with bank stabilization and the creation of riffle zones and pool zones in order to enhance the self-oxygenating potential within the flowing water and also promote habitats for aquatic life. When such a concept was proposed in public meetings with stakeholders in Columbus as a way to rejuvenate the Scioto River environment, the response was overwhelmingly positive. Fortunately within the Columbus area, many stakeholders have just witnessed

the stream restoration process upstream from downtown, with the successful Fifth Avenue Dam removal and Olentangy River Restoration adjacent to The Ohio State University campus.

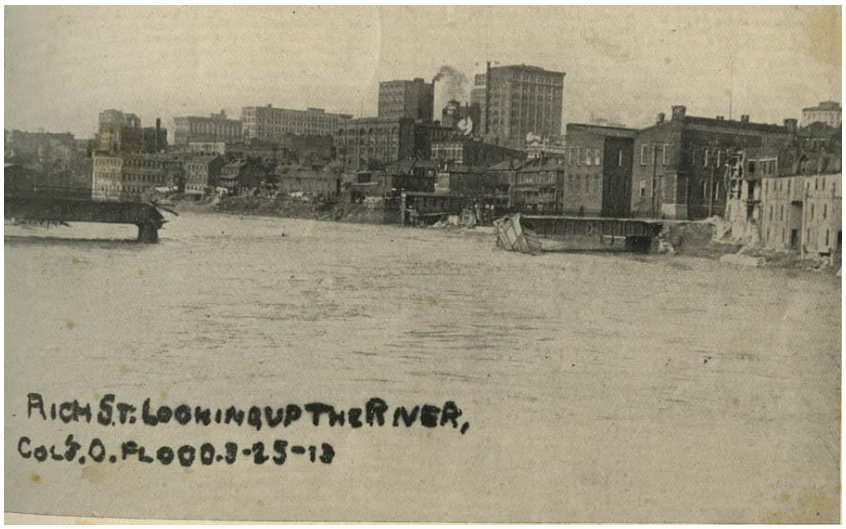

Fig. 2. Looking North on Scioto River, March, 1913 Flood

In addition to the removal of the Main Street low-head dam, the Scioto Greenways project includes the creation of over 33 acres of park green space, as the river width through downtown is narrowed from approximately 600 feet in width to 300 feet in width. The riffle-pool sequences are then constructed within this narrowed channel to enhance river life. The creation of the green space is accomplished by constructing new river banks within the project limits along both banks, requiring the placement of up to 35 feet of new embankment against the existing concrete retaining structures and a major trunk sewer structure, currently in place along the river's edge. Additionally, several park-related structures are being created to enhance the attractiveness and use of the green space and attract signature events to the river. The most notable structure is the Lower Promenade Wall, which will be positioned along the water's edge of the east river bank, opposite the Center of Science and Industry (COSI), and rise 12 feet above the normal pool water level and extend over 650 feet along the bank. This Wall feature is important in maintaining the historical context of the Civic Core as coordinated with the Ohio Historical Preservation Office (OHPO).

Subsurface Information

Significant geotechnical information was available from publically available document archives, as several downtown bridges crossing over the Scioto River within the project limits had been replaced within the past 20 years and the alignment for a new major trunk sewer tunnel passed beneath portions of the project. This existing base of subsurface knowledge provided significant information with regard to the underlying bedrock consistency and topography, resulting in the requirement for only a few additional isolated borings to be performed at select locations in order to better define support

conditions for key structures (see Fig. 3). However, securing the additional geotechnical information of the soil overburden within the river channel required drilling and sampling within the active water limits of the river. Since the water level was considered too deep in most locations to permit drive-on drill rig access, but also too shallow in many locations to depend on barge support for a floating drilling plant, Stantec mobilized a Marsh Buggy unit, from Louisiana, to the site and mounted our drill rig on the Buggy. The Marsh Buggy permitted free movement of the rig between borings within the river, no matter the depth to river bottom.

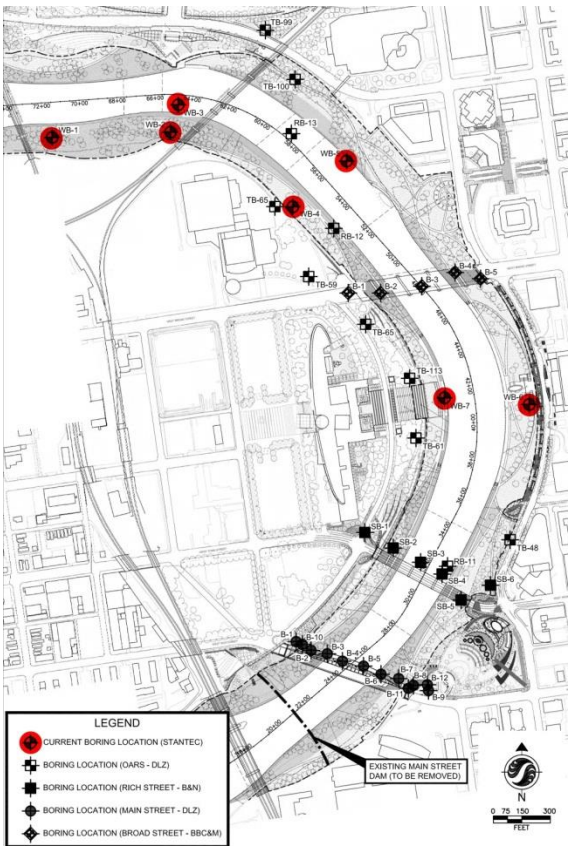

Fig. 3. Boring Locations within Scioto Greenways Project Limits

In general, the underlying bedrock within the Scioto River Basin consists of weak shale overlying fractured limestone. Fairly strong till overlies the bedrock, having been consolidated by the glaciers which advanced and then retreated over this region. The deposits overlying the till exhibit the most variation, since these deposits are the product of river-influenced deposition. The borings performed for this investigation generally confirmed the findings of previous investigations but allowed for better definition of the nearer river bottom deposits which would impact soil-structure interaction analyses. In general, the subsurface stratigraphy within the existing river limits consists of about 20 feet of recent alluvium overlying about 10 feet of the till, which is underlain by about 25 to 30 feet of weak shale over the limestone (see Fig. 4).

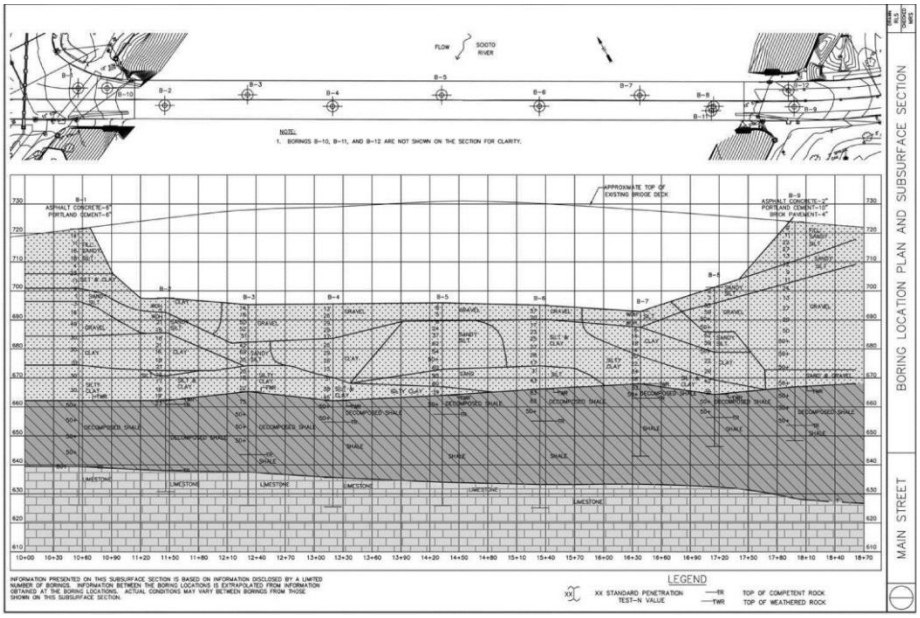

Fig. 4. Subsurface Schematic Section – Main Street Bridge, Columbus, OH

Lower Promenade Wall

One of the proposed major features associated with the Scioto Greenways project is the construction of an approximate 650-foot long wall (called the Lower Promenade Wall) along the edge of the new east bank of the river. This proposed wall will stand about 12 feet above the estimated new top of river level. The wall retains new earthen embankment fill in order to create a level plaza type of surface at the top for support of foot and bicycle traffic. The bottom of the new river channel in front of this wall varies; however, near both the north and south limits of this Lower Promenade Wall, pool sections within the new river channel are to be excavated to depths up to 15 feet below the proposed river level. As a result, the maximum vertical exposed height of retained earthen fill (behind the wall) and exposed existing river deposits totals 35 feet (including scour effects) within these pool zones.

To facilitate eventual excavation of the new river channel bottom profile and also support the Lower Promenade Wall, a drilled shaft system consisting of deep steel-reinforced drilled shafts and interfill plug shafts (see Fig. 5) was preferred, as the size of the shaft units and reinforcing could be adapted to the applied loads while still offering full earth retention of the existing deposits. A drilled shaft diameter of 5.0 feet was selected, since removal of cobbles and boulders (known to be present within the Scioto River deposits) could be better facilitated with this size drilled shaft auger, and this auger size is common within most of the drilled shaft construction industry. At the preference of the landscape architect and in accordance with the agreement with OHPO, the exposed wall was required to mimic the existing cast-in-place floodwall positioned along the east limits of the existing Scioto River in downtown. Various concepts were considered for the drilled shaft/Lower Promenade Wall/backfill configuration, with the goal of reducing the lateral load transmitted to the top of the drilled shafts as well as limiting the lateral deflection of

the drilled shafts and wall under service loads. The final selected arrangement is shown in Fig. 6, wherein a system of tiebacks anchored to deadmen buried within the new embankment fill was selected. This arrangement offered resistance to lateral loads at the top and bottom of the Lower Promenade Wall as well as at the top of the drilled shafts (at the cap). This system also offered sufficient resistance to maintain lateral deflections to within acceptable limits.

The soil-structure interaction of the structural drilled shafts was then analyzed under two conditions: 1) during drilled shaft and wall construction (with no excavation of river deposits); and 2) during service (after excavation of the pools within the river). LPILE, V6 was used to assess deflections, bending moments and shear forces within the drilled shafts as part of the drilled shaft design process. Target maximum lateral deflection at the top of the drilled shafts under Condition 1 was elected to be 0.1 inch, with the target maximum lateral deflection at the top of the shaft under service load (Condition 2) was elected to be 1.0 inch. The resultant structural drilled shaft design meeting both of these conditions included a W14X26 steel member for the majority of shafts supporting the wall and a W21X55 steel member for a few of the shafts positioned at the ends of the wall.

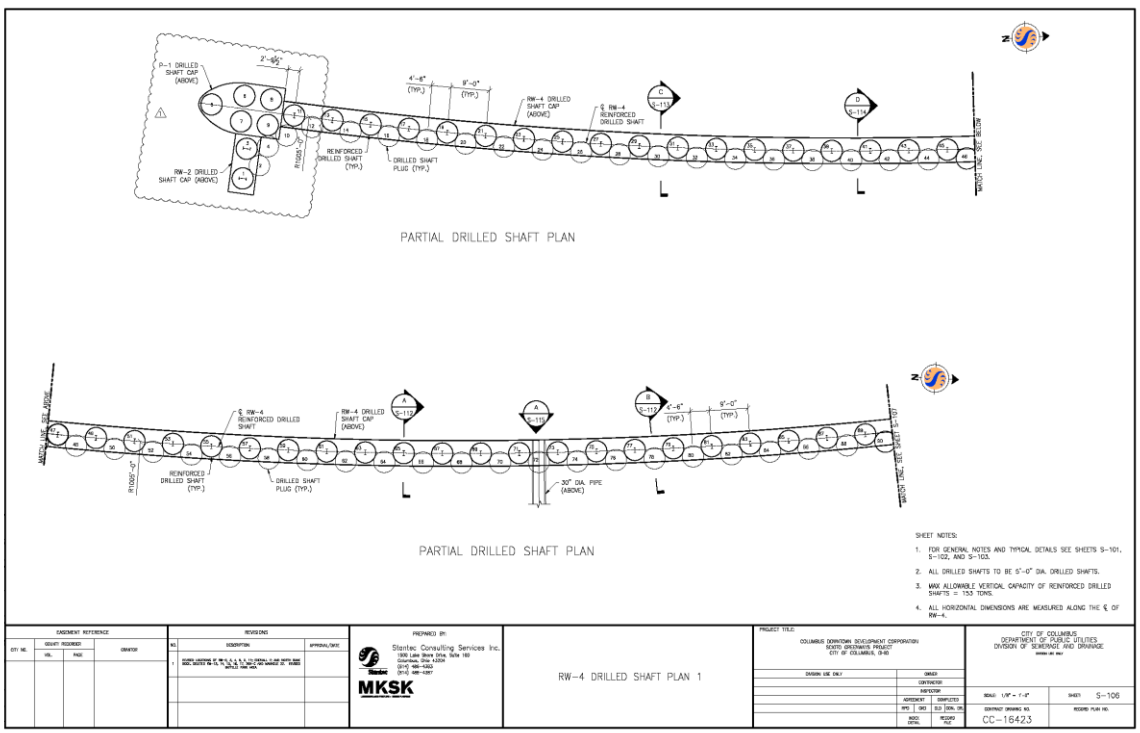

Fig. 5. Drilled Shafts Foundation Orientation – Lower Promenade Wall

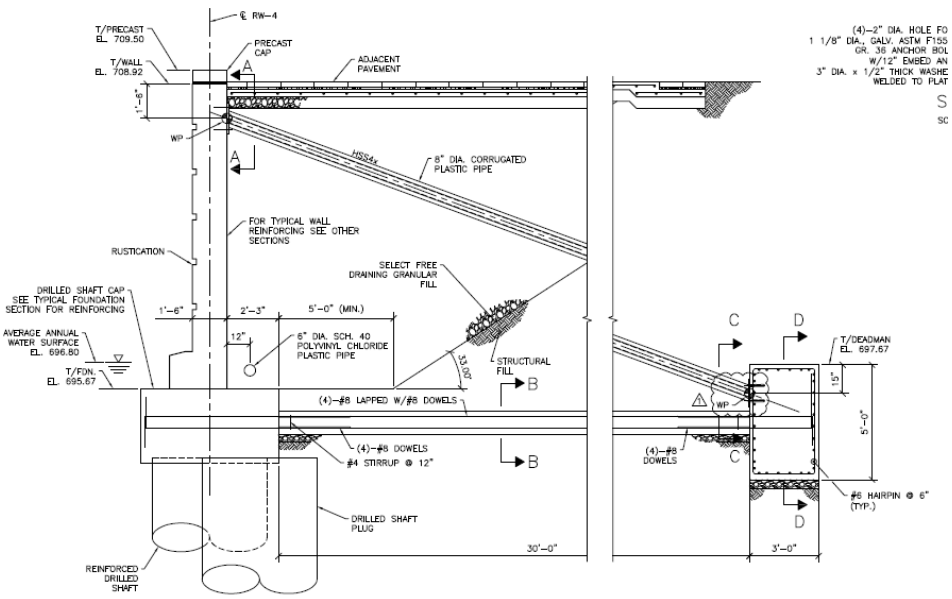

Fig. 6. Lower Promenade Wall – Deadman – Tieback Arrangement

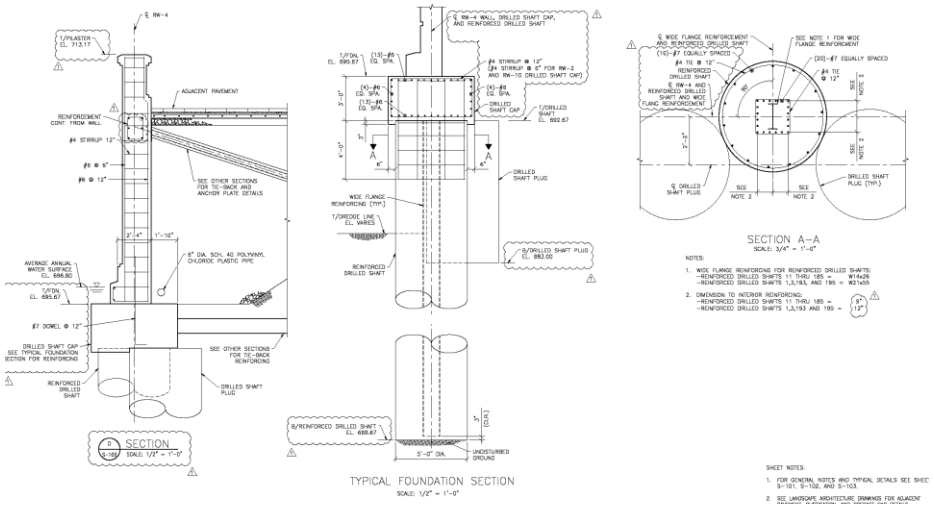

Fig. 7. Lower Promenade Wall – Drilled Shaft Structural Details

OSIS/Floodwall Analyses

The second soil-structure interaction challenge on this project involved the assessment of the impact of new embankment fill, to be placed directly against the west wall of the Olentangy Scioto Interceptor Sewer (OSIS), on the OSIS structure. Up to 35 feet of new embankment fill is to be placed against the OSIS. The OSIS collects and conveys sewer flows from a large tributary area to the east and north of the downtown Columbus area. The OSIS is a heavily-reinforced concrete structure (see Fig. 8) that was constructed in 1928. Subsequent to completion of the OSIS, the Columbus floodwall was constructed on top of the OSIS, and backfill was placed behind these structures, forming the area now termed the Scioto Mile in Columbus. Proposed new embankment fill is to be placed to

within 2 feet below the grade of the Scioto Mile within the zone to the east of the Lower Promenade Wall, and this constitutes as much as 35 feet of new earthen fill against the OSIS and floodwall.

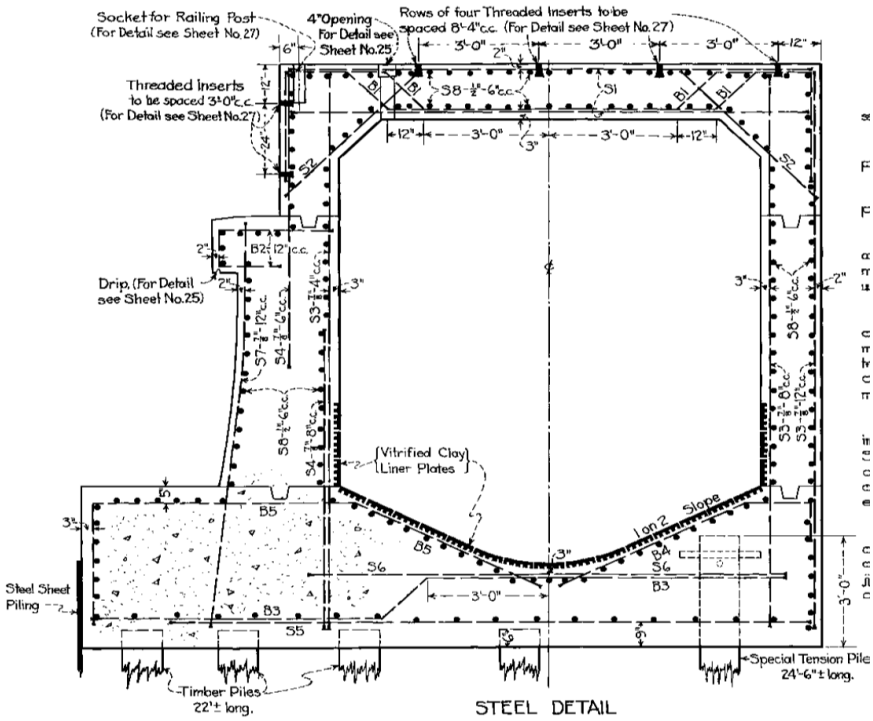

Fig. 8. OSIS – Typical Section

Consequently, the potential impact to the OSIS structure had to be addressed, since any resultant structural overstresses or cracks could have negative consequences. As the OSIS consistently conveys flows and rerouting of those flows for the purposes of facilitating detailed internal visual inspection was not considered possible, a program of exterior structural assessment coupled with soil-structure interaction modeling (using the finite difference software FLAC) was implemented.

The physical assessment of the OSIS and floodwall included the verification of reinforcing steel placement and size as well as the procurement of concrete core specimens at selected locations along the exposed west wall of the OSIS. Resource International, Inc. of Columbus was retained to provide ground penetrating radar (GPR) field measurement services (see Fig. 9) for this task, and they were supported in the field with personnel from Shelley Metz Baumann Hawk, Inc. (SMBH) structural engineers, since SMBH was knowledgeable of the OSIS/floodwall structure, having been retained for the previous Scioto Mile work. The GPR work was performed from a floating unit, with access restricted to the middle portion of the west OSIS exterior wall. As the GPR displayed/recorded the reflections, the output was also used by the team on site to physically mark the location of prospective reinforcing steel on the exposed concrete surface. Such markings were used to alert the followup coring crew as to the locations of

the reinforcement, with the goal of minimizing any long term negative impacts to the structure as part of our field work. Concrete cores were subsequently procured (see Fig. 10), with these cores used for visual assessment and strength testing purposes.

Fig. 9. Hand-Held GPR Unit (courtesy RII)

Fig. 10. OSIS – Concrete Coring

The field assessment program and lab core testing results confirmed that the in-place OSIS structure had been constructed in general accordance with the 1928 plans, with concrete strength equal to or in excess of design strength and reinforcing steel of a size and placement consistent with the plans. Consequently, straightforward structural analyses of the west wall of the OSIS was performed and indicated that the placement of the new embankment fill should not promote development of unacceptable levels of shear or bending within the wall structure and that the existing OSIS structure would still retain sufficient structural capacity.

In addition to the structural assessment of the OSIS, the soil-structure interaction assessment addressed anticipated movements (deflections or rotations) of the OSIS in response to various load cases simulating existing and proposed conditions. The finite difference program FLAC was used as a means of modeling the soil-structure interaction and computing the deflections or rotations of the OSIS/floodwall system at key locations (see Fig. 11). As noted in Figs. 8 and 11, the OSIS was constructed on a system of wood piling, largely due to the unknown but variable bearing conditions along the bank of the Scioto River in 1928, as well as due to certain temporary loading conditions that would exist during construction and backfilling of the sewer in 1928. So the soil structure interaction needed to quantify the anticipated changes associated with pile loading as well as the anticipated deflections/rotations of the OSIS under the various loading cases. Eight different loading cases were modeled:

- Existing OSIS/floodwall configuration, normal 1-year river pool level
- Existing OSIS/floodwall configuration, 100-year flood level
- OSIS/floodwall with proposed fill, normal 1-year river pool level
- OSIS/floodwall with proposed fill, 100-year flood level
- OSIS/floodwall with proposed fill, post 100-year flood, return to 1-year pool level
- OSIS/floodwall with proposed fill & Geofoam, normal 1-year river pool level
- OSIS/floodwall with proposed fill & Geofoam, 100-year flood level
- OSIS/floodwall with proposed fill & Geofoam, post 100-year flood, return to 1-year pool level

For the purposes of analyses, the displacements/rotations computed from the first load case were used as a baseline, and initialized to “zero” to indicate the existing condition. The results of all subsequent load cases reflected this initialization. The Geofoam application consisted of the placement of this material directly against the west wall of the OSIS/floodwall, and extending outward to cover the toe of the OSIS, with the intent of possibly reducing the vertical load imparted by the new fill on the OSIS toe.

The results for two of the analyses are shown in Figs. 12 and 13, with a tabular presentation of the analyses included in Table 1. In general, the most significant movement of the OSIS appeared to occur after the return of the river level to a normal 1-year pool after a 100-year flood event, wherein the east edge of the OSIS structure was computed to deflect downward as much as 0.15 inch. Also under normal loading conditions, the OSIS appears to move in a similar manner, but with smaller deflections/rotations. The addition of Geofoam appears to reduce the computed displacements, but the improvement seemed inconsequential, considering the extra costs associated with the use of Geofoam along several thousand feet of OSIS wall.

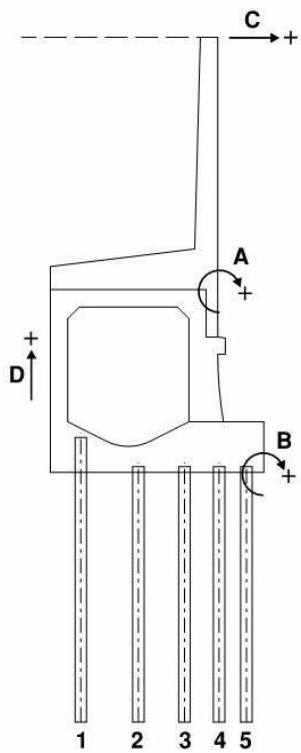

Fig. 11. OSIS/Floodwall Model

Table 1. Summary – OSIS Computed Movements vs. Load Case (FLAC Analyses)

LOAD CASE	COMPUTED MOVEMENT			
	A* (deg.)	B* (deg.)	C* (in.)	D* (in.)
Existing configuration, normal 1-year pool level	0.000**	0.000**	0.000**	0.000**
Existing configuration, 100-year flood level	0.004	0.006	-0.011	0.071
Configuration with proposed fill, normal 1-year pool level	-0.009	-0.009	-0.442	-0.151
Configuration with proposed fill, 100-year flood level	-0.008	-0.011	-0.440	-0.041
Configuration with proposed fill, post 100-year flood, return to 1-year level	-0.010	-0.012	-0.459	-0.151
Configuration with proposed fill & Geofoam, normal 1-year pool level	-0.027	-0.003	-0.425	-0.101
Configuration with proposed fill & Geofoam, 100-year flood level	-0.025	-0.004	-0.424	-0.032
Configuration with proposed fill & Geofoam, post 100-year flood, return to 1-year level	-0.028	-0.004	-0.434	-0.102

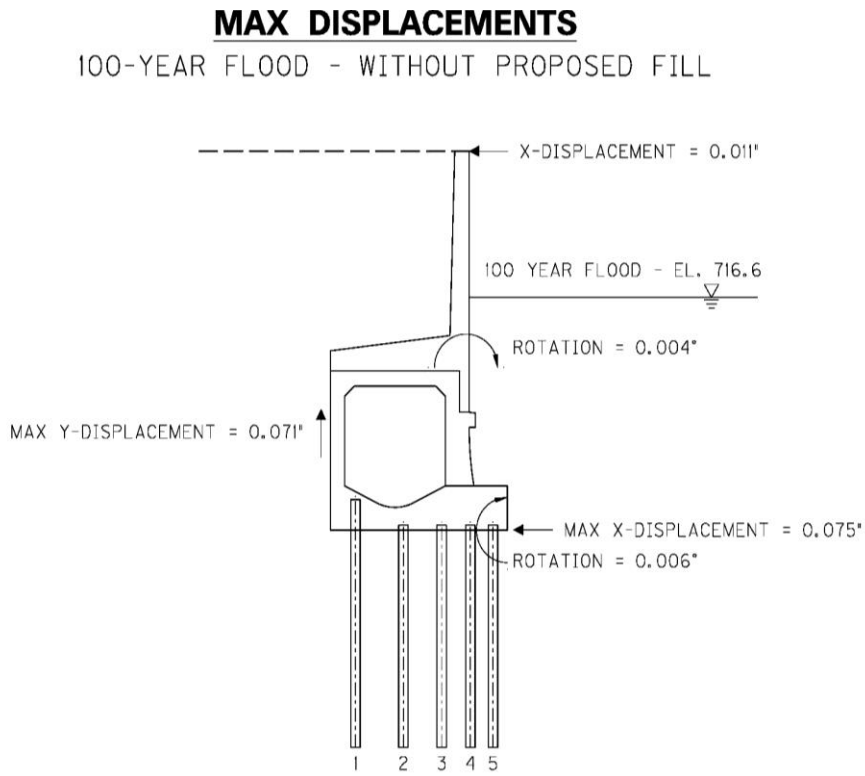

Fig. 12. OSIS/Floodwall – FLAC Output Case 2

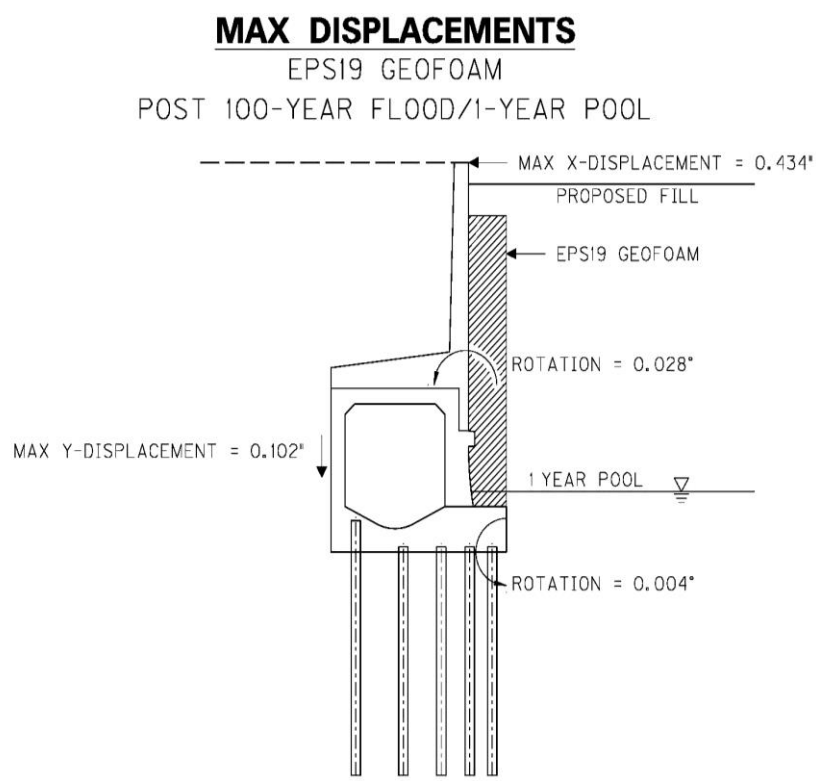

Fig. 13. OSIS/Floodwall – FLAC Output Case 8

The FLAC output also provided information with regard to load imparted to the existing piles under the various load conditions previously described. Of particular concern to the City of Columbus Department of Public Utilities (DPU) was the predicted load in Pile 1 (ref. Fig. 11), positioned at the easternmost location along the OSIS back wall, as the piles at this location were subjected to a temporary uplift load during one phase of the construction and the pile butt had been embedded higher into the OSIS foundation. For the loading conditions modeled, the computed pile loadings did display variation from the existing condition; however, no pile was computed to sustain a loading in excess of the maximum pile load (for any pile) computed in the 1928 calculations.

Construction Considerations

In response to the results of the soil-structure interaction analyses and the concerns of DPU staff, a deflection monitoring program was established at the start of construction and is on-going, with duration slated through to the end of the construction period in late 2015. Ideally, measurement of deflections within or on the OSIS structure would have been preferred. But access to the interior of the OSIS (confined space and air quality issues) for survey purposes was not permissible, and access to the OSIS rustication strip on the west exterior wall required a boat and was deemed an unacceptable location for regular monitoring. Since the FLAC output provided lateral deflection estimates for the top of the floodwall and this location would be accessible during the entire construction period, it was elected to establish monuments at multiple locations along the top of that wall for use in the monthly movement survey program. Even though the movements of the top of the floodwall do not impact the OSIS, it was felt this program would provide some relative indication as to behavior of the system, in light of the movements computed in the FLAC modeling program.

Acknowlegements

Stantec wishes to acknowledge the City of Columbus, the project owner, and the Columbus Downtown Development Corporation, the project developer, for the vision and support of this cityscape changing project. Additionally, the landscape architect, MKSK, and the construction manager at-risk, Messer Construction, were integral members of the design team working together with Stantec to deliver the Scioto Greenways vision, while maintaining the projected project budget within the level of dedicated funding sources.

Project funding partners include:

- City of Columbus
- US Environmental Protection Agency (319)
- CDDC
- Franklin County
- Columbus Foundation
- Ohio Environmental Protection Agency (WRRSP)
- Ohio Department of Transportation
- Battelle Memorial Institute
- Columbus Metro Parks

The Evolution of Levee Safety within the U.S.

Tammy L. Conforti, P.E.[1]

Abstract: Topics discussed will include national challenges such as: state of the infrastructure, development patterns, expectations and behavior, resource constraints, and other considerations. The transformation from "levees" to "levee safety" will be discussed. Other topics include: implementing the USACE Levee Safety Program Risk Framework, USACE/ FEMA joint efforts, and USACE Levee related policy updates.

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Several Relief Well Design Considerations for Dams and Levees

Erich D. Guy, Ph.D., P.G.[1], Halis M. Ider, P.E., A.M. ASCE[2], and Kenneth Darko-Kagya, Ph.D., P.E.[3]

Abstract: Seepage beneath flood protection structures can be controlled by established measures including impervious riverside blankets, cutoff walls, landside drainage trenches, seepage blankets, and relief wells. While different approaches exist for designing and evaluating relief well systems, they can be somewhat cumbersome, and computed solutions are sensitive to a number of parameters. This paper presents an overview of several concepts critical for analyses and illustrative examples from work performed at two flood protection projects (Portsmouth, Ohio and Lawrenceburg, Indiana). Appropriate effective and total stress safety factor formulations for evaluating vertical seepage-related heave and uplift potential are presented. The importance of considering well elevation- and efficiency-related aspects is discussed, and an example of well rehabilitation with transmitting capacity assessment is presented. A practical method for considering finite well line effects during system design using plan view finite element modeling is shown. Additionally, some proposed corrections to design equations presented in the widely utilized U.S. Army Corps of Engineers (USACE) relief well design manual (EM 1110-2-1914) are presented.

Relief Well Design Methods

Relief well systems are designed with the objective of reducing foundation seepage pressures to a tolerable level. Early design requirements for seepage control measures were published in TM 3-424 (USACE, 1956), and Sills and Vroman (2007) discuss the development of USACE levee seepage criteria with time. Regardless of the requirements adopted for a given project, well system design regularly focuses on providing a certain safety factor against seepage-related erosion initiation; probabilistic well design calculations (Guy et al., 2010) are also beneficial to complete and consider when possible. The establishment of a design safety factor for a project always requires thorough consideration, and the decision along with the actual design can be well informed by a risk assessment involving loading frequency and consequences consideration. An overview of factor of safety calculation methods is presented next, followed by additional well design-related discussions.

Factor of Safety Calculation

While factors of safety against heave and uplift for vertical seepage conditions are not factors of safety against structure failure (progression to failure depends on many additional factors), they provide useful information regarding stability and serve as a

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basis for relief well system (and other seepage control measures) design. Separate effective and total stress formulations have been used to calculate factors of safety, and while effective stress analyses are typically referenced in relief wells-related literature, an overview of both methods is provided here. Prior useful discussions on these methods are contained in Cedergren (1989), Doerge (2009), Duncan et al. (2011), Evans (1955), Lambe and Whitman (1969), McCook (2006), and Pabst et al. (2013); the terms "heave" and "uplift" are used in this paper as they are defined in Pabst et al. (2013).

For cohesionless soil vertical seepage conditions, the effective stress factor of safety (FS_e) against heave (condition of zero effective stress) is evaluated by comparing the vertical hydraulic ("exit") gradient (i_v) to the critical hydraulic vertical gradient (i_{cv}) as shown in Equation 1. Horizontal exit seepage conditions, which are not covered here, are addressed in Duncan et al. (2011) and O'Leary et al. (2013). For saturated confining soil (vertical) seepage conditions, the effective stress factor of safety (FS_e) against uplift is formulated in terms of weights and forces, but a mathematically equivalent solution to the gradient-based (heave) formulation is obtained (Equation 1). A difference in reality can exist, as in the cohesionless soil (heave) case there may be little frictional resistance to the erosion of materials, whereas in the confining soil (uplift) case there may be some cohesive resistance. Uplift factor of safety calculation methods do not consider soil strength (a conservative approach), nor do they directly apply to a confining soil having a defect (resulting in a concentrated seepage situation). The total stress factor of safety (FS_t) against uplift formulation yields a different solution than FS_e (except at the limit state where they agree). Duncan et al. (2011) caution against using a "misleading" total stress formulation when tail water is above the ground surface, as certain formulations (e.g., Method 2 in their paper) can yield dissimilar FS_t values when applied to separate situations where solutions should be the same. For example, with a "misleading" formulation, different FS_t values may result if applied to two cases where all conditions including the excess head value are the same (meaning computed FS_t values should be the same) but the tail water elevation is simply different between the two cases. A recent review of Doerge (2009) reveals total stress is stated in such "misleading" fashion in Fig. 1 of the paper (mentioned here for reader awareness). The below formulations for effective and total stress factors of safety (Equations 1 to 3) assume tail water at or above ground surface. The formulations and relationships assume saturated soil conditions (typically a reasonable assumption), but the formulations can be modified - for example if it is desired to consider partial confining soil saturation.

$$FS_e = \frac{i_{cv}}{i_v} = \frac{\gamma_b z}{\gamma_w h_{excess}} = \frac{FS_t(i_v+1)-1}{i_v} \tag{1}$$

$$FS_t = \frac{i_{cv}+1}{i_v+1} = \frac{\gamma_{sat} z}{\gamma_w (h_{excess}+z)} = \frac{(FS_e i_v)+1}{i_v+1} \tag{2}$$

$$i_v = \frac{(1-FS_t)}{(FS_t - FS_e)} \tag{3}$$

where γ_b is buoyant unit weight of confining soil (assumed saturated); z is vertical thickness of confining soil; γ_{sat} is total saturated unit weight of confining soil; and h_{excess} is excess head above the ground surface (or excess head above tail water elevation if it is above the ground surface). For clarity, if a piezometer has its sensing tip in a pervious foundation soil at the base of a surficial confining soil, and it reads 5.0 feet above tail water (ponded 2.0 feet above the ground surface), then the value of h_{excess} to be utilized above would be 5.0 feet. If tail water immediately receded, then h_{excess} would be 7.0 feet.

Regarding relief well system (and other seepage control measures) design, effective stress methods typically appear in guidance documents and are widely utilized, whereas little guidance exists regarding acceptable total stress factors of safety. Regardless of the analysis approach (whether evaluating FS_e or FS_t), the above formulations and Fig. 1 illustrate the relationship at a given gradient between FS_e and FS_t. For relief well system design, effective stress analysis is typically utilized and appropriate, although a corresponding FS_t directly calculated or determined using Fig. 1 can be considered if determined to be advantageous. For example, in a situation where i_v is equivalent to 0.5 and the confining layer has an average γ_{sat} of 124.8 pounds per cubic foot (corresponding to an i_{cv} of 1.0), values of FS_e of 2.0 and FS_t of 1.33 (with FS_e /FS_t = 1.5) can be computed using Equations 1 and 2 or directly obtained from their solutions which are plotted in Fig. 1.

System Geometry Determination

The design of relief well systems generally involves determining an optimized combination of geometrical and installation parameters (including well penetration, well spacing, well diameter, well screen, filter characteristics, and well discharge elevation) to provide tolerable foundation seepage pressures. Various methods such as closed form equations, superposition calculations, graphical solutions, blanket theory computations, and 2D and 3D finite element modeling can be employed along with empirical data and engineering judgment to determine what constitutes an appropriate well system for a given situation. Design computations are often based on certain assumptions (such as the boundary conditions, levee and well line being parallel to one another and of infinite length), and when a method's assumptions do not well represent the conditions being analyzed, an attempt to account for such differences should be made. As EM 1110-2-1914 (USACE, 1992) is widely utilized for relief well system design, some proposed corrections to equations contained in the manual are presented below, followed by an expanded discussion on certain infinite length assumptions and effects for relief well lines.

Design Equation Corrections

As mentioned, relief well design is often approached utilizing blanket theory and guided by methods in EM 1110-2-1914 (USACE, 1992); a few proposed corrections to the EM are presented here for reader awareness and application.

First, with regard to the theoretical well factors solutions developed by Barron (1982) and presented in Table 5-1 of the EM, the $\Delta\theta$ value for W/D (100%) is incorrectly listed as 1.0; $\Delta\theta$ is the change in θ_a and θ_m per one (1) log cycle of a/r_w. As mentioned in Guy et

al. (2010) the value should instead be listed as 0.3665, and the basis for this is documented here for reference. From Equation 5-18 of EM 1110-2-1914:

$$\theta_{av} = \frac{1}{2\pi}\ln\left(\frac{a}{2\pi r_w}\right) \tag{4}$$

where θ_{av} is the average uplift parameter, a is the design well spacing, and r_w is the effective well radius; note that θ_a and θ_{av} are both often used in the literature to represent the average uplift parameter. From Equation 4, for a given log cycle of a/r_w (100 to 1000 in this example), the correct $\Delta\theta$ is calculated as:

$$\Delta\theta = \frac{1}{2\pi}\ln\left(\frac{1}{2\pi(100)}\right) - \frac{1}{2\pi}\ln\left(\frac{1}{2\pi(1000)}\right) = 0.3665 \tag{5}$$

While this example computes $\Delta\theta$ using an average uplift parameter-based approach, a similar $\Delta\theta$ value can be computed using a midwell uplift parameter-approach, or determined graphically from Figure 5-8 of EM 1110-2-1914.

Second, this paper recommends revision of the equations for average net head in the plane of wells corrected for well losses (h_{av}) and net head midway between the wells corrected for well losses (h_m) provided in EM 1110-2-1914. As presented, Equations 7-2 and 7-4 of the EM have the average uplift parameter as an exponent; however, it should be multiplying instead, as shown in the equations below:

$$h_{av} = \frac{h\theta_{av}}{\frac{S}{a} + \theta_{av}\left(\frac{S + x_3}{x_3}\right)} \tag{6}$$

$$h_m = \frac{h\theta_m}{\frac{S}{a} + \theta_{av}\left(\frac{S + x_3}{x_3}\right)} \tag{7}$$

where S is the distance from the landside toe to the effective source entry; x_3 is the distance from the landside toe to the effective seepage exit; θ_m is the midwell uplift parameter; and the other terms are the same as defined above. It is recommended that Equations 6 and 7 be utilized rather than the net head Equations 7-2 and 7-4 in EM 1110-2-1914.

Third, it is noted the equation shown within the top stratum of the lower half of Figure 5-4 in EM 1110-2-1914 is in error. In the equation, the numerator ($S+x_3$) is multiplied by H (net head on the well system); however, the equation should multiply ($S+x_3$) by H_w (total well losses including elevation and efficiency-related losses). The proposed correction to the EM equation is shown just above the top stratum (labeled "blanket") in

Fig. 4 of this paper; note that this formulation also includes the term x_w, which is the distance of the well line downstream of the embankment toe (Guy et al., 2010).

Fourth, the y-axis of Figure 7-3 in EM 1110-2-1914 is labeled as RATIO $H_{mn}/H_{m\infty}$, (indicating plotted curves show the ratio between net head midway between the wells - for finite systems relative to an infinite well line); however, the axis label should read RATIO $h_{mn}/h_{m\infty}$ (indicating plotted curves show the ratio between net head midway between the wells corrected for well losses - for finite systems relative to an infinite well line). The basis for this correction is demonstrated by Figure 7-2 of the EM, which illustrates that the net head midway between wells (H_m) is equivalent to the sum of h_m and H_w.

Fifth and lastly, as noted in Guy et al. (2010) and repeated here for awareness, the nomograph based on approximate well factors solutions presented as Figure 5-8 of EM 1110-2-1914 does not include the required "pole" point (on the D/a line for θ_{av}). A nomograph version with the "pole" point included for use is however presented as Figure 60 of USACE (1956).

Relief Well Design Considerations

A number of considerations, which may not be explicitly incorporated into the design method(s) being employed, can often be relevant during the design of a given well system. For example, blanket theory-based design methods often include assumptions of a well line of infinite length, a homogeneous and isotropic foundation, and no hydraulic head losses for the wells. As mentioned above, when conditions for the design situation vary significantly from design methodology assumptions, then an attempt to account for such differences should be made. While the scope of this paper is not to discuss every possible significant difference between actual field conditions and design method assumptions, a few examples from work performed at two flood protection projects (Portsmouth, Ohio and Lawrenceburg, Indiana) focused on illustrating the importance of considering finite line effects and total well losses (elevation and hydraulic) are presented next.

Projects Overview

The Portsmouth Flood Protection System (Fig. 2) is located along the Ohio and Scioto Rivers in Portsmouth and New Boston, Ohio and was constructed by USACE in the 1940's. It consists of 20,000 feet of earthen levee and 21,400 feet of concrete floodwall with 12 interior pumping stations. The project's top of protection elevation is 548 feet; the maximum height is 58 feet (in the Pump Station Number 5 reach); and the 100-year flood level is 536 feet. The project was constructed on a relatively impervious top stratum underlain by a pervious sand and gravel foundation (averaging 30 feet thick). As part of levee certification-related work, the need for uplift pressure reduction (to obtain the target safety factor for the 100-year loading condition) was determined necessary in the Pump Station Number 5 reach. While toe or collector drains and ditches were originally installed along certain sections of the levee, no seepage control measures currently exist in this area. For the selected seepage control alternative, which consists of a relief well system to be installed in the Pump Station Number 5 ponding area, a brief

overview of the design approach (blanket theory and finite element modeling) is presented below.

The Lawrenceburg Flood Protection System (Fig. 3) is located along the Ohio River in Lawrenceburg, Indiana and was constructed by USACE in the early 1940's. It consists of an 18,300-foot long earthen levee having a maximum height of 44 feet; the top elevation (constructed based on a 1937 flood elevation of 503 feet) is 504 feet; and the 100-year flood level is 490 feet. The levee was constructed on a relatively impervious top stratum underlain by a pervious sand and gravel foundation (averaging 80 feet thick), and 164 (3-inch diameter) pressure relief wells were installed along the landside levee toe. While USACE (1992) indicates the first use of relief wells for uplift pressure reduction occurred during 1942-43 at Fort Peck Dam, the Lawrenceburg relief well system is another early application. The spacing of existing wells ranges from 50 to 150 feet, and their design penetration into the aquifer was to be 20 feet. The relief well system was not designed according to current methods, and recent seepage analyses have determined additional uplift pressure reduction is necessary. To estimate the existing system's potential for positively contributing to pressure reduction as part of a new seepage control system, hydraulic testing and rehabilitation (via induced resonance) were recently conducted on a sub-set of the existing relief wells. A brief overview of these relief well activities and evaluations is presented later.

Finite Line Effects

When effective seepage (entrance and exit) boundary conditions are parallel to a line of relief wells, and impervious boundaries exist at the ends of the line, the system may generally be evaluated mathematically by considering an infinite number of wells. For such conditions the flow to each well and the head distribution around each well are uniform along the line, and there is no substantial component of flow parallel to the line. Whereas dams often have a line of relief wells installed across a valley and terminating at valley walls, well lines along levees rarely terminate at impervious boundaries, and therefore finite line effects are often an important consideration. Under such conditions there is a substantial component of flow parallel to the well system and a non-uniform distribution of well discharges and heads result. Since uplift pressures between, and downstream of, wells in a finite line will exceed those in an infinite line, relief wells can be installed deeper, with reduced spacing, and/or with variable geometry at the ends of a finite line in order to achieve similar uplift reduction as an infinite line. Data are presented in USACE (1963) to evaluate various finite system geometry effects, and different modeling approaches can be employed for this purpose; however, such effects are also often empirically and judgmentally addressed in practice. In this section a brief example of utilizing blanket theory in conjunction with plan view finite element modeling, to evaluate foundation seepage conditions and design an optimized finite relief well system (as further documented in Darko-Kagya and Guy, 2014), is presented within the context of the Portsmouth Flood Protection System project.

Shown in Fig. 4 are the loading (100-year flood level) and geometrical parameters representing the Portsmouth levee Pump Station Number 5 reach (Fig. 2); variables in the figure are as defined in Guy et al. (2010) and below. As indicated in Fig. 4, the effective

seepage entrance is located at the upstream embankment toe, and a relatively impervious landward blanket results in an effective seepage exit several hundred feet downstream of the embankment. An average 30-foot deep pervious sand and gravel foundation exists at this location, and extrapolation of piezometer data with blanket theory calculation indicates greater than 15 feet of excess head would exist beneath the landward blanket for the 100-year loading. For the base condition (no relief wells installed) the computed FS_e value at this location is less than 0.5 for the 100-year loading; thus, design of a relief well system to reduce uplift pressures and provide adequate safety factors in this project reach was completed. Using blanket theory methodology as detailed in USACE (1992) and the computer program from Guy et al. (2010), the spacing for an infinite (fully-penetrating) well line to obtain an FS_e value of 1.5 (corresponding to an FS_t value of 1.2 for an i_{cv} of 1.0) for the 100-year loading condition, was determined to be 50 feet.

Given the relatively high landward elevation (and substantially thicker landward top blanket) of project reaches adjacent to the ponding area, relief wells are not required for adjacent levee reaches. However, installation of a few wells along the levee toe in the pump station vicinity alone, at the penetration and spacing computed by infinite well line methodology, would result in an actual FS_e (and FS_t) value much less than that which would be achieved by an infinite line. Therefore, plan view finite element modeling was completed to evaluate such effects, and to determine an optimized finite well system geometry for the ponding area reach, which will provide an adequate reduction in uplift pressure.

Presented in Fig. 5A is a plan view finite element model constructed using SEEP/W (GEO-SLOPE International, 2007) with parameters specified equivalent to those utilized in the above-described (infinite relief well line) blanket theory analysis (Fig. 4). While plan view analyses have certain limitations and do not have true three-dimensional capability, in this situation, direct agreement between blanket theory and plan view modeling (in terms of uplift pressures and relief well flows) is obtainable since this design involves fully-penetrating relief wells. A practical approach for utilizing plan view modeling for partial-penetration analysis will be the focus of another publication. For the model in Fig. 5A, constant head boundaries for the 100-year loading condition (pool and tail elevations of 536 and 502.5 feet, respectively) were specified along the southern and northern plan model faces (in the x-direction); boundary distances from the well line were taken as the lengths of equivalent impervious top strata computed using blanket theory. Model faces in the y-direction were represented as impervious boundaries located one-half of the well spacing from wells located at the ends of the line. The lower and upper model faces (in the z-direction) were also impervious boundaries (required condition of this modeling approach) representing confining units below and above the pervious granular foundation. A constant head boundary condition of 505.4 feet was applied to each of the relief well nodes; as discussed below, this value is equivalent to tail water elevation plus total well losses. As Fig. 5A illustrates, the resultant head distribution with an infinite well line is uniform along the line, and there is no appreciable component of flow parallel to the line. For the modeled 50-foot well spacing, heads along and downstream of the well line do not exceed elevation 507 feet, which is the maximum allowable head in the low-lying ponding area reach to achieve a FS_e value of 1.5 (FS_t value of 1.2). Well discharges of 190 gallons per minute (gpm) for

the modeled infinite line are also uniform, and along with head projections they closely agree with blanket theory results for these load conditions.

After establishing agreement between blanket theory and plan view modeling results, the plan view model was modified to determine a finite system geometry for achieving design objectives in the ponding area reach. Shown in Fig. 5B is the head distribution for only 1 relief well, and this hypothetical geometry simulates a field condition of 2 wells (fully-penetrating with 50-foot spacing) installed just beyond the levee toe in the pump station reach. Note that only one half of the 2 well system in the y-direction is required to be modeled, as head distributions would be identical in east and west directions from the plane of symmetry. The model parameters and boundary conditions are similar to the infinite well line model (Fig. 5A); however, to evaluate finite system effects, the eastern impervious model boundary (in the y-direction) was located an infinite distance (rather than one-half of the well spacing) from the well. The western boundary was kept as impervious, and again, this boundary serves as a plane of symmetry for the ponding area reach analysis. The results in Fig. 5B illustrate that for this situation, there is an appreciable component of flow parallel to the well system and a non-uniform head distribution. As a result, the installation of only 2 wells would allow uplift pressures in the ponding area that significantly exceed (by approximately 2.5 feet) those for an infinite line. Thus, such geometry would provide an FS_e value of only 1.0 for the 100-year flood level rather than the target value of 1.5. For the case of 3 wells (Fig. 5C) installed in a line with 50-foot spacing (simulating a field condition of 6 wells around the plane of symmetry) a more favorable head distribution with respect to target criteria is obtained. While heads around and downstream of the third well significantly exceed those for an infinite line, this would not be problematic because the ground elevation here is actually much higher than in the ponding area. While the modeled head is slightly higher (0.5 feet) in the ponding area than the allowable head, the results indicate a system comprised of 7 or 8 wells in a line (on 50-foot centers) just beyond the levee toe could meet design objectives. A practical concern with such an approach however, is that the high top of ground elevations adjacent to the ponding area would require very deep well housings and extensive excavation for lateral collector conduit. Therefore, a system layout further optimized with respect to practical construction concerns and cost was pursued (as discussed below).

Further comparison of resultant head distributions (total head values as measured along the finite element model north-south well line plane) for the above-discussed infinite wells, finite (1 well), and finite (3 wells) cases is shown in Fig. 6. For the infinite case, the net head midway between wells ($H_{m\_Infinite\ Wells}$), when expressed as a total head equivalent (y-axis value), is seen to be below the allowable maximum head of 507 feet. Note the magnitude of H_m for different modeled scenarios (indicated by vertical arrows in Fig. 6) represents the net head midway between the wells, with H_m equivalent to h_m (Equation 7) plus H_w. When added to tail water elevation (502.5 feet in this case), the H_m values can be expressed as total head elevation equivalents (y-axis values). For the finite (1 well) case (Fig. 5B), the net head midway between wells ($H_{m\_1\ Well}$) when expressed as a total head equivalent significantly exceeds the allowable maximum head of 507 feet in the ponding area region (Fig. 6). For the finite (3 wells) case (Fig. 5C), the net head

midway between wells ($H_{m\_3\ Wells}$) also exceeds the allowable maximum head in the ponding area region, but as mentioned such a geometry is close to an acceptable solution.

As illustrated in Fig. 5D, a finite nonlinear well geometry can be employed in the ponding area reach to achieve similar uplift reduction as that indicated by infinite well line (blanket theory) design computations. By modifying the model used to produce the results in Figs. 5B and C (keeping all parameters similar but modifying well positions and number), a head distribution can be obtained for the low-lying ponding area that is acceptable with respect to the 100-year loading design criteria. By installing a total of 8 relief wells with an approximate 50-foot spacing at similar elevation (exact positions are yet to be determined in the field, but perhaps along the 506 surface elevation contour for example) around the ponding area (Fig. 2B), it is seen in Fig. 5D that the allowable maximum head of 507 feet will not be exceeded downstream of the well system in the ponding area. The geometry of the wells alignment (in a concave shape) is a result of both the finite system effects and topography of the ponding area. Again, note that only one-half of the reach is modeled here, as the results in eastern and western directions would be symmetrical. While project objectives and available funding required focus on meeting target stability for the 100-year event at this time, and the designed well system geometry provides a computed factor of safety that is above the limit state for the top of levee (elevation 548) loading condition, the safety margin would ideally be higher for the project design loading condition (e.g., considering the amount of uncertainty associated with design data and computations). Therefore, residual stability conditions (subsequent to installation of the designed well system) for the design loading case will be further considered during future levee risk assessments, and the well system may be supplemented in the future to achieve a higher safety margin for design loading conditions. The results in Figs. 5 and 6 demonstrate the potential importance of finite system effects during relief well system design and demonstrate a methodology using plan view finite element modeling (with support from blanket theory computations) to consider them. While finite line effects can be considered and addressed by different approaches (as mentioned above), the plan view modeling approach in conjunction with blanket theory can assist in considering such effects as well as in optimizing the geometry of a well system. A total head plot of H_m values (as shown in Fig. 6 for various hypothetical well lines) is not presented for the optimized well system geometry (Fig. 5D) given its nonlinear nature; however, the plan view model head distribution results illustrate that the optimized geometry meets design objectives as stated above for the 100-year loading condition.

Elevation and Hydraulic Head Losses

Along with relief well geometry, many other factors have significant effect on the pressure reduction potential of a system. Elevation and hydraulic head loss components of the well screen, riser elevation, and discharge must be considered during design and also monitored in the future. Head losses during the flow of water through these features are important, as they result in a decrease of well discharge, and thus raise the potential along a line of wells by an amount equal to such losses. As examples, if a system has its well risers set too high, or if the screens are allowed to foul over time via lack of

maintenance, lower system discharge efficiency and higher foundation heads (along and beyond the well line) will result.

For the Portsmouth Pump Station 5 ponding area, the optimized well system geometry (Figs. 2B and 5D) was designed based on an assumed tail water elevation of 502.5 feet (ponded water located above the prevailing low ground elevation of 490 feet). For the same model, lowering the tail water elevation to 500 feet while keeping all other input parameters the same would reduce FS_e from 1.5 to 1.0 (and would reduce FS_t from 1.2 to 1.0), whereas raising the tail water above 502.5 feet would increase the FS_e value. As tail water is often an important random variable, a condition of the optimized well system design is that a minimum tail water elevation of 502.5 feet must be maintained in the ponding area during a 100-year loading condition. This should be maintained for higher loadings too (e.g., in order to achieve an FS_e value above 1.0 for the top of levee loading condition). While the required tail water can be maintained without adverse implications according to project personnel, its elevation is expected to frequently rise above elevation 504 feet (and possibly greater); therefore, potential back-flooding and optimal riser elevation were important considerations during system design. For the optimized system, well housings will be installed along the 506 foot surface elevation contour (for example, exact positions are yet to be determined in the field) of the ponding area, and well risers will be installed at elevation 505 feet. Check valves will also be installed on each riser. This approach achieves a balance between locating the wells too low (such that they are constantly inundated) and too high (such that high elevation head losses become excessive). For the designed riser elevation of 505 feet, elevation head loss is equal to 2.5 feet (riser elevation minus tail water elevation) for the 100-year loading. If the risers were located at 508 feet instead for example, and tail water was located at 502.5 feet, elevation head loss would be increased by 3 feet (relative to risers at 505 feet), the foundation potential along the well system would be raised by 3 feet, and a much lower FS_e would therefore result. Raising the tail water to 507 feet (with risers at 508 feet) would reduce uplift potential with respect to a lower tail water case; however, the FS_e would still be below design criteria.

Together with elevation head loss, hydraulic (efficiency-related) head losses were also considered during well system design for the Portsmouth ponding area. In practice, efficiency-related losses have historically been considered by estimating entrance, friction, and velocity losses for a system and then adding these to the maximum computed landside head. Note that design equations such as those in USACE (1992) generally assume well-related losses equivalent to zero, and therefore, they must be separately estimated and incorporated. Well-related losses can also be measured in the field via pump testing after well installation, and/or estimated and incorporated into design in a manner that (at least in theory, although sometimes difficult in practice) will allow post-installation measurements of well performance to be related to original design assumptions and FS_e values (Guy et al., 2010). For the Portsmouth ponding area design, a well efficiency value of 75 percent (theoretical versus actual drawdown for 100-year loading discharge) was estimated based on regional knowledge and judgment (75 percent efficiency corresponds to estimated hydraulic well losses of 0.4 feet). As with tail water and many other factors, well efficiency is also an important random variable, with the computed FS_e value ranging from 1.5 to 1.2 for the efficiency range of 75 to 50 percent.

Therefore, it is important that actual efficiency be measured (via pump testing) during wells installation to ensure it is equal to or above the design efficiency, and that actual efficiency also be monitored (and maintained as necessary) on a recurring schedule in the future to ensure it does not problematically decline. For the Portsmouth ponding area system, the total well losses were computed to be 2.9 feet for the 100-year loading condition (the sum of an elevation head loss of 2.5 feet and hydraulic head losses of 0.4 feet). The total well losses were incorporated into blanket theory design computations (Fig. 4) in a manner discussed in Guy et al. (2010). For the finite element modeling (Fig. 5) losses were included in the total head specification for each relief well. A constant head value of 505.4 feet was assigned to each well, and this value represented the tail water elevation plus the estimated total well losses. As elevation and efficiency head losses affect the net head on the well system (and vice versa with regard to efficiency), they effect the average net head in the plane of wells and net head midway between wells (parameters governing design). Therefore, it is important to consider total well losses during design and monitor/maintain efficiency-related well losses into the future. An example of well losses and well performance assessment is presented next using data acquired at Lawrenceburg Levee.

Well Performance Assessment

For the Lawrenceburg Flood Protection System (Fig. 3), underseepage analyses for different project reaches yielded FS_e values ranging from 0.6 to 2.2 for the 100-year flood loading condition (pool elevation of 490 feet). Since FS_e values along two project reaches were found to be below typical design standards, 29 new (10-inch diameter) relief wells, with designed penetrations ranging from 25 to 80 percent, have been designed for future installation in these reaches (Thelen Associates Inc., 2014). The design of the new wells was conducted in accordance with USACE (1992) and to achieve a design loading (pool elevation of 503 feet) maximum exit gradient (i_v) of 0.5, which in this case (where i_{cv} = 1.0) corresponds to an FS_e of 2.0 and a FS_t of 1.3 (Fig. 1). General information on the Lawrenceburg Flood Protection System is provided above.

To estimate the existing (early 1940's) relief well system's potential for positively contributing to seepage control as part of a newly designed system to achieve target factor of safety values, hydraulic testing before and after rehabilitation (via induced resonance technology) was performed for 7 of the existing (3-inch diameter, perforated wrought iron pipe) wells. As mentioned, Lawrenceburg Levee was constructed on a relatively impervious top stratum underlain by a pervious sand and gravel foundation (ranging from 59 to 94 feet and averaging 80 feet). The screen lengths of the wells selected for evaluation range from 4 to 19 feet, and their penetration ranges from 6 to 27 percent (Table 1); note that one of the inspected wells (RW-151) was actually found through evaluation to have no screened portion penetrating the aquifer. In terms of diameter and intake area, the existing wells have smaller diameter than those typically installed in practice today (6 to 18 inches), and their screen intake areas (ranging from 3.7 to 4.8 percent, and averaging 5 square inches of open area per foot of screen) are much less than those of modern continuous wire-wrapped screens. Frictional resistance encountered by flow being concentrated in getting to, and through, relatively small screen openings, as well as associated with flow upwards through relatively small diameter pipe,

is capable of reducing relief well discharge (and thus increasing potential along the well line). Laboratory experiments by Clark and Turner (1983) indicate that entrance velocity into a well is governed by intake area up to a value of about 10 percent, and that the performance of screens deteriorates with a decrease in open area below this value due to increasing well losses. Since increased velocity generally leads to increased turbulent losses, design methods (USACE, 1992) often recommend limiting intake velocity to 0.1 feet per second (ft/sec). While Clark and Turner (1983) did not measure significant well losses at entrance velocities up to 0.5 ft/sec, they still recommend designing for a maximum velocity of 0.1 to 0.25 ft/sec as a conservative measure. With respect to these often-recommended design criteria, the projected entrance velocities for the existing Lawrenceburg wells significantly exceed them (Fig. 7); entrance velocities were calculated by dividing discharge rate by the well screen open area. Irrespective of hydraulic testing, the limited penetration, small diameter, low open area, and projected high entrance velocities suggest that well losses associated with the existing Lawrenceburg relief wells will have a negative effect on pressure relief potential for design loading conditions.

The hydraulic testing and rehabilitation program for the 7 existing Lawrenceburg relief wells consisted of video logging, pre-rehabilitation step-drawdown testing, rehabilitation using induced resonance technology, and post-rehabilitation step-drawdown testing. Based on video logging and observations during the prior abandonment/extraction of several existing project wells, the existing wells have likely experienced performance decline over time due to silting and bio-fouling mechanisms, although historic discharge data do not exist to further determine an extent of decline. In addition to well losses associated with original well system construction details (discussed above), additional meaningful well loss components are therefore possible during design discharge rates due to a reduction in well permeability associated with clogging. The before and after rehabilitation step-drawdown tests on existing wells were generally performed at the pumping rates of 5, 10, 15, and 30 gpm with a minimum duration of 60 minutes for each step. Although it is advantageous to conduct hydraulic testing at anticipated design discharge rates (ideally the discharge rate used should sufficiently stress the entire aquifer) to reduce uncertainties associated with estimating well performance parameters at discharge values higher than those used during testing, the small diameter of existing wells prevented the use of higher capacity pumps in this case. The rehabilitation work utilized induced resonance technology ("hydropuls") in combination with pumping. The hydropuls method was performed with a high pressure nitrogen pulsing unit equipped with a pressurized hose and valve system. During the process, the impulse generator was placed in the well screen, and it released impulses of high pressure nitrogen in short, repetitive bursts. More than ten cycles (one cycle being a full pass of entire well screen up and down) of the impulse action were made for each well. The impulse actions created an air-lift effect, which in turn vibrated and loosened existing incrustation- and biofouling-related debris, such that it could be removed from the well by mechanical air-lifting and over-pumping.

Generally speaking, the total drawdown in a pumped well consists of two components: the aquifer losses and the well losses. Aquifer losses are time-dependent, and their variation is linear with time, whereas well losses are divided into linear and nonlinear

head losses. The linear total well loss potentially has several important components, including aquifer loss and other losses attributable to losses through the filter and well screen or losses associated with partial well penetration. During data analyses, if the specific capacity measured during testing is constant with increasing well discharge, then laminar flow is assumed; whereas if specific capacity decreases as discharge increases, then turbulent conditions are inferable. If an increase in specific capacity occurs during step-drawdown testing, then this usually suggests: that the well was previously under-developed; that development is occurring during testing; and that well loss parameters are not inferable from the data.

Shown in Fig. 8 (top) are step-drawdown data acquired before and after the rehabilitation of well RW-85 at Lawrenceburg Levee; data for the other site wells, which were evaluated, are contained in Thelen Associates Inc. (2014). Analyses of the data shown in Fig. 8 (bottom) were performed using the Hantush-Bierschenk method (Kruseman and de Ridder, 1994), which is based on Jacob's (1947) equation. This method of analysis involves plotting specific drawdown (s/Q) versus Q, where s is the stabilized drawdown at the end of each step and Q is the step's pumping rate. As seen in Fig. 8, the diagnostic plot data for RW-85 generally exhibit straight lines having positive slope (indicating specific capacity is decreasing with increasing Q). Therefore, the linear (B) and nonlinear (C) loss coefficients were able to be estimated using:

$$s = BQ + CQ^2 \tag{8}$$

While total linear losses could be estimated from acquired data, it was not possible to reliably estimate individual linear loss components (e.g., those potentially caused by well skin and partial penetration effects in addition to aquifer loss) without having results from constant rate pump testing (providing transmissivity and storage coefficient values). Thus, calculation of a theoretically comprehensive well efficiency (expressed as a ratio of aquifer head loss to total head losses) was not possible, and estimates of Driscoll's L_p parameter (expressed as a ratio of laminar head loss to total head losses) were made as follows:

$$L_p = \left(\frac{BQ}{BQ + CQ^2} \right) (100\%) \tag{9}$$

While L_p values provide useful well performance information, they will usually overestimate well efficiency, and they are often erroneously represented as well efficiency values in practice (Boonstra, 1999). To potentially relate field hydraulic testing results to underseepage safety factor values in a theoretically correct manner for relief well design (Guy et al. 2010), the individual components of the linear total well losses must be determined (ideally at design discharge), such that well efficiency (rather than L_p) can be computed and used in projecting uplift pressures; this will be the topic of a separate paper.

A summary of measured and projected well performance parameter values based on hydraulic testing for well RW-85 is contained in Table 2; the linear and nonlinear

coefficients measured from hydraulic testing are shown on Fig. 8 (bottom). From a comparison of the C coefficients before and after rehabilitation, an improvement in well condition due to rehabilitation is apparent. However, according to the criteria of Walton (1962), the degree of well deterioration is still "severe" after rehabilitation, and with a post-rehabilitation C coefficient value of greater than 40 sec^2/ft^5 (greater than 0.01 min^2/ft^5) it is "difficult and sometimes impossible" to restore original well capacity; Kasenow (1998) indicates that well capacity can be difficult to restore when the C coefficient value is greater than 0.0002 ft/gpm^2. The computed L_p values also illustrate well improvement due to rehabilitation; however, the values also clearly indicate increasingly poor well performance and related head losses as discharge increases, suggesting limited capacity of the well. The specific capacities were increased by rehabilitation, but even afterwards they are still an order of magnitude lower than those of many other relief wells in similar (glacial outwash) foundation materials that the authors have previously tested. Key general differences between the Lawrenceburg wells and these other wells are that the other wells have larger diameters, larger screen intake areas, and larger effective foundation penetrations. Without having accurate estimates of transmissivity and storage coefficient values from constant-rate testing, and without being able to step-test the Lawrenceburg wells at higher discharge rates often employed for this foundation-type (permeability on the order of 0.1 cm/s), inferences regarding the degree of partial penetration losses to be expected at higher discharge values could not be inferred. However, as the penetrations of all site wells are quite limited (Table 1), it would be expected that partial penetration-related losses could be measurable and significant if testing were performed with an order of magnitude increase in discharge. For this reason and others discussed above, there can be large uncertainty associated with attempting to project L_p (or well efficiency) values very far upwards beyond the highest field step. In the case of the Lawrenceburg wells, while direct reliable estimation of head losses at design discharge rates was not possible (often the case for most situations in practice), useful judgment regarding capacity of the existing wells at design loading of the levee was able to be made by considering projected L_p values in conjunction with blanket theory-based well design methodology. By incorporating projected L_p values into the well design process (Guy et al., 2010), head losses for the Lawrenceburg wells at levee design loading conditions were able to be estimated. However, it is recognized that there is uncertainty in predicted head losses for the above-discussed reasons, and that because L_p overestimates well efficiency, head losses and uplift pressures would be somewhat underestimated using L_p. From such estimates resultant uplift pressures from the existing well system were able to be predicted with consideration of well losses (a substantially better approach than one that assumed no well losses, which would be incorrect), and an overall estimate of the potential contribution towards foundation pressure reduction from the existing well system was made. These analyses were useful in determining that the existing Lawrenceburg wells, while they will not provide target stability for design loading conditions, do have some limited transmitting capacity, and will therefore provide some limited future benefit.

The above discussion regarding the testing and evaluation of the Lawrenceburg relief wells demonstrates the importance of considering well losses and efficiency concepts for both existing and new relief well systems. In the case of the existing Lawrenceburg wells, hydraulic testing has indicated that high well losses can be expected during

discharge at design loading conditions, and as well losses directly increase foundation uplift pressures, the existing system is not sufficient alone to meet target safety factors. As existing wells are structurally sound and will provide some discharge capacity though when the levee loading increases, they will still remain and provide some project benefit after installation of the (above-mentioned) newly designed Lawrenceburg relief well system.

Conclusions

This paper has provided discussion and illustrative examples concerning several concepts critical for relief well system analyses which the authors hope practitioners will find useful. Appropriate effective and total stress safety factor formulations for evaluating vertical seepage-related heave and uplift potential have been presented, and as mentioned, design decisions can be well informed by a risk assessment involving event tree, loading frequency, and consequences consideration. Some proposed revisions to design equations contained in EM 1110-2-1914 (USACE, 1992) have been presented and are recommended for future application. A practical method for considering finite well line effects during system design using plan view finite element modeling has been demonstrated, and the importance of considering well elevation- and efficiency-related aspects has been discussed. While such factors may not be explicitly incorporated into existing design methodologies, they can have a significant effect on well discharge and resultant groundwater head distribution, and thus a project's underseepage stability. Therefore, when conditions for a design situation vary significantly from design methodology assumptions, an attempt to account and design for such differences should always be made.

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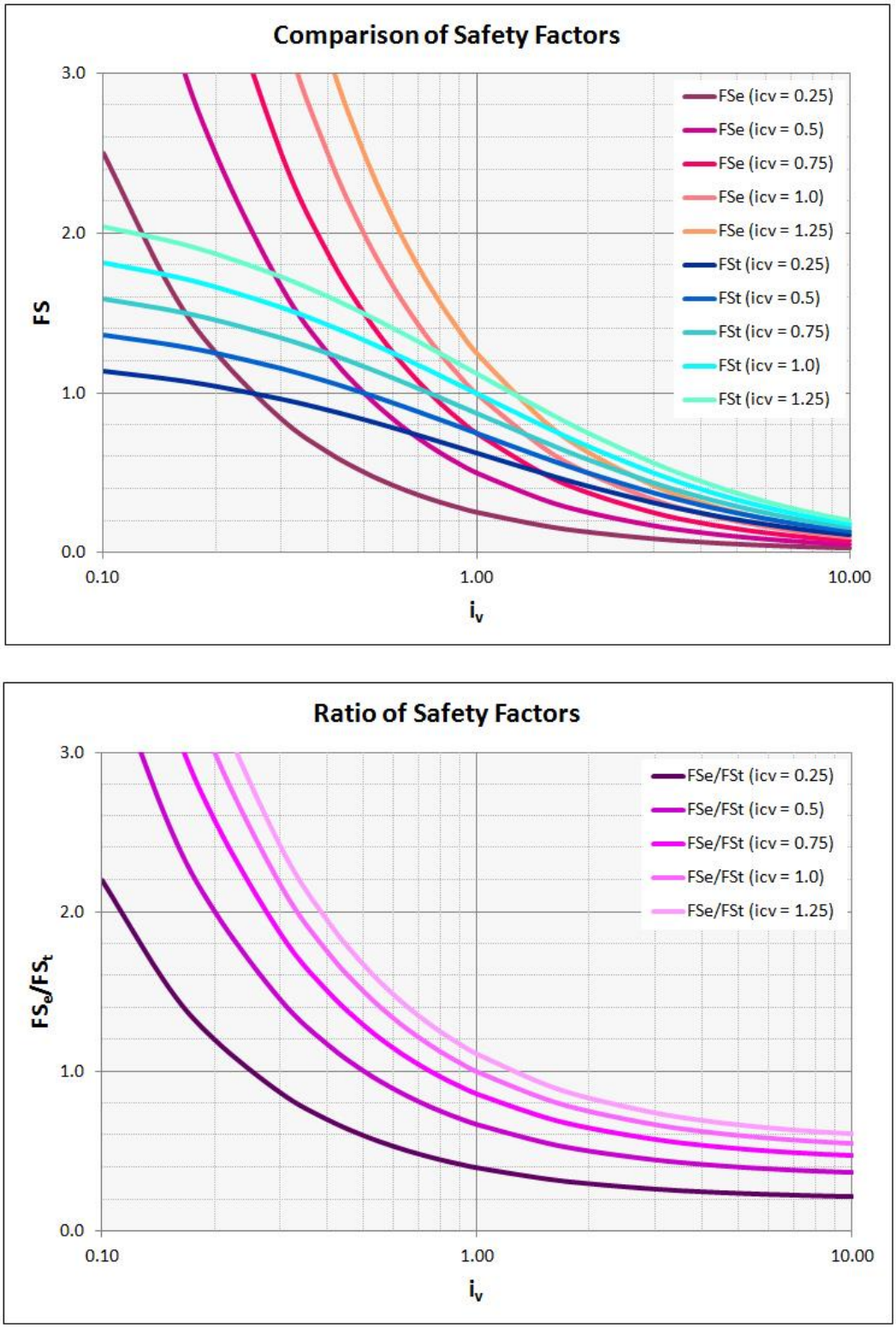

Fig. 1. Relationship between effective stress (FS_e) and total stress (FS_t) factors of safety against erosion initiation (for vertical seepage conditions) as a function of critical hydraulic vertical gradient (i_{cv}) and actual vertical hydraulic gradient (i_v).

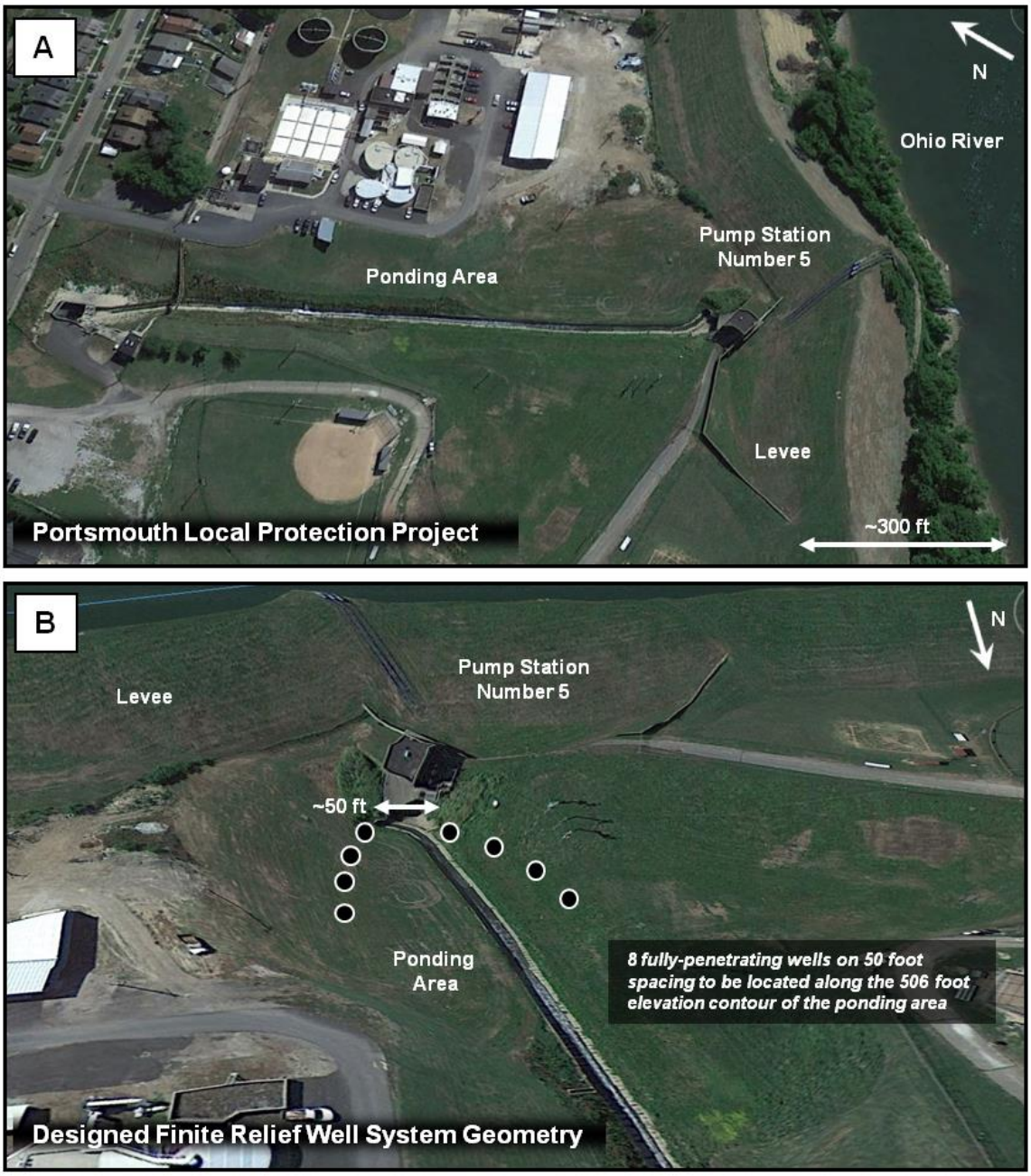

Fig. 2. Representative aerial photograph (A) of the Portsmouth Levee Pump Station Number 5 reach (looking northeast), and layout of a finite relief well system designed to reduce uplift pressures during the 100-year loading condition (B).

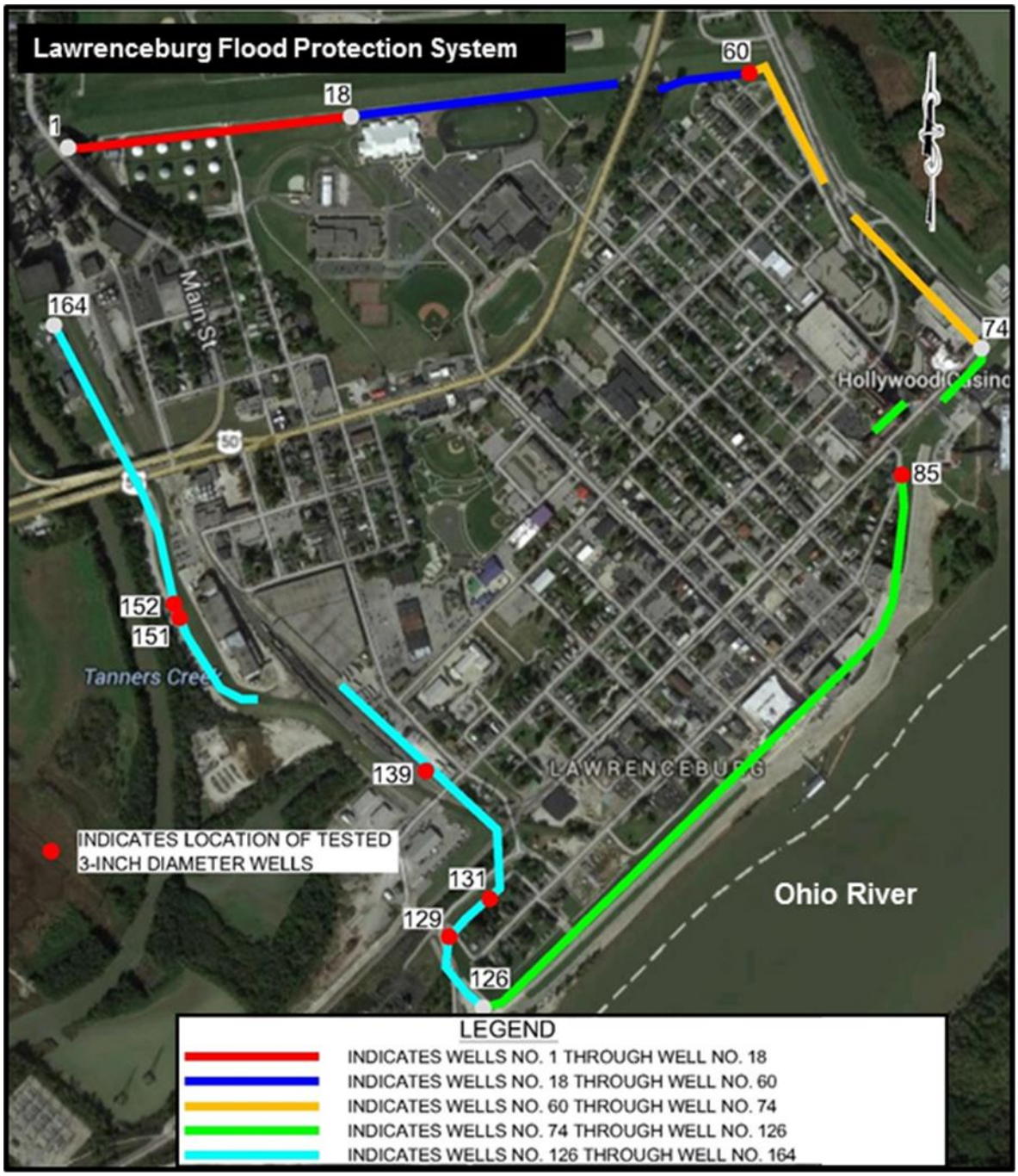

Fig. 3. Layout of existing relief wells superimposed on an aerial photograph of the Lawrenceburg Flood Protection System. The spacing between Well No. 1 through Well No. 18 is 100 feet; the spacing between Well No. 18 through Well No. 60 is 50 feet; the spacing between Well No. 60 through Well 74 is 150 feet; the spacing between Well No. 74 through Well No. 126 is 100 feet; and the spacing between Well No. 126 through Well No. 164 is 150 feet. See text for discussion.

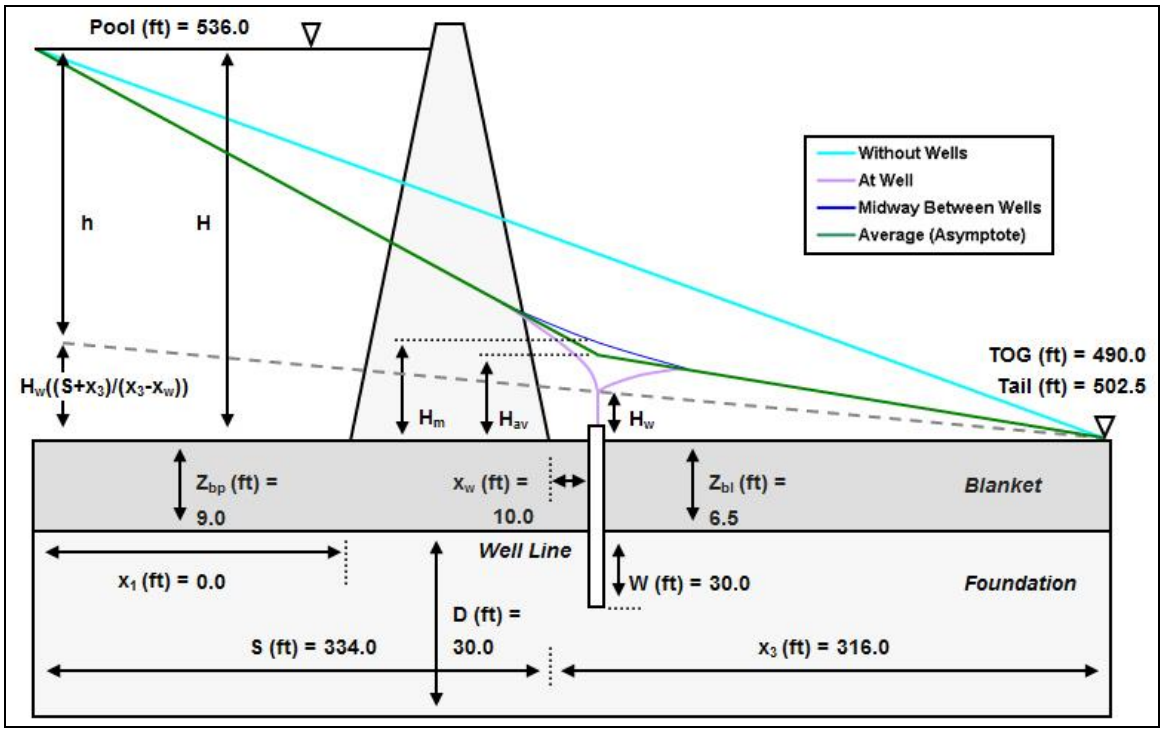

Fig. 4. Conceptual diagram illustrating relief well system design parameters for an infinite well line; variables are as defined in the text and Guy et al. (2010). Load conditions and parameter values indicated were used for the Portsmouth Pump Station Number 5 system design. This drawing is not to scale and plotted head curves are for illustration purposes.

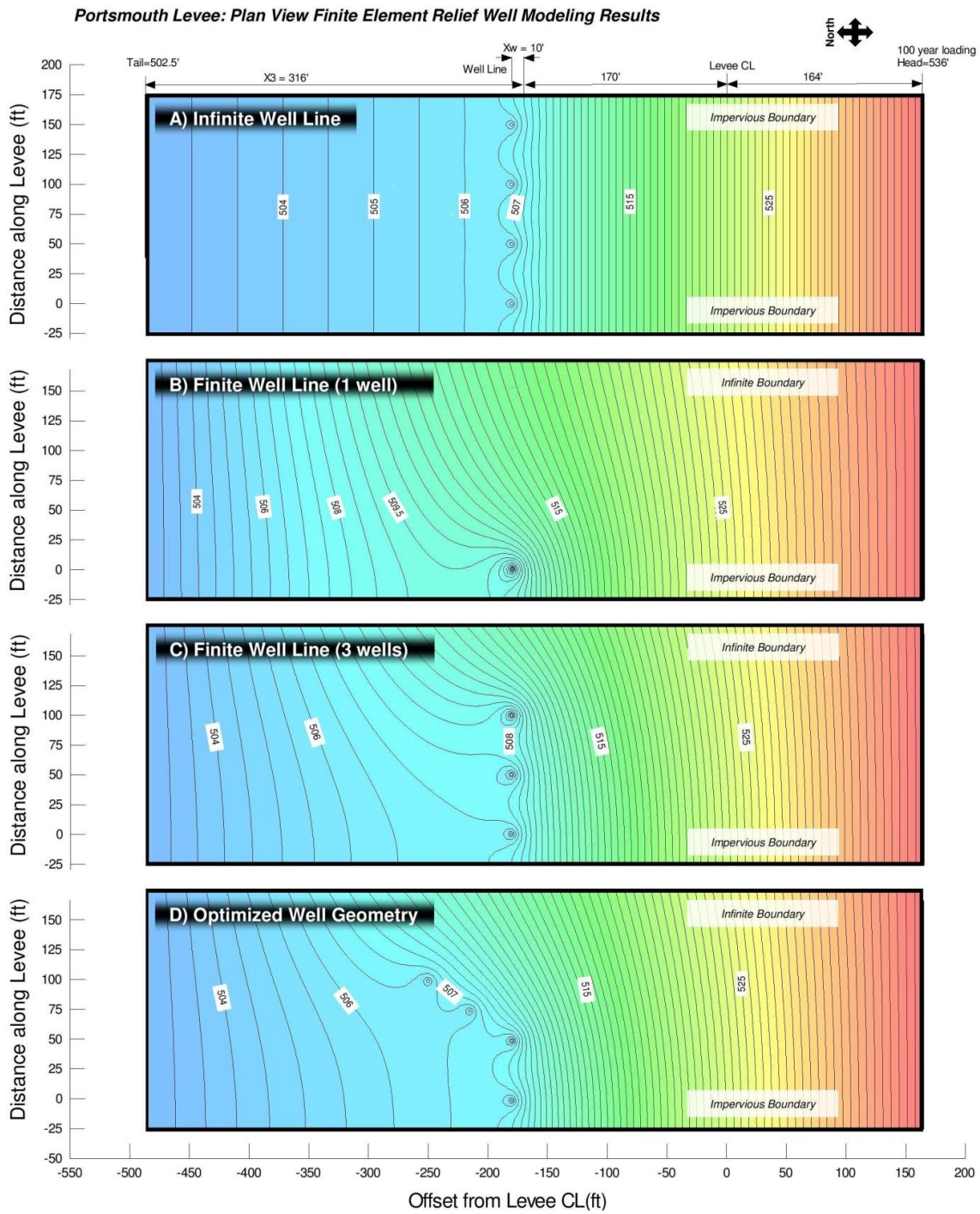

Fig. 5. Resultant head distributions for fully-penetrating relief well system geometries (computed by plan view finite element modeling) in the Portsmouth Pump Station Number 5 reach (Fig. 2A). Loading conditions represent the 100-year flood and modeled geometries include: A) an infinite line of wells with 50-foot spacing; B) 1 well simulating a field condition of 2 wells around the lower model symmetry plane; (C) 3 wells simulating a field condition of 6 wells; and D) an optimized relief well system layout consisting of 4 wells simulating a field condition of 8 wells (Fig. 2B). See text for discussion.

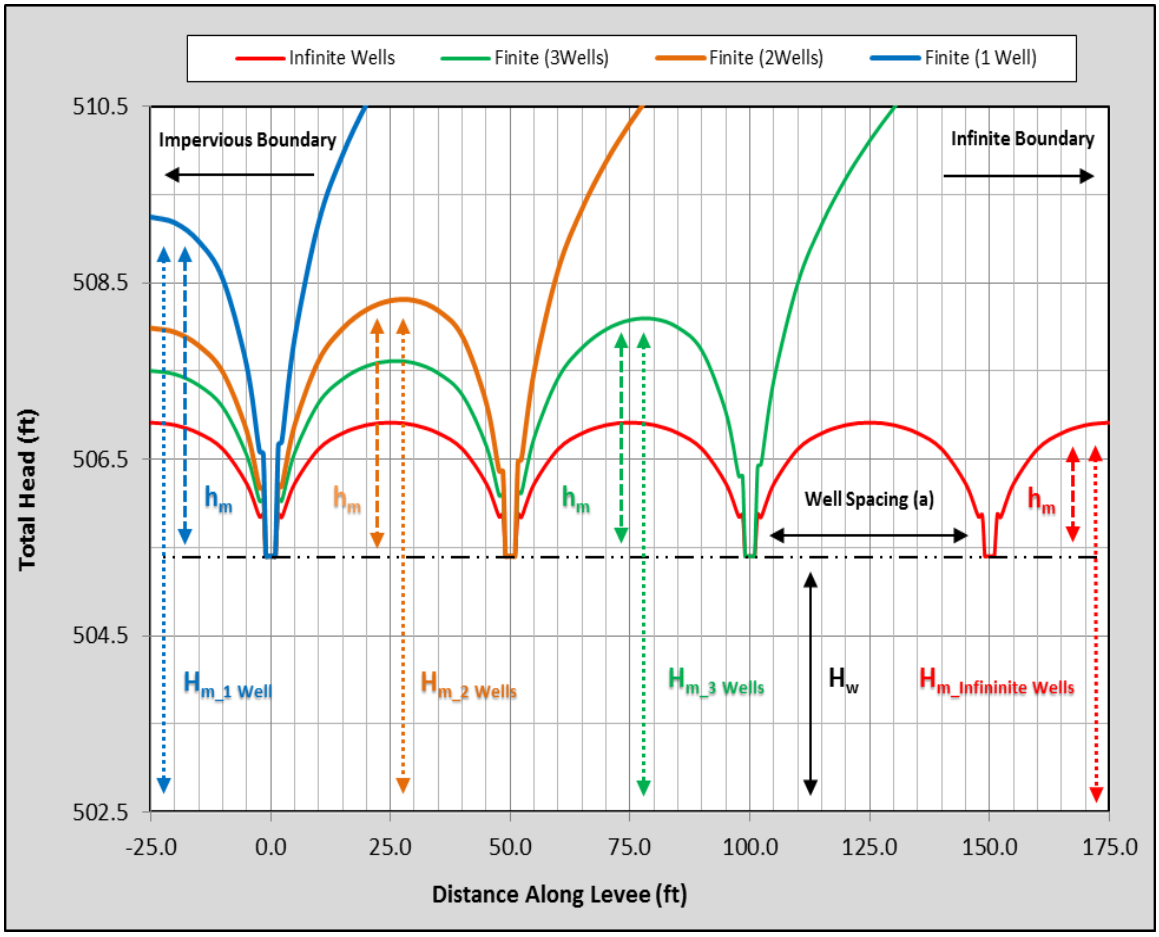

Fig. 6. Variation of total head along infinite and finite well lines modeled for the Portsmouth Levee Pump Station Number 5 reach. The infinite, finite (1 Well), and finite (3 Wells) curves correspond to node values measured along the well lines in Figs. 5A, B, and C respectively; the intermediate model used to produce the finite (2 Wells) curve is not presented in Fig. 5. Values of H_m represent net head midway between the wells and are expressed as total head elevation equivalents (by adding H_m values to the tail water elevation of 502.5 feet). Values for net head midway between the wells corrected for well losses (h_m) and total well losses (H_w) are also indicated; recall that $H_m = h_m + H_w$. The allowable total head in the ponding area just downstream of the well line for the 100-year loading condition is 507 feet. See text for further discussion.

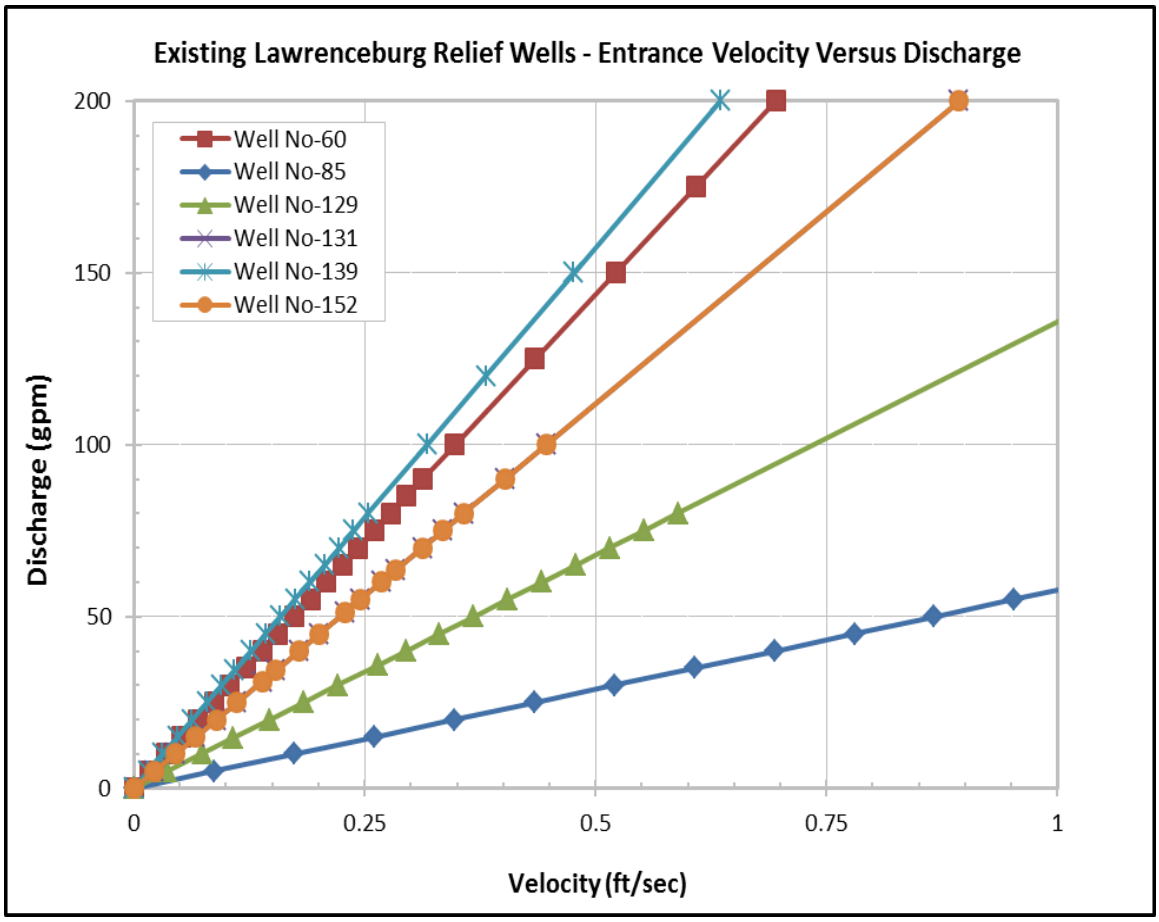

Fig. 7. Projected well entrance velocities versus discharge rate for existing Lawrenceburg relief wells (Fig. 3). The intake area per foot of screen varies from 4.2 to 5.5 square inches corresponding to open areas of 3.7 to 4.8 percent (Table 1). Entrance velocities were calculated by dividing discharge rate by open area; note results for Well No-131 and Well No-152 are similar, and results for Well No-151 are not presented because the well screen was found to not penetrate the aquifer.

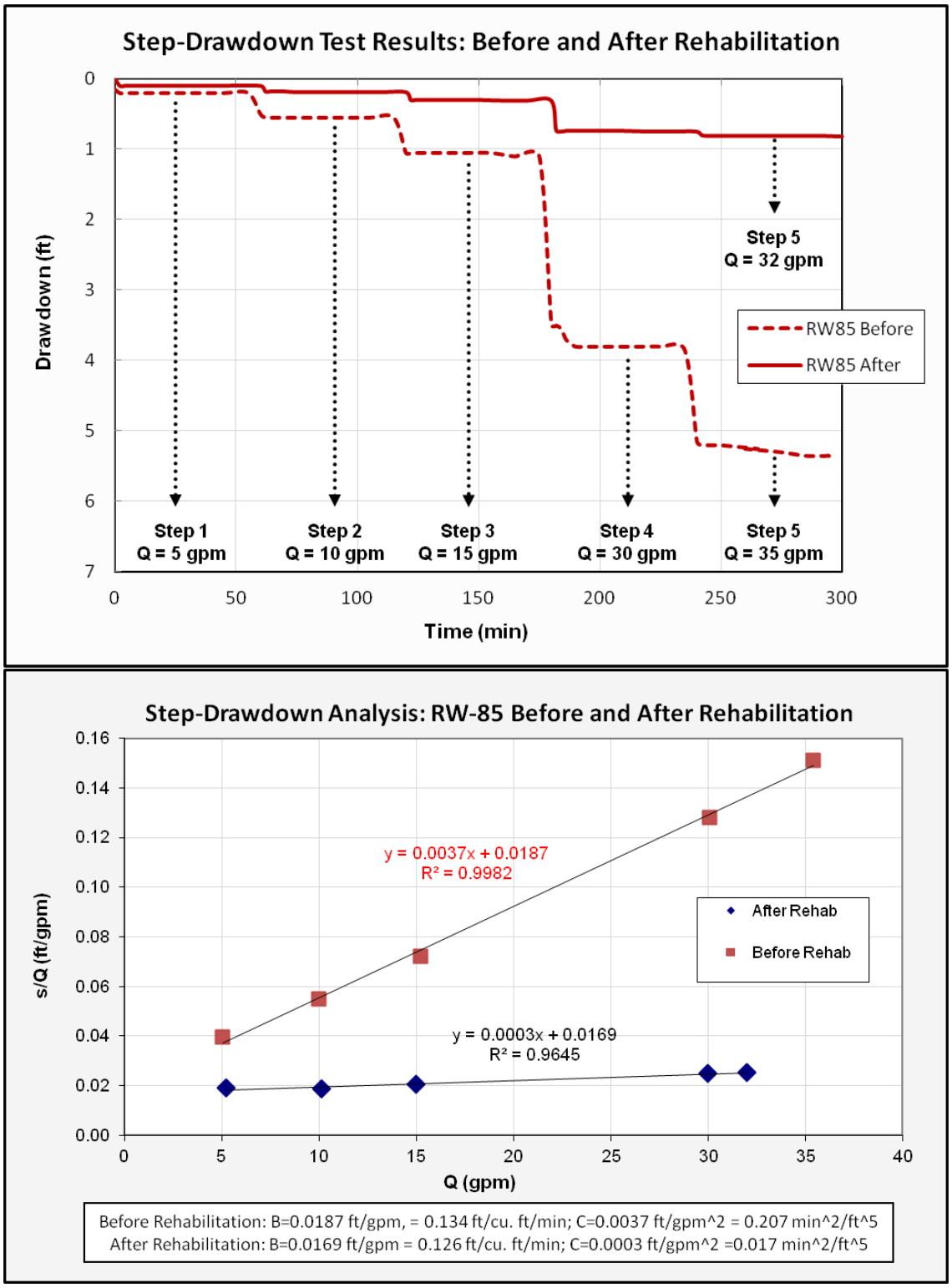

Fig. 8. Step-drawdown data acquired before and after rehabilitation for Lawrenceburg relief well RW-85 (top) with step-drawdown diagnostic plots and computed B and C loss coefficients (bottom). Values of s/Q represent specific drawdown.

Table 1. Characteristics of existing Lawrenceburg relief wells (Fig. 3) that were evaluated. Note that well RW-151 was determined through inspection to not penetrate into the aquifer; this well pumped dry at a testing discharge rate of 10 gpm.

Lawrenceburg Levee Inspected Existing Relief Well Characteristics				
Well	**Screen Length into Aquifer (ft)**	**Aquifer Penetration (%)**	**Intake Area per Foot of Screen (in^2)**	**Open Area (%)**
RW-60	18.5	23	5.0	4.4
RW-85	3.7	6	5.0	4.4
RW-129	10.4	13	4.2	3.7
RW-131	14.4	18	5.0	4.4
RW-139	18.4	27	5.5	4.8
RW-151	0.1	0	5.0	4.4
RW-152	14.3	15	5.5	4.8

Table 2. Measured and projected performance parameters for Lawrenceburg relief well RW-85, based on step-drawdown testing before and after rehabilitation. L_p represents computed Driscoll's parameter values (ratio of laminar head loss to total head loss), s_1 and s_2 represent drawdown values computed from the linear (B) and nonlinear (C) well loss coefficients (Fig. 8), and Q/s_t (discharge per total drawdown) is specific capacity computed from B and C coefficients. See text for discussion.

Lawrenceburg Levee RW-85 Performance Parameters								
		L_p (%)		**s_2 (ft)**		**s_1 (ft)**	**Q/s_t (gpm/ft)**	
Well	**Q (gpm)**	**Before Rehab**	**After Rehab**	**Before Rehab**	**After Rehab**	**After Rehab**	**Before Rehab**	**After Rehab**
RW85	5	50.4	91.9	0.09	0.01	0.08	26.9	54.3
	10	33.7	84.9	0.37	0.03	0.17	18.0	50.2
	20	20.3	73.8	1.47	0.12	0.34	10.8	43.6
	30	14.5	65.3	3.31	0.27	0.51	7.7	38.6
	40	11.3	58.5	5.89	0.48	0.68	6.0	34.6
	50	9.2	53.0	9.20	0.75	0.85	4.9	31.3

Major Roadway Landslide Repair along the Rolling Fork River in West-Central Kentucky

Adam Wynn Lewis, E.I.T., M. ASCE [1]

Abstract: This presentation includes a comprehensive step through of a major project in Nelson County, Kentucky on the Rolling Fork River – from identification through completion. In this presentation, innovative erosion control, landslide repair, and wall technologies are outlined, as well as methods available to designers and contractors for mitigation. Each solution discussed is both robust enough to repair the most challenging geohazards and specific enough to prevent unnecessary environmental impact to rivers, while minimizing delay to roadway users. Utilized technologies include traditional soil nailing, reinforced shotcrete, micropiles, horizontal drains, and geosynthetically confined soil walls.

In 2007 a landslide mechanism was noticed along KY 247 at MP 1.2. Since the damage to that point was only minor in nature, it was determined that remediation would be postponed and site monitored in the intervening years. Until 2013 only a relatively small amount of movement was identified. After a severe flooding event in the spring of 2013, the slope failed causing the road to be closed to all traffic. This presentation will present the lower cost remediation techniques that could have been utilized between 2007 and 2013, which would have stopped the slope's movement and prevented the costly repair that was ultimately required after the flooding event in 2013.

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Design and Construction of Underseepage Pressure Relief Wells along a Federal Levee

Justin Anderson, P.E.[1], Patrick Poepsel, P.E.[2], Bryan Kumm, P.E.[3], and Jeff Krist, P.E.[4]

Abstract: This paper discusses the design and construction of pressure relief wells along portions of the U.S. Army Corps of Engineers (USACE) constructed Council Bluffs Missouri River Levee System, which is a participant in the USACE Public Law 84-99 program. Two major pressure relief well construction projects were completed along this reach in recent years.

The first project consisted of replacing the deteriorated wood stave relief wells along selected portions of the levee alignment. A severe flood occurred in 2011 during the replacement project and sand boils were observed in areas where the original deteriorated relief wells had not yet been replaced and were non-functional due to the collapsed staves refilling with sand. Emergency seepage berms had to be constructed in the areas of the non-functional relief wells. The sections of levee that had already received replacement relief wells performed as intended.

The second project consisted of design and construction of a levee relocation as part of the Council Bluffs Interstate System reconstruction project and required a USACE major Section 408 permit. The total length of the relocated levee is about 1025 feet. The project consisted of designing and constructing 12 relief wells along the landward toe of the relocated levee alignment and abandoning 1 of the existing USACE constructed relief wells.

Specific design and construction challenges for the two projects included: (1) design of the relocated levee and relief well system; (2) preparation of minor and major Section 408 submittals; (3) working within the current USACE, City of Council Bluffs, and Iowa DOT requirements; (4) maintaining an Emergency Action Plan during construction in case of a flood event; and (5) completing an addendum to the Council Bluffs Missouri River Levee System Operations and Maintenance Manual.

Introduction

As the local levee sponsor for the Council Bluffs Levee System, the City of Council Bluffs implemented a replacement program in the summer of 2006 for the existing

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pressure relief wells that were constructed as part of the original levee project. The existing relief wells were severely deteriorated, and many were not functioning. The intent of the project was to replace the existing relief wells in-kind to restore the original level of flood protection provided by the project, and to abandon the deteriorated relief wells. This was one of the levee sponsor's first steps in recertifying the levee system to maintain flood insurance from the Federal Emergency Management Agency (FEMA).

A second relief well project involved relocation of a portion of the Council Bluffs Levee, further referred to as "South Levee". This work was performed as a part of the Iowa Department of Transportation (Iowa DOT) Council Bluffs Interchange System (CBIS) Reconstruction Project, which involved reconstruction of the Interstate 80 and 29 interchange in Council Bluffs, Iowa. To accommodate the interchange relocation construction, a portion of the Council Bluffs Levee System had to be relocated. A Major Section 408 submittal was prepared and submitted to the USACE for this work, which also involved the design and construction of several partially penetrating relief wells.

Original Levee Construction

The Council Bluffs Missouri River Levee System was authorized by the Flood Control Act of 1944 and provides flood protection to the City of Council Bluffs, Iowa. The levee system was designed by the U.S. Army Corps of Engineers – Omaha District. Construction, under the guidance of the Omaha District, began in 1946 and was completed in 1954 (USACE, 1981). The levee consists primarily of a zoned earthen embankment section and is located along the left (east) bank of the Missouri River including tieback levees along Indian Creek and Mosquito Creek (see Fig. 1).

The total length of the levee is about 27.2 miles and it provides flood protection to about 28 square miles and over 30,000 residents in the City of Council Bluffs. The levee is operated and maintained by the City of Council Bluffs and participates in Public Law 84-99 program.

The levee height is generally less than 20 feet above original ground surface and freeboard from the design flood (500-year) ranges from 2 to 5 feet, since some of the levees were originally constructed as agricultural levees. The levee crown is about 10 feet wide and side slopes are generally at 3H:1V. The levee section generally consist of a sand core with a 5-foot lean clay facing on the riverward slope and 2-foot lean clay facing on the landward slope. Subsurface conditions consist of a variably thick clay blanket over pervious sands, which extend to shale and limestone bedrock. The depth to bedrock is generally about 75 to 100 feet.

Where the clay blanket is thin or nonexistent, underseepage control is provided by a combination of seepage berms and relief wells. The seepage berms are generally located in the rural areas and the relief wells are generally located in the urban areas, where site constraints would not allow for seepage berms.

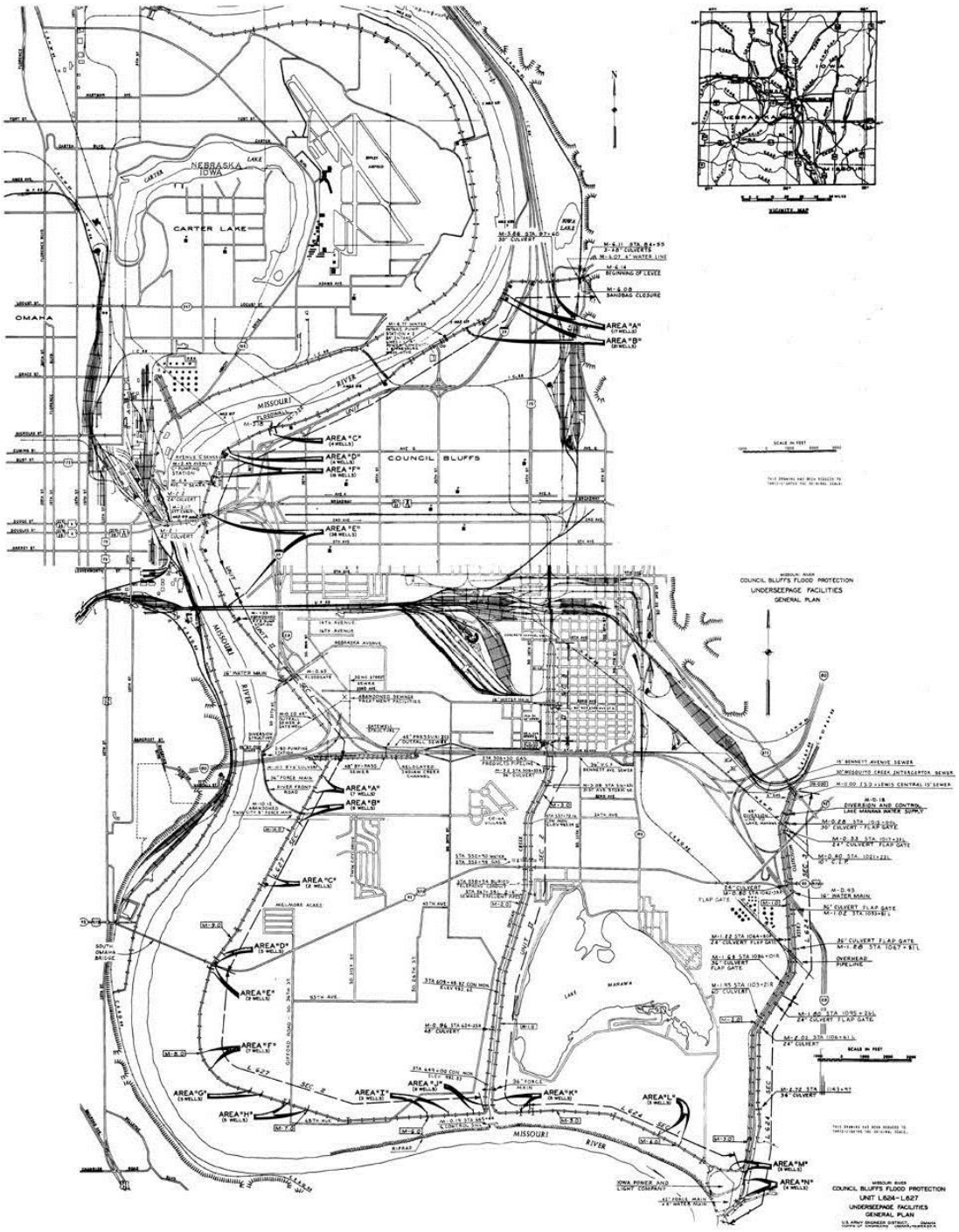

Fig. 1. Plan view of relief well locations along Council Bluffs Levee (USACE, 1981)

A total of 171 relief wells were installed as a part of the original levee construction. The relief wells are partially penetrating and are spaced at 70 to 270 feet on center. The depth of the relief wells vary from 40 to 50 feet below existing grade. The screened portion of the relief wells is provided by an 8-inch diameter Douglas fir or White pine slotted screen that is about 25 feet in length.

Risk Analysis of Relief Well Replacement Program

Since 2006, the City of Council Bluffs has been in the process of recertifying the levees in order to maintain flood insurance from the Federal Emergency Management Agency (FEMA). The first step taken in the process of recertifying the levee was to replace the existing wood stave relief wells. A risk analysis was performed in order to prioritize replacement of the relief wells. The risk analysis looked at the following factors:

- The inventory of the relief wells (missing wells, previously replaced wells);
- The current condition of the relief wells (silted in, tested below capacity, damaged);
- Proximity of the relief wells to existing landward features (homes, businesses);
- The subsurface conditions in each area;
- The spacing and depth of the relief wells and the height of the levee;
- Proximity of the relief wells to the river; and
- Areas of known past problems during high water.

The relief wells were separated into groups based on location. Each group of relief wells was rated as a high, medium, low, or no priority, based on the results of the risk analysis. At the time of the risk analysis:

- 45 relief wells had already been replaced due to being identified as non-functional prior to the risk analysis;
- 34 relief wells had been abandoned as part of the development along the riverfront;
- 28 relief wells were identified as being high priority;
- 23 relief wells were identified as being medium priority;
- 17 relief wells were identified as being low priority; and
- 24 relief wells were identified as needing further evaluation.

Prior to construction of the relief wells, a minor Section 408 submittal consisting of plans and specifications to replace the existing relief wells in-kind was provided to the USACE for review and acceptance.

Replacement of the relief wells consisted of abandoning the existing wood stave relief wells in-place and installing a new relief well adjacent to the abandoned relief well. The new depth, screened interval, diameter, and spacing of the relief wells matched that of the abandoned relief wells. A stainless steel screen and riser were used instead of the wood stave, to meet current USACE guidelines for relief wells. A sand and gravel filter pack was placed around the screen. Design of the filter pack and screen was the responsibility of the contractor and was determined by gradation tests from borings drilled by the contractor.

The estimated budget cost to construct each well was approximately $25,000. The new well construction also included clearing trees and brush adjacent to the levee; re-grading drainage ditches to pump stations; site restoration; and seeding. Some well construction also included removal of encroachments such as fences, sheds, and stored items from the base of the levee.

Fig. 2. Photo of relief wells under construction (upper left – drilling, lower left - gravel for filter pack, and right – stainless steel well screen)

Missouri River Flood of 2011

During the summer of 2011, a historic flood occurred on the Missouri River. The river was above flood stage for over 100 days and crested at a stage level of 7.29 feet above flood stage (see Fig. 3). This resulted in water up into the levee freeboard. Due to the exceptionally long duration of the flood, many portions of the levee likely experienced a steady-state seepage condition.

The USACE and City of Council Bluffs were actively flood fighting for the duration of the flood, which lasted about 3 months. At the time of the flood, a total of 76 relief wells in six well groupings had been replaced. These new relief wells performed well during the flood and did not require additional remediation to control underseepage. At the time of the flood, 61 of the existing relief wells had not been replaced. Excessive underseepage was observed at over 14 of the existing relief well grouping locations resulting in sand boils. Seepage berms consisting of 75-foot wide and four feet thick filter sand blankets had to be constructed in the area of the poorly performing relief wells to mitigate the excessive underseepage.

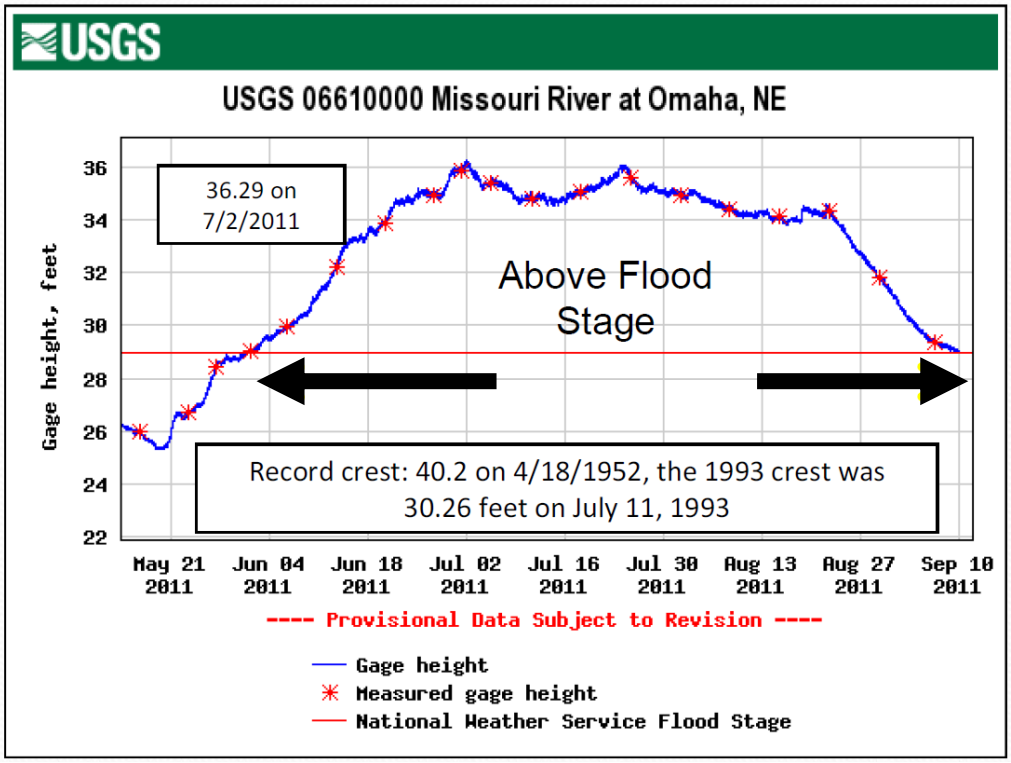

Fig. 3. USGS Missouri River Gage at Omaha, NE

Air lifting techniques were used to attempt to clean out the existing poorly performing relief wells. However, many of the poorly performing relief wells did not improve after this attempt. The photos presented in Fig. 4 below were taken along the same section of newly installed relief wells (2009) discussed previously.

Fig. 4. Photos of newly constructed relief wells during high water

The photos in Fig. 5 below were taken during the flood fight at locations where emergency granular fill seepage berms were constructed near non-functional, or underperforming relief wells that had not yet been replaced prior to the 2011 flood.

Fig. 5. Photos of emergency seepage berms constructed around existing relief wells that had not been replaced

South Levee Relocation

Design

The Iowa DOT is currently completing a greater than $1 billion reconstruction of the I-80 and I-29 Interstates through the City of Council Bluffs. This reconstruction required that a portion of the Council Bluffs Levee had to be relocated to make room for the new west interchange.

The relocated South Levee is about 1,025 feet in length and about 14 to 17 feet above existing grade. The levee crown is about 12 feet wide, and the side slopes are generally at 3H:1V. The levee consists of a zoned clay section with impervious fill (lean clay) on the outer faces and lean or fat clay on the interior random fill zone (see Fig. 6).

An extensive field and laboratory investigation was completed in the area of the relocated South Levee, which included dozens of Standard Penetration Test Borings and Cone Penetration Tests, as well as a robust laboratory soils testing program. The general subsurface profile in the area of the relocated levee consisted of a thin to nonexistent blanket of lean clay and silty sand underlain by poorly graded sand extending to bedrock. Bedrock was encountered at a depth of about 83 feet below the ground surface.

Permeability of each soil type was calculated to complete the seepage analyses and relief well design. A horizontal permeability for the poorly graded sand of 0.002 feet per second was used during the USACE design of the original levee system (USACE, 1948) and was used in the seepage analyses and design of the relief well system. A horizontal permeability for the silty sand of 0.00008 feet per second was estimated using typical

values found in published literature and Hazen's equation with a Hazen's empirical coefficient of 1.0.

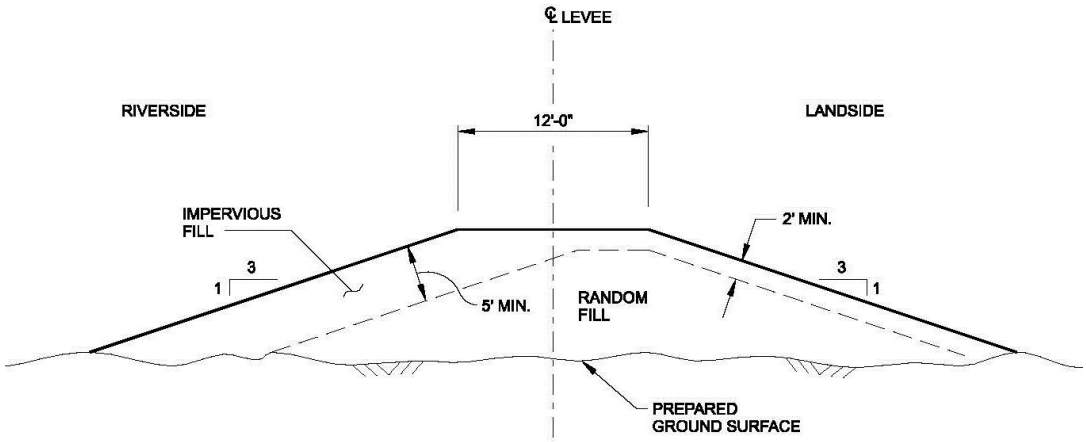

Fig. 6. Typical embankment fill zoning of proposed relocated South Levee

Analyses of underseepage were performed in general accordance with the USACE - Omaha District criteria described in *Design Guidance for Levee Underseepage*, EM 1110-2-1913 (USACE, 2005) and from *Design and Construction of Levees*, EM 1110-2-1903 (USACE, 2000). The evaluation of underseepage was performed based on the thickness and characteristics of the natural blanket, the thickness and permeability of the foundation sands, and the maximum head acting on each of the levee sections. According to these USACE documents, the following criteria for underseepage must be met, where site constraints preclude seepage berms:

- The upward gradient, i_o, through the blanket material at the landside toe of the levee should be less than 0.8 for water at the top of the levee.
- The upward gradient, i_o, through the blanket material at the landside toe of the levee should be less than 0.5 for water at the Design Water Surface Elevation (DWSE).

The cross section of the relocated South Levee is provided in Fig. 7.

The underseepage analyses of the relocated South Levee were completed using the finite element computer program SEEP/W. SEEP/W was developed by GEO-SLOPE International Ltd., and calculates equipotential lines, head drops, flow gradients and seepage pressures for the flow of water through a layered, porous and anisotropic material.

The critical analysis was for a situation where the river is sustained at the DWSE for a sufficient time for steady seepage to develop vertically and the drainage ditch at the toe of the levee is dry. The critical analysis resulted in a gradient at the drainage ditch of 1.27, which greatly exceeds the USACE criteria for a maximum allowable vertical

gradient of 0.5, thus the implementation of underseepage control measures was warranted.

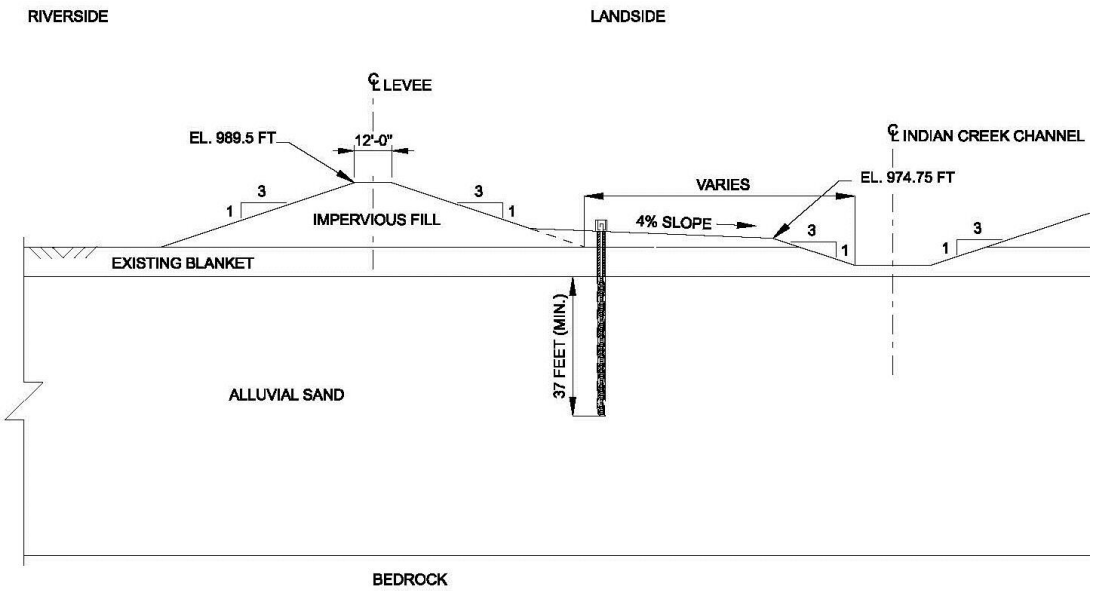

Fig. 7. Critical design section for underseepage of proposed relocated South Levee

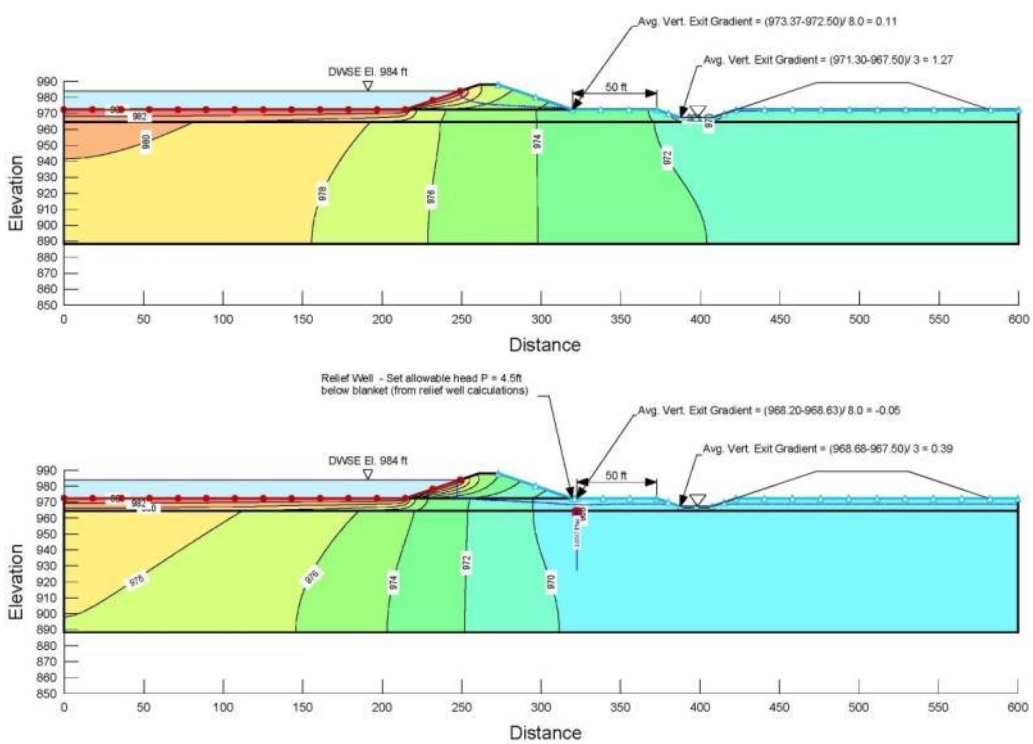

Fig. 8. SEEP/W analyses for underseepage of proposed relocated South Levee

Due to the constraint of the drainage ditch on the landward side of the levee, a landward seepage berm could not be constructed. A semi-pervious riverside blanket was considered, but not selected, because it was not a practical solution to reduce the critical gradient in the drainage ditch to an acceptable level. As such, relief wells were selected as the underseepage control measure.

The design of the relief well system for the relocated South Levee was performed in accordance with the USACE document "Design, Construction and Maintenance of Relief Wells", EM 1110-2-1914 (USACE, 1992). The design assumptions for this analysis are summarized as follows:

- An infinite line of partially penetrating relief wells.
- 8-inch diameter well screen and riser comprised of stainless steel.
- Relief well spacing, capacity and drawdown designed for water at the DWSE of 984 feet.
- A design factor of safety against uplift between relief wells of 1.5.
- An additional safety factor of 1.3 applied to the computed relief well spacing.

The results of the analyses indicated that a series of 8-inch diameter partially penetrating relief wells installed at a center-to-center spacing of 100 feet would provide the required drawdown of 6 feet between wells and a factor of safety against uplift of 1.5.

The exit gradient at the DWSE was computed to be 0.39 at the drainage ditch for analyses that modeled an infinite line of relief wells. The partially penetrating relief wells were designed to be about 57 feet deep, have a minimum screened length of 37 feet and a design capacity of 59 gallons per minute (gpm).

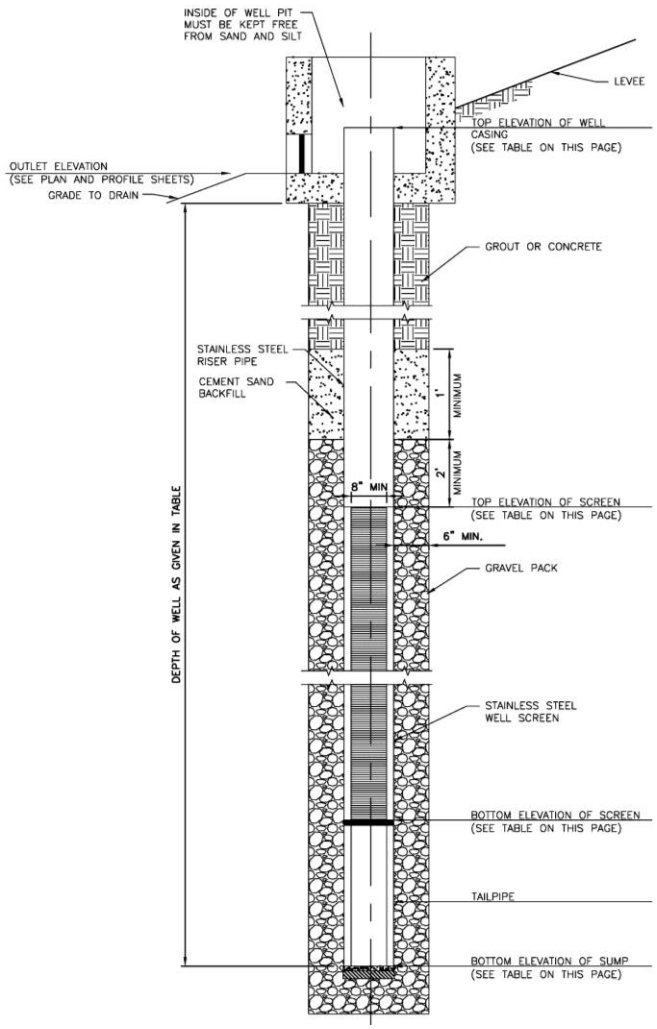

Fig. 9. Typical relief well detail

USACE Major Section 408 Submittal

The results of these analyses were submitted as part of a larger design package to the USACE in a Major Section 408 submittal. The USACE requires that realignments of levees be reviewed and accepted by the USACE prior to construction to ensure that the project has no adverse impact on the levee system.

The submittal included evaluation of the stability of the relocated South Levee, as well as underseepage, erosion control, materials, construction sequence; an assessment of the hydraulic and hydrology impacts; and requirements for operation and maintenance. Other items addressed in the submittal included real estate analysis, risk analysis, public interest determination, record of decisions made leading to the request to relocate the levee, and environmental protection compliance. The Engineer of Record for the submittal signed and sealed the document and stated that the proposed levee relocation does not adversely affect the operation or integrity of the levee system.

The submittal had to be reviewed and approved by the City of Council Bluffs, USACE Omaha District, USACE Northwest Division, USACE Headquarters, and the USACE Chief of Engineers. The approval process took about 2 years to complete and was a major component of the Iowa DOT's project schedule. Comments provided by the USACE resulted in a change to the design assumption that the landward drainage ditch would be filled with water during the design flood event. Assuming that the landward drainage ditch was dry along with steady-state seepage ultimately resulted in the need to design and construct the relief wells.

Construction

The construction of the relocated South Levee was completed in the fall of 2012. Installation of the 12 relief wells was completed during the month of September. Since this time period was during the flood season, which runs from March 1st through November 1st, the contractor was required to prepare an Emergency Action Plan, (EAP). The EAP requirements included:

- Maintaining a continuous line of protection,
- Daily Monitoring of the USGS Missouri River gage and forecasts,
- Ceasing construction operations and backfilling excavations when the river stage reaches the action level (5 feet below Flood Stage) or excessive seepage is observed, and
- Providing a list of equipment and materials that will be available should a flood occur during construction.

Prior to construction of the relief wells, the contractor was required to collect samples of the underlying sands and design the well screen and filter pack. The resulting design consisted of 0.025 slot wire wound screen and gravel filter pack.

The wells were installed with a bucket auger rig, utilizing polymer slurry to stabilize the sidewalls of the borehole during drilling and placement of the relief wells. Upon completion of the excavation, the screen and riser were lowered into the borehole and

filter pack was placed around the screen. The well was finished by placing bentonite cement grout to ground surface.

Fig. 10. Photo of well drilling with bucket auger rig

Each relief well was developed using a method called air lift/surging. An air compressor hose is attached to the bottom of a PVC pipe. Air is pumped through the hose which creates a vacuum in the pipe. The vacuum pulls water and sediment through the filter sand. The PVC pipe is lifted and lowered along the well screen for about 30 minutes.

A 6-hour pump test was completed at each well. The pump test consisted of pumping the well at a flow rate of 60 gpm for 6 hours and measuring the drawdown in the well and the two adjacent wells. As part of the pump test, a sand test was completed using a Rossum Sand Tester.

Fig. 11. Photos of well installation and completed system

One year after construction, a high water event occurred in the spring of 2014. A photo of the completed levee during this high water event is provided as Fig. 12 below.

Fig. 12. Photo of completed south levee

Operations and Maintenance Manual Update

Following construction of the South Levee, the project Operations and Maintenance (O&M) Manual needed to be updated. As with any levee modification, the as-built drawings were updated and archived for inclusion in future revisions of the O&M Manual for the levee system. Critical guidance for operation and regular maintenance for the relief wells was provided, which generally consists of drawdown flow testing on a bi-annual basis and suggested cleanout methods should the minimum acceptable values not be reached.

Conclusions and Lessons Learned

The design and construction of a relief well system is a rigorous process. Newer numerical methods using sophisticated computer software have been utilized in concert with more established blanket theory methods for the evaluation of underseepage pressures and the resulting uplift gradients below levee embankments. The newer and older methods have been found to be in general agreement and complimentary to one another.

The process involved with a Major Section 408 submittal is timely and the construction schedule needs to be able to accommodate a timeline necessary to get the submittal approved. The design and construction budget needs to be adequate for the process, with the expectation that the process will result in additional project features not initially assumed. In the case of the relocated South Levee, the Major Section 408 process resulted in the need to design and install relief wells.

The Emergency Action Plan should be reviewed with the contractor prior to construction so that the contractor is prepared to react to a flood event. Adequate time should be placed in the construction schedule to allow for several iterations during development of the EAP with the contractor.

Maintaining an updated O&M Manual that includes the complete documentation of all modifications to the levee system is a very useful resource as a permanent record of construction when evaluating underseepage control measures such as relief wells. As-built drawings and detailed recommended maintenance procedures should be included in the O&M manual immediately upon completion of levee modifications. The levee sponsor should be very familiar with the appropriate O&M procedures for both regular maintenance and flood fighting purposes.

The decision to implement the use of relief wells in a levee system should not be taken lightly, primarily due to their high costs of construction and high cost and the importance of maintenance. As made evident by field observations of sand boils in the areas of non-functioning relief wells during the 2011 Missouri River flood, the stability of the levee system is dependent on the functionality of relief wells. Relief wells should be regularly monitored to ensure functionality in the event of a flood event and levee local sponsor budgets should be structured to accommodate these requirements.

References

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Relief Well Assessment and Design, Saving Taxpayer Dollars – Southwestern Illinois Levees

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Abstract: The Southwestern Illinois levee system consists of four individual levee systems (Metro East, Wood River, Prairie DuPont and Fish Lake) on the east bank of the Mississippi River from Alton, Illinois to Columbia, Illinois, that together have approximately 60 miles of earthen levee. Underseepage was an on-going problem and identified as a design deficiency that required correction for FEMA certification. During the geotechnical analyses, value engineering, and rehabilitation design for this project, over 250 existing (circa 1950s) wood-stave pressure relief wells were evaluated for condition and performance. Creosote-contaminated water was removed from each well prior to testing, cleaned using charcoal filtration, and then each well was tested for specific capacity and sanding rate. A substantial percentage of the existing wells had good efficiency as compared to the specific capacities measured at the time of installation, and met the project criterion for less than 5 parts per million sand production.

The results were used to evaluate whether existing wells provided reliable protection for underseepage-prone areas of the levees or should be replaced or supplemented with other underseepage controls. The engineer initially prepared evaluations of each design reach using the "leaky blanket theory" as a screening tool and compared the results to current standards and project criteria. The leaky blanket theory considers an impermeable levee constructed on a 2-layer subsurface system and was judged appropriate as preliminary design tool to determine if existing well efficiencies and percent penetration were adequate. A two-dimensional finite element model was also used to evaluate numerous areas with lithologic or geometric conditions that were too complex for leaky blanket methods. The two-dimensional model was coupled with the USACE Engineer Research and Development Center hybrid analysis to calculate well losses and determine appropriate outlet elevations and spacing. As a result of the testing program and analyses, many existing relief wells were left in service, some were replaced, and additional wells were installed as indicated by the analyses. Overall, the use of many existing wells produced a significant savings in cost and time.

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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 INTRUMENTATION, 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 INFRASTUCTURE RECONSTRUCTION, October 15, 1993, Fort Mitchell, KY

CHRONOLOGY OF OHIO RIVER VALLEY SOIL SEMINARS (CONTINUED)

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
ORVSS XXVIII	UNCONVENTIONAL FILLS: DESIGN, CONSTRUCTION AND PERFORMANCE, October 10, 1997, Lexington, KY
ORVSS XXIX	PROBLEMATIC GEOTECHNCIAL 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 INFRASTRUCTUREAND 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