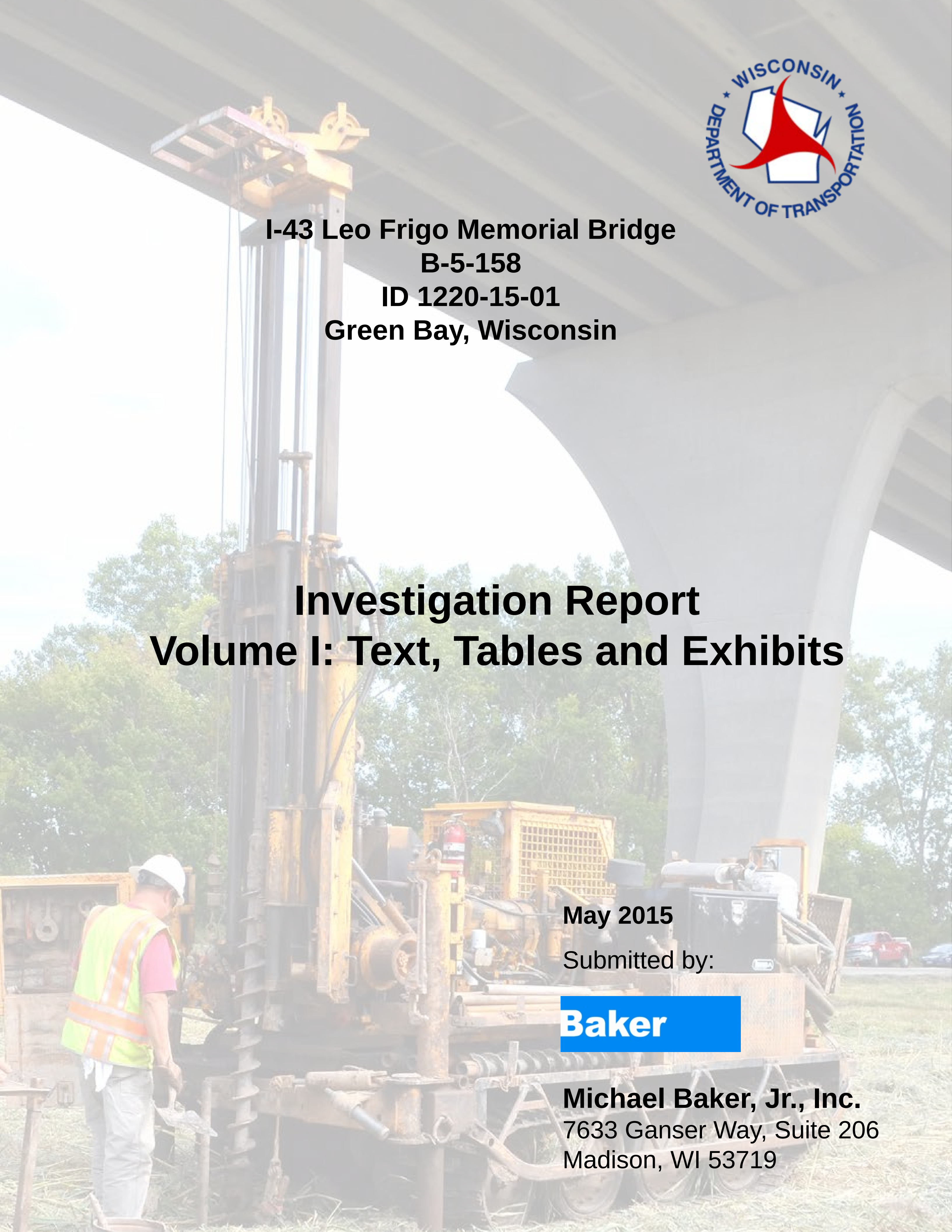


I-43 Leo Frigo Memorial Bridge
B-5-158
ID 1220-15-01
Green Bay, Wisconsin

Investigation Report
Volume I: Text, Tables and Exhibits

May 2015

Submitted by:

Michael Baker, Jr., Inc.
7633 Ganser Way, Suite 206
Madison, WI 53719

ID 1220-15-01 May 2015

I-43 Leo Frigo Memorial Bridge Investigation Report
EXECUTIVE SUMMARY

Background: This report presents the findings from the investigation, under State of Emergency Executive Order #14, to repair the I-43 Leo Frigo Memorial Bridge (Structure B-5-158), as well as to mitigate potential damage caused by a sag in the bridge deck in and about Pier 22. The sag was the result of approximately 2 feet of vertical displacement at Pier 22 that is believed to have occurred between approximately 3:00 and 3:45 AM on Wednesday, September 25, 2013, after which the bridge was closed to traffic.

2013 Investigation – Entire Bridge: A site reconnaissance was performed by the Wisconsin Department of Transportation (WisDOT) on Thursday, September 26, 2013 and then by WisDOT, Federal Highway Administration (FHWA), Michael Baker Jr., Inc. (Michael Baker), and Wiss, Janney, Elstner Associates, Inc. (WJE) on Friday, September 27. A bridge inspection of the underside of the superstructure between Piers 18 and 25 was performed and no visible damage was detected on the superstructure members. A subsurface exploration then commenced on Saturday, September 28, 2013 wherein representative sections of steel piles that support the piers were exposed and test borings were drilled. Test pits were excavated at 20 out of the 51 piers. Laboratory tests were then performed on soil and water samples to determine foundation conditions, and robotic survey instruments were installed to monitor for any additional pier displacement. These investigations determined that severe corrosion of the steel piles in the foundation that support Pier 22 was the reason for the vertical displacement. Recommendations were then developed to return the bridge to service by installing temporary shoring towers and new, supplemental permanent foundation elements.

Analysis: Based on the results of the investigation, the bridge piers were grouped into three different tiers. Tier 1 piers are locations where immediate repairs were necessary in order to return the bridge to service. These locations are characterized by severe pile corrosion that significantly reduced the structural capacity of the bridge foundation. The soils around Tier 1 piers contain a highly corrosive combination of industrial porous fly ash, high concentrations of chlorides, high sulfate concentrations, low soil resistivity, and microbial activity. The water table was typically below the bottom of the Tier 1 pier footings.

Pile Corrosion at Pier 22

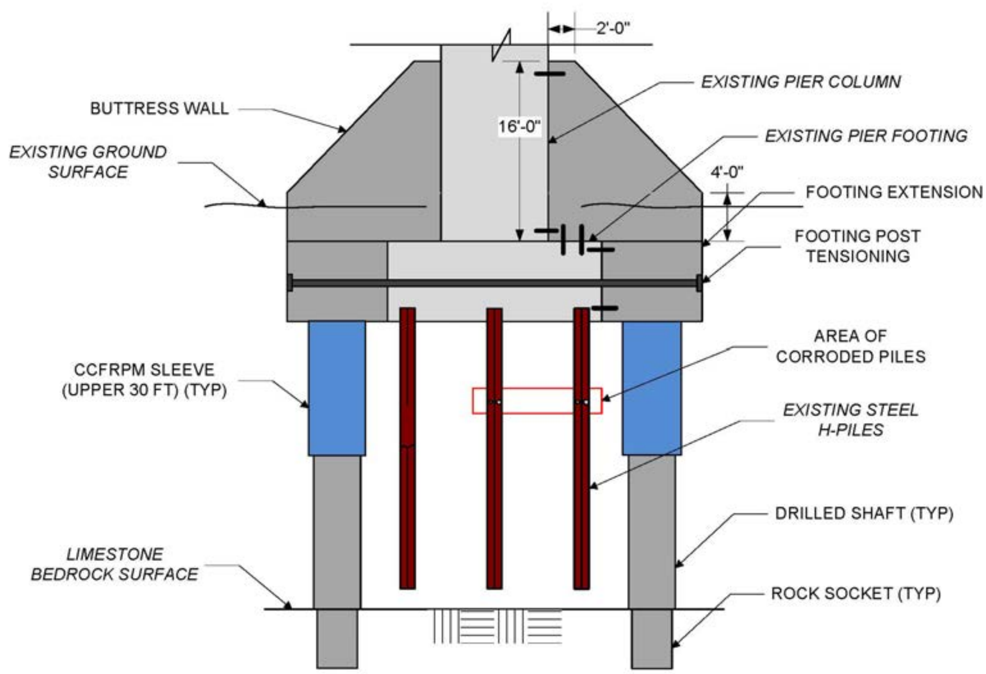
Tier 2 piers are locations where much lower levels of pile corrosion damage were observed and the soils exhibit characteristics that are potentially corrosive, but much less severe than Tier 1 locations. Tier 2 locations do not include large amounts of industrial fly ash, but do have some combination of organic material, high levels of chlorides, sulfates, or microbes; or low soil resistivity values. None of the Tier 2 locations represent an immediate safety concern. However, the locations do warrant further monitoring to determine if there is potential for long-term corrosion.

Tier 3 piers are locations where the potential for severe pile corrosion is very low. Visual inspection of exposed piles at Tier 3 locations indicated pile conditions typical for a 32+ year old bridge in normal soils. Consequently, no further investigation is warranted.

Cause of Vertical Displacement at Pier 22:
The investigation determined that several factors contributed to a highly unusual environment that caused the severe corrosion of the steel pile foundation supporting Pier 22. A corrosive environment was created by the presence of a moist, porous fly ash fill with high levels of chlorides and sulfides combined with a low resistivity. A dense clay layer was present below the porous fly ash, leading to differential oxygen concentration and differential chemical concentrations within the fill layers. Bacteria were found at many of the piers and it is likely that microbiologically influenced corrosion also played a role in the corrosion at Pier 22. An aggressive corrosion mechanism of differential oxygen concentration corrosion in the form of Accelerated Low Water Corrosion (ALWC) is likely responsible for the deterioration in the localized failure zone at Pier 22.

These site conditions led to rapid corrosion of sections of the steel piles. Once sufficient material and support were lost from enough piles, the remaining piling became unstable. Visual examination of selected Pier 22 piles indicated that the most common, perhaps only, mode of pile instability was crushing/buckling of the most heavily deteriorated sections of pile. Severe deterioration and crushing/buckling was observed on all piles that were exposed.

Tier 1 Pier Repairs: Visual inspection and soil borings confirmed the presence of fly ash surrounding severely corroded steel piles at Piers 21, 22, 23 and 25. While the piles observed at Pier 24 were substantially less corroded, Pier 24 was conservatively treated similar to Piers 21, 22, 23 and 25 based on its proximity to those critically corroded pier locations. Therefore, repairs were performed at Piers 21 through 25 prior to returning the bridge to service. Tier 1 repairs included installation of new concrete drilled shaft foundations, which are capable of supporting the entire pier design load. These new foundations were then connected to the existing piers and provided corrosion protection measures designed to offer 75 years of service life.



Schematic of Tier 1 Repair

Tier 2 Pier Monitoring: Numerous steel piles at pier locations outside the Tier 1 repair area contained localized areas of corrosion pitting and minor section loss. Seventeen of these piers were included in the Tier 2 category.

Typical Tier 2 Corrosion

In order to estimate the future corrosion rates to determine the remaining service life of the Tier 2 steel piles, monitoring probes and steel coupons (pile sample sections) were installed in 2014 adjacent to Piers 6, 13, 19, 32, 39, 44 and 50. Data will be collected from the probes during typical bi-annual bridge maintenance inspections performed by WisDOT. The probe data will be supplemented by section loss measurements conducted on the pile coupons. If the measured corrosion rates from the probes turn out to be greater than the estimated rate, the Department will have multiple treatment options available to reduce the corrosion rate and extend the service life of the bridge.

In addition to the work described above at the Tier 2 piers, monitoring probes were also installed at Pier 22. The information collected at this location can be compared with the data across the rest of the project site and provide information on what is anticipated to be the upper range of corrosion activity present at the site.

Tier 2 Overview

<u>Summary:</u> In response to the bridge sag on Wednesday, September 25, 2013, an in-depth investigation of the bridge was performed. Emergency pier repairs were completed on the bridge at five locations in January 2014. These repairs provided corrosion protection measures that will offer 75 years of service life. Future monitoring of additional Tier 2piers at the site will continue to ensure the safety of the bridge. As a result of the investigation, repairs, and future monitoring, the public should have confidence that the bridge is safe for their continued use well into the future.

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1.0 Introduction

1.1 General

Michael Baker Jr, Inc. (Baker) has been retained to conduct a subsurface investigation under State of Emergency Executive Order #14 to repair the I-43 Leo Frigo Memorial Bridge (Structure B-5-158) and to mitigate potential damage caused by a sag in the bridge deck in and about Pier 22 within Unit 7 of the bridge. (See Figure 1.1 for an illustration of the bridge units.) The sag was the result of approximately 2 feet of vertical displacement at Pier 22 that is believed to have occurred between about 3:00 and 3:45 AM on Wednesday, September 25, 2013, after which the bridge was closed to traffic.

The Wisconsin Department of Transportation (WisDOT) conducted an initial site reconnaissance on Thursday, September 26, 2013.

Subsurface exploration was conducted between September 28, 2013 and October 19, 2013 and included the installation of 49 soil borings, 25 ground water sampling points, and 39 test pits, and physical inspection of select piles combined with in situ resistivity and half-cell potential measurements. Select soil samples were tested in the laboratory to classify the soil encountered and perform soil resistivity, soil and water chemical analyses, and biological analyses. Select ground water samples were tested in the laboratory for chemical and biological analyses.

A bridge inspection of the underside of the superstructure was completed between Piers 18 and 26 between Sunday, September 29, 2013 and Tuesday, October 1, 2013.

For discussion purposes in this report, Structure B-5-158 has been sub-divided into three reaches as follows and illustrated in Figure 1.1 and Exhibit 2.

- East Approach Spans: East Abutment to Pier 27,
- Main Span: Pier 27 to Pier 28, over the Fox River (main channel)
- West Approach Spans: Pier 28 to West Abutment, which includes Piers 28 to 51

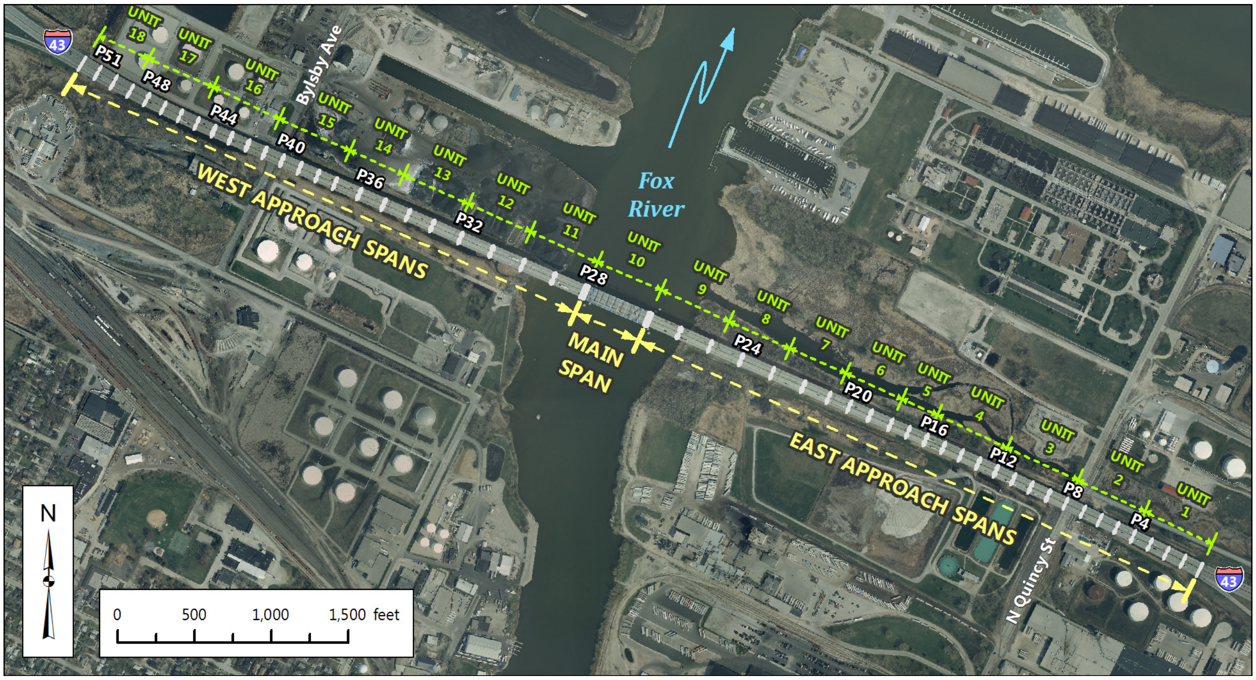


Figure 1.1: Structure Layout

1.2 Purpose

A subsurface investigation was completed to explore foundation conditions, determine the cause of footing movement, stabilize the bridge, and develop plans to repair Structure B-5-158.

1.3 Scope

At the beginning of the project the scope was not defined beyond determining what caused the vertical displacement at Pier 22 and developing plans to repair the bridge so it could be opened to traffic. Throughout the course of the investigation, additional items were identified based on the latest available information. The following list highlights the key items of work that were completed during the 2013 investigation.

- Reviewed available as-built substructure plans, pile foundation data. and other geotechnical-related information received
- Performed a field reconnaissance
- Worked with WisDOT bridge engineers to perform a bridge inspection of the underside of the bridge at/near Pier 22 (between Piers 18 and 25)
- Engaged Wiss, Janey, Elstner Associates, Inc. (WJE) to install accelerometer instrumentation to monitor movement and tilt at Piers 20, 21, 22, 23, and 25. Audible alarms were set to sound if any abrupt movement was recorded.
- Analyzed the superstructure (collaborative effort between Baker, WJE, and WisDOT BOS) to evaluate the integrity and capacity of the existing superstructure in the region of Pier 22, so that work could

proceed to investigate existing conditions and recommend mitigation measures to restore serviceability of the bridge
- Engaged WisDOT NE Region Surveys and Ayres Associates (Ayres) to install robotic survey instruments to monitor and record vertical bridge movement
- Used test pits to expose representative piles at Piers 5, 10 through 25, 32, 39, and 43 and to install lead wires to one of the piles at the southwest corner of Pier 22
- Engaged drill rigs that were provided by WisDOT and GESTRA Engineering, Inc. (GESTRA) to complete 44 auger borings and install 25 monitoring wells
- Engaged drill rigs that were provided by WisDOT and GESTRA to complete one deep boring at Pier 22 to a depth of 142 feet, from which 15 feet of bedrock was core sampled and used to install an inclinometer tube
- Utilized test boring results at the river piers (Piers 26 through 29), which were completed for TRC Environmental Corporation (TRC)
- Engaged OMNNI Associates (OMNNI) and Pace Analytical Services, Inc. (Pace) to perform laboratory testing and chemical analyses. River Valley Testing Corporation (RVT) provided laboratory testing for the river piers
- Engaged AECOM to perform an impulse echo test at two representative piles at the south column of Pier 22
- Engaged Whitlock, Dalrymple, Poston & Associates, Inc. (WDP) to evaluate corrosion potential at the project site
- Engaged The Erlin Company (TEC) to perform a petrographic examination, including scanning electron microscopy (SEM) and ancillary energy dispersive spectroscopy (EDX), on representative core samples of the existing concrete footers and corrosion product that was sampled at representative portions of exposed piling
- Performed tests to check for possible stray current from a nearby impressed current cathodic protection system and from nearby overhead electric transmission lines
- Evaluated the data obtained
- Conducted a more detailed study of exposed piling and in-situ corrosion characteristics at Piers 5, 13, 19, 20, 21, 22, 24, 25, 32, 39, and 43 to refine understanding about the corrosion characteristics of the subsurface material at the project site
- Analyzed the structural capacity of the existing steel pile bridge foundations
- Infrasense, Inc. performed Ground Penetrating Radar and Echo Impact Testing on the Polymer Asphalt Overlay
- Performed bridge inspection for the entire structure (WisDOT) including the main span cables (WJE).
- Prepared this Investigation Report, including the conclusions and recommendations presented in Section 5.0

An additional investigation of the pile conditions at Pier 22 was conducted in July 2014. The investigation further examined the nature and extent of corrosion damage that had taken place at Pier 22. These excavations and inspections provided an opportunity to better understand the failure mechanism that took place at Pier 22 and to better characterize the condition, nature and location of the failed pile sections. The following list highlights the key items of work that were completed during the 2014 investigation at Pier 22.

- Used 5 test pits to further examine the nature and extent of corrosion damage that had taken place at 6 representative piles at Pier22
- Engaged OMNNI and Pace to perform laboratory testing and chemical analyses
- Engaged WDP to evaluate corrosion potential at the project site and install corrosion monitoring instrumentation
- Evaluated the data obtained.

In order to estimate the corrosion rates more accurately and determine the remaining service life of the steel piles more reliably, monitoring probes and steel coupons (pile sample sections) were installed in July 2014. The following list highlights the key items of work that were completed during the 2014 investigation to monitor corrosion rate of the steel piling for the overall bridge.

- Installed steel coupons, typically 3 per pier, at Piers 6, 13, 19, 21, 25, 32, 35, 39, 44 and 50
- Installed corrosion monitoring instrumentation at Piers 6, 13, 19, 32, 39, 44 and 50, which included a combination of half-cell probes and electrical resistance (ER) probes
- Installed half-cell probes and ER probes at Pier 22, which can be compared with similar data that is obtained across the site and serve as an upper range of corrosion activity present at the site
- Engaged WDP to develop mass loss estimates for instrumented piers where detailed inspection and monitoring probes were installed in July 2014.

1.4 Bridge Attributes

Structure B-5-158 carries four lanes of traffic for Interstate Route I-43 over the Fox River in Green Bay, Wisconsin. Two 12-foot lanes, a 6-foot median shoulder, and a 10-foot outside shoulder make up the 40-foot clear distance between parapets in each traffic direction (see Figure 1.2). The bridge was constructed between 1978 and 1980.

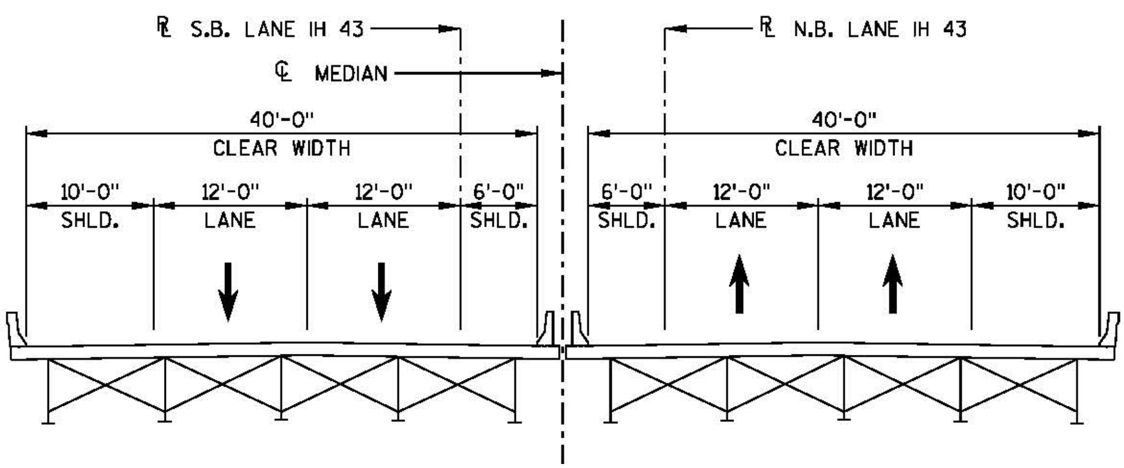


Figure 1.2: Structure Cross Section

The bridge is about 7,983 feet long from center-of-bearing at the East Abutment to center-of-bearing at the West Abutment. A Vicinity Map is presented in Exhibit 1. A Reach Layout is presented in Exhibit 2. A General Plan and Elevation is presented in Exhibit 3.

Structure B-5-158 generally runs east-westerly, with both baseline stationing and pier numbering (1 to 51) increasing from east to west.

Starting at the East Abutment and working west, the bridge structure:

- spans over a single set of railroad tracks and North Quincy Street at Span 7
- continues to the east river bank of the Fox River near Pier 26
- continues to the west river bank of the Fox River near Pier 30
- spans over a single set of railroad tracks and Bylsby Avenue at Span 39
- terminates at the West Abutment

The eastern approach spans comprise Units 1 through 9 (Spans 1 through 27). Units are defined as sections of the bridge between expansion joints. Units 1 through 5 are 70-inch prestressed girders with span lengths of 121 and 125 feet and contain Spans 1 through 18. Units 6 through 9 are steel girders with span lengths ranging from 146 to 220 feet and contain Spans 19 through 27.

The main span (Unit 10, Span 28) is a 450-foot tied arch.

The western approach spans comprise Units 11 through 18 (Spans 29 through 52). Units 11 through 14 are steel girders with span lengths ranging from 172.5 to 220 feet and contain Spans 29 through 37. Units 15 through 18 are 70-inch prestressed girders with span lengths of between 117 and 125 feet and contain Spans 28 through 52.

The piers for the prestressed girder spans (Piers 1 through 8 and 45 through 51) consist of two fluted columns, with a separate pier cap under each northbound and southbound lane (see Figure 1.3). The columns are supported by footings that range in depth from 2.5 to 4.5 feet and in size from 7x19.5 feet to approximately 15x19 feet.

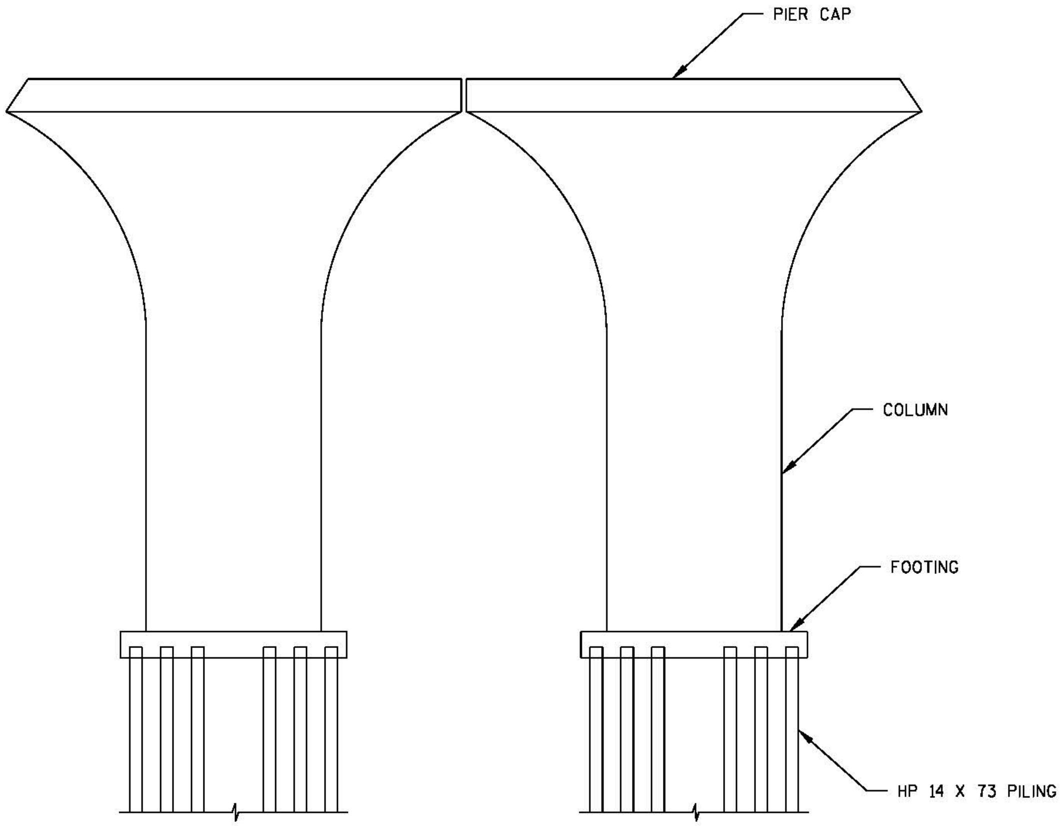


Figure 1.3: Typical Pier Detail for Prestressed Girder Spans

The piers for the steel girder spans (Piers 9 through 37) and prestressed girder spans (Piers 38 through 44) consist of two fluted columns with a continuous pier cap that spans both the northbound and southbound lanes (see Figure 1.4). The columns are supported by footings that range in depth from 4.5 to 7.75 feet and in size from 15x15 feet to 21x24.5 feet. A 6x2.5-foot concrete tie connects the two footings. The pier height (from top of footing to pier cap) varies from about 29 to 151 feet.

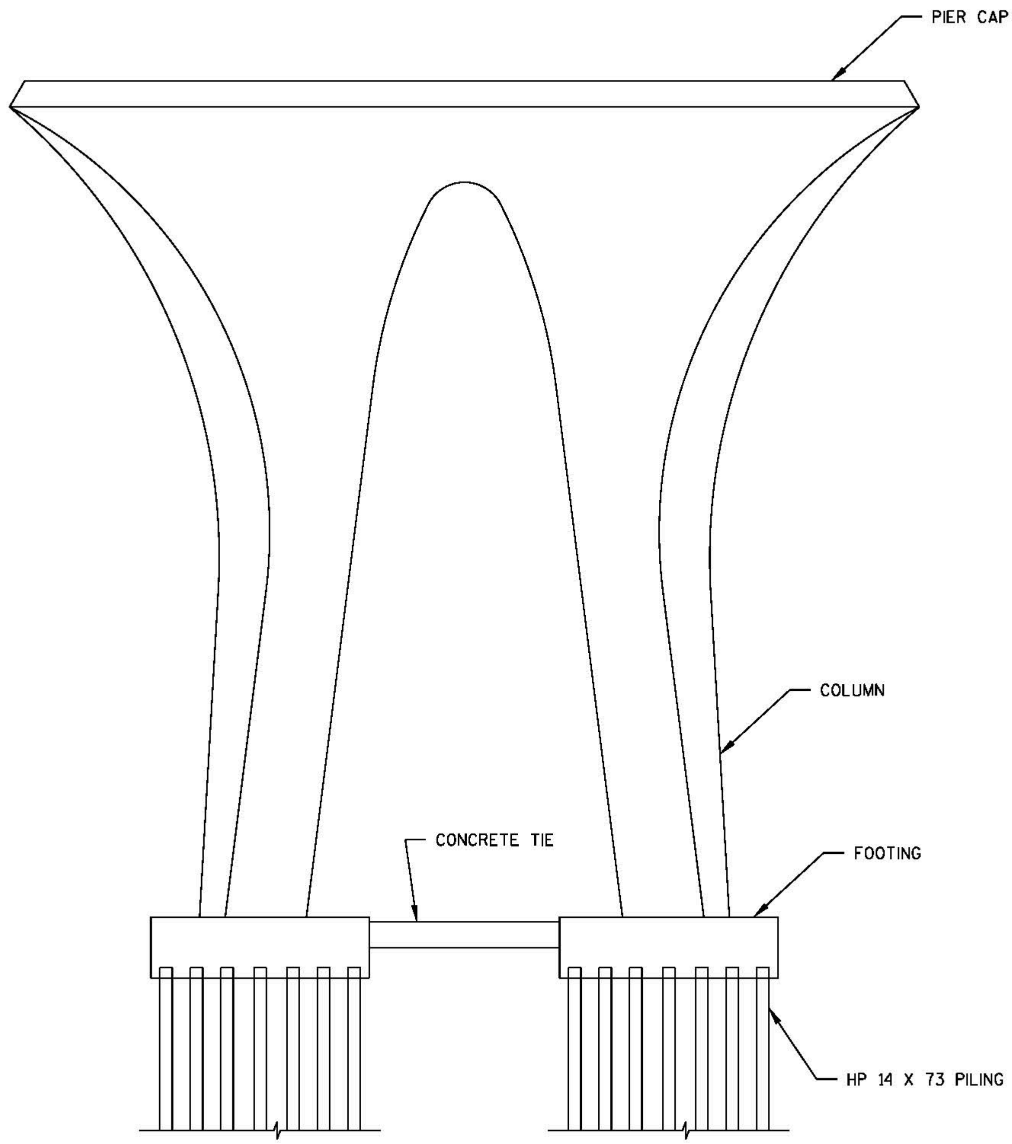


Figure 1.4: Typical Pier Detail for Steel Spans

The footings are founded on steel H-piling, which were driven to the required bearing in the underlying till. Average pile lengths per structure range from about 64 feet per pile at river Pier 28 to about 142 feet at land Pier 12. HP14x73 and HP10x42 steel bearing piles were driven to a minimum bearing value of 75 tons per pile at the West Abutment and the East Abutment, respectively. HP14x73 steel bearing piles were driven to a minimum bearing value of 100 tons per pile at river Piers 26 to 29. HP14x73 steel bearing piles were driven to a minimum bearing value of 150 tons per pile at the remaining land piers. Pile material is structural carbon steel, ASTM designation A36 (Fy = 36 ksi), supplied by the United States Steel Corporation. Allowable design stresses were based on 14 ksi for the land piers/abutments and 9 ksi for the river piers (#26 to #29) per the original geotechnical site investigation report dated May 22, 1975, and a letter from the DOT Chief Materials Engineer dated July 2, 1975. Piles were installed using the WisDOT modified ENR criteria, which was the standard of practice at the time of construction.

The prestressed girder spans have between 10 and 14 piles supporting each footing. The steel girder spans have between 14 and 34 piles supporting each footing. The piers for the main span arch are each supported by 144 piles.

Two (2) pile load tests, 1E and 2W, were completed during the original design phase in January 1975, and the results are presented in Appendix D. Pile load test 1E was completed in proximity to Pier 13 at approximate Sta. 829+09; and Pile load test 2W was completed in proximity to Pier 41 at approximate Sta.880+13. The test piles consisted of HP14x73 pile driven to refusal in rock with a Kobe K 25 diesel hammer. The test piles were proof loaded to twice the maximum design load to a maximum test load of 344 ton; from which a maximum reported vertical displacement of 1.725" and a net settlement of 0.394" were measured at Test Pile 1E, and a maximum reported vertical displacement of 1.684" and permanent set of 0.486" was measured at Test Pile 2W. Based on the test pile program, recommendations were made to increase the foundation allowable design stress to 16,000 psi for steel piling installed at the approach span units, and maintain a foundation allowable design stress of 9,000 psi for steel piling installed at the river piers. At some point during the original design, the allowable design stress was reduced from 16 to 14 ksi to design the HP14x73 pile foundations.

1.5 Location and Noteworthy Physical Features

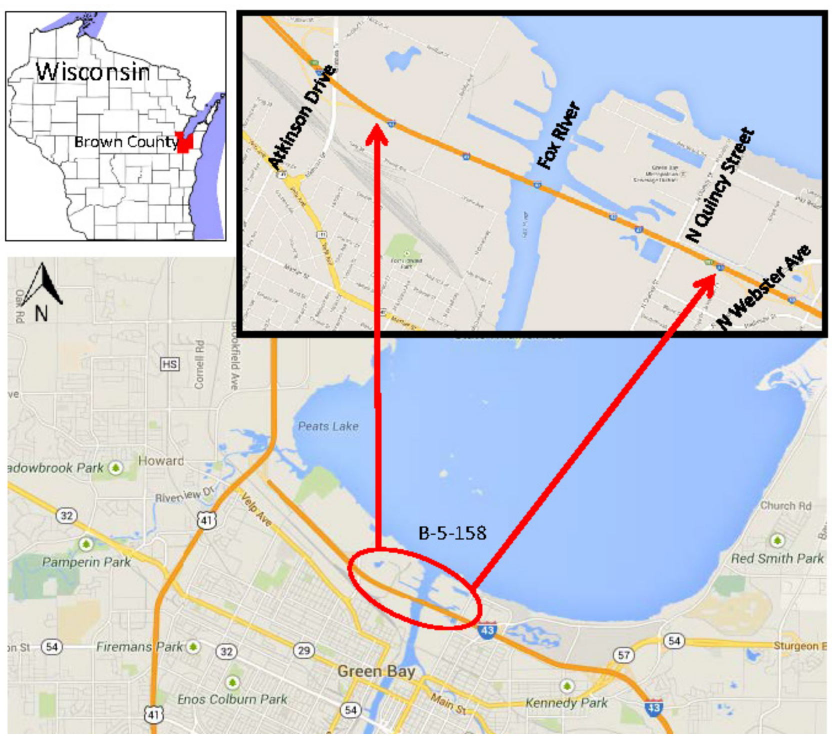


Figure 1.5: Location Map

The Leo Frigo Memorial Bridge carries I-43 over the Fox River near the south shoreline of Green Bay in the City of Green Bay, Brown County, Wisconsin. Figure 1.5 presents a site location map for this structure.

Historic Land Use Aerials (photographs) and Noteworthy Physical Features are presented in Exhibits 4 and 5, respectively. The outline of each of the bridge piers and abutments has been superimposed on the historic aerial imagery to provide points of reference in Exhibit 4.

Georgia Pacific owns and operates a landfill on the south side of the east approach between approximately Piers 8 and 24. The landfill location is depicted in Appendix A page A11 and Exhibit 5 Sections 3 through 5. The landfill includes three settling basins between about Piers 9 and 13. Based on the as-built plans, a portion of two of the settling basins was filled-in and abandoned under the bridge deck between about Piers 9 and 11, generally at the locations indicated in Exhibit 5 Section 5 and Appendix B pages B2, B5, and B6. Georgia Pacific has reported that: 1) the adjacent landfill consists of a 16-acre industrial waste disposal facility (WDNR License no. 02893), 2) the site began accepting (waste) material in 1980, and 3) to the best of Georgia Pacific's knowledge, the material placed in the landfill consists primarily of pre-existing fill material not unlike material located outside the site (in the surficial fill immediately adjacent to the landfill) and fiber-based wastewater treatment plant residuals from the paper mill. Available reports indicate that portions of the fill include chloride-containing waste from pulp and paper mills and that these fill materials probably affect local groundwater quality in the shallow zone.

Overhead power transmission lines cross over the bridge deck between Piers 12 and 13 and between Piers 32 and 33.

An underground fuel pipeline (West Shore Pipeline) with impressed direct current (DC) cathodic protection crosses under the bridge deck between Piers 22 and 23, as noted in Exhibit 5 Section 4.

The bridge site sits in a low-lying area. This area is flanked by a tributary along the north side of the bridge at the east approach.

Several depressions are filled with standing water above river pool elevation, which appear to be products of a perched water. A significant water-filled depression is located between Piers 19 and 20 at the east approach. At the west approach, water-filled depressions exist between Piers 30 and 50. Based on the as-built Construction Plans (Appendix B), the water-filled depressions at the west approach were constructed to provide wildlife habitat. These water-filled features are illustrated in Exhibit 5.

Large stockpiles of coal and salt are present along the north side of the west approach between about Piers 29 and 40. The edge of these stockpiles is within approximately 60 to 90 feet of the nearest bridge pier(s). A berm runs along the north side of the WisDOT right-of-way from Bylsby Street to the west side of the Fox River. This berm separates the piles from the WisDOT right-of-way.

Bridge scuppers discharge runoff by free-fall from the bridge deck to the ground surface immediately below. Four scuppers, located along each traffic barrier, are located approximately 10 feet uphill from each expansion joint. In the area of Pier 22, sets of scuppers are located south of Pier 21 and Pier 23. Runoff from these scuppers collects in drainage basins located below the structure. The scupper locations are illustrated in Exhibit 3 and in Exhibits 8 through 23. Bridge scupper drainage includes runoff laden with deicing agent, which is used to support snow removal operations in the winter.

1.6 Background and History

The results of a desktop literature study for Structure B-5-158 are presented in Appendix A. The desktop study revealed that the Georgia Pacific landfill was formerly owned by the James River Paper Company, Inc.; the landfill was situated in proximity to Piers 13 through 26; and landfill filling of the area started between 1938 and 1951, which covered much of the footprint of the bridge structure east of the Fox River. Historic land-use aerial mapping, which spans from 1938 to 1951, is presented in Exhibit 4. Available data suggests that the landfill content consists primarily of pre-existing fill, not unlike material located outside the (landfill) site and fiber-based wastewater treatment plant residuals from the adjacent paper mill.

As-built construction plans for the bridge substructures are presented in Appendix B. The as-built plans indicate that:

- East and West abutments, plus Piers 1 to 8, and Piers 45 to 51 were constructed under Substructure Project 1220-05-71 based on as-built plans dated July 1978
- Piers 9 through 25 were constructed under Substructure Project 1220-05-72 based on as-built plans dated May 19, 1980
- Piers 26 through 44 were constructed under Substructure Project 1220-05-73 based on as-built plans dated November 1981
- The superstructure from the east abutment through Span 18 was constructed under Superstructure Project 1220-05-74 based on as-built plans dated November, 1981
- The superstructure for Spans 19 through 27 was constructed under Superstructure Project 1220-05-76 based on as-built plans dated November 1981
- The main span arch was constructed under Superstructure Project 1220-05-77 based on as-built plans dated September 1981
- The superstructure for Spans 29 through 37 was constructed under Superstructure Project 1220-05-78 based on as-built plans dated November 1981
- The superstructure for Span 38 through the west abutment was constructed under Superstructure Project 1220-05-75 based on as-built plans dated May 1980
- An asphaltic concrete overlay was placed, modular joints were replaced, and pin and hangers were rehabilitated as part of bridge maintenance project 1220-15-71 in 2012-2013

Available data indicates that the coal stockpiles (at the north side of the west approach) pre-date the bridge construction. The large stockpiles of salt have been present for at least the past several years on adjacent property along the north side of the west approach.

2.0 Subsurface Conditions

2.1 Geologic Setting

The project site lies within the Fox River Lowland, near the confluence of the Fox River and Green Bay in Green Bay, Wisconsin. The area is underlain by Pleistocene glacial deposits and deeper dolostone bedrock (dolomite) from the Galena Formation Sinnipee Group Ordovician geologic period. Based on the Wisconsin Geological and Natural History Survey UW Extension Open-File Report 2011-02, the glacial deposits in the region generally consist of glacial till, outwash, and glaciolacustrine sediment.

Structural geology is dominated by the Niagara escarpment, which is located immediately to the east of the project site. Bedrock dips in an easterly direction as indicated in Figure 2.1.

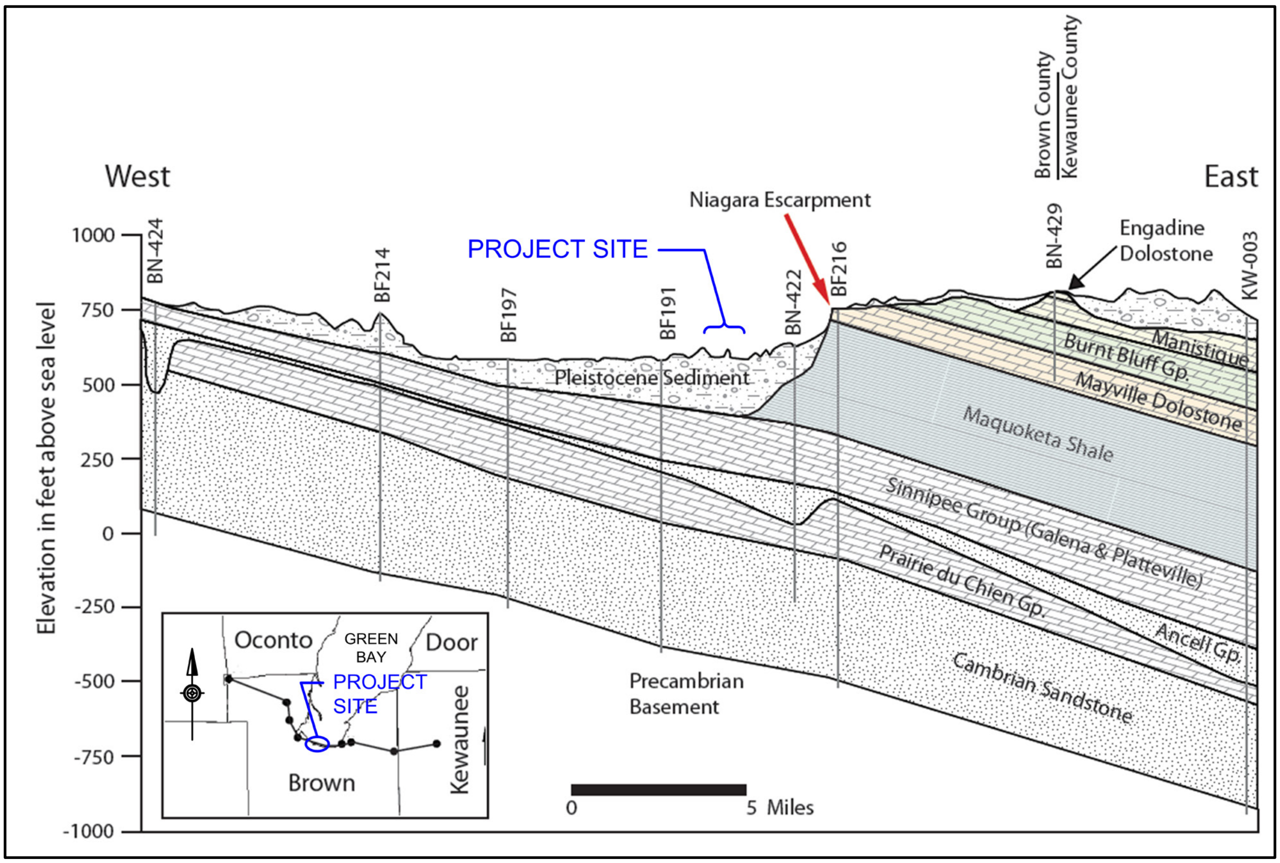


Figure 2.1: Geologic Profile (looking north)

2.2 Subsurface Profile

Subsurface conditions generally consist of fill underlain by alluvial soil and deeper till, all underlain by dolomite bedrock. A generalized subsurface profile and an enlarged profile of the surficial fill are presented in Figures 2.2 and 2.3, respectively on the following page. These figures are based on the original and recent borings, which correlate well.

Based on available data, top of rock varies between approximate Elevations 440 and 470 at the east approach spans, at approximate Elevation 465 at the main span over the Fox River, and between approximate Elevations 465 and 485 at the west approach spans.

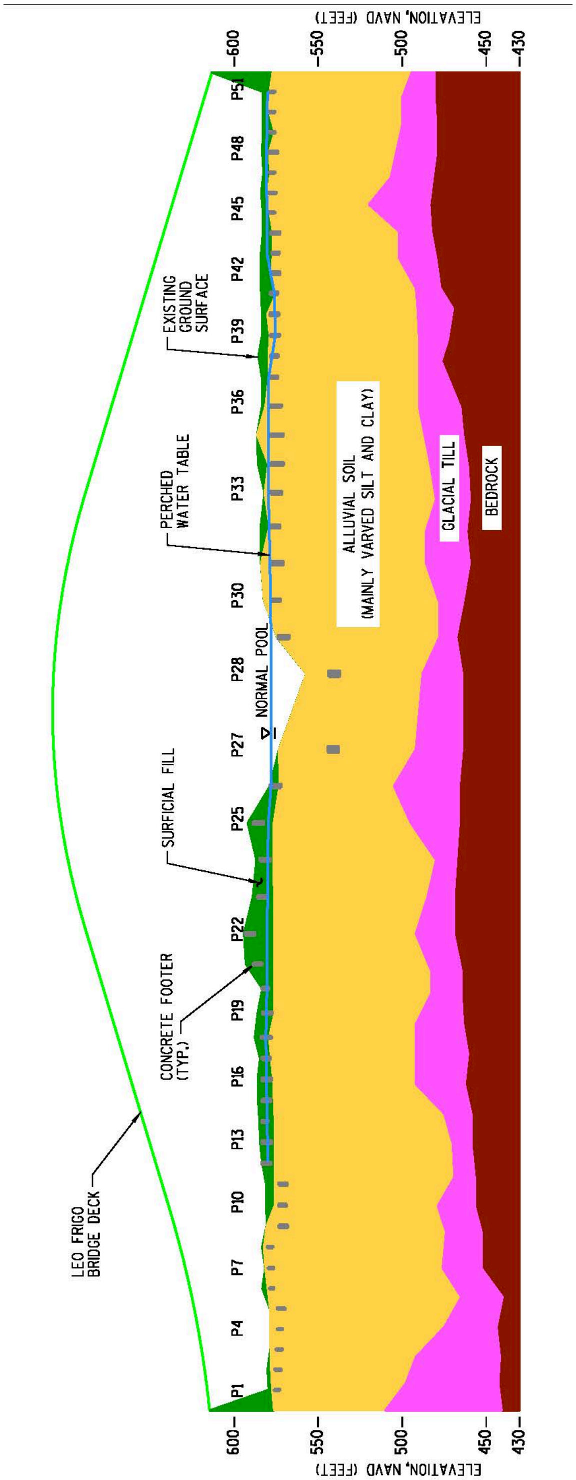


Figure 2.2: Generalized Subsurface Profile (looking south)

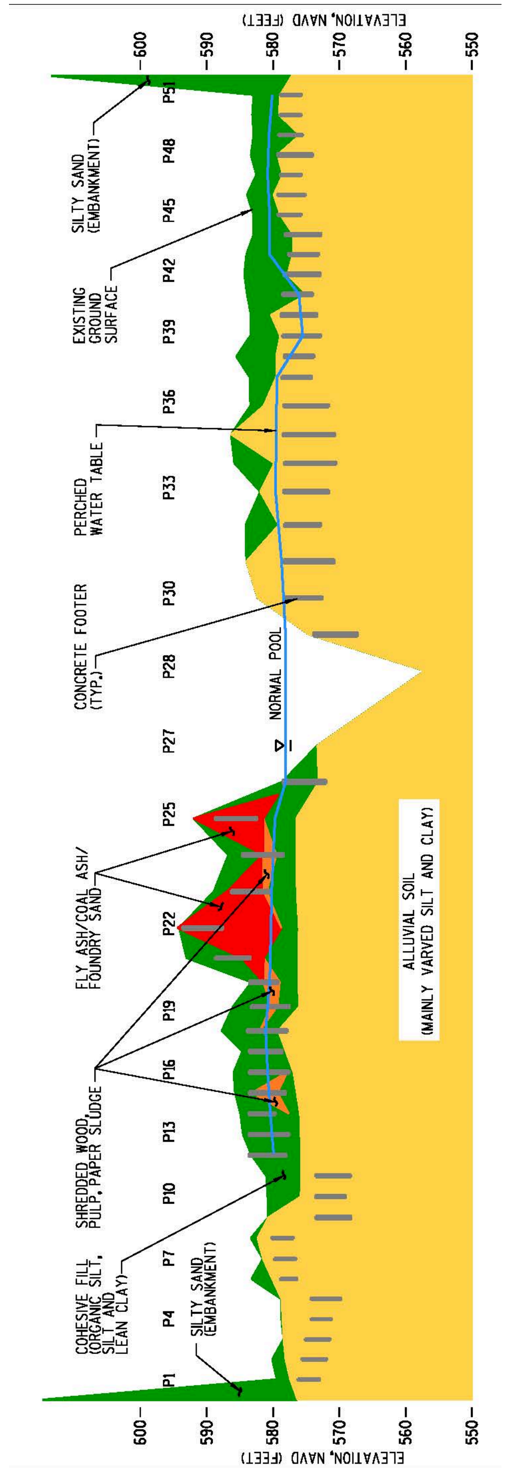


Figure 2.3: Enlarged Profile of Surficial Fill (looking south)

2.3 Fill Conditions

A surficial fill caps the majority of the project site and is composed of a variety of materials related to past land use. The base of the fill generally varies between approximate Elevations 573 and 586. A generalized profile of the surficial fill is presented in Figure 2.3 on the preceding page.

A significant portion of the fill is cohesive and generally consists of a soft to very stiff silt and lean clay.

At both abutments, the fill consists of medium dense to dense fine sand with variable amounts of silt.

At Piers 21 through 25, the fill is predominantly very loose to loose fly ash, coal ash, and foundry sand. The thickness of the fly ash, coal ash, and foundry sand stratum varies from approximately 0 feet around most of Pier 24 to a maximum of about 19 feet at Pier 22. At Piers 22 and 25, the fly ash is exposed at the ground surface.

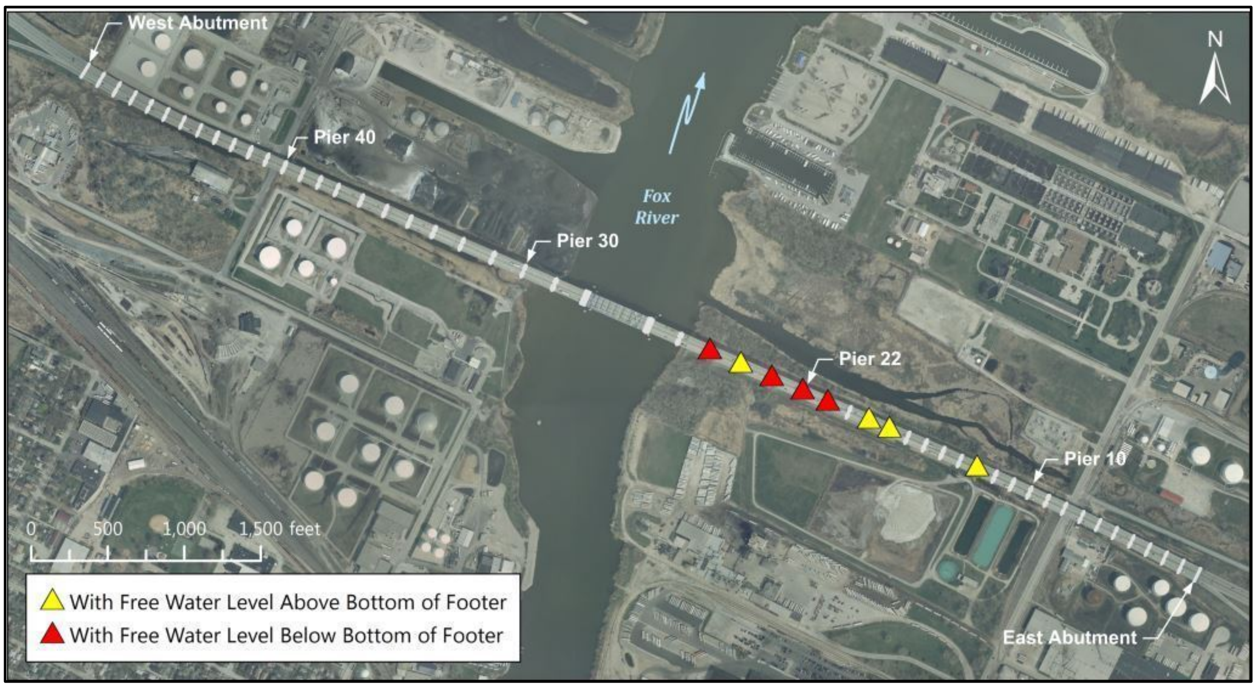


Figure 2.4: Plot of Fly Ash Locations Encountered

A discontinuous stratum of shredded wood and pulp was encountered between Piers 15 and 25. At Piers 15 and 16, a 4-foot stratum was encountered between approximate Elevations 483 and 479. At Piers 19, 20, and 21, a 1- to 3.5-foot-thick stratum of very loose shredded wood, wood pulp, and paper sludge was encountered above about Elevation 578.

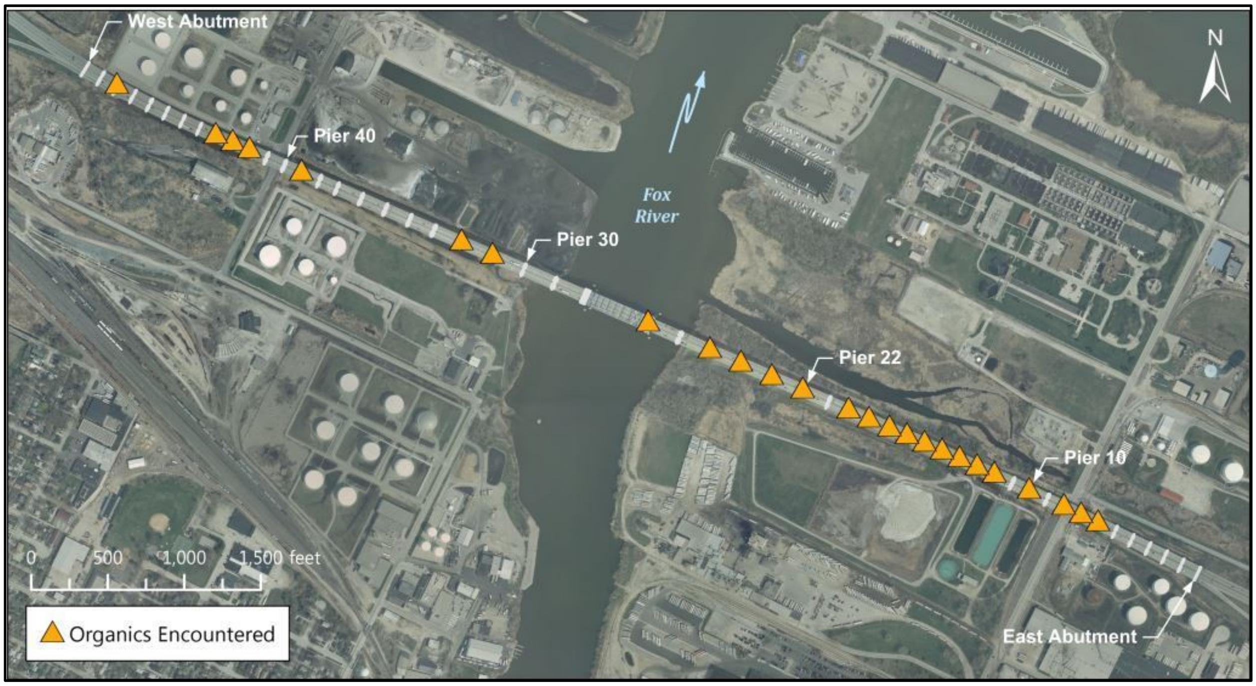


Figure 2.5: Plot of Organics Encountered

2.4 Alluvial Soil and Glacial Till Conditions

The fill is underlain by alluvial soil and deeper glacial till. These strata are shown in Figure 2.2.

The alluvial soil generally consists of a soft to medium stiff varved silt and clay, with an unconfined compressive strength generally ranging from 0.25 to 1.0 tons per square foot (TSF) and Standard Penetration Test (SPT) N-values generally ranging from 3 to 8 blows per foot. The alluvium is generally classified as a lean clay (CL) and silt (ML) according to the Unified Soil Classification System.

A discontinuous stratum of soft peat was encountered in the alluvium at Piers 2, 8, 18, and 48. The peat was generally 1 to 2 feet thick and was typically encountered between approximate Elevations 571 and 579.

The deeper till generally consists of hard silt and clay with discontinuous strata of very dense fine sand, based on the conditions encountered at the test borings. The thickness of the till encountered varies from about 10 to 50 feet, with an average of about 22 feet. The top of the till varies from approximate Elevations 465 to 520. N-values in the glacial till generally range from approximately 15 to in excess of 100 blows per foot.

2.5 Bedrock Conditions

The bedrock generally consists of a hard limestone. Rock cored at Pier 22 consists of a fine-grained medium bedded fossiliferous dolomite, with a core recovery of 100 percent, and a core Rock Quality Designation (RQD) ranging from about 83 to 90 percent. Based on historic borings that were drilled for the original bridge construction (Appendix D page D189), RQDs ranged from about 30 to 92 with almost all above 70, and core recoveries ranged from about 80 to 100 percent. A subsurface profile, which denotes where rock was cored, is presented in Exhibit 6 and in Appendix B pages B14 through B20.

A total of seven core samples were transported to the WisDOT and Pace laboratories and tested for unconfined compressive strength (UCS), which ranged from 10,110 to 35,330 psi with an average of about 18,700 psi. Photographs of the rock cored at Pier 22 are presented in Appendix L page L9.

2.6 Ground Water Conditions

The regional ground water table is controlled by pool fluctuations in the Fox River and Green Bay. Based on 33 years of NOAA data (1980 to 2013) obtained for Tide Station 9087079, which is located in the project area near the confluence of the Fox River with Green Bay, the mean daily river pool level has been in decline over about the past three decades, as indicated in Figure 2.6. Based on the NOAA data, the mean daily pool for Fox River near the project site varied from a high of Elevation 583.7 (NAVD) to a low of Elevation 576.0 (NAVD) over 33 years, with an average of Elevation 579.6 (NAVD).

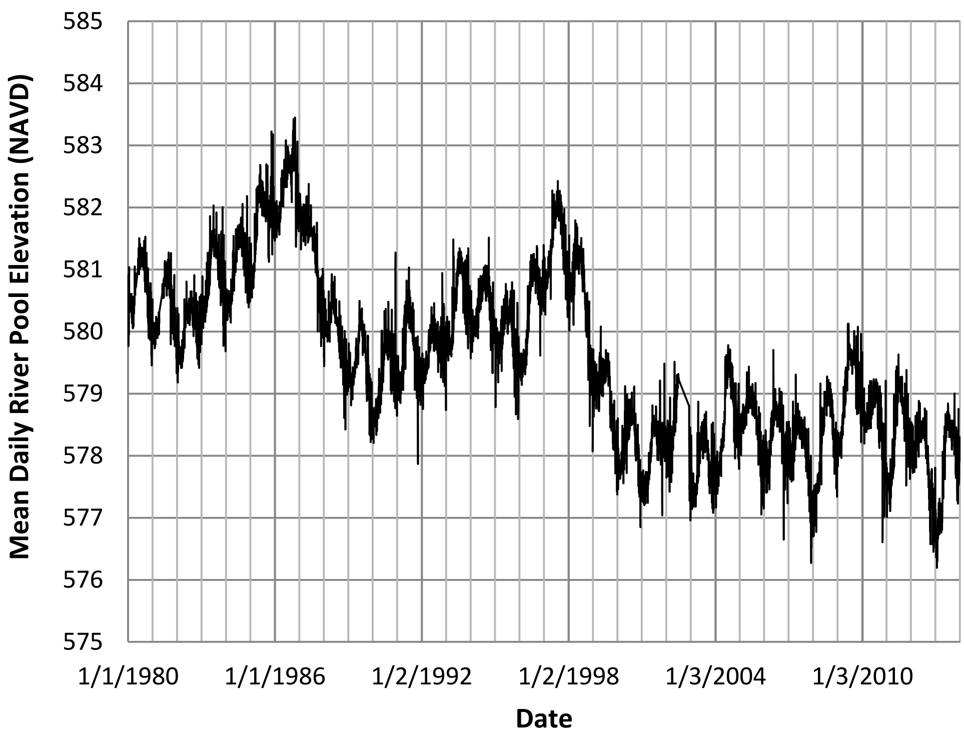


Figure 2.6: Mean Daily River Pool Elevation (NAVD 88) Between 1980 and 2013

Perched water was encountered at both the east and west bridge approaches. River pool was at approximate Elevation 578.3 on October 18, 2013 and generally varied by a few tenths of a foot over the duration of the field investigation in late-September and October 2013. Ponded surface water was noted at a slightly higher elevation at the locations identified in Exhibit 5. The water at the land piers is generally believed to be perched based on the subsurface conditions encountered at the test pits and test borings and the characteristic discharge of free water from a granular base at the bottom of concrete footer(s) where test pits were dug. The origin of the perched water is believed to be primarily a combination of surface runoff and bridge deck drainage from the scuppers overhead. The elevation of the perched water is illustrated graphically in Exhibit 8. The elevation of the perched water with respect to the bottom of concrete footer is noted in Table 4 and shown for Piers 13 through 26 in Figure 2.7

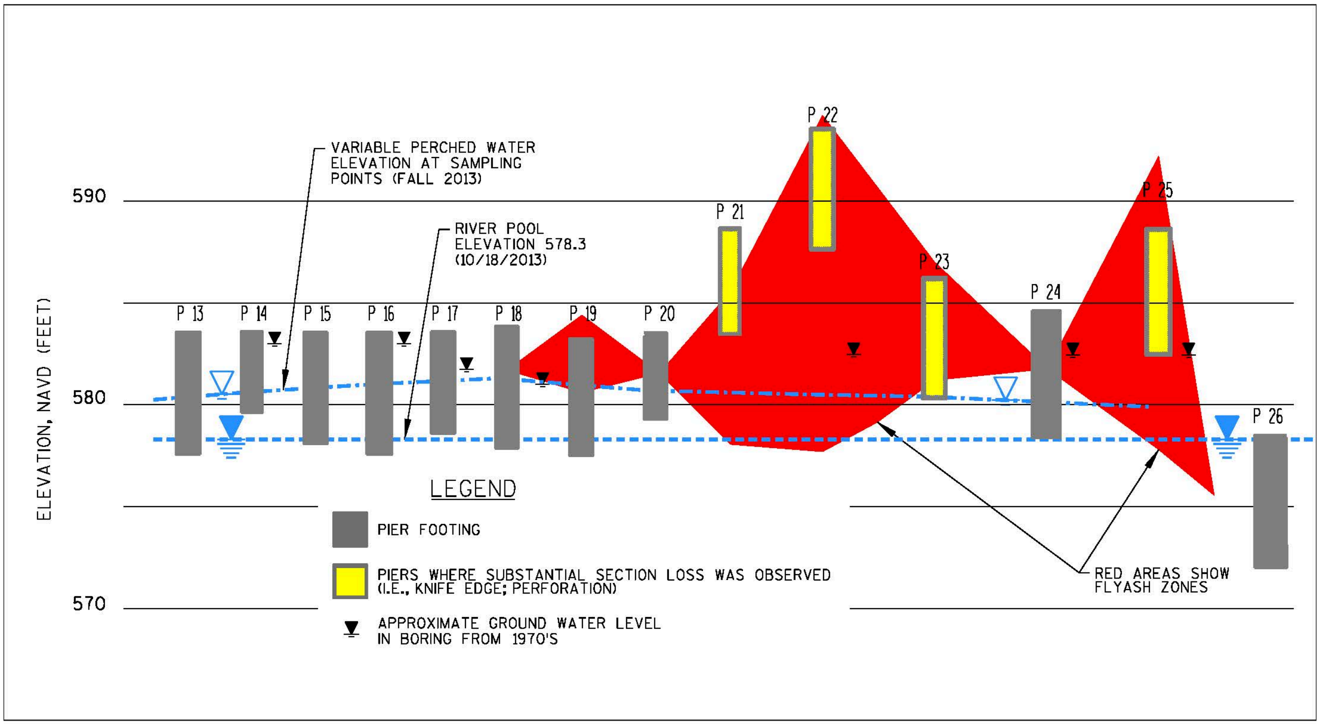


Figure 2.7: Ground Water and Footing Elevations

The elevation of the regional ground water table (river pool) and the perched water are both significant. Available data indicates that the river pool can fluctuate on the order of 3 feet annually and that the river pool elevation has varied by about 7 to 8 feet over the past 30 years. The perched water elevation is expected to also vary seasonally and is dependent generally on the magnitude of rainfall and snowmelt.

It is worth noting that the perched water at Pier 22 was reaffirmed at test pits dug in July 2014. The perched water at Pier 22 was at elevation 583.3 (NAVD 88) on September 29, 2013 at Boring B22A, and was at elevation 584.1 (NAVD 88) on July 23, 2014 at Test Pit TP-22SSE (about a 0.8 foot elevation difference, which is well within what would be expected for seasonal change).

3.0 Data Obtained

3.1 2013 Site Reconnaissance

On Thursday, September 26, 2013, WisDOT conducted an initial site reconnaissance. On Friday, September 27, Baker conducted an initial site reconnaissance and met with WisDOT representatives to obtain first-hand observations of visible distress, obtain relevant information, review available plans and construction records, develop an exploration plan to commence investigation of existing foundation conditions, and develop an approach to arrest and mitigate foundation movement.

3.2 2013 Exploration Plan

The initial exploration and investigation plan was developed after the site visit on Friday September 27. As safety was the top priority throughout the investigation, a “Red Zone” was established around Pier 22 where activity was restricted. The Red Zone was initially established between Piers 20 and 24. One of the first goals was to determine if there was any continuing movement of Pier 22 and to determine how stable the structure was in its current condition. Several actions were initiated to determine the answers to these questions.

- Accelerometers and tilt meters were installed on Piers 21, 22, and 23. Audible alarms would sound if any abrupt movement occurred
- Survey elevations were taken twice a day on Pier 22 and adjacent piers to monitor the magnitude and rate amount of vertical movement; robotic survey equipment was installed for real time monitoring of elevations
- Field survey was completed to ascertain the bridge deck profile; comparison with available data indicated that the vertical substructure displacement was isolated to Pier 22
- Inspection of Units 6, 7, and 8 was completed to determine if there was any visible damage to the structural steel, piers, and deck resulting from the vertical displacement; discussions of the results of the bridge inspection are included in the following sections and in Appendix F
- Structural analysis was completed to determine the stresses on the structure resulting from the vertical displacement; when the stress levels were found to be acceptably within the elastic range, further analysis was completed to determine how much additional vertical displacement would result in permanent deformation

Once the structure was determined to be in a stable and safe condition, the Red Zone was opened for additional investigation.

While the above items were being investigated, a geotechnical exploration plan was initiated. Starting on September 28, 2013, test borings were drilled at Piers 16, 18, 19, 20, and 22, and test pits were dug at Piers 11, 13, 15, 17, and 19. The test borings were drilled to characterize the type and thicknesses of surficial fill encountered. Water sampling points were installed to develop a detailed understanding of the perched water and to establish the relative position of the perched water with respect to the footing elevations at each of the piers. The water sampling points were used to collect water samples for subsequent chemical analyses in the laboratory. The locations of the test pits and borings are shown in Exhibit 6. The borings typically alternated between the north and south sides of the piers. Test pits were generally excavated on the south side of the piers. Test pits on the north pier footings were excavated in areas to collect additional information at and around pier 22.

Figure 3.1: Plan Location of Ground Water and River Pool Sampling Points

The test pits were used to expose representative piling and to characterize the type and condition of fill in direct contact with the piling. Representative soil samples were collected from the test pits and tested in the laboratory to classify the soil and determine soil moisture content, specific gravity, organic content, and soil resistivity. Soil and water chemical analyses were performed to determine pH; specific conductance; bromide, chloride, phosphorous, and sulfate concentrations; and total acidity, alkalinity, and iron concentrations. Chemical analyses were also performed to determine fecal coliform and fluoride content to assist with future determination of possible sources of contaminant. An impulse echo test was conducted on the exposed piling at Pier 22 to identify potential anomalies (artifacts) in the tested piles.

Arrangements were initiated to drill a series of deep test borings to determine the type and condition of soil overburden, determine the type and condition of bedrock, and install inclinometer casing for subsequent geophysical testing, including 3D crosshole tomography and electrical resistivity surveys around Pier 22. Crosshole seismic tomography involves subsurface seismic imaging that can delineate anomalies, including the presence of piling between a series of boreholes. Electrical resistivity surveys involve placement of a sensor at various depths in a borehole that is used to determine where the steel piling does or does not exist (i.e., detect the depth of the existing piling pile tip).

The exploration plan was updated periodically during the course of the investigation based on concurrent interpretation of data obtained both in the field and in the laboratory as it became available. Additional testing was included, such as metallurgical tests on a scrap piece of HP14x73 piling that was recovered at Pier 15 from Test Pit 15S, to determine chemical composition and strength.

The exploration plan was also expanded to conduct a detailed study of the corrosive characteristics of the surficial fill at representative piers. Half-cell potential measurements were obtained, piling was exposed and measured to assess extent of corrosion loss, and representative samples of soil and corrosion product were collected for subsequent laboratory testing. Water samples were also collected and analyzed.

The exploration plan was also expanded to include field tests to measure possible stray current induced corrosion at Pier 22 from both high voltage AC transmission lines and the cathodically protected underground pipeline. The laboratory investigation was expanded to include microbial analyses of soil samples for both aerobic and anaerobic bacteria strains and scanning electron microscopy (SEM) and ancillary energy dispersive spectroscopy (EDX) on representative samples of corrosion product to refine the understanding of the type and extent of corrosion mechanism at the project site.

Additional investigation included coring representative samples of the existing concrete footers at Piers 21 and 25 for petrographic examination to assess the quality of the existing concrete footers and determine the extent of chloride penetration. Representative footing cores were obtained from the south side of the south column at Piers 21 and 25.

The limits of exploration also expanded throughout the course of the investigation. Eventually, test pits and soil borings were conducted at select piers throughout the length of the structure.

When it became evident that corrosion near the perched water was the likely cause of the vertical displacement, additional deep exploration and geophysical testing activities were suspended.

3.3 2013 Bridge Inspection

An inspection of the underside of the bridge was performed to assess any potential superstructure damage due to vertical displacement of Pier 22. The inspection was conducted from a man-lift and from existing pier catwalks. The inspection encompassed Unit 7, along with the adjacent Units 6 and 8 (between Piers 18 and 26). Results for the bridge inspection are presented in Appendix F. Except as noted in Appendix F, there was no apparent sign of unusual distress due to pier vertical displacement, such as bearing rotation, plastic member deformation, or paint cracking. The bridge inspection did note the following items that were not noted on previous inspection reports:

Pier 22

a) One horizontal angle of a vertical cross-brace that was slightly buckled by 3/8-inch (see Figure 3.2)
b) Very minor spalling at the top flange/bottom of the deck interface at girders 1 and 2

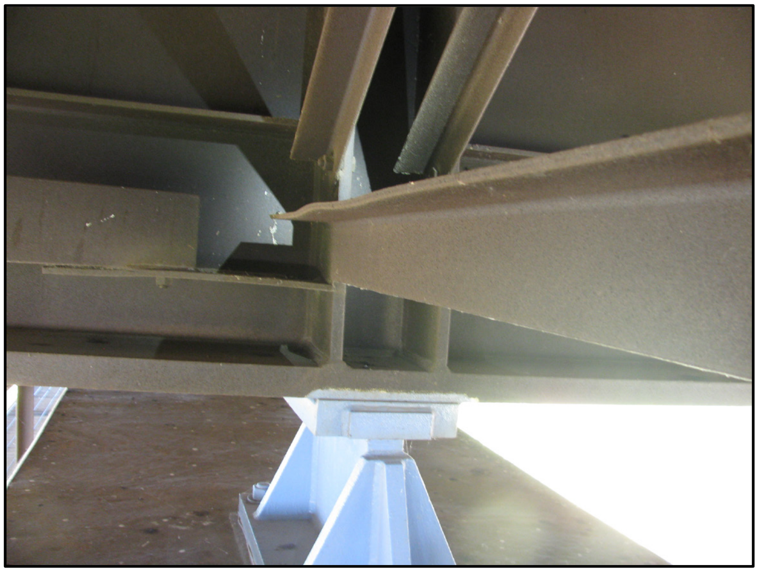


Figure 3.2: Buckled Angle at Vertical Bracing between Girders 6 & 7

Pin and hangers in Span 21

c) The vertical hangar plates were rotated approximately 3.5 degrees (see Figure 3.3)

d) The bottom of the wind lock plates were not in contact with the top of the bottom flange of the adjacent girder with an average gap of approximately 5/8-inch (see Figure 3.4)

Figure 3.3: Span 21 Pin & Hanger at Girder 8 (looking north)

Figure 3.4: Span 21 Wind Lock Plates at Girder 9

Pin and hangers in Span 23

e) The vertical hanger plates were vertical and flat (see Figure 3.5)

f) The wind lock plates were in contact with the top of the bottom flange of the adjacent girder at the end of the adjacent girder. However, at the end of the wind lock plate itself, there was an average gap of approximately 3/16-inch (see Figure 3.6)

Figure 3.5: Span 23 Pin & Hanger at Girder 2

Figure 3.6: Span 23 Pin & Hanger Wind Lock Plates at Girder 6

In general, the underside of the bridge was found to be in good condition despite the considerable sag in the superstructure due to the settled pier. There are no immediate concerns regarding the safety or serviceability of the superstructure. Detailed results for the bridge inspection are presented in Appendix F.

A bridge inspection of the deck surface of the bridge was performed to complete a damage inspection to assess the effects that the superstructure had experienced due to vertical displacement of Pier 22. A visual inspection was performed on Friday September 27, 2013. There was no apparent sign of unusual distress due to pier vertical displacement, such as cracking of the deck overlay or parapets or severe misalignment of the joints.

Prior to jacking the structure, a Ground Penetrating Radar (GPR) scan was completed on November 11, 2013 on Unit 7 of the bridge deck (between Piers 21 and 23). The asphalt overlay, top mat of rebar, and bottom deck are all in good condition throughout. There were no signs of damage in the raw data for this deck, and very little was revealed through the analysis. The full GPR testing report is presented in Appendix P.

After the superstructure was jacked back into place, echo testing was completed on January 3, 2014, on Unit 7 of the bridge deck (between Piers 21 and 23). Of the 120 tests covered under this evaluation, three were found to contain evidence of overlay debonding. The full echo testing report is presented in Appendix Q. Based on the results of the GPR and echo testing of the deck, no immediate action is recommended for the overlay. Bridge Maintenance will continue to monitor the few areas of minor concern.

While the structure was closed to traffic, an in-depth bridge inspection of the entire structure was completed by WisDOT NE Region Bridge Maintenance staff, and inspection of the main-span cables was completed by Wiss, Janey, Elstner Associates, Inc. on November 25 and 26, 2013.

3.4 2013 Subsurface Investigation

Based on available data, an exploration program was crafted at a meeting held on September 27, 2013 to conduct an initial investigation at the east approach from Piers 10 through 25. The initial investigation was later expanded to include representative sampling at the remaining piers and bridge abutments. Results presented herein are based on the expanded investigation. A summary of subsurface explorations is presented in Table 1.

Equipment was mobilized, and subsurface exploration commenced on Saturday, September 28, 2013. Two drill rigs were utilized, one owned and operated by WisDOT and the other by GESTRA. A backhoe and bulldozer with operators were provided by the Brown County Highway Department.

A subsurface plan and profile is presented in Exhibit 6. An enhanced subsurface plan and profile, with expanded vertical scale, is presented in Exhibit 7, which highlights the type and depth of surficial fill encountered. Test pit logs, with representative photographs of piling that was exposed, are presented in Appendix G. Test boring logs are presented in Appendix H. A generalized subsurface profile and a generalized profile of the surficial fill are presented in Figures 2.2 and 2.3, respectively, to aid with discussion of key points presented herein.

Soil and ground water samples were collected from the test pits and test borings and transported to the laboratory for testing and soil and water chemical analyses.

At Pier 22 Test Pit 22S, an impulse echo test was performed on two H-piles, #15 and #14. Results from Pile #15 were inconclusive. It was hoped that the impulse echo test results would corroborate the historic pile tip elevations and identify potential anomalies over the length of pile. Results from Pile #14 were initially concluded to show a reflection at a depth of approximately 95 feet (roughly Elevation 491, at a height of approximately 11 feet above the as-built pile tip elevation reported in Appendix D page D93). However, they were later concluded to indicate that measured anomalies were more likely to be artifacts generated by multiple reflections from the superstructure above the piles. Detailed results are presented in Appendix I.

In-situ resistivity tests at multiple depths were performed at Piers 5, 13, 19, 24, 25, 32, 39, and 43 and reported in Table 4. Test pits were opened in a progressive manner such that resistivity measurements could be recorded in order to identify abrupt changes in resistivity versus depth. Generally, resistivity tests were performed at the ground surface prior to beginning test pit excavation, at the elevation of the top of the concrete footer, at the elevation of the bottom of the concrete footer, and at the bottom of the test pit excavation (typically 4 to 6 feet below the bottom of the footer). Measurements were recorded with an electrode spacing of 4 feet at a distance of approximately 5.5 feet from the face of the pier footer being exposed. This electrode spacing was selected to accommodate resistivity testing with respect to a reasonable excavation size at its deepest point. Additional measurements were recorded with a larger spacing at the surface approximately 20 feet from the face of the pier footer being exposed.

Half-cell potential measurements were performed at Piers 5, 13, 19, 24, 25, 32, 39, and 43 with a copper sulfate electrode using a hand-held voltmeter and are tabulated below at the end of this paragraph, as well as in Table 4 and Appendix R. Testing was conducted with potentials being recorded between exposed steel piles and the soil immediately adjacent to the pile. Measurements were recorded at 6-inch intervals along the length of exposed piles. Typically, half-cell measurements were recorded over a depth of 3 to 5 feet of the exposed pile. The results of these tests are highly negative potentials. While a specific value associated with likely corrosion is difficult to determine based on the number of soil strata on site, a large range in recorded potential values is an indication of the occurrence of a macro cell, which is required for corrosion to occur between the anode and the cathode. At Pier 25, a range of approximately 280 mV was recorded along the length of a single pile and over a distance of 18 inches. This large range in all likelihood indicates aggressive macro cell corrosion.

Maximum Difference in Half-Cell Potentials Measured at Test Pits

Pier	Test Pit	Maximum Range (mV)*
5	TP-5NE	9
13	TP-13NE	37
19	TP-19NE	30
24	TP-24NNW	56
25	TP-25NE	282
32	TP-32NE	31
39	TP-39SE	41
39	TP-39NW	18
43	TP-43NE	17

* Potential range measured over a 2 to 3 foot length of pile.

Multiple exposed piles exhibited visual signs of corrosion, but half-cell potential results were relatively constant over the tested pile height, masking areas where localized corrosion was occurring. The potential magnitudes at these locations were indicative of a high risk of corrosion; however, the small shifts in recorded values indicate a microcell-type phenomenon is occurring. The corrosion occurring at these piles is less detectable by potential measurements, but nonetheless, the corrosion has in some cases resulted in deep pitting and some loss of cross-sectional area. In these locations, the localized corrosion observed, coupled with largely negative potential values led to an evaluation of microbially influenced corrosion (MIC).

Figure 3.8 shows the results of visual ratings on average pile section loss observed. A visual rating range of 1 (poor condition) to 10 (good condition) were assigned based on the condition of pile exposed. Typical photographs for the condition of piling with respect the visual rating assigned is provided in Figure 3.7 below. Table 5 present detailed rating results. In general, a visual rating of 1 was assigned to the exposed piling at Pier 22 where section loss was visually estimated to exceed 40%; visual ratings of 2 to 4 were associated to piling where the average section loss was estimated to be approximately 20 to 30%; visual ratings of 5 to 6 were associated to piling where the average section loss was estimated to be approximately 5 to 10%; and visual ratings of 7 to 10 were associated to piling where the average section loss was estimated to be approximately 0 to 5%.

Pier 25 South Column Pile 17
(Visual Condition Rating 2)

Pier 39 North Column Pile 20
(Visual Condition Rating 6)

Pier 13 North Column Pile 33
(Visual Condition Rating 8)

Pier 5 North Column Pile 10
(Visual Condition Rating 10)

Figure 3.7: Photographs of Typical Corrosion on Exposed Piles

Tests were also performed to assess the potential for stray current induced corrosion at Pier 22, from both high voltage AC transmission lines and the cathodically protected underground pipeline. These test results are documented in Appendix N. Test leads, which were attached to an exposed pile at the south column at Pier 22, were used to conduct these tests. Results from the stray current monitoring tests indicate no recorded influence on the monitored piling at Pier 22 from the cathodically protected pipeline or the overhead transmission lines at the time of testing.

Select piles were exposed at test pits, and representative samples of corrosion product were collected for subsequent laboratory testing, including petrographic and electron microscopic examination. The number, location, and visual rating of the pile condition are summarized in Table 4 and shown in Figure 3.8.

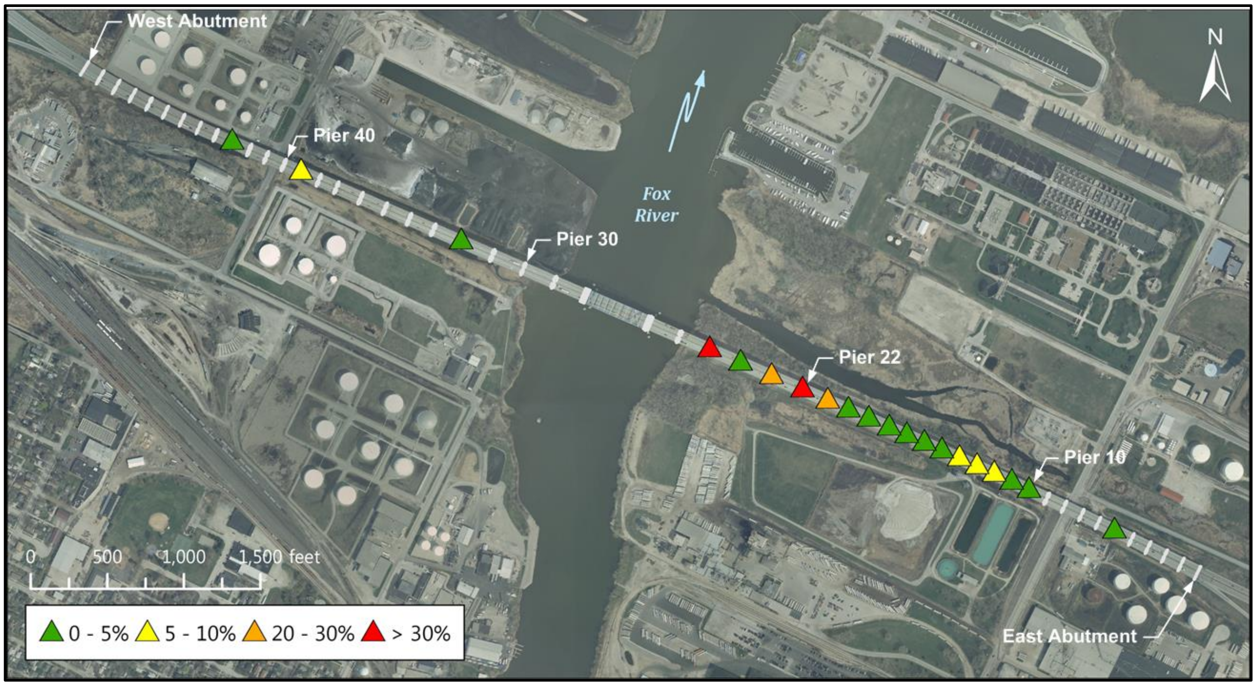


Figure 3.8: Plot of Average Section Loss Observed Over Exposed Pile Length (Visually Rated)

3.5 2013 Laboratory Investigation

Laboratory tests were completed on soil samples to classify the type of soil encountered. Chemical analyses were completed on both ground water and soil samples. At the land piers, laboratory testing was completed by both OMNNI and Pace. At the river piers, laboratory testing was completed by both RVT and Pace.

The laboratory investigation generally entailed physical testing to classify the soil and fill encountered, chemical analyses of both soil and water samples, unconfined compressive strength tests for both rock and concrete core samples, metallurgical tests on coupon samples taken from a representative steel pile cutoff-remnant that was recovered from the test pits, microbial diagnostic testing to determine the presence and type of biologic microbes, petrographic examination of representative samples of the existing concrete footings and corrosion product, and quality control validation testing.

3.5.1 Soil Classification

Laboratory tests completed to classify representative soil samples generally consisted of moisture content determination, sieve and hydrometer analyses, and Atterberg limits. In addition, soil samples were tested in the laboratory to determine soil resistivity and total organic content. A tabular summary of soil classification, soil resistivity, and chemical analyses for test pit and test boring samples is presented in Table 2. Soil types generally ranging from sand to silt to lean clay, including organic silt and ash were classified. Of particular interest was the types of soil classified versus soil resistivity and liquid limit ratios (without sample drying versus with sample drying) along with specific gravities to assist with classification of organic silts encountered.

3.5.2 Chemical Analyses

Chemical analyses were performed on representative soil samples in the laboratory to determine pH; specific conductance; and concentrations of sulfate, chloride, bromide, fluoride, and phosphorus.

Chemical analyses were performed on representative water samples in the laboratory to determine pH; alkalinity; total acidity; and concentrations of chloride, sulfate, iron, bromide, fluoride, and phosphorus. In addition, representative water samples were tested to determine specific conductance.

The following tabular values are provided for comparison of the laboratory test results with chemical values that would typically be expected for the project site.

Parameter	Test Method	Typical Values*	Range of Laboratory Test Results
Soil pH	ASTM G 51	5.5 to 8.5	7.0 to 9.1
Water pH	EPA 9040	≥ 5.5	5.4 to 8.0
Soil Resistivity	ASTM G 57	> 2,000 ohm-cm	40 to 5,700 ohm-cm
Soil Chloride Concentration	AASHTO T 291	< 500 mg/kg	55 to 11,400 mg/kg
Soil Sulfate Concentration	AASHTO T 290	< 1,000 mg/kg	< 27 to 21,800 mg/kg
Water Specific Conductance	EPA 120.1	< 500 umhos/cm	980 to 18,150 umhos/cm
Water Chloride Concentration	EPA 300.0	< 500 mg/L	56 to 27,100 mg/L
Water Sulfate Concentration	EPA 300.0	< 500 mg/L	7 to 2,140 mg/L

* Reference: AASHTO LRFD Bridge Design Specifications, 6^{th} edition including the 2013 interim update.

Of particular note was the high sulfate concentration of 21,800 mg/L, which was collected at the buckled piling at Pier 22, north column. A retest was conducted in the laboratory, which confirmed the sulfate concentration of 21,800 mg/L. High sulfide concentrations may be indicative of microbiological activity at the site, in addition to the presence of an ash-derivative from burning low-grade coal.

Also noteworthy were the high chloride concentrations near the piles on the north side of the west approach. The high chloride concentration is of particular interest with respect to potential corrosivity of the subsurface conditions at the project site. These results are graphically depicted in Exhibits 10 and 13.

A tabular summary of field pH, field conductivity, and chemical analyses conducted on water samples at ground water monitoring points is presented in Table 3. Detailed results for laboratory tests completed for samples collected at land piers and rivers piers are presented in Appendices J and K, respectively. Detailed graphic plots for ground water elevations, minimum soil resistivity, soil chloride, soil sulfate, ground water pH, maximum water chloride, maximum water sulfate, and maximum water alkalinity are presented in Exhibits 8 through 15, respectively; and graphic summary plots for the same are presented below in Figures 3.9 through 3.15.

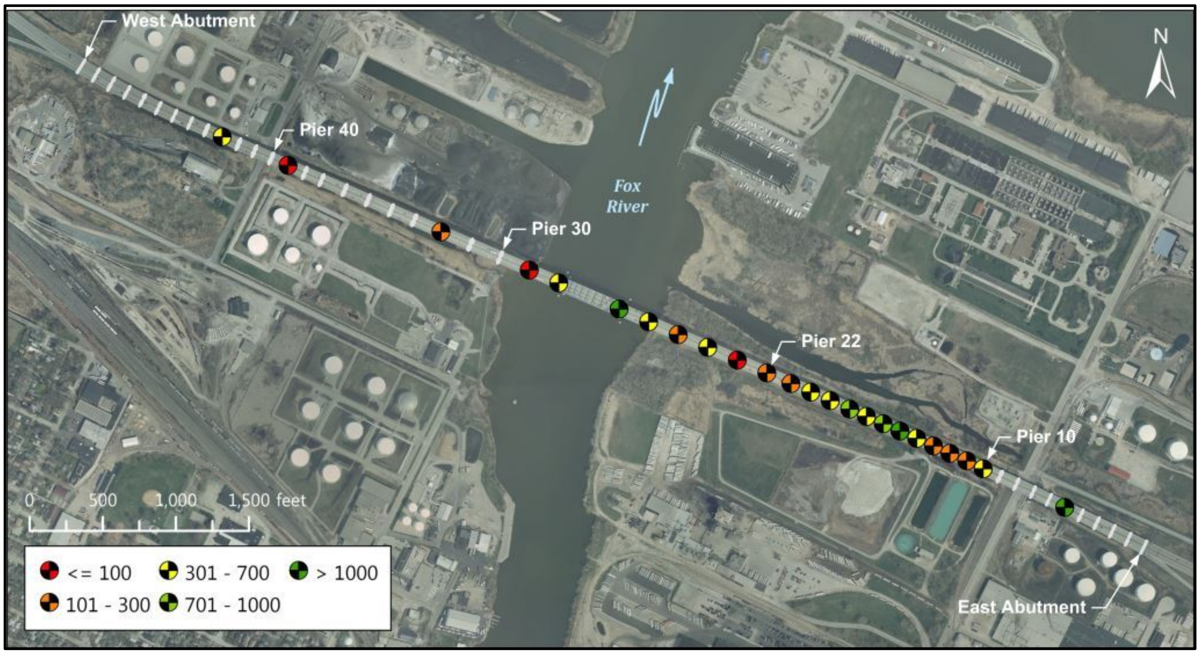


Figure 3.9: Plot of Minimum Laboratory Soil Resistivity Measurements (ohms-cm)

The soil with the lowest minimum soil resistivity was sampled at Pier 23 on the east side of the Fox River, and at Piers 29, 32 and 39 on the west side of the Fox River (between Bylsby Avenue and the Fox River).

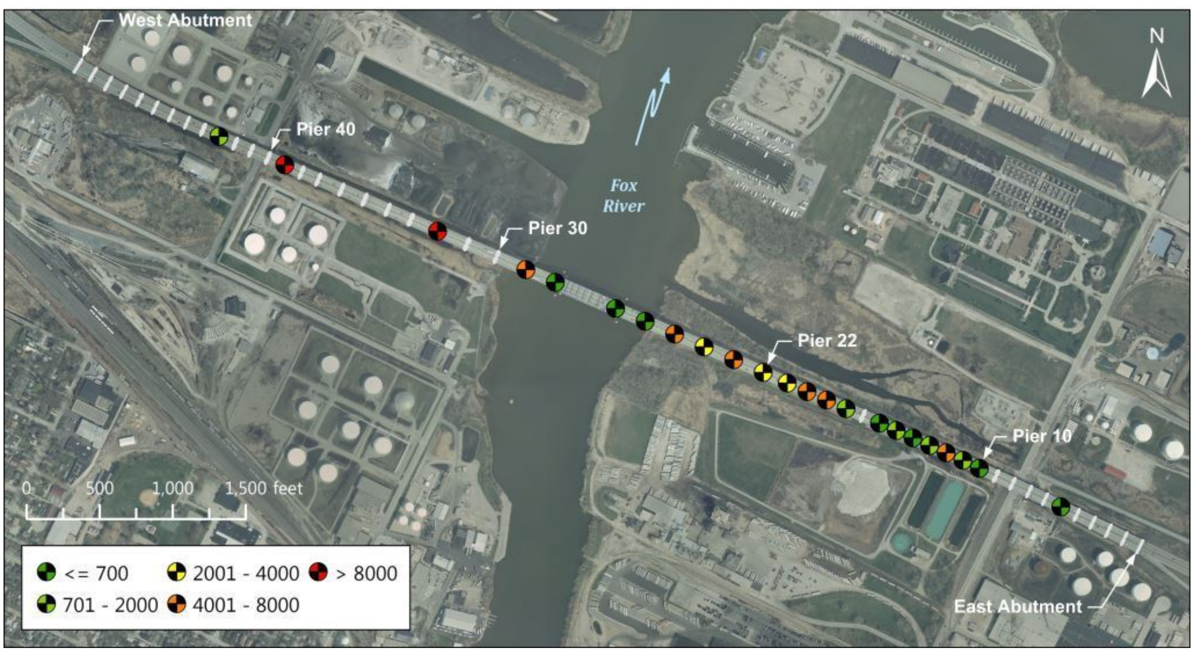


Figure 3.10: Plot of Maximum Soil Chloride Concentrations (mg/kg)

East of the Fox River, the highest soil chloride concentration was at Piers 19, 20 and 23. West of the Fox River, the highest soil chloride concentration was at Piers 29, 32 and 39.

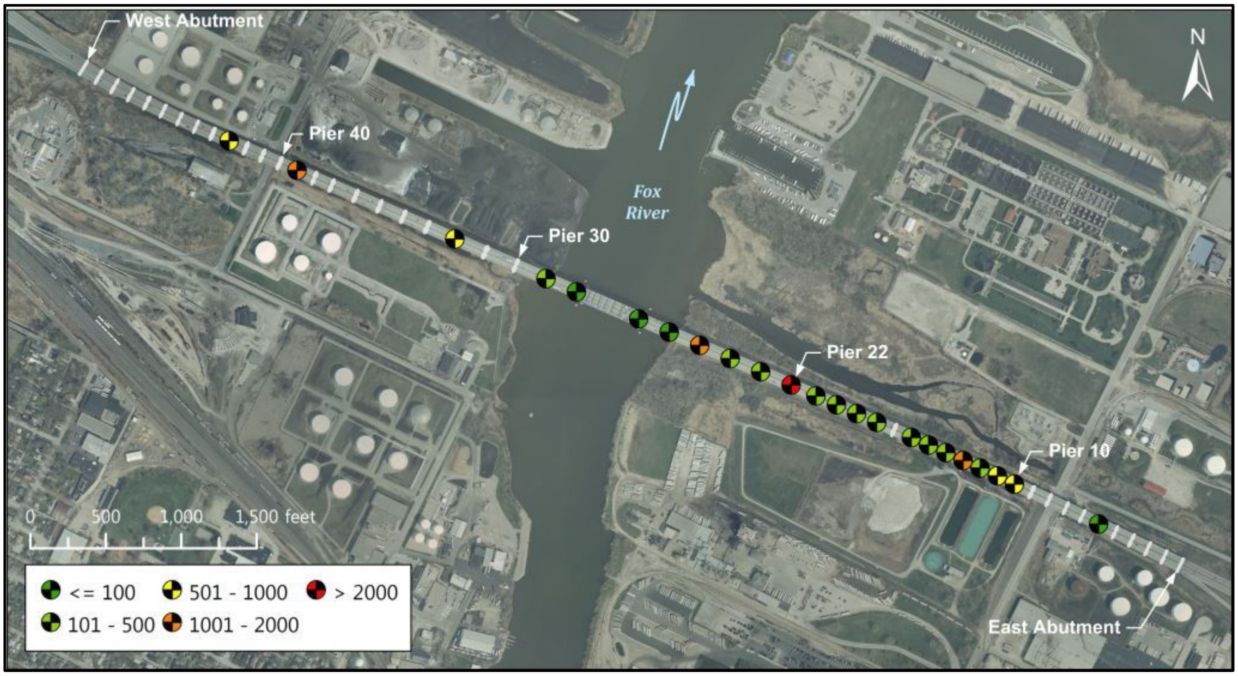


Figure 3.11: Plot of Maximum Soil Sulfate Concentrations (mg/kg)

The highest soil sulfate concentration was measured at Pier 22.

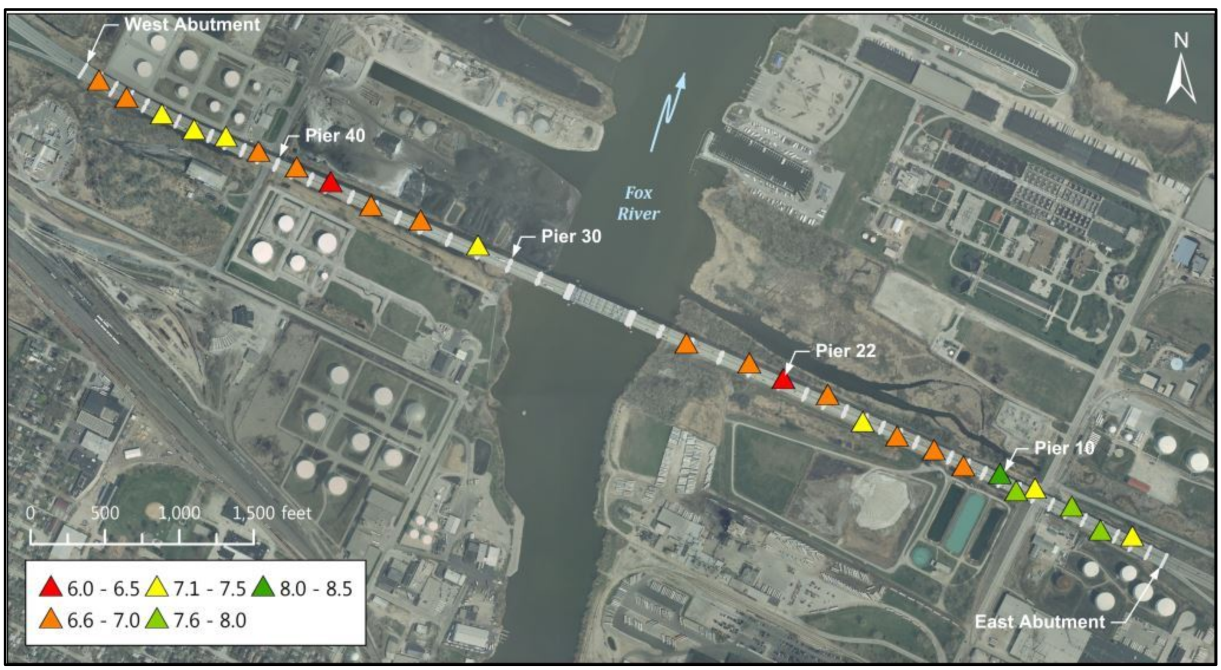


Figure 3.12: Plot of Ground Water pH at Sampling Points

Ground water pH did vary across the project site, but was neither excessively acidic nor alkaline.

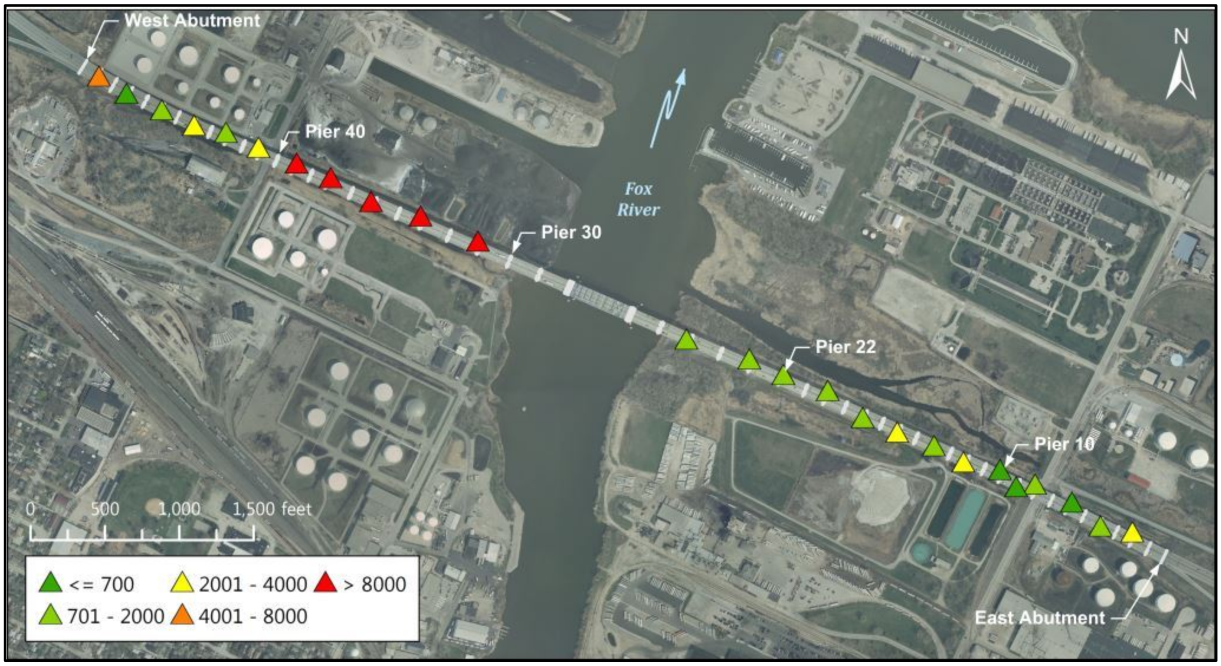


Figure 3.13: Plot of Maximum Water Chloride Concentrations (mg/L)

The highest water chloride concentrations were determined for perched water samples that were collected on the west side of the Fox River near existing stockpiles of salt (present on an adjacent parcel).

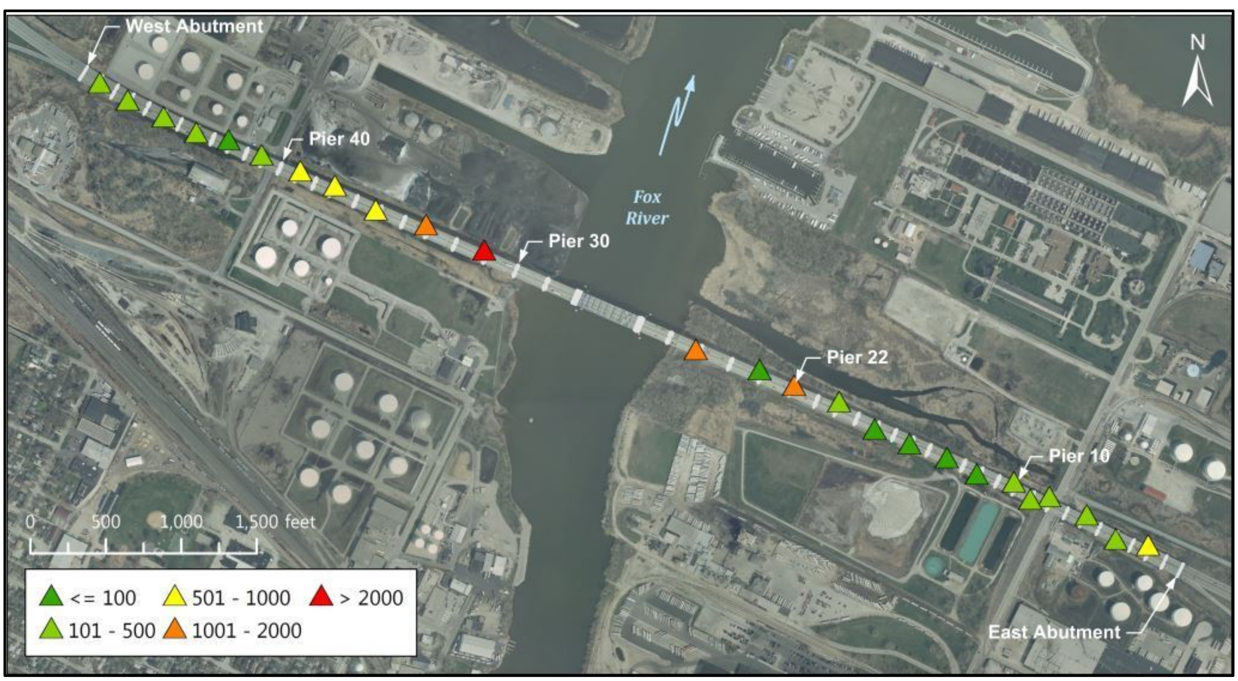


Figure 3.14: Plot of Maximum Water Sulfate Concentrations (mg/L)

The highest water sulfate concentration was obtained from a perched water sample that was collected on the west side of the Fox River near existing stockpiles of coal (present on an adjacent parcel).

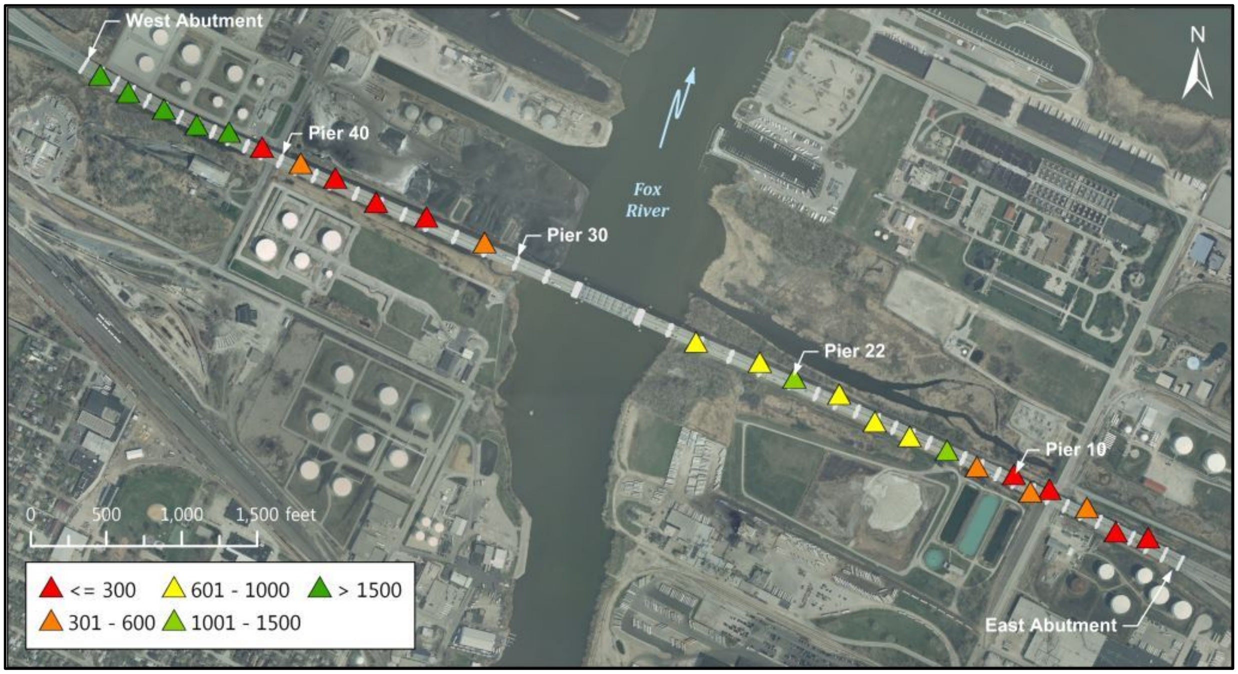


Figure 3.15: Plot of Maximum Water Alkalinity (mg/L $CaCO_3$)

The lowest water alkalinities were obtained at the easterly end of the bridge on the east side of the Fox River, and generally between Bylsby Avenue and the river on the west side of the Fox River. It is particularly noteworthy that the highest alkalinities were identified on the east side of the river opposite the landfill and at the westerly end of the bridge.

The chemical analyses confirm that the subsurface conditions over a significant portion of the project site exhibit atypically low soil resistivity and high chloride concentrations. The soil resistivity measured in the laboratory for samples obtained from land piers ranged from a low of 80 ohm-cm (Pier 39 Test Pit TP39NW, sandy silt with organics) to a high of 5,700 ohm-cm (Pier 22 Test Pit TP22S, pit run sand & gravel base material at bottom of footer). The soil chloride concentration ranged from a low of 55 mg/kg (Pier 24 Test Pit TP24NW, silty sand) to a high of 11,400 mg/kg (Pier 39 Test Pit TP39SE, sandy silt with organics). Water sample chloride concentration ranged from a low of 56 mg/L (Pier 10 Sampling Point MW10) to a high of 27,100 mg/L (Pier 31 Sampling Point MW31). Low resistivity soils in combination with high chloride concentration are particularly prone to support development of corrosion macrocells, especially where piling penetrates through aerated soil/fill into saturated clay.

Figure 3.16 illustrates the correlation of soil resistivity with available chorides/sulfates in relation to the position of the perched water and bottom of the pile-supported concrete footings.

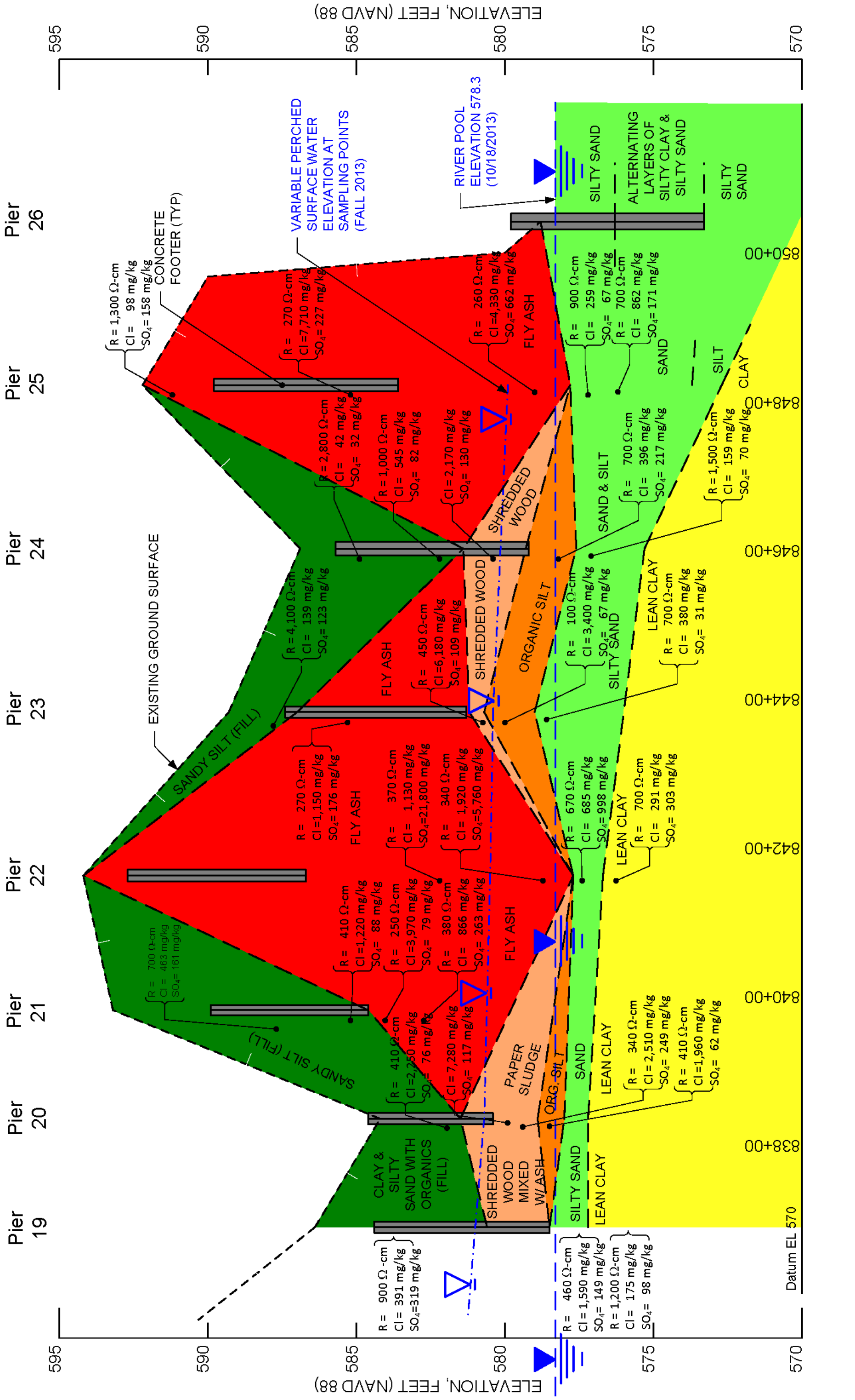


Figure 3.16: Distribution of Select Representative Chemical Analysis Results at Piers 19 through 26

3.5.3 Unconfined Compression Tests

Unconfined compression tests were performed on representative concrete and rock core samples to provide data for design of foundation repair. A total of 6 concrete core samples were tested, respectively. The unconfined compressive strength of the concrete core samples tested ranged from 7,920 to 9,500 psi, with an average of 8,655 psi.

A total of 7 rock core samples were tested. The unconfined compressive strength of the rock core samples tested ranged from 10,110 to 35,330 psi, with an average of 18,678 psi. Based on a typical 7,000 to 19,000 psi range for the unconfined compressive strength of dolostone, the laboratory strength test results obtained are considered to be representative of more competent end of the strength spectrum for dolostone. Detailed results for the unconfined compression tests are presented in Appendix L.

3.5.4 Metallurgical Testing

Metallurgical tests were performed on a scrap piece of HP14x73 piling that was recovered at Pier 15 from Test Pit 15S. Coupon samples from the pile web and flange were tested for chemical composition and strength. The test results indicate that the steel tested generally consists of mild/low carbon steel similar to ASTM A36 steel, with yield strengths of 37,200 and 31,100 psi at the pile flange and web, respectively. This is in general conformance with the material requirements of the original design.

3.5.5 Microbial Diagnostic Testing

Diagnostic test kits were used to check for the presence of microbes in soil samples with regard to microbially influenced corrosion (MIC). MIC involves biological microbes that react and cause the corrosion or influence other corrosion processes of metallic materials. Physical evidence of MIC at the project site included the presence of sulfide gas (rotten-egg smell, which is a by-product of sulfate reducing bacteria) as well as biofilm, which was observed at some of the piles that were exposed in the test pits. MIC tests were performed to check for aerobic bacteria, including Low Nutrient Bacteria (LNB) and Iron Reducing Bacteria (IRB), and to check for anaerobic bacteria, including Anaerobic Bacteria (AB), Acid Producing Bacteria (APB), and Sulfate Reducing Bacteria (SRB). APB feed on organic nutrients and excretes organic acid, which can chemically react with chlorides and form a strong acid that can lead to severe pitting corrosion. The MIC test results are presented in Table 4.

The microbial samples were either collected from the surface of the steel, the corrosion product, or from the soil adjacent to the exposed piles. The results reported herein indicate the microbial results after 7 to 15 days of incubation in a laboratory environment.

Summary - Distribution and Range of MIC Test Results
(Number of Piers out of 17 Piers where MIC Samples Were Collected and Tested)

Description		No Colonies Recorded	1-10 colonies per mL	10-100 colonies per mL	1,000-10,000 colonies per mL	10,000-100,000 Colonies Per mL	> 100,000 colonies per mL
Aerobic	Low Nutrient Bacteria	0	0	3	8	--	5
	Iron Reducing Bacteria	1	5	7	3	--	0
Anaerobic	Anaerobic Bacteria	0	0	2	6	--	8
	Acid Producing Bacteria	0	1	3	5	1	7
	Sulfate Reducing Bacteria	1	4	8	4	--	0

Detailed graphic plots of test results for Low Nutrient Bacteria (LNB), Iron Reducing Bacteria (IRB), Anaerobic Bacteria (AB), Acid Producing Bacteria (APB), and Sulfate Reducing Bacteria (SRB) are presented in Exhibits 16 through 20, respectively; and graphic summary plots for the same are presented below in Figures 3.17 through 3.21.

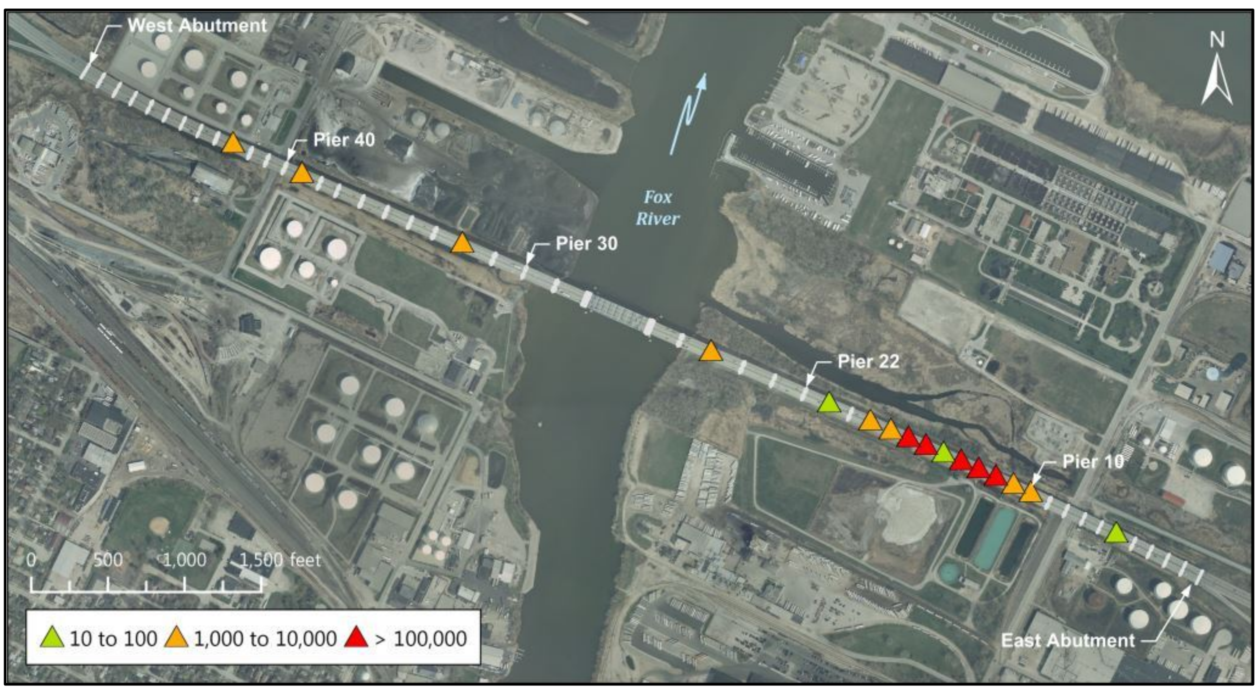


Figure 3.17: Plot of Low Nutrient Bacteria (colonies/mL)

The highest concentration of low nutrient (aerobic) bacteria was detected on the east side of the Fox River between Piers 10 and 19.

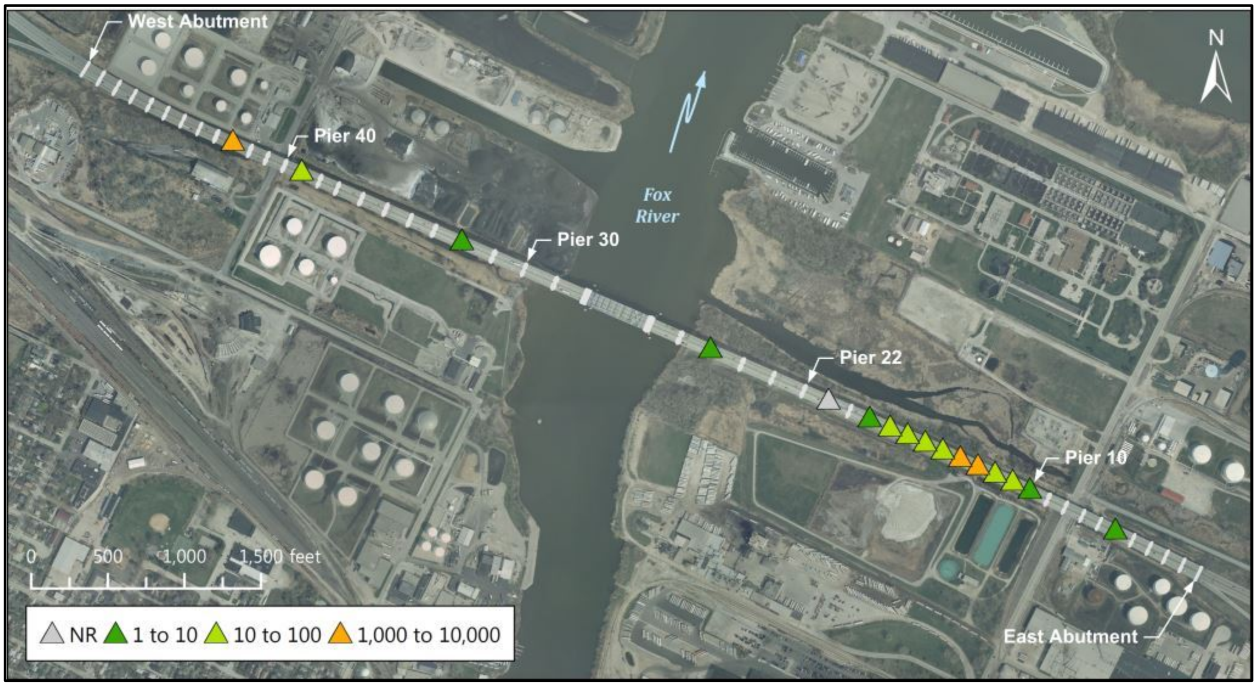


Figure 3.18: Plot of Iron Reducing Bacteria (colonies/mL)

The most significant concentration of iron reducing (aerobic) bacteria was detected at Piers 13, 14 and 43.

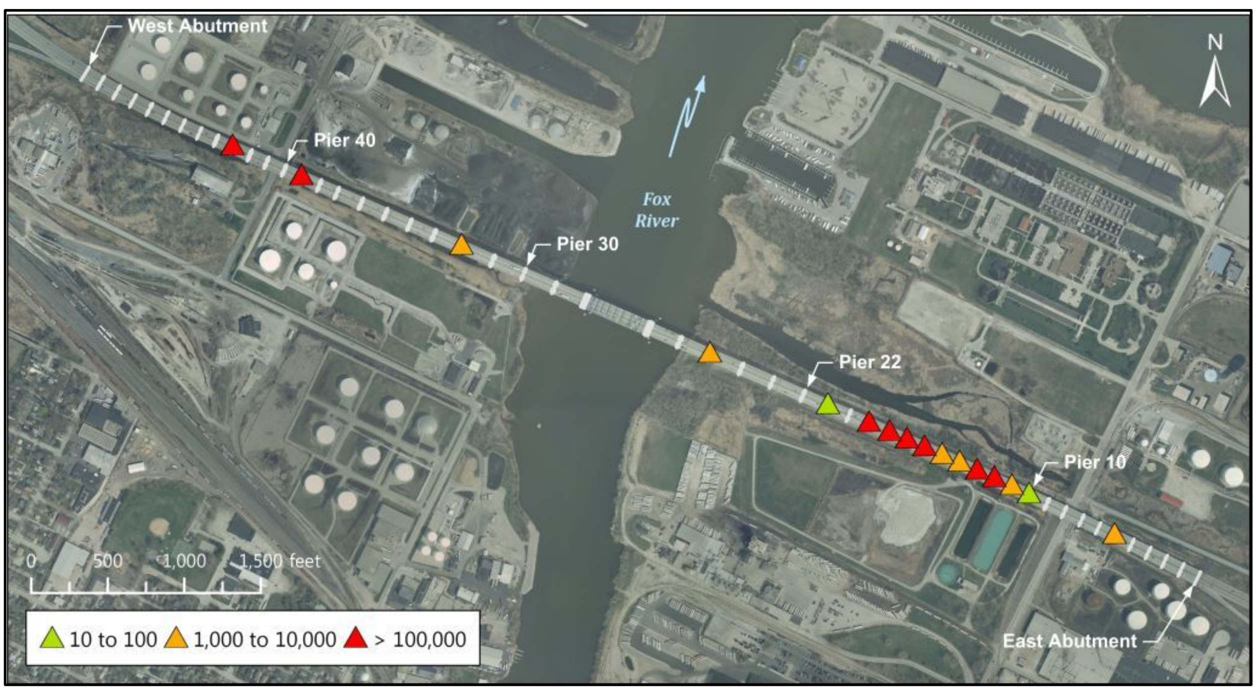


Figure 3.19: Plot of Anaerobic Bacteria (colonies/mL)

The presence of anaerobic bacteria was substantial at all of the piers sampled, except for Piers 10 and 21.

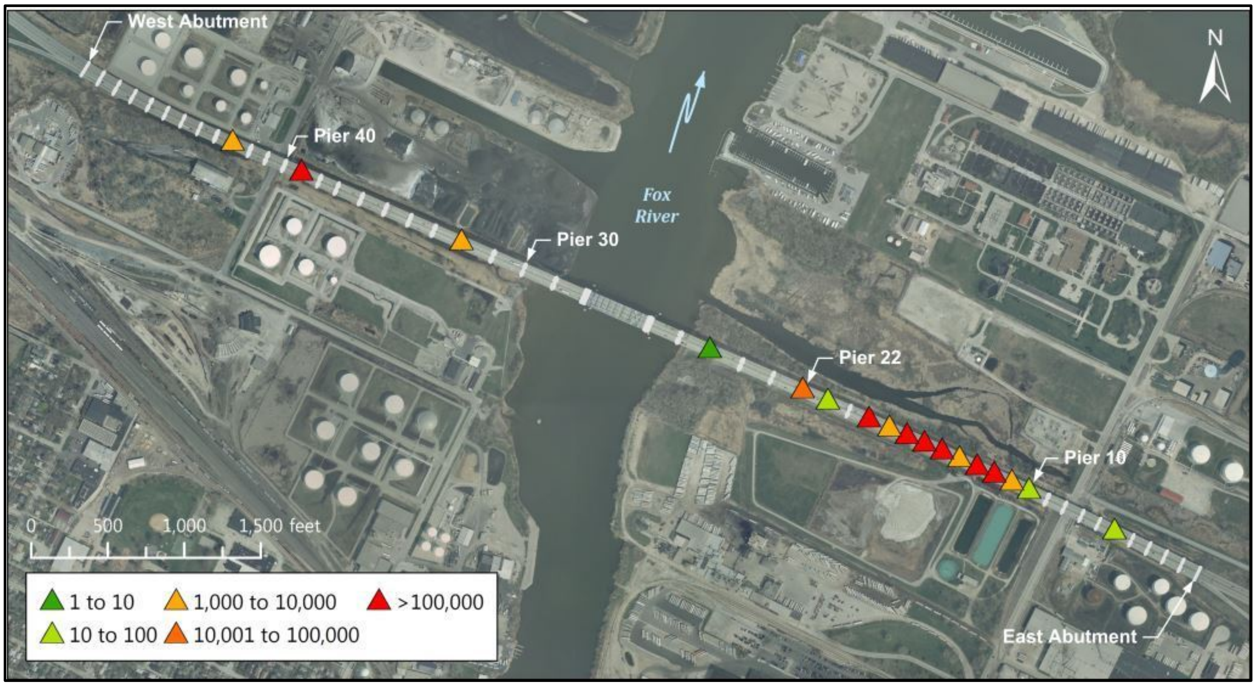


Figure 3.20: Plot of Acid Producing Bacteria (colonies/mL)

Acid producing bacteria was particularly noteworthy on the east side of the Fox River between Piers 11 and 19.

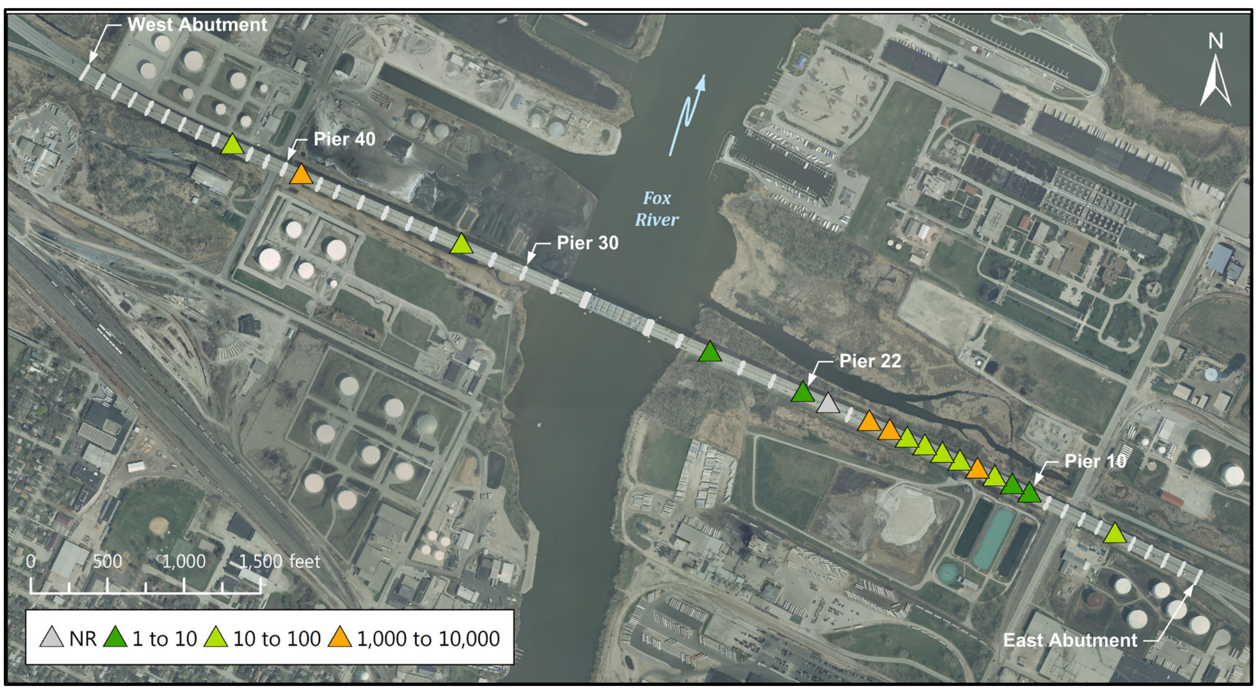


Figure 3.21: Plot of Sulfate Reducing Bacteria (colonies/mL)

The most noteworthy concentration of sulfate reducing bacteria was encountered at Piers 13, 18, 19 and 39. It should be noted that the microbial growth of Sulfate Reducing Bacteria typically requires at least 28 days before an accurate colonial count can be estimated. Given that the maximum incubation times for the samples collected was 15 days, the results shown in Figure 3.21 can be used to verify the presence or absence of SRB at a certain pier location. However, the data presented substantially underestimate the colonial count of the SRB strains.

The microbial diagnostic testing was particularly useful to demonstrate that microbial activity is exceptionally active at the project with numerous samples showing in excess of 100,000 colonies/ml; with microbes that substantially more active, than what would otherwise be expected for a typical bridge site.

3.5.6 Petrographic Examination

Petrographic examinations, including scanning electron microscopy (SEM and ancillary energy dispersive spectroscopy (EDX, were conducted on representative concrete core samples that were obtained from the existing footer at the Pier 21 south column and the Pier 25 south column. Photographs and details about the concrete core samples are presented in Appendix L. Based on the petrographic examination, there was no evidence of chemical alteration or physical deterioration of the concrete sampled, and there was no evidence of sulfate attack. Chloride profiles indicate that there has been a chloride uptake at Pier 21 to at least a depth of 1-1/2 inches. The other noteworthy observation from the petrographic analysis was an elevated base level of chloride content at a depth of 14 inches, which infers that elevated levels of chloride may have been present in the mixing water when the concrete was originally placed. Detailed results for the petrographic examination are presented in Appendix O.

Petrographic examination, including SEM and EDX, was also conducted on 10 representative samples of corrosion product and soil to assist with interpretation and evaluation of the type of corrosion process that has taken place. The samples were obtained from the surface of exposed piling and adjacent soil at Piers 19, 21, and 25. The samples analyzed indicate that the soils that surround the piles at Piers 19, 21, and 25 contain natural sand intermixed with products from the combustion of coal that contain iron sulfides that have oxidized and contributed to sulfuric acid attack to the steel H-piles. The petrographic examination detected sulfate content in excess of 4 percent (40,000 mg/kg) and a chemical alteration of the steel piling itself to various forms of iron oxide including iron sulfate. Based on the petrographic examination, the compositional properties of the ash correspond to the mullite series of minerals and are judged to represent fly ash residue from burning low grade coal. Pier 19 differs from Piers 21 and 25 in that there is a surficial layer of fill that contains significant silt, which serves as a cap to restrict the flow of oxygen from the surface. Availability of an oxygen source is a key component for the corrosion process that is acting at these piers. Detailed results for the petrographic examination are presented in Appendix O.

3.6 2013 Sampling Procedures and Quality Control (QC) / Validation Testing

Select samples were collected during the field exploration and transported to the laboratory for testing. Samples were containerized and transported directly to the laboratory in accordance with relevant test methods. Bulk soil samples were quartered and processed in the laboratory. Chain of Custody forms are presented in Appendix J.

Test results were reviewed in the laboratory and screened for consistency, completeness, and identification of possible outlier results. When a possible outlier result was identified, a retest was performed to validate the test result. The retests were particularly useful to validate the abnormally high soil sulfate and soil chloride content determinations, which occasionally exceeded 5,000 mg/L.

Supplemental tests were conducted on representative samples of ash and organic silt to determine the effect of drying temperature on moisture content determination, between samples dried at 60, 85, and 110 degrees centigrade. Average moisture loss of 1.2 and 2.4 percent were determined when the drying temperature was increased from 60 to 85 and from 60 to 110 degrees centigrade, respectively. Moisture change with respect to variance in drying temperature for the ash and organic silt was less than anticipated and hence management of the drying temperature to determine moisture content for the other samples tested was deemed sufficient for quality control purposes.

3.7 2014 Tier 2 Investigation

In order to estimate corrosion rates more accurately and determine the remaining service life of the Tier 2 steel piles more reliably, steel coupons (pile sample sections) were installed (typically 3 per pier) adjacent to Piers 6, 13, 19, 21, 25, 32, 35, 39, 44 and 50; and monitoring probes were installed at test pits dug at Piers 6, 13, 19, 32, 39, 44 and 50 (between July 14 and July 23, 2014). The monitoring probes included 35 half-cell probes and 49 electrical resistance (ER) probes. The monitoring probes are anticipated to be inspected every three months for the first few years, then every six months for the next year or two, and then every two years moving forward. The steel coupons will be assessed after 5, 10 and 15 years. Furthermore, additional soil testing and chemical analyses were conducted during the installation of the probes to better assess the existing soil conditions at each of these piers. Apparent corrosion rate and corrosion potential measurements will be collected from the probes during typical bi-annual bridge maintenance inspections performed by WisDOT. Data from the probes will be supplemented by section loss measurements conducted on the pile coupons. If the measured corrosion rates from the probes turn out to be greater than the estimated rate, the Department will have multiple treatment options, such as cathodic protection or modification of the environmental settings through jet grouting, available to reduce the corrosion rate and extend the service life of the bridge.

In addition to the work described above at the Tier 2 piers, monitoring probes were installed at Pier 22. The information collected at this location can be compared with the data across the rest of the project site and serve to provide information on what is anticipated to be the upper range of corrosion activity present at the site.

A detailed inspection and condition assessment of the corrosion damage at Piers 6, 13, 19, 32, 39, 44 and 50 was conducted in July 2014. The assessment included visual evaluation, thickness measurements, half-cell potential surveys, and in-situ acid spray tests.

Due to the variability of the soil conditions along the Leo Frigo Memorial Bridge site, the severity of the observed corrosion damage was highly variable between the seven Tier 2 piers included in the July 2014 investigation. While evidence of corrosion damage was observed at all the examined piles, pitting was more pronounced at pier locations where the soil exhibited high sulfate ion concentrations, high chloride ion concentrations, or both.

The monitoring system installed at each instrumented Tier 2 pier incorporates two different corrosion sensor technologies: half-cell electrodes and electrical resistance (ER) probes. The half-cell potential technique evaluates the corrosive nature of the soil medium and provides a qualitative measure of the corrosion risk. The ER probes measure the rate of corrosion at the point of embedment, and can be used to assess the cross sectional loss over time.

At each of the monitored piers, multiple half-cell (Figure 3.22) and ER (Figure 3.23) probes were embedded in undisturbed native soil and at the vicinity of the inspected steel piles. The probes were installed at different elevations to assess the changes in the electrochemical characteristics over the depth of the piles. Figure 3.24 provides a schematic representation of the corrosion monitoring system installed at the bridge. Data obtained during test pitting and installation of monitoring probes in July 2014 is presented in Appendix V.

Figure 3.22: Half-Cell Electrode Embedded In Undisturbed Soil

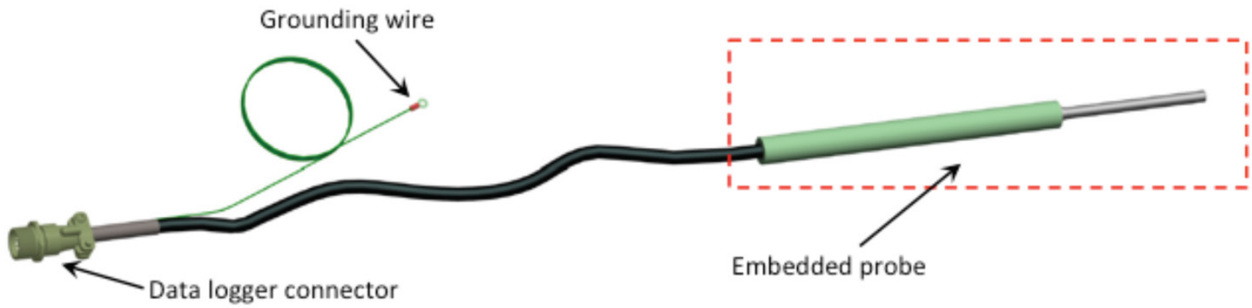


Figure 3.23: Schematic Showing the Primary Components of the ER Probe

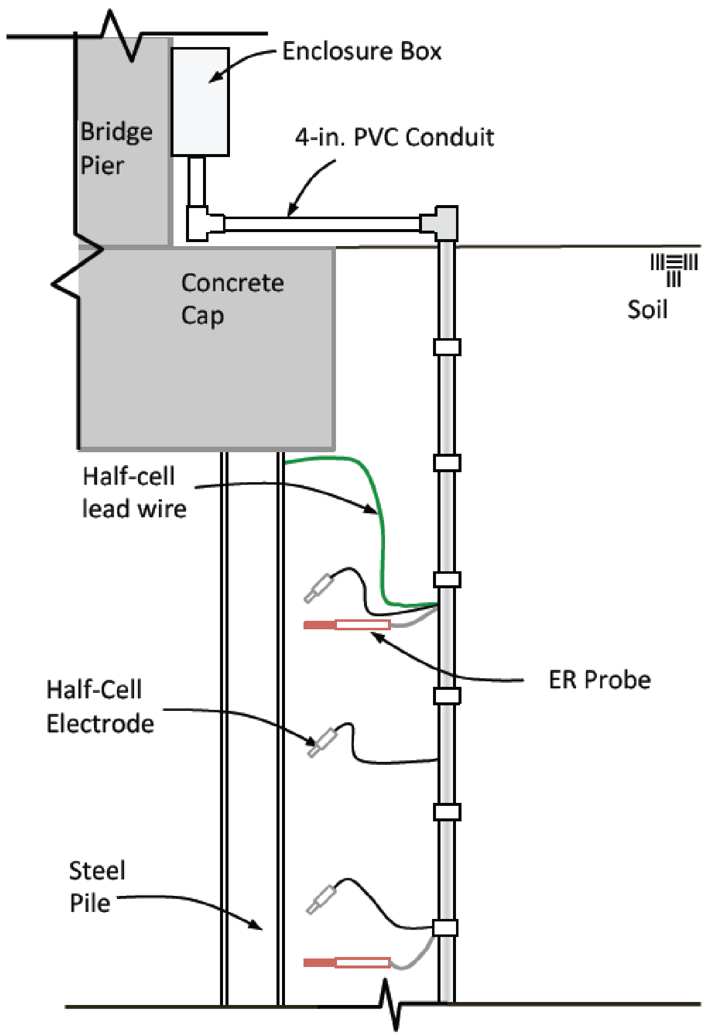


Figure 3.24: Schematic Showing the Primary Components of the Monitoring System

4.0 Engineering Considerations

The test pits that were excavated in 2013 and 2014 at selected pile-supported footers, to expose the upper portion of the steel piles, revealed varying conditions and degrees of corrosion. At many locations, the corrosion appeared to be minimal surface corrosion or very modest pitting, which has not resulted in any significant loss of steel section or capacity. At some pier locations, specifically 21, 22, 23, and 25, the H-piling has experienced corrosion resulting in significant loss of steel cross section of the piles inspected. It is noted, though, that not all of the piles at any individual pier experienced the same degree of corrosion. The term “severe corrosion” in this report should be taken to mean corrosion that has significantly reduced the bridge foundation’s structural capacity.

The position of the perched water, soil type, soil stratification, and soil resistivity provide relevant information regarding the corrosiveness of the soil medium. The perched water is a temporary accumulation of water over a relatively impermeable layer (aquitard) above the regional water table (Fox River) that is prone to localized fluctuation and that has a direct impact on moisture and oxygen availability, which are two key factors to sustain pile corrosion. In addition, chlorides and sulfates tend to decrease the resistivity of the soil, especially over depths where the soluble salt concentration varies, such as where soluble salt from external sources becomes concentrated within a porous fill over relatively impermeable fine grained soil. Relevant data is summarized on a substructure-by-substructure basis in Table 4. In particular, a plot of maximum soil chloride, soil sulfate, water chloride, and water sulfate concentrations are presented in Exhibits 10, 11, 13, and 14, respectively.

The elevation of the regional ground water table (river pool) is significant. Available data indicates that the river pool can fluctuate on the order of 3 feet annually and that the river pool elevation has varied by about 7 to 8 feet over the past 30 years (elevation 576.0 to 583.7). This water table elevation is expected to vary seasonally and is dependent generally on the magnitude of rainfall and snowmelt. The significance of the regional water table is that piling located below this elevation is submerged, which restricts the amount of oxygen that is in contact with the steel piling. At depths where the steel piling is permanently inundated, or is submerged for the vast majority if its service life, the steel piling is protected against aggressive corrosion at this site.

A plot of minimum soil resistivity measurements is presented in Exhibit 9. Additional information is presented in Figures 3.9, 3.16 and 4.7. Measured soil resistivity was of particular interest at the east approach near fill areas with high concentrations of ash (Piers 21 to 23 and 25), near settling basins for the adjacent landfill (Piers 11 through 13), and at the west approach near the salt stockpiles that were placed during the recent past (Piers 29, 32, and 39).

In the area of Pier 22, sets of bridge scuppers are located south of Pier 21 and Pier 23. Runoff from these scuppers collects in drainage basins located below the structure. Past bridge drainage has contributed to elevate the chloride content of the in situ soil under the bridge, which has contributed to create a chloride-rich subsurface condition that can promote accelerated corrosion. Although, in principle, diversion of salt-laden drainage from bridge scuppers is potentially beneficial, diversion of bridge scupper drainage at this point in time is qualitatively not expected to cause a significant reduction in the chloride content in the existing soil under the bridge that is sufficient to make a substantial reduction in the corrosivity of the existing surficial soil.

The forces applied to the Pier 22 piles are different than on the adjacent piers. Although subsurface conditions with regard to corrosion are similar at Piers 21, 22, 23, and 25, Pier 22 is the only pier of these four with fixed bearings, which result in a different magnitude of forces applied to Pier 22 than to the other three piers, which have expansion bearings. Wind loads, braking forces, and thermal loads are all different at fixed piers compared to adjacent expansion piers. Due to Pier 22 being a fixed pier, both lateral and longitudinal (parallel to the bridge centerline) restraint was provided by the bridge superstructure at the top of the pier. An expansion bearing would provide lateral restraint, but would not provide the same level of longitudinal restraint. This restraint at the top of the pier helped to prevent the pier cap from rotating longitudinally and toppling over.

COMPARISON OF MAXIMUM FORCES AND MOMENTS BETWEEN PIER 21 AND PIER 22 AT BOTTOM OF ONE COLUMN

LIMIT STATE	MAXIMUM FORCE OR MOMENT	PIER 21 (EXPANSION BEARINGS)	PIER 22 (FIXED BEARINGS)
SERVICE ENVELOPE	FORCE ACTING DOWNWARD (KIPS)	3,704	4,130
	MOMENT ABOUT THE BRIDGE LONGITUDINAL DIRECTION (KIPS-FT)	1,821	3,404
	MOMENT ABOUT THE BRIDGE TRANSVERSE DIRECTION (KIPS-FT)	4,781	8,285
	Number of Piling per Footing	17	20
	Required Pile Capacity (KIPS)	300	300

The discussion below makes reference to a tiered metric, which was employed to assist with categorizing the project site in regions with similar subsurface setting. Further detail about the tiered metric is presented below in Section 4.6.

4.1 Potential Failure Mechanisms Considered

On September 25, 2013, between about 3:00 and 3:45 AM, Pier 22 displaced vertically approximately 24 inches. The south edge of the 90-foot-wide pier cap was about 4 inches lower than the north edge, with an apparent transverse slope of about 1 inch in 22.5 feet. There has been no measurable tilt in the longitudinal direction at Pier 22. The majority of the Pier 22 vertical displacement occurred in very likely less than one hour, and possibly over a period of seconds. Based on surveying completed since the initial vertical displacement, Pier 22 sustained an additional vertical movement of about 1 inch until the temporary stabilization was complete (a period of just over 6 weeks), with no measurable tilt in either direction. No additional movement was recorded at Pier 22 through the completion of the bridge repairs. There has been no measurable vertical displacement or tilt in any of the adjacent piers.

There has been no sign of structural damage to Pier 22 or its pile-supported footings. The fixed bearings at the top of Pier 22 remained fully engaged. Due to continuity of the steel girders over Pier 22, after vertical pier displacement occurred the load from the superstructure onto Pier 22 decreased substantially. This load was shed to the adjacent piers so that the loading then increased at Piers 21 and 23.

Each column at Pier 22 is supported by 20 HP14x73 steel end-bearing piles, which were driven through fill and underlying soft to medium stiff varied silt and clay into a deeper hard glacial till. At Pier 22, construction driving records (Appendix D pages D90, D91, D92, and D107) indicate that a Kobe K13 single acting diesel hammer followed by a Kobe K25 single-acting diesel hammer were both used to drive a total of 40 HP14x73 steel piles to an allowable bearing capacity ranging from about 162 to 183 tons, as determined by the WisDOT modified Engineering News Record Formula (Appendix C page C23). Test pile records for Pier 22 Pile 30 (Appendix D pages D3 through D6) indicate that a Kobe K13 hammer was used to drive the 1st 55 feet of pile, and then a Kobe K25 hammer was "used for bearing" to drive the piling from a depth of 55' to the final pile tip elevation. Test pile records for Pier 22 (Appendix D pages D4, D5 and D6) indicate that approximately 20 percent of the pile capacity was developed in the fill and varved silt and clay, above approximate Elevation 493, and the remaining 80 percent of the allowable bearing capacity was developed in the deeper glacial till, as indicated in Figure 4.1. Figure 4.1 shows a side-by-side correlation of the test pile results, generalized subsurface profile, graphic log for Boring 16A, and graphic log for Boring Pier 22B, all completed at Pier 22. It is worthy to note that both the bottom of the surficial fill elevation and the top of the dolomite bedrock elevation for both Boring 16A (circa 1970's) and Boring Pier 22B (post-pier vertical displacement) are at nearly the same elevation.

Based on the available construction records, the pile tips at Pier 22 were driven to between about Elevation 487.4 and 476.4, with an average tip Elevation of about 483.6 (NAVD88) that is approximately 15 feet above the top of bedrock and within the glacial till.

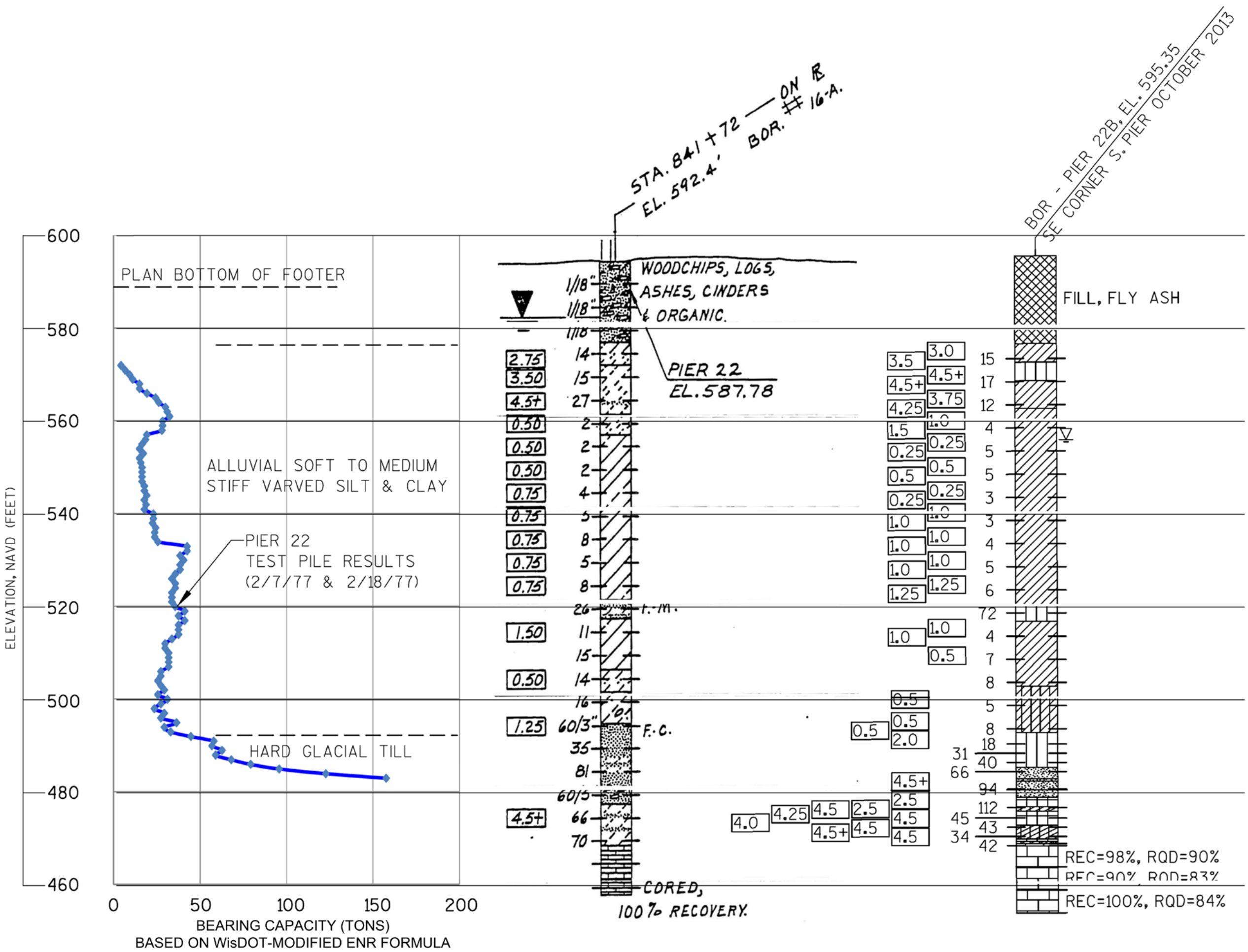


Figure 4.1: Test Pile Bearing Graph

The following was considered to evaluate the failure mechanism that occurred at Pier 22.

a. Loss of bearing resistance within the glacial till due to ground loss into underlying solution cavities within the bedrock.
b. Loss of bearing resistance due to a rising regional ground water potentiometric head within the bedrock formation, with subsequent reduction in the effective overburden pressure.
c. Redistribution of bearing resistance due to non-uniform bearing in the glacial till, with some piles bearing on possible bouldery material.
d. Vertical displacement of the pile group.
e. Structural section loss of the piling due to stray current induced corrosion, from either an underground pipeline with impressed current cathodic protection or overhead high voltage alternating current transmission lines.
f. Structural section loss of the piling due to corrosion.

Loss of bearing resistance within the glacial till due to ground loss into underlying solution cavities within the bedrock was considered. Based on the pile driving records, the HP14x73 piles at Pier 22 were terminated in the till, approximately 6 to 22 feet above the recorded top of bedrock. This failure mechanism, if present, would most likely result in a softening and reduction in consistency of the glacial till near top of rock due to ground loss. However, the subsurface conditions encountered at Boring Pier 22B suggest that the consistency of the glacial till has generally remained the same over the life of the bridge; hence, possible ground loss into underlying solution cavities is considered unlikely. In addition, 10-foot-deep bedrock cores obtained from the initial 1973 to 1975 site investigations for the structure and from the 2013 deep boring (B-22B) 14.5-foot bedrock core indicate fair to excellent rock quality (RQD data) at Pier 22 and the remaining piers, with no indication of solution enlarged joints or voids. Further, ground loss would most likely be progressive over a period of time, with gradual reduction in bearing resistance; a gradual reduction in bearing resistance does not explain why an abrupt pier displacement occurred.

Loss of bearing resistance was also considered due to a rising regional ground water potentiometric head within the bedrock formation, with subsequent reduction in the effective overburden pressure. However, the pile bearing layer is a very hard cohesive till and, similar to the limestone bedrock, has a compressive strength that is not likely influenced by a change in the effective overburden pressure caused by a rise of the regional ground water potentiometric head. If the bearing layer were granular, then a change in the effective overburden pressure could have influenced the bearing resistance, but this is not the case. In addition, a rising ground water potentiometric head would most likely affect multiple piers and does not account for the isolated vertical displacement at Pier 22, with no apparent movement at the adjoining piers. Further, if this were the mode of failure, it would be expected to have resulted in a progressive change in frictional resistance at the pile-soil interface resulting in a progressive change in support, which does not account for the sudden vertical displacement of Pier 22. Thus, this possible failure mechanism was dismissed from consideration.

Redistribution of bearing resistance due to non-uniform bearing in the glacial till, with some piles bearing on possible boulder material, was also considered. Redistribution of bearing resistance could result in an increase in axial compression load at a fraction of the piles within the pile group. However, a similar redistribution would need to occur at both columns to result in the lack of noticeable tilt of the pier. Further, it would be expected that any such redistribution of bearing resistance would have manifested itself at a much earlier stage in the life of the bridge. In addition, only one boring (at the east abutment) of the original 47 bridge borings (and the recent deep Pier 22B boring) indicate the presence of boulders near the pile tip elevation, and none indicate the presence of cobbles. Although possible, redistribution of bearing resistance to such a degree to cause 2 feet of abrupt vertical pier displacement was considered to be highly unlikely.

Settlement of the two pile groups at Pier 22 was also considered. It is possible that each pile group could act as an equivalent raft foundation, which would apply load on the deeper hard glacial till. Since the piles for this bridge get nearly 80% of their capacity from end bearing, significant settlement or compression of the underlying till is highly unlikely. Consolidation settlement would be expected to occur gradually over a period of time, which is not consistent with abrupt movement. Thus, this mode of failure was dismissed from further consideration.

Structural section loss of the piling due to stray current induced corrosion was also considered. The stray current would have been from either the West Shore underground pipeline, which is cathodically protected with an impressed current corrosion protection, or adjacent overhead transmission lines. The cathodic protection (CP) system associated with the West Shore Pipeline operates on direct current (DC) and can contribute to stray current interference and detrimental corrosion to nearby facilities and structures. The high voltage alternating current (AC) power transmission lines can introduce electromagnetic fields that can induce voltages and current on aboveground and underground structures. Based on field test results, which are reported in Appendix N, it was concluded that there was no measured influence from the West Shore Pipeline CP system or from induced AC on the piling at Pier 22. Thus, this mode of failure was dismissed from further study.

Structural section loss of the piling due to corrosion was also considered. Corrosion of piling can lead to reduced structural capacity and can result in local buckling and/or crushing of steel piling. Further discussion is presented below about this failure mechanism.

4.2 Corrosion Mechanisms Affecting Steel Piles

Based on the visual observations and the results of the testing conducted on soil and corrosion product samples, it is evident that a number of corrosion mechanisms are affecting the steel piling at the I-43 Leo Frigo Memorial Bridge. The following paragraphs describe the corrosion mechanisms that very likely led to corrosion damage at the bridge site. However, due to the complexity of the soil profile and exposure conditions surrounding the piling combined with a lack of thorough testing at multiple pier locations, the exact corrosion mechanism leading to the severe damage at Piers 21, 22, 23, and 25 cannot be precisely identified with 100% certainty.

Aerobic corrosion is the most common corrosion mechanism affecting steel structures. In neutral environments, steel corrodes if oxygen and moisture are available in the surrounding medium. Aerobic corrosion is most likely to occur in porous soils where the steel piles are partially submerged, including but not limited to the piles at Piers 21, 22, 23, and 25. At Pier 25, an air gap was encountered immediately below the bottom of pier footing (Figure 4.2). The corrosion damage within the air gap is indicative of uniform aerobic corrosion damage. The presence of severe localized pitting, which is not characteristic of aerobic corrosion, suggests that other corrosion mechanisms were active in the severely corroded piles at Pier 22.

Galvanic cell corrosion occurs between soil layers when there is a significant difference in the characteristics of the layers. Soil composition, pH, moisture content and oxygen content are all factors in this type of corrosion. The pile length located in the fill zone is the cathode in the galvanic corrosion cell and the pile length below the fill is the anode (corrodes). The location of most severe galvanic corrosion will be at the water table where the corrosion cell is most intense. Galvanic cell corrosion is one form of corrosion found at at the Leo Frigo site.

Figure 4.2: Air Gap Below Bottom of Pier Footing at Test Pit TP-25NE

An oxygen differential corrosion cell can form when the pile passes through a porous fill from an oxygen rich environment above the water table into an oxygen deprived environment below the water table. This type of corrosion cell exists at Leo Frigo within the surficial fly ash fill at the ground water table.

Corrosion cells can also form when piling penetrates from one soil type into another or when there are chemical differences within a soil stratum. An example of this is a pipeline that passes through different soil types or soils having different salt contents. This type of corrosion cell also exists at the Leo Frigo Bridge caused by the different chemistries of the soil and fill.

Differences in pH from one zone to another can trigger corrosion. A pH differential cell could be formed between the steel piling embedded in the concrete footing and the steel piling in the fill. Steel piling in sound uncarbonated concrete develops a passive film due to the high pH of the concrete (pH>10) making it the cathode. Steel piling in soil with a neutral or more acidic pH is the anode. Corrosion is concentrated in the steel piling embedded in soil adjacent to the concrete interface. No significant carbonation in the concrete was noted during the investigation, which suggests that pH differential corrosion was not seen at Leo Frigo. Since there was no significant corrosion at the concrete interface and the worst corrosion was well below the pile/concrete interface, pH differential corrosion is not considered to be a significant factor at Leo Frigo.

Stray current from pipeline cathodic protection systems can result in corrosion due to errant current flow through the bridge piling, which originates from a nearby induced current source. The Pier 22 failure occurred in an area where a buried cathodically protected 10-inch petroleum pipeline crosses under the bridge deck between Piers 22 and 23 and where high voltage alternating current overhead electric transmission lines exist. The cathodic protection system for the 10-inch pipeline uses an impressed direct current (D-C), which can contribute to stray current interference and detrimental corrosion damage to nearby bridge piling. High voltage A-C power transmission lines can introduce electromagnetic fields that can induce voltages and current in above ground and underground structures, like bridge piling. Stray D-C current interference can result in very high corrosion rates while A-C interference can result in moderately high corrosion rates. However, based on the cyclic testing that was performed on the pipeline, the cyclic test results indicate that there was no influence from the West Shore Pipeline impressed D-C cathodic protection system at the time of test and that there was no induced A-C affecting Pier 22.

The porous fly ash with high levels of chloride, sulfide and low soil resistivity resulted in aerobic corrosion at Leo Frigo. The presence of sulfates can lead to production of sulfuric acid, which can cause steel to corrode.

Microbiologically Influenced Corrosion (MIC) is common in most environments. There are many different types of bacteria that can cause corrosion, including anaerobic, aerobic, acid producing and metal oxidizing. A particularly common example of MIC is sulfate-reducing bacteria (SRB). Typical environments where SRB's occur are wet, swampy areas, and clay soil with decaying vegetation. SRB are anaerobic, but it is important to note that chemical conditions within a bacteria colony can be widely different than in the surrounding environment. Acid producing bacteria (APB) produce either inorganic or organic acids that attack metal. Acid producing bacteria can also promote the growth of SRB by providing the environment and nutrients for SRB growth. If MIC alone were the cause of the pile failures, the degree of corrosion would be similar along the length of the piles exposed in the fly ash. Even if MIC were not present, the composition of the fly ash is aggressive enough to cause severe metal loss in the piles as discussed above, but this corrosion should also be present more uniformly along the length of the pile exposed to the fly ash. The localized corrosion observed in the failed areas of the piles in Pier 22 suggests that another mechanism is active.

Combinations of the above corrosion mechanisms can and do occur. A prime example is Accelerated Low Water Corrosion (ALWC) on steel piles at marine structures in the presence of sulfates, where both differential oxygen concentration corrosion and MIC is responsible for local "concentrated corrosion cells just below the low water mark. Complex biofilm colonies of both aerobic and anaerobic microbes can affix themselves to the pile surface to produce oxygen deprived regions that are particularly prone to very aggressive differential oxygen concentration corrosion. These complex biofilm colonies can develop in both aerobic and anaerobic conditions (above as well as below the water table). Sulfates are converted by sulfate reducing bacteria (SRB) into hydrogen sulfide which in turn leads to production of sulfuric acid and subsequent anaerobic corrosion on steel surfaces. At the low water mark, the corrosion process is exacerbated by an electrochemical process that lends itself to differential oxygen corrosion. The presence of ALWC is reported by the Maritime Board to be characterized by the presence of an orange surface that is underlain by grey sludge and shiny pitted steel surface, which is consistent with that observed at Leo Frigo.

4.3 Failure Mechanism at Pier 22

As discussed above there were various types of corrosion activity present at the site, but it appears that Accelerated Low Water Corrosion (ALWC) was mainly responsible for premature section loss of the steel piling at Pier 22 that led to crushing/buckling of the steel HP14x73 foundation piles as described below.

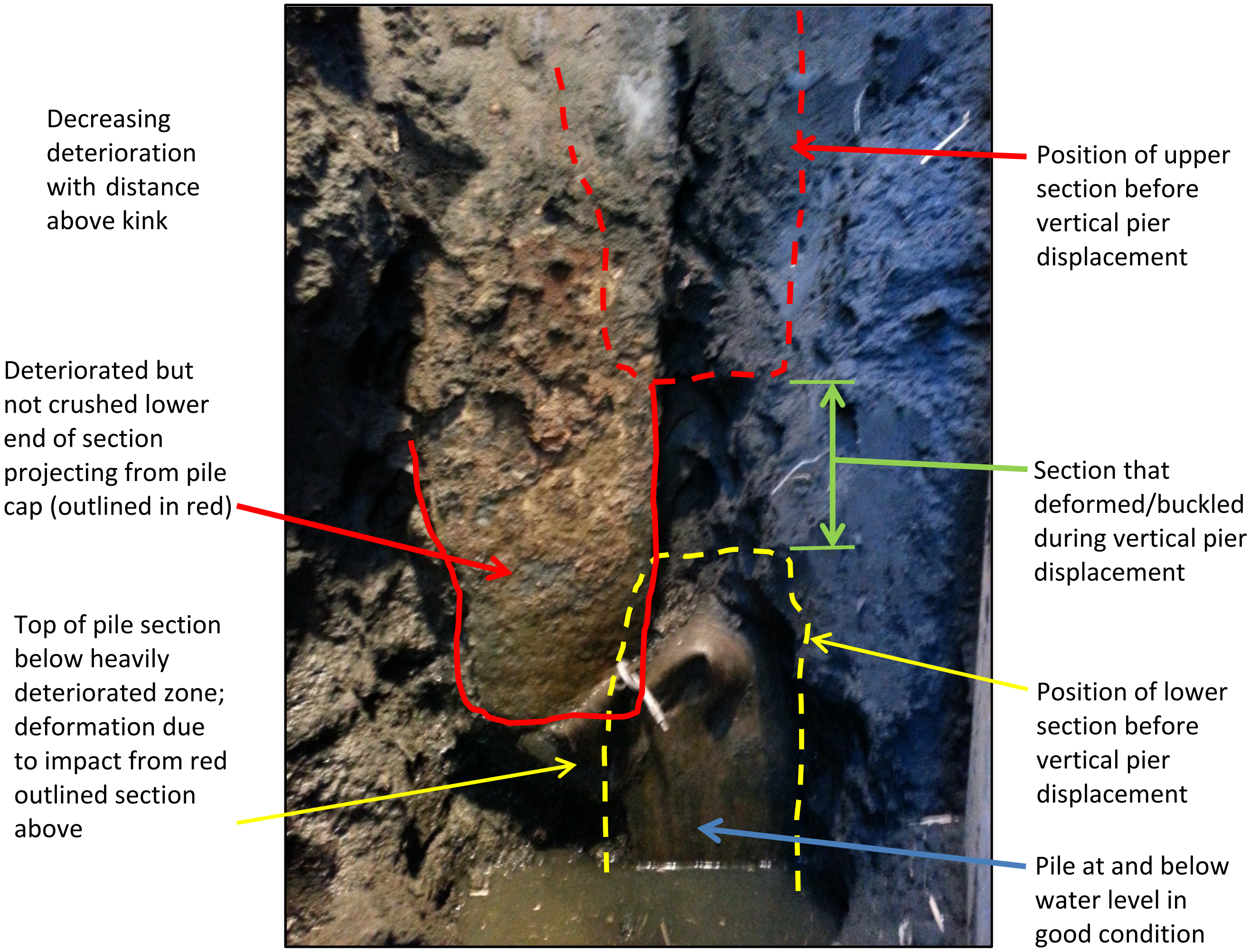


Figure 4.3: Buckled Pile #27 at Pier 22

The outboard flanges of two piles located near the southwest corner of the south footer (Pile # 1 and Pile #2) were exposed to a similar depth as Pile #27. Figure 4.3 shows the lower section of Pile #2. In each case, a small offset in the flange was visible about 6 to 7 feet below the bottom of the footer, and the offset represented the interface between two sections of pile that had been widely separated before the recent vertical displacement occurred. As with Pile #27, a section of pile between the currently visible portions above and below the offset had crushed. As shown in Figure 4.3, the upper and lower sections of pile cut several inches into each other, causing significant overlap in the exposed remains of the outboard flange.

Complete section loss of outer portions of flange

Bottom of upper section is several inches below top of lower section; the sections have cut deeply into each other

Figure 4.4: Crushed Pile #2 at Pier 22

Two piles (one at southeast corner of south footer - Pile #14 - and one immediately north (Pile #15) had sustained modest section loss at some areas of the exposed portion; the overall loss at any particular cross section appeared to be well below 20 percent. However, this excavation did not go as deep as the excavations for Piles #1, #2, and #27. Consequently, there may have been more heavily deteriorated and/or crushed or buckled sections below the exposed areas.

It is worth pointing out that similar buckled/crushed piling was observed at Pier 22 in July 2014 at the 6 piling that were exposed during the Tier 2 investigation. Representative photographs of the buckled/crushed piling from the 2014 investigation are presented in Appendix S Pages S195, S198, S216, S229, S238, S246, S252, S294 and S300.

Structural section loss of the piling was considered to evaluate the failure mechanism that occurred at Pier 22. Given the extent of corrosion noted in sections of Pier 22 piles, deterioration of many Pier 22 piles resulted in load redistribution, which caused the loading in less deteriorated piles to increase. As this process progressed, the stabilities of all piles decreased. Relatively intact piles became more likely to plunge, while heavily deteriorated piles approached crushing limits, even as they shed load (i.e., their capacities decreased faster than the load they were carrying).

Eventually, the stability of the pile group was compromised, either by plunging of a critical intact pile, or by crushing of a critical deteriorated pile. Either way, the associated redistribution of load from the critical pile to surrounding piles was enough to cause further pile failures (plunging, crushing, or both), which led to downward movement of the pier.

The settling pier came to a stop due to one or more of the following factors:

1. As the pier lowered, the load imparted by the superstructure decreased.
2. As the pier lowered, additional foundation resistance was mobilized in crushing zone piles when less deteriorated sections above and below the crushed zone came to bear on one another.
3. As the pier lowered, additional foundation resistance was mobilized as load was redistributed to plunging intact piles.

4.4 Corrosion Rate at Pier 22

The corrosion rate was variable at Pier 22. At Pier 22, Pile #2 was crushed and had corroded to a point that the exposed flange and web was “feather thin”, as noted in Figure 4.4 above; and Pile #27 was buckled with measured flange thicknesses ranging from 0.259 to 0.434 inches, which yields a flange thickness loss of approximately 0.07 to 0.25 inches based on a theoretical flange thickness of 0.505 inches. Based on the pile driving records the Pier 22 piles were driven in February 1977, approximately 36.6 years prior to the 2-foot vertical displacement at Pier 22. This yields an average section loss of up to a maximum averaged corrosion rate of about 7 mils per year x 2 flange sides for a total of 14 mils per year (0.36 mm per year) at Pier 22 Pile #2, and up to a maximum average total of about 7 mils per year (0.18 mm per year) at Pier 22 Pile #27).

Typical corrosion rates are tabulated below for comparative use. The following corrosion rates are based on values published on page 188 of the National Association of Corrosion Engineers (NACE) Corrosion Engineer’s Reference Book, Third Edition dated 2002. The minimum soil resistivity measured in the laboratory for the fly ash encountered at Pier 22 near Piles #2 and #27 ranged from about 260 to 370 ohms-cm. Based on comparison with the typical corrosion rates tabulated above, the measured section loss at Pier 22 Piles #2 and #27 corresponds with the average to maximum pitting rate range listed below for soil with a resistivity of less than 1,000 ohms-centimeter (Ω-cm).

Effects of Environmental Factors on Corrosion of Steel in Soil [a]

Environmental Factor	Overall Corrosion Rate (mm/year)			Maximum Pitting Rate (mm/year)		
	Maximum	Minimum	Average	Maximum	Minimum	Average
Resistivity (Ω-cm)						
< 1,000	0.063	0.018	0.033	0.31	0.11	0.20
1,000 to 5,000	0.058	0.006	0.017	> 0.45 (b)	0.05	0.14
5,000 to 12,000	0.033	0.005	0.018	0.23	0.06	0.14
> 12,000	0.036	0.003	0.014	0.26	0.03	0.11
Drainage						
Very poor	0.058	0.038	0.046	> 0.45 [b]	0.16	0.28
Poor	0.037	0.010	0.024	0.23	0.05	0.14
Fair	0.063	0.018	0.022	0.31	0.08	0.16
Good	0.022	0.003	0.010	0.18	0.03	0.11
Air-pore space (%)						
< 5	0.033	0.010	0.021	0.20	0.05	0.13
5 to 10	0.063	0.009	0.024	0.31	0.10	0.17
10 to 20	0.037	0.006	0.017	0.26	0.05	0.15
20 to 30	0.058	0.012	0.025	> 0.45 [b]	0.10	0.20
> 30	0.038	0.004	0.013	0.23	0.03	0.09

[a] Original data are based on NBIS field tests on open-hearth steel for 12 years at 44 locations in the United States.

[b] Perforated.

Source: M. Romanoff, *Underground Corrosion*, NIST, 1957.

There is insufficient information available to reliably differentiate published typical corrosion rates for sites where ALWC was and was not a significant contributing factor.

4.5 Temporary Stabilization

After the initial abrupt settlement of the foundation at Pier 22, daily monitoring indicated that the pier was continuing to settle at a rate of less than 0.01 feet per day. Additional settlement of Pier 22 negatively affects the structure in several ways. First, as the pier continues to settle, load on the pier is shed to the adjacent piers. As the exposed piling for piers 21 and 23 also had considerable corrosion, there was concern about adding additional loading to these piers. Second, the approximate 2-foot deflection experienced by the superstructure was within the elastic stress range of the steel girders; i.e., the structure could be jacked back into place without any permanent deformation. Temporary stabilization of the structure was determined to be the appropriate action to prevent further settlement and deflection.

Two temporary towers were constructed on each side of Pier 22. The towers were supported on steel H-piling that was installed in prebored holes to minimize vibrations. The piling for the temporary supports was driven to the same bearing stratum as the original piling. The capacities for these piling (250 tons at the west support and 200 tons for the east support) were determined using modified Gates (modified ENR was used for the original piling). Steel trusses were constructed and placed on top of the towers. Jacks were installed on top of the trusses to provide support to the superstructure and reduce the load on Pier 22 (as well as Piers 21 and 23). The following figures illustrate the plan, elevation and section views of the temporary support system.

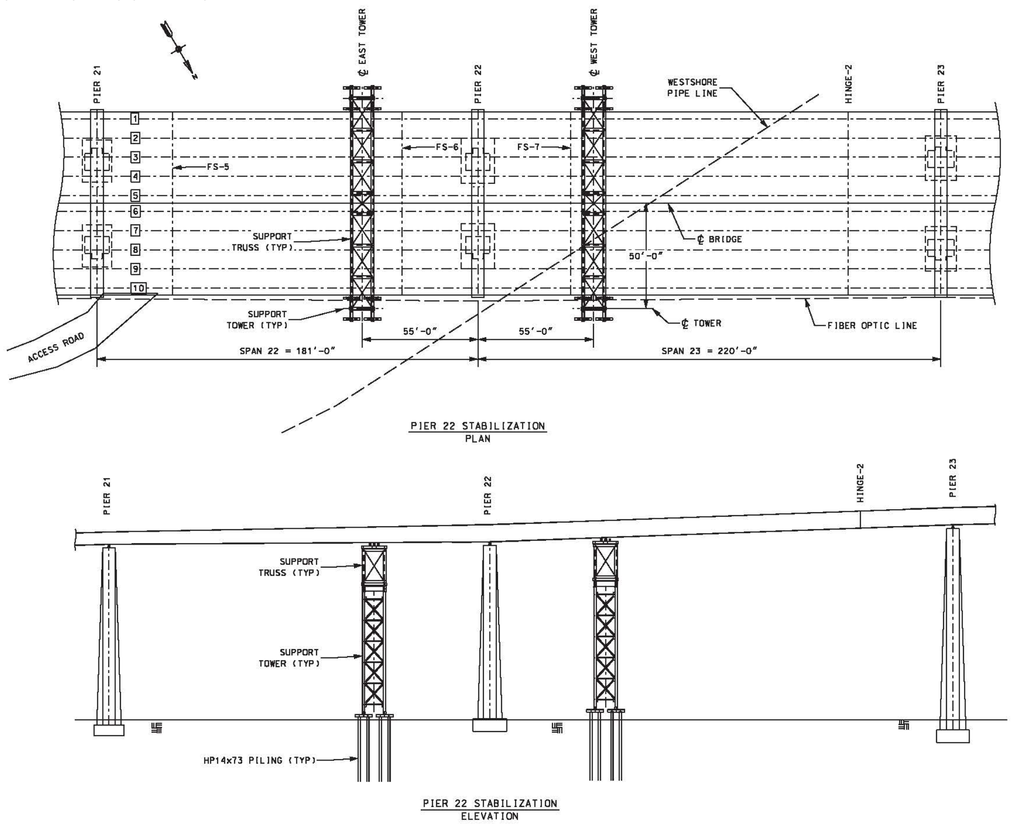


Figure 4.5: Plan and Elevation Views of Temporary Support

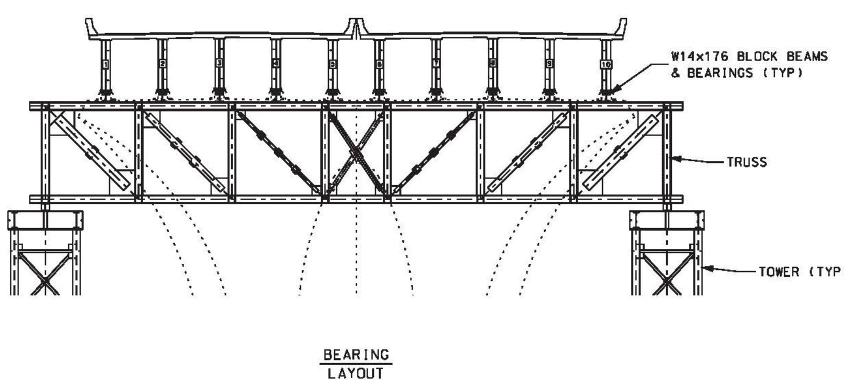


Figure 4.6: Cross Section of Temporary Support

4.6 Tiered Subsurface Setting

A project-specific tiered metric was employed to assist with categorizing the project site into regions with similar corrosion-related pier groupings. Three tiers were established, namely Tier 1, Tier 2, and Tier 3. The tiered metric was based on a combination of soil type, position of the perched water with respect to bottom of footer, minimum soil resistivity, and visual condition rating of exposed piling. A summary of pier-by-pier tier levels is presented in Table 5.

Tier 1 piers are locations where immediate repairs need to be performed in order to return the bridge to service. These locations are characterized by severe pile corrosion that has significantly reduced the bridge foundation's structural capacity. Tier 1 soils contain a toxic combination of porous industrial fly ash, perched water, low resistivity and significant ALWC. The Tier 1 foundation setting generally included substructures where steel piles are embedded in ash; **and** where the free water surface is below the bottom of footing; **and** where fine grained soil with a minimum resistivity of less than 300 ohm-cm exists near the piling; **and** the visual condition of the exposed piling ranked 1 to 4, out of a possible scale of 1 to 10, with 1 being the most severe and 10 being the least severe. The following piers are included in the Tier 1 setting: 21, 22, 23, and 25. It should be noted that, while Pier 24 did not exhibit all of the above attributes associated with the Tier 1 classification, based on its proximity to the critically corroded pier conditions, it has been conservatively included with the Tier 1 repairs.

Tier 2 piers are locations where, even though no evidence of severe pile corrosion was found, the soils are potentially more corrosive than normal. Tier 2 locations do not include large amounts of industrial fly ash or evidence of perched water levels below the bottom of the footings, but these piers exhibit some combination of organic material, abnormally high levels of chlorides, sulfates, or microbes; or low levels of resistivity. None of the Tier 2 locations represent an immediate safety concern, but the locations do deserve further investigation and inspection to determine if there is a future long-term reduction in the foundation's structural capacity. Tier 2 included two different criteria, both of which were combined to form a Tier 2 foundation setting. The first Tier 2 criteria generally included substructures where fibrous organics were encountered at test pits and/or test borings; **and** where fine grained soil with a minimum resistivity of less than 1,000 ohm-cm exists near the piling; **and** the visual condition of the exposed piling ranked 5 to 8; **and** where active microbes with > 1,000 colonies/ml were measured in proximity to the pier.

The second Tier 2 criteria generally included substructures where fibrous organics were encountered at test pits and/or test borings; **and** where fine grained soil with a minimum resistivity of less than 1,000 ohm-cm exists near the piling; **and** piers where no visual condition rating was performed; **and** where active microbes with > 1,000 colonies/ml were measured in proximity to the pier. The following piers are included in the Tier 2 setting: 6, 7, 8, 12, 13, 14, 18, 19, 20, 24, 31, 32, 39, 42, 44, 48, and 50.

The Tier 3 foundation setting included the remaining substructures, which did not qualify as Tiers 1 or 2. At these locations, from the data obtained, the soil condition and pile deterioration were considered more typical for a 32+ year old bridge.

4.7 Tier 1 Repairs

The extent of corrosion observed at the Tier 1 locations warranted a repair of the substructure units. The repair for Piers 21 through 25 is illustrated in Figure 4.7. The highly corrosive nature of the site was considered during design and a design life of 75 years was used for the new foundation components. Drilled shafts were placed into the underlying bedrock. The top portions of the drilled shafts contained a centrifugally-cast fiberglass-reinforced polymer mortar (CCFRPM) sleeve for added corrosion protection. The drilled shafts were designed to resist the entire structure load. Footing extensions were post tensioned to the existing footing, and concrete buttress walls were added to improve load transfer to the drilled shafts.

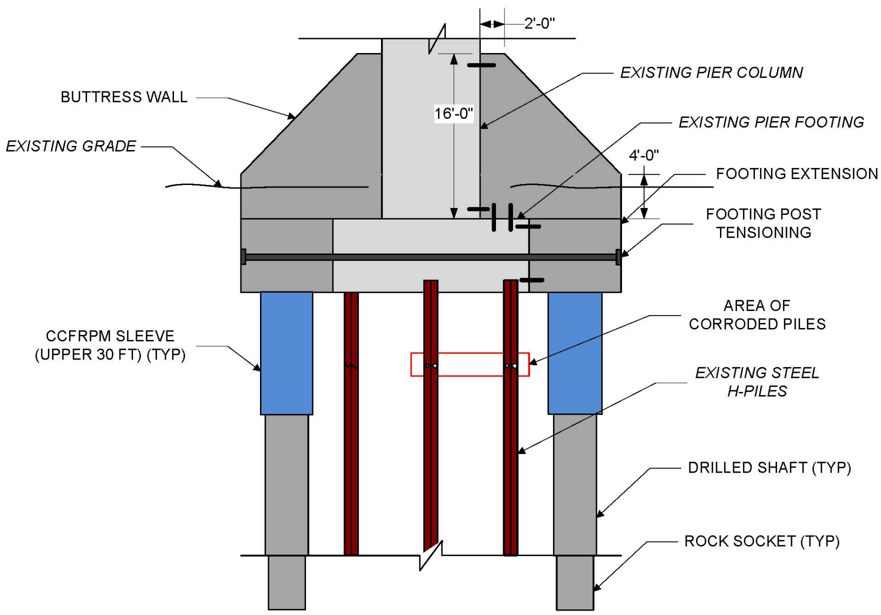


Figure 4.7: Tier 1 Pier Repair

4.8 Future Proposed Work at Pier 22

Work was discussed with WisDOT to further examine the extent of corrosion that had taken place at Pier 22. New drilled shaft foundations with reinforced caps have been installed and post-tensioned to the Pier 22 column footers to replace the load carrying capacity of the existing HP14x73 pile foundations. The opportunity exists to expose the corroded piling under the concrete footers to examine the condition of more piling than was examined under the 2013 investigation. This additional investigation at Pier 22 was completed in the summer of 2014.

Structural analyses were completed in 2013 to evaluate the feasibility of excavating to a depth of 12 feet beneath the bottom of concrete footers at Pier 22 to observe all of the exposed piling. There is a certain amount of uncertainty involved in the excavation of all of the soil from underneath Pier 22. It is unknown how much of the load is currently being carried by the steel piling and how much has been transferred to the drilled shafts. While the analysis performed demonstrated the adequacy of the drilled shafts to carry all of the structural loads in the final configuration, the sequence by which load will transfer from the piles to the drilled shafts cannot be known. In the unexcavated state, the existing H-piles are considered continuously braced. During the excavation, this bracing is removed, which will begin to reduce the pile load carrying resistance. As the unbraced length of the severely corroded H-piles increases due to excavation, the H-piles will continue to transfer load to the drilled shafts. This transfer may occur suddenly and without warning due to flexural or local buckling of the piles. The proposed excavation is acceptable structurally but does represent a safety concern to workers while performing the excavation.

In July 2014, an additional investigation of the pile conditions at Pier 22 was conducted by Baker and WDP. The investigation further examined the nature and extent of corrosion damage that had taken place at Pier 22. Inspection pits were excavated and dewatered to a depth of approximate 12 feet beneath the bottom of the concrete footing(s) at Pier 22 to expose piling for direct observation from within stacked trench boxes. These excavations and inspections provided an opportunity to better understand the failure mechanism that took place at Pier 22 and to better characterize the condition, nature and location of the failed pile sections. Following discussion with the Department, it was agreed to expose piling on the northerly and southerly face of both the north and south columns at Pier 22, and leave the soil under the column footers intact. An attempt was made to extract the corner pile (#14) at the southeasterly corner of the south column at Pier 22 (Appendix S Page S130) with an acetylene torch; however, the presence of moisture hampered the extraction process and efforts to remove a test section of Pile #14 were abandoned. A total of six failed piles were exposed during the additional exploration work, which reaffirmed the characteristic section loss, corrosion damage and buckled/crushed pile condition that was noted during the 2013 investigation at Pier 22.

4.9 Tier 2 Structural Pile Capacity Analysis

For the piers included in Tier 2, a structural analysis was performed on the existing HP14x73 pile section to determine the magnitude of section loss at which the existing pile capacity would no longer satisfy Operating Rating criteria in general conformance with the AASHTO Manual for Bridge Evaluation. The Operating Rating describes the maximum permissible live load to which the structure may be subjected. If a structure does not satisfy Operating Rating criteria, it is a candidate for a load posting limitation. Piers 12, 13, 14, 18, 19, 20, and 39 were analyzed as representative samples of piers with Tier 2 subsurface conditions. These locations were chosen based upon Piers 12-14 representing Tier 2 locations with high levels of microbial activity. Piers 18-20 are located adjacent to the Tier 1 locations and are among the taller Tier 2 piers, and Pier 39 represented the most advanced Tier 2 soil conditions on the west side of the Fox River. In general, the analysis determined that the piles would need to experience a 35 percent to 40 percent section loss before the Operating Rating criteria was not satisfied.

After it was determined that a 35 percent to 40 percent section loss would result in a potential load posting of the bridge, this section loss value was then used as the maximum allowable pile corrosion. The initial pile section loss value is taken as the maximum current field-measured section loss. An estimated future annual pile corrosion rate was then added to the current section loss. A pile’s total cumulative section loss at each year in the future can then be determined. The estimated future year when the cumulative section loss reaches the 35 percent to 40 percent value that would result in a load posting is then known. The pier’s remaining service life is then defined as the estimated number of years remaining until the bridge would need to be load posted.

The pile's Operating Rating considers the smaller of both the structural capacity and the geotechnical capacity. The structural capacity refers to the capacity of the steel pile itself while the geotechnical capacity refers to the capacity of the soil surrounding the pile. The lowest controlling capacity of the two is then used to determine the Operating Rating.

The FB-MultiPier program, which is a non-linear finite element analysis software that is commonly used to analyze bridge foundations, was used to perform a lateral soil-structure interaction analysis. The analysis considered the P-Y, T-Z, and Q-Z behaviors of the foundation. P-Y curves are a force-displacement analysis that is commonly used to determine the lateral capacity of a pile group, while T-Z curves model frictional pile behavior, and Q-Z curves model pile tip bearing behavior. This finite element analysis determined that the axial capacity was the controlling condition for the foundation as opposed to the lateral capacity. Maximum lateral deflection of the foundation was conservatively determined to be 1.3 inches for the controlling Service Limit State loading.

The Service Load Operating Rating capacity of an HP14x73 pile with no section loss was calculated to be 453 kips structurally and 375 kips geotechnically. The geotechnical resistance shown on the original bridge plans is 300 kips, which is taken as the Inventory Rating resistance. To determine the Operating Rating geotechnical resistance, this value was increased by 2.12/1.70, which is the ratio of Inventory Rating to Operating Rating factors of safety. The four structural pile deterioration cases that were investigated are described below:

1. Uniform Section Loss applied to all Piles: this refers to a uniform corrosion applied to the web and both flanges of all piles. Determine deterioration where Operating Rating is not satisfied at one pile.

2. Uniform Section Loss to all Piles: this refers to a uniform corrosion applied to the web and both flanges of all piles. Determine deterioration where Operating Rating is not satisfied for the entire pile group after load redistribution.

3. Non-Uniform Section Loss to the most heavily loaded pile: this refers to a uniform corrosion of the pile flanges only with no web corrosion. Determine deterioration where Operating Rating for structural capacity is not satisfied at one pile. The effects of localized instability of the flange are considered in determining the pile's axial resistance.

4. Non-Uniform Section Loss to the most heavily loaded pile: this refers to a uniform corrosion of the pile flanges only with no web corrosion. Determine deterioration where Operating Rating for structural capacity is not satisfied for the entire pile group after load redistribution. The effects of localized instability of the flange are considered in determining the pile's axial resistance.

Case 1 examines applying increasing uniform corrosion to the web and both flanges of all piles, simultaneously, within a substructure footing until the operating rating is not satisfied at the most heavily loaded pile. The analysis determined that piles in Case 1 typically exceeded Operating Rating capacities when approximately 35 percent to 45 percent uniform section loss over the entire pile has occurred.

Case 2 examines applying increasing corrosion to the web and both flanges of all piles, simultaneously, within a substructure footing. When the load at the most heavily loaded pile exceeds a pile's operating capacity due to section loss, all future loads are then redistributed to the remaining piles within the group. This first pile retains 100% of the load up to its operating capacity and then receives no additional deterioration or loading. An additional case was investigated with the first pile retains 50% of the load up to its operating capacity and then received no additional deterioration or loading. The remaining piles then continue to experience deterioration. The pile retaining 100% of its load prior to redistribution was chosen to be similar to other AASHTO strength limit state resistance theories. The pile retaining only 50% of its load prior to redistribution was chosen as a conservative case to represent a situation of additional load shedding. Following this procedure, a uniform section loss over all piles of 35 percent to 45 percent results in one pile not satisfying operating capacity, just like in Case 1. When all additional loads are then redistributed to the pile group, an additional two to four piles could reach their operating capacity before the entire pile group does not satisfy its Operating Rating capacity.

Case 3 examines applying increasing uniform corrosion to both flanges of all piles, simultaneously, within a substructure footing until the operating rating is no longer satisfied at the most heavily loaded pile. The pile's structural capacity decreases as the pile's flange thickness decreases due to section loss. The analysis determined that local flange section losses ranging from 34 percent to 44 percent result in Operating Ratings not being satisfied.

Case 4 examines applying increasing uniform corrosion to both flanges of the most heavily loaded pile only within a substructure footing. The pile's structural capacity decreases while reducing flange thicknesses from corrosion, similar to Case 3, but then allows for load redistribution to the remaining pile group after the operating capacity is exceeded at one pile, similar to Case 2. After the most heavily loaded pile exceeds the Operating capacity, the pile receives no additional load or deterioration. The second most heavily loaded pile then received increasing uniform corrosion to both flanges until its Operating Rating is exceeded, and so on until the redistributed load causes all piles to exceed their Operating Rating. Similar to Case 2, cases were investigated where the original pile retained both 50% and 100% of its load prior to redistribution. Local flange section losses ranging from 34 percent to 44 percent result in Operating Ratings not being satisfied for one pile, just like in Case 3. When all additional loads are then redistributed to the remaining pile group, an additional two to three piles would exceed operating capacity before the entire pile group does not satisfy its Operating Rating capacity.

These calculated section losses, which result in piles exceeding Operating Rating limits, are then compared to current estimated section loss combined with estimated future corrosion rates to assess the remaining service life of the piers. This process determined that the Tier 2 piles will not satisfy Operating Rating criteria in approximately 20 years. To arrive at that conclusion, assumptions were made based on the data obtained for the current corrosion present on the piles as well as the future pile deterioration rate.

Field investigation data was used to determine initial localized section loss resulting from pitting. In the limited number of Tier 2 piles exposed during the field investigations, the two localized flange areas of maximum section loss observed were 5 x 0.088 inches and 3 x 0.181 inches. A schematic diagram and photographs are included in Figure 4.8. These maximum values were taken as two scenarios for the current pile localized flange section loss. Various future local corrosion rates of between 3 and 10 mils per year (0.003 to 0.010 inches per year) were then applied to the local section loss based upon historical deterioration data for steel in corrosive environments.

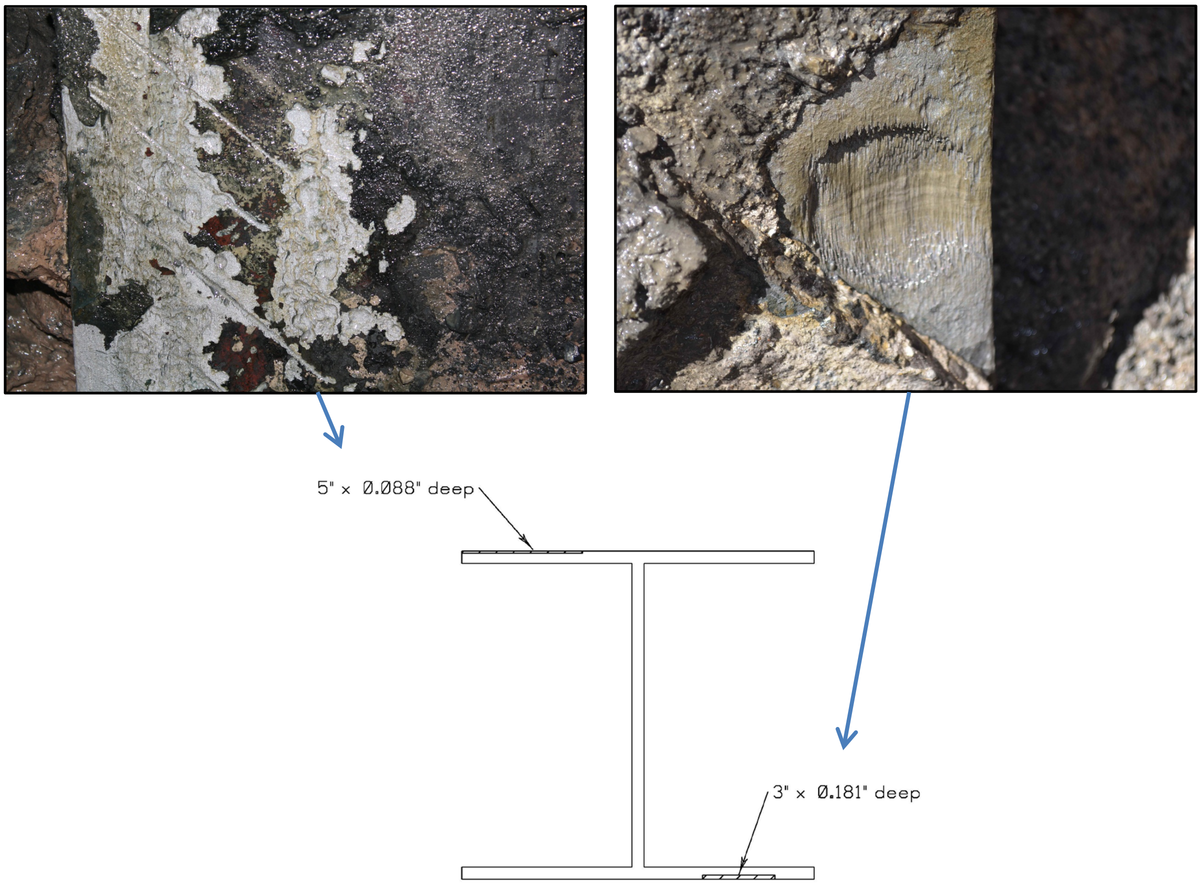


Figure 4.8: Observed Corrosion and Initial Pile Section Loss Illustration

In addition to the observed local section loss, an initial uniform baseline section loss over the entire pile section was taken as 5 percent based upon the field investigation data. Over the original 35 years of the structure life, this represents a baseline overall uniform section corrosion rate of 0.14 percent per year. The two corrosion values are added together to determine the overall corrosion on the section at various future dates.

It was determined that it would take approximately 20 years of 10 mil local corrosion rate per year and 0.14 percent baseline corrosion to result in a flange section loss of 35 percent, which would result in Operating Rating criteria to not be satisfied. To provide perspective for the same flange section loss of 35 percent, it is estimated that it would take 30 to 40 years of 3 mil local corrosion per year for the Operating Rating Criteria to not be satisfied. It was determined that the Case 1 & 2 uniform section loss criteria did not govern the remaining service life, and that the localized flange section loss in Cases 3 & 4 was the controlling criteria.

For simplicity, a linear average corrosion rate was considered for these determinations. The actual corrosion rate is not linear. The results from the installation of the inspection probes will provide information about the actual corrosion rate over time.

4.10 2014 Investigation at Tier 2 Piers

The accuracy of predicted corrosion rates for this site is limited due to the lack of data points. Thus, consideration has been given to installing devices to measure the existing corrosion rate and provide more data points to improve prediction rates about future corrosion and remaining service life for the steel piling. Once more accurate corrosion rates and future remaining service life are determined, potential remedies, including cathodic protection or modification of the environmental setting, could be implemented to extend the service life of the structure.

4.10.1 Inspection Probes

The corrosion damage observed at the Leo Frigo Memorial Bridge is primarily due to ALWC, which is driven by localized corrosion activity.

In order to establish more data points about the apparent corrosion rate at representative pier locations and to reasonably characterize the nature and extent of the conditions and type of corrosion occurring at those representative pier locations, two inspection methodologies have been considered for the Leo Frigo bridge. The first methodology employs steel coupons that are embedded in the soil and extracted after a few years to assess the degree and extent of corrosion that has taken place on the exposed steel coupon. The second methodology employs buried corrosion sensors that can detect the corrosion mechanisms and provide timely information regarding in-situ corrosion rate.

The following paragraphs provide a brief description of the inspection methodologies that have been used for the Leo Frigo Memorial Bridge.

4.10.1.1 *Steel Coupons*

Low-carbon steel coupons that mimic the electrochemical behavior of the in-service piles have been installed to assess the corrosion rate and detect the mechanisms driving the corrosion process. Sets of three new 20-foot long HP10x42 steel coupons were driven in-place in relative proximity to each instrumented pier. Priority was given to choosing locations that had not been previously disturbed during the investigation process. Prior to placement in the soil, the steel coupons were imprinted with registration marks and cross-sectional dimensions of each steel coupon were measured at 1-foot incremental depths along 4 separate reference lines (1 line on each side of coupon web on both flanges, for a total of 4 lines) using ultrasonic (UT) thickness measurement equipment. The UT thickness measurements provided baseline data for subsequent use to determine section loss and calculate corrosion rate, when the steel coupons are extracted and inspected at a later point in time. Installation data is presented in Appendix V.

After five years from initial placement, one steel coupon at each instrumented pier will be extracted and inspected to assess the extent of corrosion damage. The extracted steel coupons will be inspected and the registration marks will be used to reestablish baseline locations along the length of each steel coupon. The baseline locations will be used to identify locations where the thickness of each steel coupon was originally measured, so that follow-up UT thickness measurements can be obtained and used to compute thickness loss, and hence determine the apparent corrosion rate that has taken place over the period that the steel coupon was buried in the ground. The difference between the baseline thickness(es) and the thickness(es) measured after five

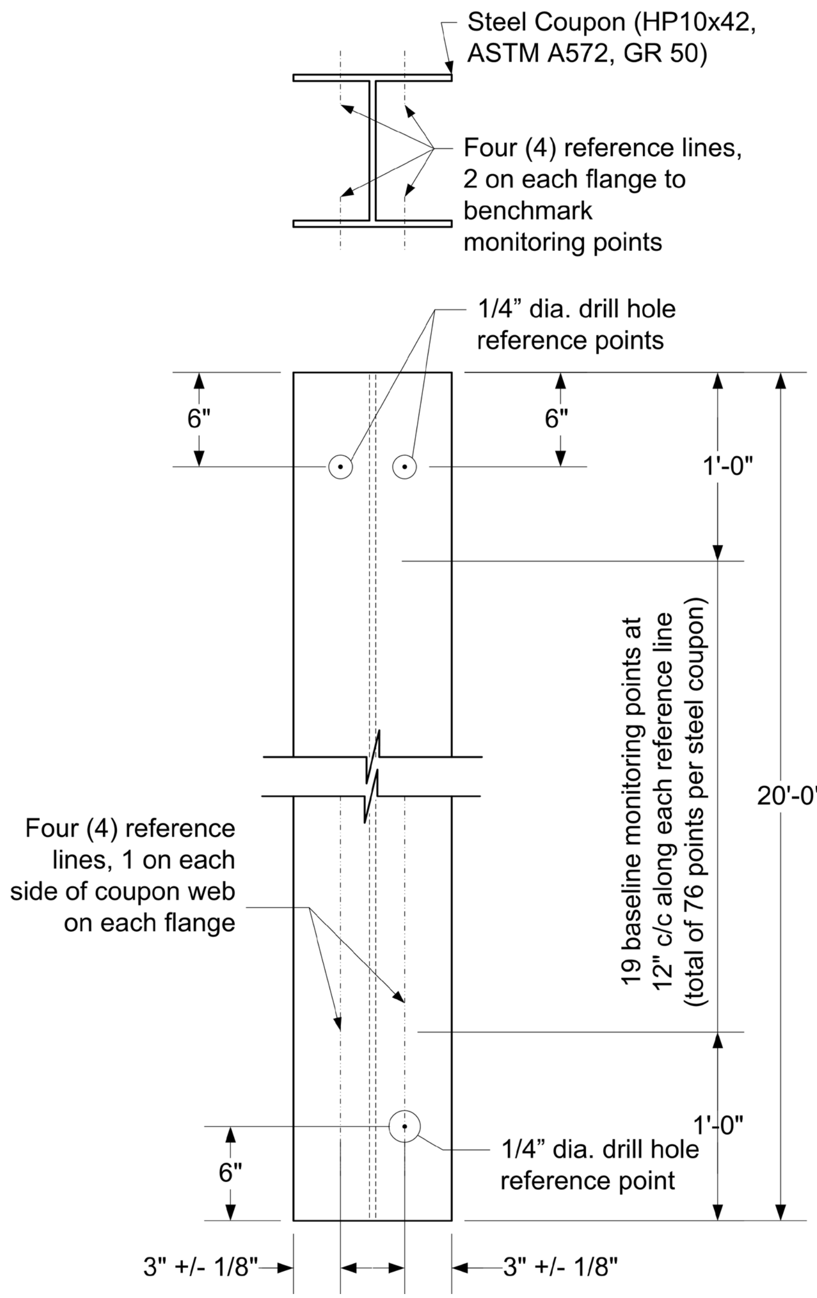


Figure 4.9: Steel Coupons

years of exposure will indicate the corrosion rates at different depths and can be used to assess the condition of the in-service piles. The remaining second and third steel coupons will be extracted after 10 and 15 years, respectively. The additional coupons will provide critical information regarding the change in corrosion rate over time. After each steel coupon is inspected and measured to determine section loss, the extracted coupons may be driven back into place for possible future use, at locations that were not previously disturbed during the subject investigation/inspection. Following discussion with the Department about the potential advantages and disadvantages of reusing steel coupons due to extraction disturbance, it was agreed that the benefits of reinstalling the coupons for possible future reuse outweighed the possible disadvantages that the disturbance would cause.

4.10.1.2 *Corrosion Sensors*

While the steel coupon methodology provides the most direct measurement of corrosion rate and are expected to capture the corrosion mechanism/type that is at work at each instrumented pier location, several years or more of incremental inspection is required to discern a predictable trend in the apparent corrosion rate. Hence, a second methodology was crafted to provide supplemental data about corrosion loss at buried sensors to collect real-time data, especially until such time that steel coupons are exhumed. The sensors were installed in July 2014 during the Tier 2 investigation.

The corrosion observed at the Leo Frigo Memorial Bridge is a result of a number of corrosion mechanisms. Hence, the inspection scheme incorporated a suite of different sensors enabling an assessment of the corrosion activity. Based on a literature review, a monitoring suite that consists of half-cell electrodes and electrical resistance (ER) probes at each point of measurement was chosen to can provide an assessment of the corrosion state. Both technologies are relatively inexpensive and have shown great success in field applications. When used together, the corrosion risk and corrosion rates can be evaluated.

The half-cell and ER probes required test pits to bury the probes at multiple depths to identify and monitor zones where corrosion is the most aggressive. The major advantage of the half-cell and ER probes is the rate and frequency at which corrosion inspection can be performed. This will permit the opportunity to evaluate and improve predictions about the ongoing corrosion rate.

The following paragraphs provide a brief summary of the envisioned system.

Half-Cell Electrodes: The half-cell method is an invaluable and cost-effective technique for identifying the corrosion risk within the soil environment. Half-cell potential is an electrochemical measure that quantifies the ease of removing electrons from a metal surface. Hence, the half-cell method measures the willingness not the rate of a steel member to corrode.

Figure 4.10: Half-Cell Electrode

In order to measure the corrosion potential of a steel member embedded in soil, the steel member is electrically connected to a high-impedance DC voltmeter. The voltmeter is also attached to a half-cell electrode. The reference electrode is installed generally perpendicular to the existing HP14x73 pile flange and 1 to 4 inches away from the edge of flange. Sections of the H-pile that are passive generally exhibit more positive voltage readings. More negative potentials are recorded in regions where active corrosion occurs.

Research and practical experience have shown that the assessment of the corrosion risk is best achieved by examining the spatial variation of the corrosion potentials rather than sole use of the absolute potential values. Hence, multiple silver-silver chloride half-cell electrodes were embedded at variable depths to assess the corrosion activity in a pile. The method can be used to successfully detect the formation of a macrocell mechanism. Cathodic regions exhibit half-cell potential values that are significantly more noble (positive) than those recorded in the anodic regions. During the investigation, half-cell potential differences of more than 250 mV were recorded at Pier 25, which exhibited extensive corrosion damage due to macrocell activity. Exposed portions of inspected piling at Leo Frigo that exhibited limited corrosion damage had potential differences of less than 50 mV.

Electrical Resistance (ER) Probes: ER probes are widely used in the gas and petrochemical industry to monitor corrosion rate for underground pipelines. ER probes measure the increase in electrical resistivity of a sacrificial metal strip. The increase in resistance is attributed to the corrosion-induced cross-sectional loss of the sacrificial strip. The mass loss of the sacrificial strip can be evaluated using the ER measurements, and the corrosion rate can then be evaluated by detecting the change in strip mass over time.

The sacrificial metal strip is fabricated from a material that exhibits electrochemical properties that are similar to the pile material. Furthermore, the ER probe is intended to be installed close to the existing H-pile in a similar soil/chemical environment. The design intent is to expose the sacrificial metal to a corrosive condition that is similar to that affecting the existing steel H-piles. Hence, changes in the response of the ER probe(s) can be used to assess the apparent corrosion rate.

Figure 4.11: Electrical Resistance (ER) Probe and Handheld Data Logger

The ER probes are point sensors that can provide information regarding the corrosion rate at the location of embedment. Due to the stratified nature of the soil conditions at Leo Frigo, ER probes were installed at multiple depths at each instrumented pier location. Furthermore, at locations where the soil conditions can promote macrocell corrosion, duplicate ER probes were installed at the cathode and anode locations. The duplicate probes were linked with electrical wire and used to assess the increase in corrosion rate due to macrocell activity.

Schematic of Main Sensor Components. For each instrumented pier location, the sensors were embedded at multiple depths in undisturbed soil. To this end, soil was removed from the vicinity of the pier without exposing the piles being monitored. Pilot holes were augered and sensors were pushed into place at the appropriate depth. Signal cable from each half-cell and ER probe was routed to a common instrumentation cabinet at each instrumented pier. PVC electrical duct was installed to protect the signal cable from physical damage during installation and backfilling of the test pit. After the signal cable was installed, the electrical conduit was filled with cement grout to restrict possible seepage of surface water along the conduit length.

Instrumented Pier Locations and Inspection Frequency. Probe maintenance is typically low and the majority of the probe cost is associated with excavation cost to install the probes. Inspection is straight forward and Department staff was trained to obtain periodic measurements. It is anticipated that inspection frequency would initially be of shorter duration, for the first few years, to establish a trend in corrosion rate. Inspection frequency would then continue at a longer duration, such as semi-annually for the next year or two and then to every two years, so that it would become part of the NBIS bridge inspection program. Half-cell and ER probes are widely accepted and used in the corrosion field. The probes were installed at representative locations among the Tier 2 piers. The following piers were selected to get a representative sample of the rates of corrosion throughout the project site: Piers 6, 13, 19, 32, 39, 44, and 50. Table 5 shows the piers that were categorized as Tier 2. A pier from each cluster was selected as the representative pier for that group. For example, Piers 6, 7, and 8 are three adjacent Tier 2 piers. Pier 6 was selected for the probe location to represent this group. Pier 6 was selected in order to keep the probe off of railroad right-of-way. The following table illustrates the groupings and representative pier.

Tier 2 Pier Number	Representative Pier for Probe Installation	Remarks
6 7 8	6	Representative Tier 2 pier between the East Abutment & N. Quincy St. Establish corrosion rate for the easterly end of Leo Frigo Bridge & establish what is expected to be a lower limit for the measured corrosion rate.
12 13 14	13	Representative Tier 2 pier between N. Quincy St. & Fox River. Maximum soil chloride (Cl) & sulfate (SO4) concentrations > 1,000 mg/kg. Acid producing bacteria (APB) & sulfate reducing bacteria (SRB) > 1,000 colonies/mL.
18 19 20	19	Representative Tier 2 pier between N. Quincy St. & Fox River. Cl & SO4 concentrations > 1,000 mg/kg & < 700 mg/kg, respectively. APB & SRB > 1,000 colonies/mL.
31 32	32	Representative Tier 2 pier between Fox River & Bylsby Avenue. Cl & SO4 concentrations > 10,000 mg/kg & < 700 mg/kg, respectively. APB > 1,000 colonies/mL, SRB < 100 colonies/mL.
39	39	Representative Tier 2 pier between Fox River & Bylsby Avenue. Cl & SO4 concentrations > 10,000 mg/kg & > 1,000 mg/kg, respectively. APB & SRB > 1,000 colonies/mL.

42	44	Representative Tier 2 pier between Bylsby Avenue & the West Abutment. Cl & SO4 concentrations > 700. APB > 1,000 colonies/mL, SRB < 100 colonies/mL.
44		
48	50	Representative Tier 2 pier near the West Abutment. Provide corrosion rate data for the westerly end of the Leo Frigo bridge.
50		

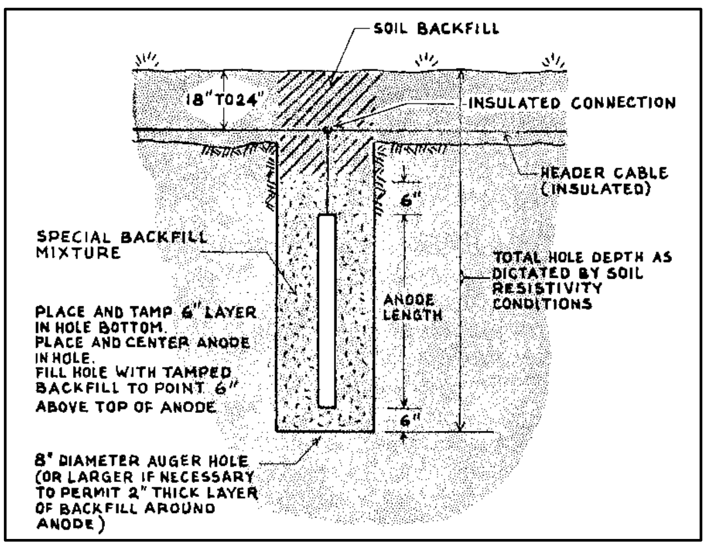
To interpret instrumented data at piers where instrument probes are installed, subsurface conditions were characterized. The characterization included laboratory tests to classify the physical and chemical characteristics of the actual soil within which the instruments were installed. The physical tests included soil moisture, gradation, plasticity and organic content determinations. Chemical analyses included determination of pH, soil resistivity, chloride content and sulfate content determinations.

The measured rates of corrosion can be compared to the values assumed as part of the analysis and used to determine if preventative maintenance will be required during the expected life of the structure.

Cathodic protection and modification of the environmental setting through jet grouting have been identified as viable alternatives to reduce the rate of corrosion at the site.

4.10.2 Cathodic Protection

Cathodic protection (CP) has been considered to reduce the rate of corrosion at the existing steel H-piles. Both impressed current and passive anode CP system types are available. To reduce future maintenance demand, the passive anode CP system type is preferred for the project site. A ring of passive deep sacrificial anodes would be installed to encircle each designated pile-supported footer. The anodes would create a dissimilar metal corrosion cell with respect to the piling, such that the anodes would corrode and render the piling cathodic to the externally placed anodes. The more active anodes would corrode, in lieu of the piling, and discharge current in the process. Over a period of time, the anodes would require replacement. The number and spacing of anodes can be adjusted to minimize the replacement frequency. When new anodes are required, a new ring of anodes could be installed outside the existing ring.



The CP system will slow down the corrosion rate but will not completely eliminate the corrosion. This treatment is not effective in reducing other forms of corrosion, such as macro cell corrosion. There is no information available to predict the corrosion rate post-CP other than that the corrosion rate would be expected to be significantly reduced. It is recommended to use an 80 percent reduction in the expected corrosion rate after the CP application at locations other than at piers where there is a concentration of shredded wood in direct contact with the piling; and it is recommended to use a 60 percent reduction in the expected corrosion rate after the CP application at locations where there is a concentration of shredded wood in direct contact with the piling. The inspection probes are recommended to assist with collecting data to monitor the future corrosion rate.

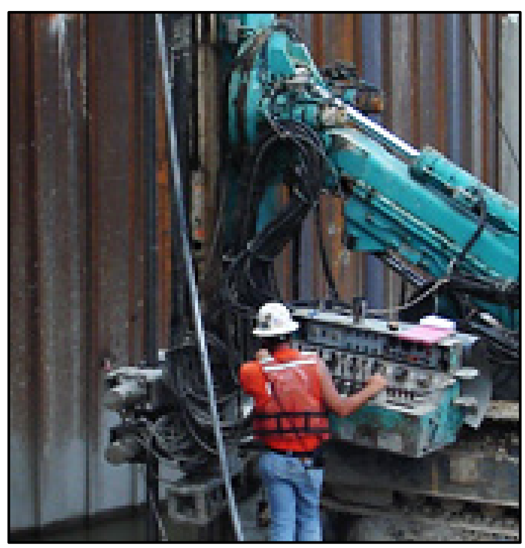
Current from the CP system will be diminished at localized anomalies where the shredded wood is in direct contact with the steel piling. Where the shredded wood is in direct contact with the steel piling, the covered portion of piling that is directly under the shredded wood will continue to be oxygen deprived, which will continue to promote the development of a microcell that can accelerate localized pitting. Hence, where there is a propensity of fibrous organic matter, such as at areas with a propensity of shredded wood, jet grouting would be required to supplement cathodic protection to reduce the rate of corrosion.

Cathodic protection of buried structures is a mature field and is well documented. Cathodic protection is used in several states to extend the serviceable life of pile-supported bridges, including but not limited to Florida, Virginia, New York, and California. The estimated cost to install the cathodic protection system is estimated to be approximately $50,000 per pier.

4.10.3 Modification of Environmental Setting

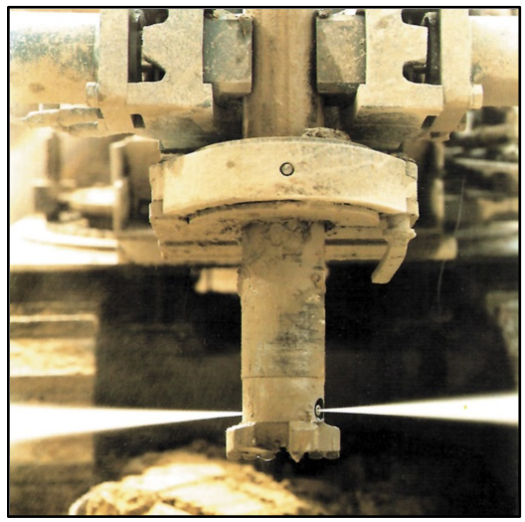
Consideration has been given to modifying the environmental setting to slow down the microbial activity and thereby reduce the perpetuation of ALWC. Primary considerations to modifying the environmental setting have been to remove fibrous organic matter in proximity to the steel H-piling, increase the pH of the soil, and inject cementious material in and around the piling within the upper 10 feet of soil beneath pile supported footings.

Modification of the environmental setting will aid in isolating fibrous organic matter, like shredded wood, and in separating fibrous organic matter from direct contact with the steel piling. Where the shredded wood is in direct contact with the steel piling, the fibrous wood creates a local anomaly that is oxygen deprived that promotes the development of a microcell that can accelerate localized pitting. Evidence of this was observed at exposed piling in the test pits where an impression of shredded wood had been imprinted on corroded piling.



Consideration has also been given to constructability and challenges associated with removal/replacement of organic laden soil under the existing pile supported footers. Based on discussion with contractors and concerns raised about the logistics required to remove/replace material under the pile-supported piers with recognition that the existing bridge is in service, it was deemed more desirable to seek other options to modify the environmental setting.

Further study brought forth the option to jet grout the existing soil beneath pile-supported footings in-place. The primary advantages of jet grouting include increasing the pH of the soil, which is anticipated to diminish microbial activity; encasing fibrous organic matter to reduce available organic nutrients, which will also help to curtail microbial activity; and injecting a cementious grout that will help to encase and protect the steel piling. Jet grouting will also permit workers to operate equipment from beside rather than under pile supported footings. The disadvantage is that some of the fibrous organic material will remain, although encased in cement grout. A temporary excavation is anticipated to be required to a depth on the order 10 to 15 feet below the bottom of pile footer to allow the operation of horizontal drilling equipment. Sump pumps are anticipated to be required to dewater perched water within the temporary excavation. Temporary sheeting would be used to form a shield at the edge of footer to contain the liquefied soil-cement mass. The sheeting should be installed with a bond breaker so that the temporary sheeting could be retrieved and reused at other designated pier locations for similar treatment. Horizontal holes would be drilled at approximately 4-foot spacing and used to inject a cementious grout under high pressure to liquefy and mix with the surrounding soil to produce a soil-cement mass directly under the pile-supported footer.



The jet grouting will slow down the MIC corrosion rate, but it will not completely eliminate the corrosion. This treatment is not effective in reducing other forms of corrosion, such as macro cell corrosion. There is no information available to predict the corrosion rate post-jet grouting other than that the corrosion rate would be expected to be diminished. At locations where there is a concentration of shredded wood in direct contact with the piling, and where jet grouting is used in combination with cathodic protection, it is recommended to use a 70 percent reduction in the expected corrosion rates. The inspection probes are recommended to assist with collecting data to monitor the future corrosion rate.

Although jet grouting is a mature technology that has been used in the United States for more than 30 years, and longer abroad, its use to mitigate ALWC beneath pile-supported footings is an unconventional, creative solution that meets the project challenges. The estimated cost for the jet grouting mitigation combined with cathodic protection is estimated to be on the order of $350,000 per pier.

4.10.4 Additional Testing

Additional testing is recommended to diminish data gaps and reduce spatial variability, to refine understanding quantitatively about the distribution/variability of corrosion-related parameters. The additional data will improve clarity about the degree and extent of corrosion at the project site.

The existing data is a product of an adaptive investigation that has been focused on identifying solutions to restore bridge serviceability. Methodology has been adapted to primarily support efforts to restore bridge serviceability. For instance, data acquisition to-date has been focused on providing solutions in proximity to the Tier 1 piers, namely Piers 10 through 25 on the east side of the Fox River, where the spatial density of the data obtained is generally greatest. The spatial density over the remainder of the bridge is less. At the river piers, 26 through 29, the spatial density of the chemical analyses is high but no MIC diagnostic test data exists since test pitting was not performed in the river to collect samples of corrosion product for diagnostic testing. On the west side of the Fox River, test pitting has been performed to expose piling at a total of five out of 22 piers, and as such the spatial density to sample corrosion product is substantially less than that completed to date on the east side of the Fox River.

Representative samples were collected, especially on the west side of the Fox River, when the inspection probes were installed at Tier 2 piers. At the Tier 2 inspection probe locations (tabulated in Section 4.10.1.2), the surrounding soil, ground water and pile condition was classified to characterize the environment in proximity to where the steel coupons and inspection probes were installed. Data obtained at the instrumented Tier 2 piers assisted to diminish data gaps and reduce spatial variability. Data obtained at the instrumented Tier 2 piers generally included detailed documentation of the exposed stratigraphy, footing and H-piling, in-situ soil resistivity and half-cell potential measurements, representative sampling of soil, water and corrosion product for subsequent physical testing and chemical analyses in the laboratory. Chemical analyses generally included pH, soil resistivity, chloride content and sulfate content determinations.

Other than at the instrumented Tier 2 piers, no additional testing is planned to fill-in data gaps. Regardless, the additional testing that is planned will help to reduce data gaps as noted below.

Location		Recommended Pier Locations Where Additional Samples and Testing will Contribute to Diminish Data Gaps and Reduce Spatial Variability	Remark
East Side of Fox River	East Abutment to Pier 4	---	Tier 3 Piers
	Piers 6 to 8	Pier 6	Pier 6 (Tier 2) at location of recommended inspection probe installation
	Pier 22	---	In conjunction with discussion in Section 4.8
River Piers	Piers 26 to 29	---	River piers with submerged footers well below normal pool
West Side of Fox River	Pier30 to 32	---	Pier 32 at location of recommended inspection probe installation
	Piers 33 to 38	---	Tier 3 Piers
	Piers 40 to 42	---	Bounded by data that has been collected to-date at Piers 39 and 43
	Piers 45 to 47	---	Tier 3 Piers
	Piers 48 to West Abutment	Pier 50	Pier 50 at location of recommended inspection probe installation

5.0 Conclusions and Recommendations

The investigation determined that several factors contributed to a highly unusual environment that caused the severe corrosion of the steel pile foundation supporting Pier 22. An unusual corrosive environment was created by the presence of a moist, porous fly ash fill with high levels of chlorides and sulfides combined with low resistivity. A dense clay layer was present below the porous fly ash, leading to differential oxygen concentration and differential chemical concentration within the fill layers. Bacteria were found at many of the piers and it is likely that microbiologically influenced corrosion also played a role in the corrosion at Pier 22. An aggressive corrosion mechanism of differential oxygen concentration corrosion in the form of Accelerated Low Water Corrosion (ALWC) is likely responsible for the deterioration in the localized failure zone at Pier 22.

These site conditions led to rapid corrosion of sections of the steel piles. Once sufficient material and support were lost from enough piles, the remaining piling became unstable. Visual examination of selected Pier 22 piles indicated that the most common, perhaps only, mode of pile instability was crushing/buckling of the most heavily deteriorated sections of pile. Severe deterioration and crushing/buckling was observed on all piles that were exposed at this pier.

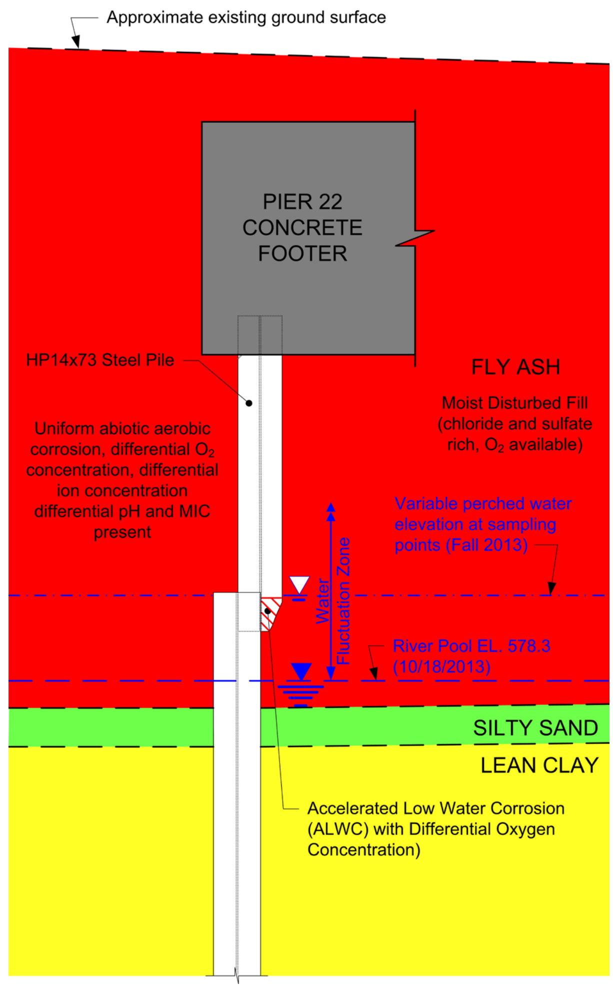


Figure 5.1: Pier 22 Summary Graphic

Visual inspection and soil borings confirmed the presence of fly ash surrounding severely corroded steel piles at Piers 21, 22, 23 and 25. While the piles observed at Pier 24 were substantially less corroded, Pier 24 was conservatively treated similar to Piers 21, 22, 23 and 25 based on its proximity to those critically corroded pier locations. Therefore, repairs were performed at Piers 21 through 25 prior to returning the bridge to service. Tier 1 repairs included installation of new concrete drilled shaft foundations, which are capable of supporting the entire pier design load. These new foundations were then connected to the existing piers and provided corrosion protection measures designed to offer 75 years of service life.

Based on the results of the structural analysis, there is no near-term structural concern at the substructures in Tier 2. A long term inspection program was recommended and is now in progress to record the apparent corrosion rates at representative piers along the length of the structure. This program involves the installation of Electrical Resistance (ER) probes to measure corrosion rates at representative Piers 6, 13, 19, 32, 39, 44, and 50 for comparison with predicted corrosion rates, and then to later implement preventive treatments if required. At each of these pier locations, test pits were dug to install inspection probes in the surficial fill and in the deeper natural cohesive soil. Testing at the time of installation included baseline readings and characterization (physical and chemical) of the material type and chemistry at the ER probe locations.

Inspection and recording of the results for the corrosion monitoring instrumentation should occur at a minimum frequency of every two years to coincide with the standard bridge inspection program. However, increased inspection frequency (such as twice per year) will provide additional data points that will aid in the projection of future corrosion. Recorded corrosion rates should be compared to the rates projected as part of this study for the Department's evaluation of future maintenance activity.

Past bridge drainage has contributed to elevate the chloride content of the in-situ soil under the bridge, which has contributed to create an electrolyte-rich subsurface condition that can promote accelerated corrosion. Although, in principle, diversion of salt-laden drainage from bridge scuppers is potentially beneficial, the potential for establishing significant corrosion macrocells at pier locations that are fully submerged is minimal. Diversion of bridge scupper drainage nor filling-in water-filled depressions next to the piers at this point in time are not expected to result in a significant reduction in the chloride content in the existing soil under the bridge that is sufficient to make a substantial reduction in the potential corrosivity of the existing surficial soil.

6.0 Closing

Opinions expressed in this report are based on the results of field explorations, office and laboratory investigation, and engineering judgment. The test pits and test borings depict soil, rock, and groundwater conditions encountered at the specific locations and time during which they were performed. The soil, rock, and groundwater conditions at other locations on the site may differ from those occurring at the test pit and boring locations. Conditions that vary significantly from those described should be brought to the attention of Baker so that a determination can be made as to whether any changes need to be made to the conclusions and recommendations.

The professional services have been performed, findings obtained, and recommendations prepared in accordance with generally accepted geotechnical engineering principles and practice. No other warranty, expressed or implied, is made as to the professional advice included in this report. Michael Baker Jr., Inc. is not responsible for the conclusions, opinions, and recommendations made by others based upon the data included herein.

Tables

Table 1- Summary of Subsurface Explorations

Designation					Depth Below Existing Ground Surface to Base of Apparent Surficial Fill (feet)		
Substructure	Test Pit	Shallow Auger Boring	Deep Boring	Sampling Point	Test Pit	Shallow Auger Boring	Deep Boring
East Abutment		B-AE				38	
Pier 1		B-1				0	
Pier 2		B-2		MW-2		2	
Pier 3		B-3				3	
Pier 4		B-4		MW-4		0	
Pier 5	TP-5NE	B-5			0	2	
Pier 6	TP-6SW	B-6		MW-6	4.7	0	
Pier 7		B-7				5.5	
Pier 8		B-8		MW-8		7.5	
Pier 9		B-9		MW-9		0	
Pier 10	TP-10S	B-10		MW-10	12.3	5	
Pier 11	TP-11S				9.4		
Pier 12	TP-12S	B-12		MW-12	4.0	0	
Pier 13	TP-13NE				6.0		
Pier 13	TP-13S				7.2		
Pier 13	TP-13SE				8.8		
Pier 14	TP-14S	B-14		MW-14	5.5	4	
Pier 15	TP-15S				7.7		
Pier 16	TP-16SE	B-16		MW-16S	7.1	7.5	
Pier 17	TP-17S				5.5		
Pier 18	TP-18SE	B-18		MW-18S	10.4	8.5	
Pier 19	TP-19NE	B-19A			7.9	7.5	
Pier 19	TP-19NW	B-19B			7.3	5.5	
Pier 19	TP-19S				6.7		
Pier 19	TP-19SE				8.6		
Pier 19	TP-19SW				7.2		
Pier 19	TP-19SW-A				7.3		
Pier 20	TP-20NW	B-20		MW-20N	6.0	5	
Pier 20	TP-20S				5.3		
Pier 20	TP-20SE				3.5		
Pier 20	TP-20SW				5.0		
Pier 21	TP-21NW				> 14.0		
Pier 21	TP-21S				> 12.1		

Table 1- Summary of Subsurface Exploration (continued)

Designation					Depth Below Existing Ground Surface to Base of Apparent Surficial Fill (feet)		
Substructure	Test Pit	Shallow Auger Boring	Deep Boring	Sampling Point	Test Pit	Shallow Auger Boring	Deep Boring
Pier 22	TP-22N			MW-22N	16.5		
Pier 22	TP-22S	B-22A			> 11.0	16	
Pier 22	TP-22SW		B-22B		17.0		19
Pier 22	TP-22SSE				15.9		
Pier 22	TP-22SNW				15.9		
Pier 22	TP-22SNE				15.9		
Pier 22	TP-22NSW				16.5		
Pier 22	TP-22NSE				16.5		
Pier 23	TP-23S	B-23		MW-23N	8.6	11	
Pier 24	TP-24NE				8.5		
Pier 24	TP-24NW				6.0		
Pier 24	TP-24S				7.6		
Pier 24	TP-24SE				9.1		
Pier 24	TP-24SW				9.4		
Pier 25	TP-25NE	B-25		MW-25S	14.4	13.5	
Pier 25	TP-25NW				>12.5		
Pier 25	TP-25S				> 10.4		
Pier 26		B-26					
Pier 27		B-27					
Pier 28		B-28					
Pier 29		B-29					
Pier 30		B-30				0	
Pier 31		B-31		MW-31		0	
Pier 32	TP-32NE	B-32			9.0	5	
Pier 32	TP-32SW				7.6		
Pier 32	TP-32NW				9.2		
Pier 33		B-33		MW-33		0	
Pier 34		B-34				6	
Pier 35		B-35		MW-35		0	
Pier 36		B-36				2	
Pier 37		B-37		MW-37		4	
Pier 38		B-38				6	
Pier 39	TP-39NW	B-39		MW-39	6.3	4.5	

Table 1- Summary of Subsurface Exploration (continued)

Designation					Depth Below Existing Ground Surface to Base of Apparent Surficial Fill (feet)		
Substructure	Test Pit	Shallow Auger Boring	Deep Boring	Sampling Point	Test Pit	Shallow Auger Boring	Deep Boring
Pier 39	TP-39SE				7.5		
Pier 39	TP-39SW				11.9		
Pier 40		B-40				3	
Pier 41		B-41		MW-41		9	
Pier 42		B-42				6.5	
Pier 43	TP-43NE	B-43		MW-43	8.5	7	
Pier 44	TP-44NW	B-44			11.9	6	
Pier 45		B-45		MW-45		4	
Pier 46		B-46				4	
Pier 47		B-47		MW-47		0	
Pier 48		B-48				5	
Pier 49		B-49		MW-49		7	
Pier 50	TP-50SE	B-50			5.4	4	
Pier 51		B-51		MW-51		4	
West Abutment		B-AW				37.5	

Note: Temporary sampling points were generally installed using PVC riser pipe, screen section, sand pack and bentonite seal at the ground surface. The sampling points were installed to observe ground water levels and collect samples for subsequent laboratory testing. For sampling points installed at odd-numbered piers between Pier 31 and Pier 51, the same bore hole was used to both conduct SPT sampling and install the temporary sampling point.

Table 2 - Summary of Results - Soil Classification, Soil Resistivity and Chemical Analyses for Test Pit and Test Boring Samples

	DESCRIPTION					SOIL																			WATER										
Test Pit / Test Boring Designation	Sample ID	Lab Sample ID	Soil Classification: Visual	Soil Classification: Unified	Soil Classification: USDA	Bulk Sample Moisture Content	Grain Size Distribution (% Passing)			D_{10}	Soil Resistivity	pH	Specific Gravity	Liquid Limit	Liquid Limit (Not Dried)	Plastic Limit	Plasticity Index	LL / LL (Not Dried)	Organic Content	Bromide	Chloride	Fluoride	Sulfate	Phosphorus	Fecal Coliform	Specific Conductance	pH	Total Acidity	Alkalinity	Iron	Bromide	Chloride	Fluoride	Sulfate	Phosphorus
						(%)	#4	#40	#200	(mm)	(ohms-cm)	(---)	(---)	(---)	(---)	(---)	(---)	(---)	(%)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(CFU/100 ml)	(umhos/cm)	(---)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)
TP05NE	GS1	TP05NE-1	Sandy silt, reddish brown	SANDY SILT, reddish brown (ML)	Loam	-	100	100	59.3	-	1,100	8.2	2.69	NP	-	NP	-	-	-	-	197	-	76.8	-	-	-	-								
TP06SW	GS01	TP06SW-01	Lean clay/topsoil (Fill)			26.1															1,590		278.0												
TP06SW	GS02	TP06SW-02	Shredded wood (Fill)			152.8															4,860		233.0												
TP06SW	GS03	TP06SW-03	Organic clay	SILT w/SAND, black (ML)	Silt Loam	82.8	100	95	71.1	<.001	200	7.9	2.26	NP		NP	NP		21.6		2,620		122.0												
TP06SW	GS04	TP06SW-04	Silty sand	SILTY SAND, fine grained, very dark gray	Sandy Loam	22.9	96	88	29.9	0.004	500	7.9	2.62	NP		NP	NP		1.9		510		44.8 J												
TP06SW	GS05	TP06SW-05	Silt/silty clay			15.7															236		54.8												
TP06SW	GS06	TP06SW-06	Fat clay	LEAN CLAY, dark brown (CL)	Silty Clay Loam	28.4	100	100	97.8	<.001	800	8.4	2.66	44		18	26		2.7		601		30.9 J												
TP06SW	GS07	TP06SW-07	Silt/silty clay	SILT, dark brown (ML)	Silt Loam	13.3	100	100	90.4	<.001	1,100	8.2	2.65	21		NP	NP		1.1		264		37.3 J												
TP06SW	GS08	TP06SW-08	Silt/silty clay			13.6																													
TP10S	-	TP10S	Test pit water sample	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	<2.44	9530	7.6	102J	115	0.068J	<1.0	2870	1.5J	1150	<0.052
TP10S	GS1	TP10S-1	Lean clay w/ organics, reddish brown	LEAN CLAY w/SAND, reddish brown (CL)	Silt Loam	18.1	91	86	70.4	<.002	470	7.8	2.64	29	-	16	14	-	2.2	<2.4	670	4.0J	543.0	557											
TP10S	GS2	TP10S-2	Silty clay, reddish brown	LEAN CLAY, reddish brown (CL)	Silty clay loam	18.9	100	100	96.5	<.001	600	7.9	2.74	29	-	16	13	-	1.2	<2.4	560	<2.4	409.0	534											
TP11S	GS1	TP11S-1	Lean clay, strong brown	LEAN CLAY w/ SAND, strong brown (CL)	Clay loam	20.4	94	89	72.5	<.001	220	7.8	2.59	34	-	16	18	-	1.8	<2.4	1,400	3.6J	646.0	473	-										
TP11S	GS2	TP11S-2	Sandy silt, dark brown	SILT, dark brown (ML)	Silt loam	21.8	98	97	87.5	<.001	800	8.3	2.70	31	-	NP	-	-	0.7	<2.3	281	2.9J	50.9	387	<2.44	4750	8.0	-	144	<0.10	<0.20	1450	<0.20	288	0.65
TP12S		TP12S	1 1/2: Clear Crushed Stone	"GRAVEL, gray (GP)"	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	<3.22	9040	7.0	<66.7	738	<0.052	<1.0	2680	1.2J	60.8	<0.052
TP12S	GS1	TP12S-1	Organic silt w/ sand & clay, black	SILTY SAND w/ ORGANIC FINES, fine to medium grained, black (SM)	Sandy loam	85.6	93	81	41.3	<.002	160	9.0	2.29	68	100	NP	NP	0.68	14.4	<22.1	5,370	<22.1	286J	639											
TP12S	GS2	TP12S-2	1 1/2" clear crushed stone	GRAVEL, gray (GP)	-	-	3.5	2.6	1.9	11	-	-	-	-	-	-	-	-	-	-	-	-	-	-											
TP13NE	GS1	TP13NE-1	Silty Sand, black	SILTY SAND, fine to medium grained, black (SM)	Sandy loam	-	91	74	34.7	-	300	7.7	-	NP	-	NP	-	-	-	-	1,870	-	1,600.0	-											
TP13NE	GS2	TP13NE-2	Lean Clay, dark brown	SANDY LEAN CLAY, dark brown (CL)	Clay loam	-	96	91	69.2	-	1,200	7.9	-	-	-	-	-	-	-	-	268	-	51.8	-											
TP13NE	GS3	TP13NE-3	Sand, grayish brown	SAND, fine grained, brown (SP)	Sand	-	100	96	0.9	0.170	1,600	8.0	-	-	-	-	-	-	-	-	277	-	29.6J	-											
TP13NE	GS4	TP13NE-4	Clay, reddish brown	LEAN CLAY w/SAND, reddish brown (CL)	Silty clay loam	-	98	96	83.5	-	1,300	8.1	-	-	-	-	-	-	-	-	125	-	26.7J	-											
TP13NE	GS5	TP13NE-5	Silty Sand, black	SAND w/SILT, fine to medium grained, black (SP-SM)	Sand	-	100	62	8.3	0.090	300	7.4	2.18	-	-	-	-	-	-	-	1,320	-	484.0	-											
TP13NE	GS6	TP13NE-6	Sandy Silt, black	SILTY SAND w/ GRAVEL, fine grained, black (SM)	Silt loam	-	80	65	39.6	-	410	8.1	-	-	-	-	-	-	-	-	1,050	-	176.0	-											
TP13NE	GS7	TP13NE-7	Silty sand, very dark gray	SILTY SAND w/ GRAVEL, fine grained, very dark gray (SM)	Loam	-	82	71	37.9	-	530	7.9	2.37	NP	-	NP	-	-	-	-	1,590	-	145.0	-											
TP13S	GS1	TP13S-1	Organic silty sand, black	SANDY SILT, black (OL)	Loam	76.2	95	84	53.5	<.001	410	7.8	2.38	NP	NP	NP	-	-	15.2	<3.8	1,550	13.3	101.0	399	-	3830	6.9	-	570	0.36	<1.0	991	<1.0	30.3	0.72
TP13S	GS2	TP13S-2	3/4" Crushed Stone, dark grayish brown	GRAVEL w/ SILT and SAND, dark gray (GP)	-	9.5	43	16	9.6	0.080	800	8.1	-	-	-	-	-	-	1.2	-	-	-	-	-											
TP13S	GS3	TP13S-3	Silt with Sand, dark grayish brown	SILTY SAND w/ ORGANIC FINES, fine grained, dark grayish brown (SM)	Loam	84.7	97	86	46.4	<.003	430	7.9	2.07	84	117	NP	NP	0.72	10.4	<5.2	1,800	8.1J	138.0	469											
TP13SE	GS01	TP13SE-01	Black silt (Fill)	SILTY SAND, fine grained, black (SM)	Loam	31.7	91	80	44.6	0.009	210	8.9	2.27	NP		NP	NP		9.9		1,430		923.0												
TP13SE	GS02	TP13SE-02	Shredded plant debris			53.1															3,080		2,330.0												
TP13SE	GS03	TP13SE-03	Silty sand			17.3																													
TP13SE	GS04	TP13SE-04	Sandy silt/ash/roots/wood debris (Fill)			96.1																													
TP13SE	GS05	TP13SE-05	Lean clay	LEAN CLAY, reddish brown (CL)	Silty Clay Loam	18.4	99	97	89.8	<.001	1,300	8.3	2.69	29		17	12		1.6		296		121.0												
TP14S	-	TP14S	Test pit water sample	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	3.22	4210	7.0	<66.7	1190	0.39J	<0.20	804	<1.0	6.3	0.56
TP14S	GS1	TP14S-1	Clay, sand, silt & organics, brown to very dark gray	SILTY SAND, fine to medium grained, brown to very dark gray (SM)	Sandy loam	29.6	87	72	31.4	.003	700	8.0	2.58	27	-	NP	-	-	4.3	<13.8	298	<13.8	174J	279											
TP14S	GS2	TP14S-2	Organic silty sand, black	SILTY SAND w/ ORGANIC FINES, fine	Sandy loam	76.4	94	82	39.7	<.002	800	7.6	2.41	34	102	NP	NP	0.33	6.4	<3.7	381	5.1J	43.3J	326											
TP14S	GS3	TP14S-3	3/4" crushed stone, grayish brown	"GRAVEL w/ SILT and SAND, grayish brown (GP-GM)	-	9.2	50	16	8.7	0.110	1,500	8.0	-	-	-	-	-	-	1.2	-	-	-	-	-											
TP15S	GS1	TP15S-1	Shredded wood debris, black	-	-	354.5	-	-	-	-	-	-	-	-	-	-	-	-		<3.6	896	8.2	396.0	461	-	4080	6.9		824	0.31	<1.0	1270	<1.0	10.7J	0.75
TP15S	GS2	TP15S-2	3/4" Crushed Stone, dark gray	GRAVEL w/ SILT and SAND, dark gray (GP-GM)	-	10.4	47	16	8.2	0.150	1,300	8.2	-	NP	-	NP	-	-	1.6	-	-	-	-	-											
TP15S	GS3	TP15S-3	Silty sand w/ organics, very dark grayish brown	SAND, fine grained, very dark grayish brown (SP)	Sand	30.9	99	95	4.5	0.130	1,100	8.1	-	NP	-	NP	-	-	0.7	<2.4	140	3.5J	27.3J	123											

Table 2 - Summary of Results - Soil Classification, Soil Resistivity and Chemical Analyses for Test Pit and Test Boring Samples

DESCRIPTION						SOIL																			WATER										
Test Pit / Test Boring Desig-nation	Sample ID	Lab Sample ID	Soil Classification: Visual	Soil Classification: Unified	Soil Classification: USDA	Bulk Sample Moisture Content	Grain Size Distribution (% Passing): #4	#40	#200	D_{10}	Soil Resistivity	pH	Specific Gravity	Liquid Limit	Liquid Limit (Not Dried)	Plastic Limit	Plasticity Index	LL / LL (Not Dried)	Organic Content	Bromide	Chloride	Fluoride	Sulfate	Phosphorus	Fecal Coliform	Specific Conductance	pH	Total Acidity	Alkalinity	Iron	Bromide	Chloride	Fluoride	Sulfate	Phosphorus
						(%)				(mm)	(ohms-cm)	(---)	(---)	(---)	(---)	(---)	(---)	(---)	(%)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(CFU/100 ml)	(umhos/cm)	(---)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)
TP16SE	-	TP16SE	Test pit water sample	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	16.4	4560	6.8	<66.7	799	0.75J	<1.0	1050	1.4J	10.8J	1.9
TP16SE	GS1	TP16SE-1	Organic lean clay, black	SILTY SAND w/ ORGANIC FINES, fine grained, very dark gray (SM)	Sandy loam	37.9	93	81	36.0	<.002	900	8.0	2.43	30	56	NP	-	0.53	8.8	<15.2	259J	19.0J	214J	474											
TP16SE	GS2	TP16SE-2	Sand, brown	SAND, fine grained, dark grayish brown (SP)	Sand	23.4	100	98	4.5	0.130	800	7.8	-	-	-	-	-	-	0.3	<2.4	430	3.0J	26.9J	80.1											
TP16SE	GS3	TP16SE-3	Lean Clay, brown	LEAN CLAY w/ SAND, reddish brown (CL)	Clay	21.1	99	96	76.9	-	800	8.1	2.74	30	-	14	16	-	1.5	<2.4	259	3.5J	27.7J	341											
TP16SE	GS4	TP16SE-4	Crushed Stone, very dark gray	GRAVEL w/ SILT and SAND, very dark gray (GP-GM)	-	9.1	52	18	9.2	0.090	800	8.3	-	-	-	-	-	-	0.9	-	-	-	-	-											
-	-	MW16S	Sampling point water sample	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	<32.1	8240	6.7	-	2110	8.1	<0.20	2530	<0.20	7.4	5.9
TP17S	GS1	TP17S-1	Organic silt, black	SILT w/ SAND, black (ML)	Silt Loam	67.1	99	97	73.1	<.002	440	8.0	2.62	35	45	NP	NP	0.78	3.3	-	-	-	-	-											
TP17S	GS2	TP17S-2	Sandy Silt, brown	SANDY SILT, brown (ML)	Silt Loam	18.5	96	89	60.1	<.001	900	8.1	2.71	16	-	NP	-	-	0.8	-	-	-	-	-											
TP17S	GS3	TP17S-3	3/4" Crushed Stone, dark gray	SAND w/ SILT AND GRAVEL, coarse to medium grained, very dark gray (SP-SM)	Sandy loam	13.0	57	20	11.3	0.045	700	8.0	2.83	19	-	NP	-	-	1.0	-	-	-	-	-											
TP18SE	-	TP18SE	Test pit water sample	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	<6.25	5290	7.3	<66.7	975	<0.10	<0.10	1280	1.2J	63.3	0.26J
TP18SE	GS1	TP18SE-1	Organic sand w/ roots, black	SILTY SAND, fine grained, black (SM)	Loam	105.8	96	90	47.4	-	1,400	8.9	2.31	NP	-	NP	-	-	19.7	<18.9	222J	26.4J	291.0J	478											
TP18SE	GS2	TP18SE-2	Sand w/ silt & gravel, fine to medium grained, gray	SAND w/ SILT and GRAVEL, fine to medium grained, brown (SP-SM)	Sand	24.0	76	57	5.0	0.110	800	8.1	-	-	-	-	-	-	1.0	<5.1	1,230	7.1J	123.0	136											
TP18SE	GS3	TP18SE-3	Crushed stone, dark gray	GRAVEL w/ SILT and SAND, dark gray (GP-GM)	-	8.4	57	16	9.1	0.090	1,700	8.1	-	-	-	-	-	-	0.9	-	-	-	-	-											
TP18SE	GS4	TP18SE	White organic mass	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	<18.3	1,980	25.6J	518.0	33.0J											
-		MW18S	Sampling point water sample	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	<16.4	9770	7.1	-	1200	<0.10	<0.20	3180	<0.20	87.2	3.3
TP19NE	GS1	TP19NE-1	Sandy Silt with Organics, black	SILTY SAND, fine grained, black (SM)	Sandy loam	-	95	84	34.5	-	900	7.9	2.33	NP	-	NP	-	-	-	-	391	-	319.0	-											
TP19NE	GS2	TP19NE-2	Sandy silt w/ organics, black	SILTY SAND, fine to medium grained, black (SM)	Sandy loam	-	100	85	43.7	-	570	7.8	2.38	NP	-	NP	-	-	-	-	2,130	-	81.0J	-											
TP19NE	GS3	TP19NE-3	Silty sand with Organics, black	SILTY SAND w/ORGANIC FINES, fine to medium grained, black (SM)	Loamy Sand	-	95	51	13.2	<0.07	460	7.9	2.16	75	118	NP	-	0.64	-	-	1,590	-	149.0	-											
TP19NE	GS4	TP19NE-4	Clay, strong brown	LEAN CLAY w/ SAND, strong brown (CL)	Silty clay loam	-	98	96	78.3	-	1,200	8.1	-	-	-	-	-	-	-	-	175	-	97.8	-											
TP19NW	GS1	TP19NW-1	Sandy silt w/ organics, black	SILTY SAND, fine grained, black (SM)	Sandy loam	-	93	82	38.9	-	700	7.6	-	-	-	-	-	-	-	-	607	-	176J	-											
TP19NW	GS2	TP19NW-2	Silt, brown	SILT w/ SAND, brown (ML)	Silt loam	-	100	94	73.2	-	1,100	7.7	-	-	-	-	-	-	-	-	283	-	27.1J	-											
TP19S	GS1	TP19S-1	Sandy Organic Silt w/ gravel, black	SILTY SAND, fine grained, black (SM)	Loam	65.2	89	81	44.9	<.001	700	7.9	2.31	NP	-	NP	-	-	19.7	<4.2	1,740	12.7	215.0	486											
TP19S	GS2	TP19S-2	Shredded wood debris, very dark brown	-	-	261.4	-	-	-	-	-	-	-	-	-	-	-	-	-	<8.8	4,060	<8.8	268.0	266	<9.01	5110	6.8	-	786	<0.10	<0.20	1460	<0.20	2.8J	<0.052
TP19S	GS3	TP19S-3	2" Crushed stone, dark yellowish brown	GRAVEL w/ SILT and SAND, dark yellowish brown (GP-GM)	-	10.7	46	17	8.1	0.150	700	8.0	-	-	-	-	-	-	1.1	-	-	-	-	-											
TP19S	GS4	TP19S-4	Sand w/ silt and gravel, brown	SAND, fine grained, brown (SP)	Sand	27.3	97	87	3.4	0.110	1,500	7.6	-	-	-	-	-	-	0.3	<2.4	287	2.9J	32.3J	113											
TP19S	GS5	TP19S-5	Lean clay w/ sand, brown	LEAN CLAY w/ SAND, reddish brown (CL)	Clay	19.0	96	92	75.9	-	900	8.4	2.70	31	-	14	17	-	1.4	<2.5	258	4.7J	38.6J	439											
TP19SW	GS01	TP19SW-01	Lean clay	SANDY LEAN CLAY, dark brown (CL)	Clay Loam	16.6	98	93	65.5	<.001	1,100	8.4	2.80	25		13	12		1.3		183		54.9												
TP19SW	GS02	TP19SW-02	Varved lean clay	LEAN CLAY, dark brown (CL)	Silty Clay Loam	14.9	100	99	98.1	<.001	1,100	8.2	2.70	30		17	13		1.6		104		357.0												
TP20S	GS1	TP20S-1	Sandy silt w/ organics, dark yellowish brown & black	SILTY SAND, fine to medium grained, dark yellowish brown w/ black mottling (SM)	Loam	51.0	95	78	46.4	<.001	410	7.4	2.29	44	-	NP	-	-	13.6	<3.7	2,250	13.4	76.2	722											
TP20S	GS2	TP20S-2	Sludge	-	-	232.8	-	-	-	-	-	-	-	-	-	-	-	-	75.3	<9.4	7,280	423.0	117J	496											
TP20S	GS3	TP20S-3	Crushed stone, grayish brown	GRAVEL w/ SILT and SAND, brown (GP-GM)	-	8.4	50	20	8.7	0.100	700	8.1	-	-	-	-	-	-	0.9	-	-	-	-	-											
TP20S	GS4	TP20S-4	Crushed stone, dark grayish brown	GRAVEL w/ SILT and SAND, some organics, yellow brown mottled black (GP-GM)	-	21.1	34	14	5.8	0.200	340	8.0	-	-	-	-	-	-	7.8	<21.2	2,510	31.8J	249J	460											
TP20S	GS5	TP20S-5	Organic silt, black	SILT w/ SAND, black (OL)	Silt loam	77.2	96	94	73.6	<.002	410	7.8	2.55	45	53	27	18	0.84	5.7	<3.6	1,960	11.1	62.4J	543											
TP21NW	GS1	TP21NW-1	Silty sand, black	SILTY SAND, fine grained, black (SM)	Loam	-	96	89	49.2	-	370	8.4	2.23	NP	-	NP	-	-	-	-	1,890	-	462.0	-											
TP21S	GS1	TP21S-1	Sandy silt, brown	SANDY SILT, brown (ML)	Loam	18.4	96	88	65.8	<.001	700	8.4	2.66	28	-	NP	-	-	3.9	<11.4	463	14.9J	161.0J	354											
TP21S	GS2	TP21S-2	Silty sand, brown/gray	SANDY SILT, dark gray (ML)	Loam	34.0	98	93	57.0	<.002	410	8.1	2.65	25	-	NP	-	-	3.3	<2.7	1,220	3.2J	87.7	379											
TP21S	GS3	TP21S-3	Ash, very dark gray	SILTY SAND, fine grained, very dark gray (SM)	Loam	76.6	99	91	48.7	0.002	250	8.0	2.18	NP		NP	-	-	9.6	<3.8	3,970	7.1J	79.2	351											
TP21S	GS4	TP21S-4	Ash, black	SANDY SILT, very dark gray (ML)	Silt loam	71.6	99	95	65.8	<.002	380	8.1	2.25	NP	-	NP	-	-	5.2	<3.6	866	7.7	263.0	350											

Table 2 - Summary of Results - Soil Classification, Soil Resistivity and Chemical Analyses for Test Pit and Test Boring Samples

	DESCRIPTION					SOIL																			WATER										
Test Pit / Test Boring Desig-nation	Sample ID	Lab Sample ID	Soil Classification			Bulk Sample Moisture Content	Grain Size Distribution (% Passing)			D_{10}	Soil Resistivity	pH	Specific Gravity	Liquid Limit	Liquid Limit (Not Dried)	Plastic Limit	Plasticity Index	LL / LL (Not Dried)	Organic Content	Bromide	Chloride	Fluoride	Sulfate	Phosphorus	Fecal Coliform	Specific Conductance	pH	Total Acidity	Alkalinity	Iron	Bromide	Chloride	Fluoride	Sulfate	Phosphorus
			Visual	Unified	USDA	(%)	#4	#40	#200	(mm)	(ohms-cm)	(---)	(---)	(---)	(---)	(---)	(---)	(---)	(%)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(CFU/100 ml)	(umhos/cm)	(---)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)
-	-	MW22N	Sampling point water sample	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	45.2	5220	7.2	-	1030	0.65	<0.40	1380	<0.40	1580	5.1
TP22N	-	TP22N	Test pit water sample	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	5910	5.4	937.0	101	581	<2.0	1460	<0.20	1690	0.11J
TP22N	GS1	TP22N-1	Ash, black	SILTY SAND, fine grained, black (SM)	Sandy loam	85.8	99	93	45.4	0.007	370	7.9	2.24	NP	-	NP	-	-	18.8	<22.9	1,130	<22.9	21,800.0	299											
TP22N	GS2	TP22N-2	Ash, black	SILTY SAND, fine grained, black (SM)	Sandy loam	131.3	99	93	49.6	0.008	340	7.6	2.22	NP	-	NP	-	-	8.5	<5.4	1,920	7.3J	5,760.0	264											
TP22N	GS3	TP22N-3	Organic silty sand, dark gray - black	SILTY SAND, fine grained, dark gray to black (SM)	Sandy loam	38.7	99	95	33.8	0.001	670	7.7	2.51	NP	-	NP	-	-	2.6	<2.8	685	3.9J	998.0	233											
TP22N	GS4	TP22N-4	Lean clay, reddish brown	LEAN CLAY w/ SAND, reddish brown (CL)	Silty clay loam	20.2	98	95	79.5	<.001	700	8.1	2.66	35	-	16	20	-	1.5	<2.4	291	3.8J	303.0	463											
TP22S	GS1	TP22S-1	Ash, black	CLAYEY SAND, fine grained, black(SC)	Sandy loam	65.8	90	78	42.4	-	300	7.7	2.24	NP	-	NP	-	-	17.7	<2.6	1,720	5.8	2,970.0	216											
TP22S	GS2	TP22S-2	Ash, black	SILTY SAND, fine grained, black (SM)	Sandy loam	88.0	96	87	39.5	0.011	290	8.2	2.32	NP	-	NP	-	-	13.1	<3.9	1,120	7.9	5,270.0	295											
TP22S	GS3	TP22S-3	Ash, black	SILTY SAND, fine grained, black (SM)	Silt loam	107.3	98	89	49.7	0.005	260	8.9	2.36	NP	-	NP	-	-	18.8	<4.1	158	6.0J	1,660.0	253											
TP22S	GS4	TP22S-4	Pit run sand & gravel dark yellowish brown	SAND w/ GRAVEL, coarse to fine grained, dark yellowish brown (SP)	Sand	2.0	58	12	2.2	0.350	5,700	9.1	-	-	-	-	-	-	0.7	<2	29.8J	2.4J	33.8J	402											
TP22S	GS5	TP22S-5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	<4.3	2,870	<4.3	3,150.0	197											
TP22S	GS1	TP22S-M1	Pile corrosion	-	-	25.4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
TP22SW	GS1	TP22SW-1	Ash, very dark gray	SILTY SAND, fine grained, black (SM)	Sandy loam	55.4	95	87	47.3	0.008	230	8.0	2.26	NP	-	NP	-	-	13.7	<3.3	971	7.6	5,840.0	276											
TP22SW	GS2	TP22SW-2	Sand, fine grained, gray	SAND, fine grained, dark yellowish brown (SP)	Sand	20.4	100	97	3.2	0.120	700	7.9	-	-	-	-	-	-	0.3	<2.4	249	2.9J	230.0	107											
TP22SW	GS3	TP22SW-3	Lean clay w/ sand, reddish brown	LEAN CLAY w/ SAND, brown w/ black mottling (CL)	Silty clay loam	18.6	98	96	80.6	<.002	900	7.8	2.66	36	-	14	22	-	1.6	<2.5	229	3.5J	274.0	457											
TP23S	GS1	TP23S-1	Silty sand, some gravel, brown	SAND w/ SILT, fine grained, dark brown (SP-SM)	Sand	9.6	96	93	7.4	0.090	4,100	8.9	-	-	-	-	-	-	1.0	<11.2	139J	<11.2	123.0J	219											
TP23S	GS2	TP23S-2	Ash, black	SAND w/ GRAVEL, fine to medium grained, black (SP)	Sand	70.4	84	61	3.8	0.090	270	8.1	-	-	-	-	-	-	22.5	<3.7	1,150	23.5	176.0	237											
TP23S	GS3	TP23S-3	Shredded wood debris	-	-	93.3	-	-	-	-	-	-	-	-	-	-	-	-	-	<5.9	6,180	8.9J	109.0J	193	<90	11800	7.2	-	1320	<0.21	<1.0	3270	<1.0	13.3J	2.8
TP23S	GS4	TP23S-4	Fine grained silty sand, brown	SILTY SAND, fine grained, yellowish brown (SM)	Sandy loam	22.0	93	89	24.9	0.002	700	8.3	2.62	NP	-	NP	-	-	0.5	<2.4	380	3.3J	31.4J	138											
TP23S	GS5	TP23S-5	Foundry sand, dark gray	SAND w/ SILT, fine to medium grained, dark gray (SP-SC)	Sand	113.0	99	69	10.1	0.070	100	7.1	2.11	-	-	-	-	-	11.6	<4.1	3,400	6.5J	67.3J	142											
TP23S	GS6	TP23S-6	Fat clay, reddish brown	LEAN CLAY w/ SAND, reddish brown (CL)	Silty clay loam	34.1	97	90	75.3	-	450	8.5	-	41	-	29	12	-	1.6	-	-	-	-	-											
-	-	MW23N	Sampling point water sample	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	<9.01	5430	6.8	-	635	0.15J	<0.20	1190	<0.20	13.6	0.57
TP24NW	GS1	TP24NW-1	Sand, yellowish brown	SILTY SAND, fine grained, yellowish brown (SM)	Loamy sand	-	100	97	13.4	-	4,200	7.3	-	-	-	-	-	-	-	-	34.2 J	-	26.9J	-											
TP24NW	GS2	TP24NW-2	Shredded Wood debris	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	626	-	67.3J	-											
TP24NW	GS3	TP24NW-3	Silt and Clay, dark grayish brown mottled dark yellowish brown	LEAN CLAY w/ SAND, dark grayish brown w/ mottled dark yellowish brown (CL)	Silt loam	-	97	92	74.5	-	1,700	7.5	-	-	-	-	-	-	-	-	91	-	26.4J	-											
TP24NW	GS4	TP24NW-4	Silt, yellowish brown	SILT, yellowish brown (ML)	Silt	-	100	99	97.3	-	1,500	8.0	-	-	-	-	-	-	-	-	143	-	32.8J	-											
TP24NW	GS5	TP24NW-5	Lean Clay, dark brown	LEAN CLAY w/SAND, dark brown (CL)	Silty clay loam	-	98	95	78.5	-	1,200	8.1	-	-	-	-	-	-	-	-	106	-	37.4J	-											
TP24S	GS1	TP24S-1	Silty sand w/ gravel & organics, dark yellowish brown	SAND w/ SILT, fine grained, dark yellowish brown (SP-SM)	Sand	14.9	99	92	6.8	0.080	2,800	8.3	-	-	-	-	-	-	1.5	<2.3	42.4J	3.5J	32.3J	174											
TP24S	GS2	TP24S-2	Organic silty sand w/ gravel, black	SILTY SAND w/ GRAVEL, fine to medium grained, black (SM)	Sandy loam	76.7	83	52	25.0	0.006	1,000	7.7	2.16	NP	-	NP	-	-	28.3	<3.3	545	10.4	81.6	472											
TP24S	GS3	TP24S-3	Shredded wood debris, very dark brown	-	-	223.8	-	-	-	-	-	-	-	-	-	-	-	-	0.7	<7.5	2,170	9.7J	130.0J	444											
TP24S	GS4	TP24S-4	3/4" Crushed Stone, dark gray	GRAVEL w/ SILT and SAND, dark gray (GP-GM)	-	8.4	41	12	6.6	0.300	1,000	7.8	-	-	-	-	-	-	1.5	-	-	-	-	-											
TP24S	GS5	TP24S-5	Organic silty sand, black	SANDY SILT, black (ML)	Loam	39.1	100	95	56.9	<.002	700	7.5	2.42	34	-	NP	-	-	5.0	<13.8	396	16.5J	217.0J	313											
TP24S	GS6	TP24S-6	Fine sand w/ silt & some organics, dark grayish brown	SAND w/ SILT, fine grained, dark grayish brown (SP-SM)	Sand	19.3	90	82	10.3	0.070	1,500	8.3	-	-	-	-	-	-	0.3	<2.3	159	3.2J	70.3	132											

Table 2 - Summary of Results - Soil Classification, Soil Resistivity and Chemical Analyses for Test Pit and Test Boring Samples

	DESCRIPTION					SOIL																			WATER										
Test Pit / Test Boring Designation	Sample ID	Lab Sample ID	Soil Classification: Visual	Soil Classification: Unified	Soil Classification: USDA	Bulk Sample Moisture Content (%)	Grain Size Distribution (% Passing) #4	#40	#200	D_{10} (mm)	Soil Resistivity (ohms-cm)	pH (---)	Specific Gravity (---)	Liquid Limit (---)	Liquid Limit (Not Dried) (---)	Plastic Limit (---)	Plasticity Index (---)	LL / LL (Not Dried) (---)	Organic Content (%)	Bromide (mg/kg)	Chloride (mg/kg)	Fluoride (mg/kg)	Sulfate (mg/kg)	Phosphorus (mg/kg)	Fecal Coliform (CFU/100 ml)	Specific Conductance (umhos/cm)	pH (---)	Total Acidity (mg/L)	Alkalinity (mg/L)	Iron (mg/L)	Bromide (mg/L)	Chloride (mg/L)	Fluoride (mg/L)	Sulfate (mg/L)	Phosphorus (mg/L)
-	-	MW25S	Sampling point water sample	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	9.01	5230	6.6	-	1190	0.30J	<0.20	1030	<0.20	373	0.78
TP25NE	GS1	TP25NE-1	Silty Sand with Clay and Organics, black	SILTY SAND, fine grained, black (SM)	Sandy loam	49.8	95	86	40.7	-	1,300	8.4	2.42	-	-	-	-	-	-	-	98	-	158.0	-											
TP25NE	GS2	TP25NE-2	Silty sand with Organics, black	SILTY SAND, fine grained, black (SM)	Sandy loam	-	98	88	33.2	-	270	7.0	2.11	-	-	-	-	-	-	-	7,710	-	227.0	-											
TP25NE	GS3	TP25NE-3	Silty sand with Organics, black	SILTY SAND, fine grained, black (SM)	Sandy loam	185.2	98	88	35.2	-	260	7.7	2.23	NP	-	NP	-	-	-	-	4,330	-	662.0	-											
TP25NE	GS4	TP25NE-4	Silty Sand, black mottled dark grayish brown	SAND w/ SILT, fine grained, black mottled dark grayish brown (SP-SM)	Sand	30.9	98	95	7.0	0.110	900	8.0	2.61	NP	-	NP	-	-	-	-	259	-	66.9	-											
TP25NE	GS5	TP25NE-5	Silty Sand, light brown	SAND, fine grained, light brown (SP)	Sand	-	98	90	2.7	0.16	700	8.0	-	-	-	-	-	-	-	-	862	-	171.0	-											
TP25NW	GS1	TP25NW-1	Silty Sand w/ Organics, black	SILTY SAND, fine to medium grained, black (SM)	Sandy loam	-	99	86	35.3	-	550	7.8	2.28	NP	-	NP	-	-	-	-	1,410	-	1,150.0	-											
TP25S	GS1	TP25S-1	Ash, black	SILTY SAND, fine grained, black (SM)	Sandy loam	90.1	95	84	46.2	0.005	370	8.2	2.40	NP	-	NP	-	-	19.9	<3.9	3,980	12.1	248.0	358											
TP25S	GS2	TP25S-2	Ash, black	SILTY SAND, fine grained, black (SM)	Sand loam	120.2	95	87	45.6	0.006	450	8.1	2.31	NP	-	NP	-	-	22.3	<4.6	3,780	7.1J	277.0	326											
TP32NE	GS1	TP32NE-1	SAND w/ SILT, dark grayish brown	SANDY SILT, dark greyish brown (ML)	Loam	-	98	94	60.1	-	190	7.6	-	-	-	-	-	-	-	-	2,610	-	664.0	-											
TP32NE	GS2	TP32NE-2	Fill - Sand, Silt, Clay, topsoil mix with Organics, brown	SILTY SAND, fine grained, brown (SM)	Loam	-	96	88	48.7	-	160	8.1	-	-	-	-	-	-	-	-	3,130	-	158.0	-											
TP32NE	GS3	TP32NE-3	Silty Lean Clay, reddish brown	LEAN CLAY w/ SAND, dark reddish brown (CL)	Silty clay loam	-	98	96	82.2	-	250	8.0	-	-	-	-	-	-	-	-	2,640	-	143.0	-											
TP32NE	GS4	TP32NE-4	Pile corrosion	-		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	10,300	-	226.0	-											
TP32NE	GS5	TP32NE-5	Silty Lean Clay, reddish brown	LEAN CLAY, reddish brown (CL)	Silty clay		98	96	87.3	<.001	160	7.8	2.70	41	-	18	23	-	-	-	3,460	-	70.3	-											
TP32NE	GS6	TP32NE-6	Sandy Silt, brown	SILTY SAND, fine grained, brown (SM)	Sandy loam	-	100	99	50.0	0.015	3,500	8.5	2.70	NP	-	NP	-	-	-	-	55	-	50.9	-											
TP32NW	GS01	TP32NW-01	Silt/topsoil/clay (Fill)			21.0															2,570		97.7												
TP32NW	GS02	TP32NW-02	Lean clay	LEAN CLAY w/SAND, reddish brown (CL)	Silty Clay Loam	19.6	98	96	79.4	<.001	350	8.0	2.66	35		15	20		2.2		1,630		148.0												
TP32NW	GS03	TP32NW-03	Sandy cilt			14.7																													
TP32NW	GS04	TP32NW-04	Lean clay			19.1																													
TP32NW	GS05	TP32NW-05	Lean clay-Instrument hole			22.6															1,640		117.0												
TP32NW	GS06	TP32NW-06	Lean clay-Instrument hole			23.0																													
TP32NW	GS07	TP32NW-07	Lean clay-Instrument hole			22.1															118		46.8 J												
TP32NW	GS08	TP32NW-08	Lean clay & peat -Instr hole	FAT CLAY, dark brown (CH)	Clay	37.6	100	98	94.9	<.001	1,000	8.4	2.69	64		27	37		3.8		258		50.0 J												
TP39NW	GS1	TP39NW-1	Lean Clay with Sandy Silt, reddish brown to dark brown	SANDY LEAN CLAY, reddish brown to dark brown (CL)	Loam	-	95	89	60.6	-	370	7.8	-	-	-	-	-	-	-	-	1,110	-	322.0	-											
TP39NW	GS2	TP39NW-2	Lean Clay with Sandy Silt, reddish brown to brown mottled dark grayish brown	SANDY LEAN CLAY, reddish brown to brown mottled dark grayish brown (CL)	Loam	-	95	88	59.6	-	170	7.6	-	-	-	-	-	-	-	-	2,890	-	278.0	-											
TP39NW	GS3	TP39NW-3	Lean Clay, brown	LEAN CLAY w/ SAND, brown (CL)	Loam	-	99	96	71.9	-	170	7.9	-	-	-	-	-	-	-	-	3,170	-	218.0	-											
TP39NW	GS4	TP39NW-4	Silt, Sand with stone and organics, black	SANDY SILT, black (ML)	Silt loam	-	91	81	58.5	<.001	80	8.3	2.69	24	-	NP	-	-	-	-	7,850	-	333.0	-											
TP39NW	GS5	TP39NW-5	Sandy Silt/Silty Sand, strong brown	SILTY SAND, fine grained, strong brown (SM)	Sandy loam	-	99	98	41.4	-	90	7.6	-	-	-	-	-	-	-	-	3,730	-	184.0	-											
TP39SE	GS1	TP39SE-1	Silty Sand with some Organics, dark yellowish brown mottled with brownish yellow and very dark gray	SANDY SILT, dark yellowish brown mottled w/ brownish yellow and very dark gray (ML)	Silt loam	-	95	88	58.4	-	300	8.0	-	-	-	-	-	-	-	-	1,150	-	1,140.0	-											
TP39SE	GS2	TP39SE-2	Clay, strong brown	LEAN CLAY w/ SAND, strong brown (CL)	Clay loam	-	99	97	72.1	<.001	200	7.9	2.74	26	-	14	13	-	-	-	1,590	-	140.0	-											
TP39SE	GS3	TP39SE-3	Silty sand with Organics, black	SANDY SILT, black (ML)	Loam	-	99	88	50.4	-	170	7.5	-	-	-	-	-	-	-	-	11,400	-	226.0	-											
TP39SE	GS4	TP39SE-4	Silt, dark grayish brown	SILT, dark grayish brown (ML)	Silt loam	-	100	99	98.4	-	1,500	7.9	-	-	-	-	-	-	-	-	266	-	70.2	-											
TP39SW	GS01	TP39SW-01	Topsoil (Fill)			11.8															1,820		2,520.0												
TP39SW	GS02	TP39SW-02	Sandy lean clay/sandy silt (Fill)			16.5															1,080		105.0												
TP39SW	GS03	TP39SW-03	Sandy lean clay/sandy silt (Fill)	SANDY SILTY CLAY, brown (CL-ML)	Clay Loam	16.0	97	90	56.7	<.001	620	8.0	2.62	20		14	6		1.7		1,050		30.8 J												
TP39SW	GS04	TP39SW-04	Sandy silt			12.2																													
TP39SW	GS05	TP39SW-05	Sandy lean clay			16.5																													
TP39SW	GS06	TP39SW-06	Lean clay	SILTY CLAY w/ SAND, brown (CL-ML)	Silt Loam	19.1	100	98	84.0	<.001	230	8.5	2.68	21		15	5		2.0		2,410		169.0												
TP39SW	GS07	TP39SW-07	Silt	SILT w/SAND, dark brown (ML)	Silt Loam	18.3	100	99	75.1	<.001	320	8.0	2.68	NP		NP	-		1.7		2,010		160.0												
TP39SW	GS08	TP39SW-08	Varved fat clay	LEAN CLAY, brown (CL)	Silty Clay	22.4	100	99	97.3	<.001	470	8.2	2.70	38		18	20		2.4		2,050		199.0												
TP43NE	GS1	TP43NE-1	Silty, Sandy Clay w/ Stone, dark grayish brown	SILTY SAND, fine grained, dark grayish brown (SM)	Loam	-	89	80	48.9	-	310	8.0	-	-	-	-	-	-	-	-	962	-	875.0	-											
TP43NE	GS2	TP43NE-2	Fill, Sandy Silty, Stone, dark grayish brown	SILTY SAND w/ GRAVEL, fine grained, dark grayish brown (SM)	Sandy loam	-	72	60	21.1	0.012	390	7.8	2.66	NP	-	NP	-	-	-	-	1,190	-	52.6	-											
TP43NE	GS3	TP43NE-3	Sandy Silt, dark gray	LEAN CLAY w/ SAND, dark gray (CL)	Silt loam	-	99	97	81.8	-	350	8.1	-	-	-	-	-	-	-	-	1,130	-	40.2J	-											
TP43NE	GS4	TP43NE-4	Gray clay w/ black & brown organics	LEAN CLAY, dark grayish brown (CL)	Silty clay loam	-	100	99	98.6	<.001	900	8.1	2.63	36	-	21	15	-	-	-	443	-	< 27.9	-											
TP43NE	GS5	TP43NE-5	Silt, dark grayish brown	LEAN CLAY, dark grayish brown (CL)	Silty clay	-	100	97	94.1	-	800	7.9	-	-	-	-	-	-	-	-	400	-	< 27.0	-											

Table 2 - Summary of Results - Soil Classification, Soil Resistivity and Chemical Analyses for Test Pit and Test Boring Samples

Test Pit / Test Boring Designation	DESCRIPTION: Sample ID	Lab Sample ID	Soil Classification: Visual	Unified	USDA	SOIL: Bulk Sample Moisture Content	Grain Size Distribution (% Passing) #4	#40	#200	D_{10}	Soil Resistivity	pH	Specific Gravity	Liquid Limit	Liquid Limit (Not Dried)	Plastic Limit	Plasticity Index	LL / LL (Not Dried)	Organic Content	Bromide	Chloride	Fluoride	Sulfate	Phosphorus	WATER: Fecal Coliform	Specific Conductance	pH	Total Acidity	Alkalinity	Iron	Bromide	Chloride	Fluoride	Sulfate	Phosphorus
						(%)				(mm)	(ohms-cm)	(---)	(---)	(---)	(---)	(---)	(---)	(---)	(%)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(CFU/100 ml)	(umhos/cm)	(---)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)
TP44NW	GS01	TP44NW-01	Sandy silt (Fill)			15.6															362		1,850.0												
TP44NW	GS02	TP44NW-02	Silt (Fill)	SANDY SILT, very dark grayish brown (ML)	Loam	23.4	91	85	58.5	<.001	800	8.0	2.65	NP		NP	-		3.0		211		116.0												
TP44NW	GS03	TP44NW-03	Silt	LEAN CLAY, dark yellowish brown (CL)	Silty Clay Loam	28.7	98	98	97.1	<.001	1,000	8.2	2.70	31		18	13		2.0		273		79.4												
TP44NW	GS04	TP44NW-04	Soft fat clay	LEAN CLAY, brown (CL)	Silty Clay Loam	35.7	100	100	99.1	<.001	1,500	8.2	2.72	40		20	20		2.1		174		92.1												
TP44NW	GS05	TP44NW-05	Mottled fat flay	LEAN CLAY, brown (CL)	Silty Clay	20.2	100	99	98.3	<.001	1,600	8.4	2.68	39		20	19		1.8		101		77.2												
TP44NW	GS06	TP44NW-06	Soft fat clay			34.0																													
TP50SE	GS01	TP50SE-01	Topsoil (Fill)			24.1															1,980		795.0												
TP50SE	GS02	TP50SE-02	Silt (Fill)	SANDY SILT, brown (ML)	Clay Loam	22.0	95	87	56.1	<.001	570	8.3	2.61	20		NP	-		1.2		1,750		859.0												
TP50SE	GS03	TP50SE-03	Silt	SANDY SILT, reddish brown (ML)	Loam	19.8	95	85	53.9	<.001	200	8.2	2.71	20		NP	-		2.1		555		188.0												
TP50SE	GS04	TP50SE-04	Lean clay	LEAN CLAY w/ SAND, strong brown (CL)	Silty Clay Loam	17.2	99	98	83.7	<.001	1,100	8.2	2.69	32		14	18		1.5		161		168.0												
TP50SE	GS05	TP50SE-05	Lean clay			17.5																													
TP50SE	GS06	TP50SE-06	Sandy lean clay	SANDY LEAN CLAY, reddish brown (CL)	Loam	16.8	98	93	57.3	<.001	1,700	8.3	2.68	28		13	15		0.7		155		97.1												
TP50SE	GS07	TP50SE-07	Lean clay			17.5																													
Pier 26	ST-1	0-2.5	FILL - SILTY SAND W/ WOOD DEBRIS	SILTY SAND (SM), TRACE GRAVEL		-	-	-	-	-	-	-	-		-	-	-																		
Pier 26	ST-5	10-12.5	NATIVE -SILTY SAND	SILTY CLAYEY SAND (SC-SM)		24.0	100	100	23.0	0.003	700	-	-		NP	NP	NP																		
Pier 26	ST-6	12.5-15.0	NATIVE - SILTY SAND, LITTLE CLAY	SILTY SAND (SM)		-	-	-	-	-	-	-	-		-	-	-																		
Pier 26	ST-9	20.0-22.5	NATIVE - SILTY SAND, OCCASIONAL SHELL	SILTY SAND (SM)		27.0	100	99	33.0	0.002	700	-	-		NP	NP	NP																		
Pier 26	ST-10	22.5-25	NATIVE - SILT, OCCASIONAL SHELL	SILT, with SAND (ML)		-	-	-	-	-	-	-	-		-	-	-																		
Pier 26	ST-12	27.5-30.0	NATIVE - SILT, OCCASIONAL SHELL	SILT, with SAND (ML)		28.0	100	99	77.0	0.002	800	-	-		24	22	2																		
Pier 27	ST-2	8.5-11.0	SEDIMENT - w/ WOOD & FILL DEBRIS	SILTY SAND (SM)		-	-	-	-	-	-	-	-		-	-	-																		
Pier 27	ST-3	11.0-12.7	SEDIMENT - w/ WOOD & FILL DEBRIS	SILTY SAND (SM)		63.0	100	94	21.0	0.006	1,400	-	-		NP	NP	NP																		
Pier 27	ST-6	18.5-20.6	NATIVE - SILTY SAND, VERY FINE SAND	POORLY GRADED SAND WITH SILT (SP-SM)		25.0	100	96	8.4	0.08	3,700	-	-		NP	NP	NP																		
Pier 27	ST-8 & 9	23.5-28.0	NATIVE - SOFT SILT	SILT, with SAND (ML)		52.0	100	100	80.0	0.0015	1,100	-	-		46	27	19																		
Pier 27	ST-10	28.5-31.0	NATIVE - SOFT SILT	SILT, with SAND (ML)		--	--	--	--	--	-	-	-		--	--	-																		
Pier 28	ST-2	25.0-27.5	SEDIMENT - w/OCCASIONAL SHELL	SANDY SILT (ML)		-	-	-	-	-	-	-	-		-	-	-																		
Pier 28	ST-3	27.5- 30.0	SEDIMENT - w/OCCASIONAL SHELL	SANDY SILT (ML)		58.0	100	99	67.0	0.0015	800	-	-		NP	NP	NP																		
Pier 28	ST-5	32.5-34.0	SEDIMENT - w/OCCASIONAL SHELL	SANDY SILT (ML)		-	-	-	-	-	-	-	-		-	-	-																		
Pier 28	ST-7	37.5-40.0	NATIVE - CLAY	LEAN CLAY (CL)		55.0	100	100	86.0	<0.001	800	-	-		46	24	22																		
Pier 28	ST-11	47.5-50.0	NATIVE - CLAY	LEAN CLAY (CL)		-	-	-	-	-	-	-	-		-	-	-																		
Pier 28	ST-12	50.0-52.5	NATIVE - CLAY	FAT CLAY (CH)		68.0	100	100	100.0	<0.001	600	-	-		58	28	30																		
Pier 29	ST-3	10.0-12.5	SILTY SAND - w/OCCASIONAL SHELL	SILTY SAND (SM)		41.0	100	98	26.0	0.002	40	-	-		NP	NP	NP																		
Pier 29	ST-6 & 7	19.2-22.5	SILT - w/OCCASIONAL SHELL	SANDY SILT (ML)		-	-	-	-	-	-	-	-		-	-	-																		
Pier 29	ST-8	22.5-25.0	SILT - w/OCCASIONAL SHELL	SANDY SILT (ML)		58.0	100	97	69.0	<0.001	900	-	-		37	36	1																		
Pier 29	ST-11	30.0-35.0	SILT - w/OCCASIONAL SHELL	SANDY SILT (ML)		33.0	100	98	66.0	<0.001	400	-	-		NP	NP	NP																		

Note:
NP = Non-plastic

Note:
J - Estimated concentration above the adjusted method detection limit and below the adjusted method reporting limit.

Table 3 - Summary of Results - Water Level Elevations and Chemical Analyses for Sampling Points

Sample Point ID	Elevation (ft.) Ground Surface	Screen Section Top	Screen Section Bottom	Field Tests Date	Water Level Elev. (ft.)	pH	Specific Conduc-tance (µmhos/cm)	Laboratory Tests Total Acidity (mg/L)	Total Alkalinity, CaCO3 (mg/L)	Bromide (mg/L)	Chloride (mg/L)	Fecal Coliform (CFU/100mL)	Fluoride (mg/L)	Ferrous Iron (mg/L)	pH	Phos-phorus (mg/L)	Specific Conduc-tance (µmhos/cm)	Sulfate (mg/L)
MW2	579.0	570.0	560.0	10/07/13	561.5													
				10/18/13	573.9													
				10/22/13	575.8	7.43	3,750	< 40.0	218	-	2,760	-	-	-	-	-	8,800	680
MW4	578.9	569.8	559.8	10/07/13	562.2													
				10/18/13	567.3													
				10/22/13	568.5	7.62	3,040	< 40.0	154	-	800	-	-	-	-	-	3,120	331
MW6	581.5	572.8	562.8	10/07/13	562.3													
				10/18/13	563.4													
				10/22/13	563.9	7.97	1,340	< 40.0	304	-	116	-	-	-	-	-	1,400	377
MW8	583.5	573.5	563.5	10/18/13	565.7													
				10/22/13	567.5	7.24	2,660	< 40.0	235	-	722	-	-	-	-	-	2,760	106
MW9	580.9	571.9	561.9	10/12/13														
				10/18/13														
				10/22/13	562.2	7.82	1,400	< 40.0	516	-	88.4	-	-	-	-	-	1,330	391
MW10	581.0	557.4	547.4	10/12/13	548.9													
				10/18/13	549.9													
				10/22/13	550.5	8.15	980	< 40.0	49.8	-	56.1	-	-	-	-	-	869	326
MW12	584.3	575.6	565.6	10/12/13	578.1													
				10/18/13	580.0													
				10/22/13	580.1	6.92	11,320	< 40.0	592	-	3,650	-	-	-	-	-	11,500	92.7
MW14	585.1	582.7	572.7	10/12/13	580.5													
				10/18/13	580.6													
MW16S	586.3	584.8	574.8	09/28/13		6.64	9,460	-	2,110	< 0.20	2,530	< 32.1	< 0.20	8.1	6.7	5.9	8,240	7.4
				09/29/13	581.1	7.24	2,800											
				09/30/13	581.1													
				10/01/13	581.1													
				10/02/13	579.8													
				10/12/13	581.1													
				10/18/13	581.1													
				10/22/13	581.0	6.97	4,230	< 40.0	791	-	2,900	-	-	-	-	-	9,540	13.1
MW18S	587.9	583.6	573.6	09/28/13	581.3	6.96	11,050	-	1,200	< 0.20	3,180	< 16.4	< 0.20	< 0.10	7.1	3.3	9,770	87.2
				09/29/13	581.4	6.96	6,030											
				09/30/13	581.4													
				10/01/13	581.3													
				10/02/13	581.3													
				10/12/13	581.5													
				10/18/13	581.5													
				10/22/13	581.3	7.09	2,800	< 40.0	924	-	1,060	-	-	-	-	-	4,980	80.9
MW20	583.4	576.7	566.7	10/22/13	580.7	6.87	6,400	< 40.0	958	-	1,440	-	-	-	-	-	6,110	235
MW22N	594.0	584.0	574.0	09/30/13	580.6	6.61	7,960	-	1,030	< 0.40	1,380	< 45.2	< 0.40	0.65	7.2	5.1	5,220	1,580
				10/01/13	580.6													
				10/12/13	581.2													
				10/18/13	580.5													
				10/22/13	580.5	6.32	7,530	< 40.0	1,030	-	1,390	-	-	-	-	-	7,440	1,600
MW23N	589.1	581.5	571.5	09/29/13	580.4	6.77	5,320	-	635	< 0.20	1,190	< 9.0	< 0.20	0.15 J	6.8	0.57	5,430	13.6
				09/30/13	580.5													
				10/01/13	580.4													
				10/12/13	580.4													
				10/18/13	580.5													
				10/22/13	580.4	6.66	5,710	< 40.0	634	-	1,540	-	-	-	-	-	5,840	22.7
MW25S	592.2	582.2	572.2	09/29/13	579.9	6.67	5,760	-	1,190	< 0.20	1,030	< 9.01	< 0.20	0.30 J	6.6	0.78	5,230	373
				09/30/13	580.0													
				10/01/13	580.0													
				10/12/13	579.8													
				10/18/13	579.9													
				10/22/13	579.9	6.65	6,470	< 40.0	967	-	1,130	-	-	-	-	-	6,540	1,030
MW31	582.2	572.6	562.6	10/08/13	578.1	6.99	3,000										-	
				10/09/13	579.1													
				10/22/13	578.9	7.46	3,160	< 20.0	449	-	27,100	-	-	-	-	-	88,600	2,140
MW33	582.1	571.9	561.9	10/09/13	579.8													
				10/22/13	579.8	6.68	>19,990	< 0.10	265	-	20,400	-	-	-	-	-	68,700	1,390

Table 3 - Summary of Results - Water Level Elevations and Chemical Analyses for Sampling Points

Sample Point ID	Elevation (ft.)			Field Tests				Laboratory Tests										
	Ground Surface	Screen Section Top	Screen Section Bottom	Date	Water Level Elev. (ft.)	pH	Specific Conduc-tance (µmhos/cm)	Total Acidity (mg/L)	Total Alkalinity, CaCO3 (mg/L)	Bromide (mg/L)	Chloride (mg/L)	Fecal Coliform (CFU/100mL)	Fluoride (mg/L)	Ferrous Iron (mg/L)	pH	Phos-phorus (mg/L)	Specific Conduc-tance (µmhos/cm)	Sulfate (mg/L)
MW35	586.5	576.5	566.5	10/09/13	579.4													
				10/22/13	579.7	6.79	>19,990	< 10.0	142	-	10,400	-	-	-	-	-	34,700	685
MW37	583.7	573.4	563.4	10/09/13	578.5													
				10/22/13	579.6	6.39	>19,990	< 10.0	257	-	18,300	-	-	-	-	-	59,900	901
MW39	583.6	573.8	563.8	10/09/13	569.3													
				10/22/13	575.7	6.70	14,400	< 20.0	453	-	9,810	-	-	-	-	-	33,400	592
MW41	584.1	575.6	565.6	10/22/13	576.3	6.68	5,790	< 40.0	136	-	2,460	-	-	-	-	-	7,630	275
MW43	584.2	575.1	565.1	10/07/13	581.6	7.02											-	
				10/08/13	581.3													
				10/09/13	581.1													
				10/22/13	580.7	7.19	5,340	< 40.0	3,310	-	1,470	-	-	-	-	-	5,810	59.3
MW45	583.1	574.4	564.4	10/07/13	579.5	6.88	8,490										-	
				10/08/13	580.1													
				10/09/13	580.8													
				10/22/13	580.7	7.02	7,390	< 40.0	2,940	-	2,080	-	-	-	-	-	7,280	222
MW47	582.7	573.9	563.9	10/08/13	581.2	7.02	4,870										-	
				10/09/13	581.1													
				10/22/13	581.0	7.33	4,780	< 40.0	2,570	-	1,150	-	-	-	-	-	4,720	255
MW49	583.1	576.0	566.0	10/08/13	580.6	6.88	3,590										-	
				10/09/13	580.9													
				10/22/13	580.8	6.84	3,240	< 40.0	2,410	-	532	-	-	-	-	-	3,020	375
MW51	583.1	575.2	565.2	10/08/13	579.9	6.07	18,150										-	
				10/09/13	580.2													
				10/22/13	580.3	6.67	16,940	< 40.0	3,240	-	5,680	-	-	-	-	-	17,400	146

Table 4 - Corrosion-Related Data

Description		Units	East Abutment	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8	Pier 9	Pier 10	Pier 11	Pier 12	Pier 13	Pier 14	Pier 15	Pier 16	Pier 17
Noteworthy Surficial Soil/Fill Conditions Encountered			silty sand over lean clay	lean clay	lean clay	lean clay	lean clay	silt and lean clay	silt and lean clay with	silt and lean clay with	silt and lean clay with	lean clay	lean clay	lean clay	silty sand with	silty sand with	silty sand with	organic sandy silt with wood pulp over	organic silty sand	organic silt with
			--	--	--	--	--	--	organics	organics	organic silt / peat	--	organics	--	organics	organics	organics	shredded-wood debris	wood	wood pulp
			--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Half-Cell Potential, Maximum Range over 2 to 3 feet of Pile Length		(mV)	--	--	--	--	--	9	40	--	--	--	--	--	--	37	--	--	--	--
Soil Resistivity	In-Situ	(ohm-cm)	--	--	--	--	--	550 to 1,440	--	--	--	--	--	--	--	800 to 1,560	--	--	--	--
	Lab	(ohm-cm)	--	--	--	--	--	1,100	200 to 1,100	--	--	--	470 to 600	220 to 800	160	300 to 1,600	700 to 1,500	1,100 to 1,300	800 to 900	440 to 900
Chloride Concentration	Water	(mg/L)	--	--	2,760	--	800	--	116	--	722	88	56	--	3,650	--	838	--	2,900	--
	Soil	(mg/L)	--	--	--	--	--	197	236 to 4,860	--	--	--	560 to 670	281 to 1,400	5,370	125 to 3,080	298 to 381	140 to 896	259 to 430	--
Sulfate Concentration	Water	(mg/L)	--	--	680	--	331	--	377	--	106	391	326	--	93	--	84	--	13	--
	Soil	(mg/L)	--	--	--	--	--	77	37 to 278	--	--	--	409 to 543	51 to 646	286	27 to 2,330	43 to 174	27 to 396	27 to 214	--
Microbial (MIC) test, time elapsed since inoculation		(days)	--	--	--	--	--	11	--	--	--	--	7	7	7	11	7	7	7	7
Aerobic	Low Nutrient Bacteria (measured to-date)	(colonies/mL)	--	--	--	--	--	10 to 100	--	--	--	--	1,000 to 10,000	1,000 to 10,000	>100,000	>100,000	>100,000	10 to 100	>100,000	>100,000
	Iron Reducing Bacteria	(colonies/mL)	--	--	--	--	--	1 to 10	--	--	--	--	1 to 10	10 to 100	10 to 100	1,000 to 10,000	1,000 to 10,000	10 to 100	10 to 100	10 to 100
Anaerobic	Anaerobic Bacteria	(colonies/mL)	--	--	--	--	--	1,000 to 10,000	--	--	--	--	10 to 100	1,000 to 10,000	>100,000	>100,000	1,000 to 10,000	1,000 to 10,000	>100,000	>100,000
	Acid Producing Bacteria	(colonies/mL)	--	--	--	--	--	10 to 100	--	--	--	--	10 to 100	1,000 to 10,000	>100,000	>100,000	1,000 to 10,000	>100,000	>100,000	>100,000
	Sulfate Reducing Bacteria	(colonies/mL)	--	--	--	--	--	10 to 100	--	--	--	--	1 to 10	1 to 10	10 to 100	1,000 to 10,000	10 to 100	10 to 100	10 to 100	10 to 100
Bottom of Footing Elevation		(feet)	601.7 *	574.2 *	573.1 *	572.6 *	572.4 *	571.4 *	577.6 *	577.7 *	578.1 *	569.4 *	570.1	569.1	578.8	578.6	580.7	579.2	578.6	579.6
Elevation of Water Level (measured on 10/22/2013)		--	--	--	575.8	--	568.5	--	563.9	--	567.5	562.2	550.5	--	580.1	--	580.6	--	581.0	--
Approximate Elevation of Water Seep(s) at Time of Test Pitting		(feet)	--	--	--	--	--	569	--	--	--	--	570	569	579	579	581	581	581	580
No. of Piles Visually Inspected		(piles)	--	--	--	--	--	6	3	--	--	--	4	2	2	12	2	2	2	2
No. of Piles Per Pier		(piles)	--	--	--	--	--	24	20	--	--	--	30	36	32	34	32	36	34	34
Exposed Pile - Visual Condition Rating		---	--	--	--	--	--	10	10	--	--	--	10	10	7	8	7	10	10	10
Average Section Loss Observed (visually rated) ***		(percent)	--	--	--	--	--	0 to 5	0 to 5	--	--	--	0 to 5	0 to 5	5 to 10	5 to 10	5 to 10	0 to 5	0 to 5	0 to 5

Notes:

* Bottom of footing elevation estimated based on as-built construction plans (circa 1970's/1980's), which includes correction for a 1.1 foot datum equality to match the current (2013) project datum [e.g. as-built EL 570.0 (circa 1980) = current EL 571.1 (2013)].

** River pool at EL 578.0 based on field survey conducted on 10/12/2013.

*** Visual Rating of Pile Condition: 1 = poor condition; 10 = fair condition.

Description		Less Corrosive	←	→		More Corrosive
Soil Resistivity	(ohm-cm)	> 1,000	701 to 1,000	301 to 700	101 to 300	≤ 100
Chloride Concentration	(mg/L)	≤ 700	701 to 2,000	2,001 to 4,000	4,001 to 8,000	> 8,000
Sulfate Concentration	(mg/L)	≤ 100	101 to 500	501 to 1,000	1,001 to 2,000	> 2,000
Bacteria Concentration	(colonies/mL)	NR	1 to 10	10 to 100	1,000 to 10,000	> 100,000
Half-Cell Potential, Maximum Range	(mV)	--	<100	100 to 300	301 to 800	> 800
Visual Rating of Pile Condition ****	--	--	9 to 10	6 to 8	1 to 4	--

Table 4 - Corrosion-Related Data (continued)

Description			Pier 18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
Noteworthy Surficial Soil/Fill Conditions Encountered			--	silt and clay	silt over	organic silt over	--	sand over	sandy silt over	--					silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay
			peat	shredded-wood debris below	paper sludge	--	wood pulp	shredded-wood debris	shredded-wood debris	wood fibers	River Pier	River Pier	River Pier	River Pier	--	with wood	with organic seams	--	--	--
			fly ash/ coal ash/ foundry sand	fly ash/ coal ash/ foundry sand	--	fly ash/ coal ash/ foundry sand	fly ash/ coal ash/ foundry sand	fly ash/ coal ash/ foundry sand	fly ash/ coal ash/ foundry sand	fly ash/ coal ash/ foundry sand					--	--	--	--	--	--
Half-Cell Potential, Maximum Range over 2 to 3 feet of Pile Length		(mV)	--	30	--	--	--	--	56	282	--	--	--	--	--	--	31	--	--	--
Soil Resistivity	In-Situ	(ohm-cm)	--	650 to 1,190	--	--	--	--	870 to 14,330	180 to 980	--	--	--	--	--	--	230 to 1,080	--	--	--
	Lab	(ohm-cm)	800 to 1,700	460 to 1,500	340 to 700	250 to 700	230 to 5,700	100 to 4,100	700 to 4,200	260 to 1,300	700 to 800	1,100 to 3,700	600 to 800	40 to 900	--	--	160 to 3,500	--	--	--
Chloride Concentration	Water	(mg/L)	1,060	--	1,440	--	1,390	1,540	--	1,130	--	--	--	--	--	27,100	--	20,400	--	10,400
	Soil	(mg/L)	222 to 1,980	104 to 4,060	1,960 to 7,280	463 to 3,970	30 to 2,870	139 to 6,180	34 to 2,170	98 to 7,710	34 to 257	<25 to 46	<35 to 51	161 to 6,430	--	--	55 to 10,300	--	--	--
Sulfate Concentration	Water	(mg/L)	81	--	235	--	1,600	23	--	1,030	--	--	--	--	--	2,140	--	1,390	--	685
	Soil	(mg/L)	123 to 518	27 to 357	62 to 249	79 to 462	34 to 21,800	31 to 176	26 to 217	67 to 1,150	41 to 87	<25 to <35	<30 to 44	<31 to 246	--	--	51 to 664	--	--	--
Microbial (MIC) test, time elapsed since inoculation		(days)	7	11	--	11	15	--	--	11	--	--	--	--	--	--	10	--	--	--
Aerobic	Low Nutrient Bacteria (measured to-date)	(colonies/mL)	1,000 to 10,000	1,000 to 10,000	--	10 to 100	--	--	--	1,000 to 10,000	--	--	--	--	--	--	1,000 to 10,000	--	--	--
	Iron Reducing Bacteria	(colonies/mL)	10 to 100	1 to 10	--	NR	--	--	--	1 to 10	--	--	--	--	--	--	1 to 10	--	--	--
Anaerobic	Anaerobic Bacteria	(colonies/mL)	>100,000	>100,000	--	10 to 100	--	--	--	1,000 to 10,000	--	--	--	--	--	--	1,000 to 10,000	--	--	--
	Acid Producing Bacteria	(colonies/mL)	1,000 to 10,000	>100,000	--	10 to 100	10,000 to 100,000	--	--	1 to 10	--	--	--	--	--	--	1,000 to 10,000	--	--	--
	Sulfate Reducing Bacteria	(colonies/mL)	1,000 to 10,000	1,000 to 10,000	--	NR	1 to 10	--	--	1 to 10	--	--	--	--	--	--	10 to 100	--	--	--
Bottom of Footing Elevation		(feet)	581.7	578.4	580.4	584.5	586.6	581.3	579.2	583.6	573.2 *	539.1 *	538.5 *	568.5 *	573.8 *	572 *	573.9 *	572.8 *	571.7 *	571.9 *
Elevaton of Water Level (measured on 10/22/2013)		--	581.3	--	580.7	--	580.5	580.4	--	579.9	--	--	--	--	--	578.9	--	579.8	--	579.7
Approximate Elevation of Water Seep(s) at Time of Test Pitting		(feet)	581.5	579	580	581	580	580	579	580	578.0 **	578.0 **	--	578.0 **	--	--	578	--	--	--
No. of Piles Visually Inspected		(piles)	2	24	13	8	11	2	24	15	--	--	--	--	--	--	15	--	--	--
No. of Piles Per Pier		(piles)	28	32	32	34	40	40	44	44	--	--	--	--	--	--	46	--	--	--
Exposed Pile - Visual Condition Rating		---	8	8	8	3	1	4	8	2	--	--	--	--	--	--	8	--	--	--
Average Section Loss Observed (visually rated) ***		(percent)	0 to 5	0 to 5	0 to 5	20 to 30	> 40	20 to 30	0 to 5	30 to 40	--	--	--	--	--	--	0 to 5	--	--	--

Notes:

* Bottom of footing elevation estimated based on as-built construction plans (circa 1970's/1980's), which includes correction for a 1.1 foot datum equality to match the current (2013) project datum [e.g. as-built EL 570.0 (circa 1980) = current EL 571.1 (2013)].

** River pool at EL 578.0 based on field survey conducted on 10/12/2013.

*** Visual Rating of Pile Condition: 1 = poor condition; 10 = fair condition.

Description		Less Corrosive	←		→	More Corrosive
Soil Resistivity	(ohm-cm)	> 1,000	701 to 1,000	301 to 700	101 to 300	≤ 100
Chloride Concentration	(mg/L)	≤ 700	701 to 2,000	2,001 to 4,000	4,001 to 8,000	> 8,000
Sulfate Concentration	(mg/L)	≤ 100	101 to 500	501 to 1,000	1,001 to 2,000	> 2,000
Bacteria Concentration	(colonies/mL)	NR	1 to 10	10 to 100	1,000 to 10,000	> 100,000
Half-Cell Potential, Maximum Range	(mV)	--	<100	100 to 300	301 to 800	> 800
Visual Rating of Pile Condition ****	--	--	9 to 10	6 to 8	1 to 4	--

Table 4 - Corrosion-Related Data (continued)

Description			Pier 36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	West Abutment
Noteworthy Surficial Soil/Fill Conditions Encountered			silt & lean clay	silt, clay & fine sand	sand over clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silt & lean clay	silty sand over lean clay
			--	--	--	with organics and trace black slime	--	--	with organics	with organics	with organics	--	--	--	with peat	--	with organics	--	--
			--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Half-Cell Potential, Maximum Range over 2 to 3 feet of Pile Length		(mV)	--	--	--	41	--	--	--	17	--	--	--	--	--	--	18	--	--
Soil Resistivity	In-Situ	(ohm-cm)	--	--	--	210 to 4,770	--	--	--	510 to 1,850	--	--	--	--	--	--	--	--	--
	Lab	(ohm-cm)	--	--	--	80 to 1,500	--	--	--	310 to 900	800 to 1,600	--	--	--	--	--	200 to 1,700	--	--
Chloride Concentration	Water	(mg/L)	--	18,300	--	9,810	--	2,460	--	1,470	--	2,080	--	1,150	--	532	--	5,680	--
	Soil	(mg/L)	--	--	--	266 to 11,400	--	--	--	400 to 1,190	101 to 362	--	--	--	--	--	155 to 1,980	--	--
Sulfate Concentration	Water	(mg/L)	--	901	--	592	--	275	--	59	--	222	--	255	--	375	--	146	--
	Soil	(mg/L)	--	--	--	70 to 2,520	--	--	--	<27 to 875	77 to 1,850	--	--	--	--	--	97 to 859	--	--
Microbial (MIC) test, time elapsed since inoculation		(days)	--	--	--	8	--	--	--	10	--	--	--	--	--	--	--	--	--
Aerobic	Low Nutrient Bacteria (measured to-date)	(colonies/mL)	--	--	--	1,000 to 10,000	--	--	--	1,000 to 10,000	--	--	--	--	--	--	--	--	--
	Iron Reducing Bacteria	(colonies/mL)	--	--	--	10 to 100	--	--	--	1,000 to 10,000	--	--	--	--	--	--	--	--	--
Anaerobic	Anaerobic Bacteria	(colonies/mL)	--	--	--	>100,000	--	--	--	>100,000	--	--	--	--	--	--	--	--	--
	Acid Producing Bacteria	(colonies/mL)	--	--	--	>100,000	--	--	--	1,000 to 10,000	--	--	--	--	--	--	--	--	--
	Sulfate Reducing Bacteria	(colonies/mL)	--	--	--	1,000 to 10,000	--	--	--	10 to 100	--	--	--	--	--	--	--	--	--
Bottom of Footing Elevation		(feet)	572.8 *	575.4 *	575 *	574 *	574.6 *	575.3 *	574.1 *	574.4 *	574 *	576.9 *	576.3 *	576.9 *	575.2 *	576.8 *	577 *	576.9 *	601.5 *
Elevaton of Water Level (measured on 10/22/2013)		--	--	579.6	--	575.7	--	576.3	--	580.7	--	580.7	--	581.0	--	580.8	--	580.3	--
Approximate Elevation of Water Seep(s) at Time of Test Pitting		(feet)	--	--	--	578	--	--	--	578	--	--	--	--	--	--	--	--	--
No. of Piles Visually Inspected		(piles)	--	--	--	15	--	--	--	4	3	--	--	--	--	--	3	--	--
No. of Piles Per Pier		(piles)	--	--	--	34	--	--	--	32	36	--	--	--	--	--	24	--	--
Exposed Pile - Visual Condition Rating		---	--	--	--	6	--	--	--	10	10	--	--	--	--	--	10	--	--
Average Section Loss Observed (visually rated) ***		(percent)	--	--	--	5 to 10	--	--	--	0 to 5	0 to 5	--	--	--	--	--	0 to 5	--	--

Notes:
* Bottom of footing elevation estimated based on as-built construction plans (circa 1970's/1980's), which includes correction for a 1.1 foot datum equality to match the current (2013) project datum [e.g. as-built EL 570.0 (circa 1980) = current EL 571.1 (2013)].
** River pool at EL 578.0 based on field survey conducted on 10/12/2013.
*** Visual Rating of Pile Condition: 1 = poor condition; 10 = fair condition.

Description		Less Corrosive	⟵	⟶		More Corrosive
Soil Resistivity	(ohm-cm)	> 1,000	701 to 1,000	301 to 700	101 to 300	≤ 100
Chloride Concentration	(mg/L)	≤ 700	701 to 2,000	2,001 to 4,000	4,001 to 8,000	> 8,000
Sulfate Concentration	(mg/L)	≤ 100	101 to 500	501 to 1,000	1,001 to 2,000	> 2,000
Bacteria Concentration	(colonies/mL)	NR	1 to 10	10 to 100	1,000 to 10,000	> 100,000
Half-Cell Potential, Maximum Range	(mV)	--	<100	100 to 300	301 to 800	> 800
Visual Rating of Pile Condition ****	--	--	9 to 10	6 to 8	1 to 4	--

Table 5 - Foundation Setting

Description		East Abutment	Pier																																																	West Abutment
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	
Organics Encountered in Test Pit(s) and/or Test Boring(s)		--	--	--	--	--	--	organics		organic silt/peat	--	organics	--	organics			shredded-wood debris, pulp, paper slude and/or peat						--	wood pulp	shredded-wood debris		wood fibers	River Piers				--	wood	organic seams	--	--	--	--	--	--	organic, trace black slime	--	--	organics			--	--	--	peat	--	organics	--	--
Ash Encountered in Test Pit(s) and/or Test Borings		--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	fly ash/coal ash/foundry sand		--	fly ash/ coal ash/ foundry sand					River Piers				--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Soil Resistivity - In-Situ	(ohm-cm)	No Data, Assumed Similar to Pier 5					550 to 1,440	No Data, Assumed Similar to Pier 5			--	--	--	--	800 to 1,560	--	--	--	--	--	650 to 1,190	--	--	--	--	870 to 14,330	180 to 980	--	--	--	--	--	--	230 to 1,080	--	--	--	--	--	--	210 to 4,770	--	--	--	510 to 1,850	--	--	--	--	--	--	--	--	--
Soil Resistivity - Lab	(ohm-cm)	No Data, Assumed Similar to Pier 5					1,100	200 to 1,100	No Data, Assumed Similar to Pier 6		No Data, Assumed Similar to Pier 10	470 to 600	220 to 800	160	300 to 1,600	700 to 1,500	1,100 to 1,300	800 to 900	440 to 900	800 to 1,700	460 to 1,500	340 to 700	250 to 700	230 to 5,700	100 to 4,100	700 to 4,200	260 to 1,300	700 to 800	1,100 to 3,700	600 to 800	40 to 900	No Data, Assumed Similar to Pier 29	No Data, Assumed Similar to Pier 32	160 to 3,500	No Data, Assumed Similar to Pier 32			No Data, Assumed Similar to Pier 39			80 to 1,500	No Data, Assumed Similar to Pier 39		No Data, Assumed Similar to Pier 43	310 to 900	800 to 1,600	No Data, Assumed Similar to Pier 44			No Data, Assumed Similar to Pier 50		200 to 1,700	No Data, Assumed Similar to Pier 50	--
Active Microbial Activity with > 1,000 colonies / mL	(yes/no)	No Data, Assumed Similar to Pier 5					yes	No Data, Assumed Similar to Pier 5			No Data, Assumed Similar to Pier 10	yes	yes	yes	yes	yes	yes	yes	yes	yes	yes	No Data, Assumed Similar to Pier 19	no	No Data, Assumed Similar to Pier 25			yes	--	--	--	--	No Data, Assumed Similar to Pier 32		yes	No Data, Assumed Similar to Pier 32			No Data, Assumed Similar to Pier 39			yes	No Data, Assumed Similar to Pier 39		No Data, Assumed Similar to Pier 43	yes	No Data, Assumed Similar to Pier 43								
Bottom of Footing Elevation	(feet)	601.7 *	574.2 *	573.1 *	572.6 *	572.4 *	571.4 *	577.6 *	577.7 *	578.1 *	569.4 *	570.1	569.1	578.8	578.6	580.7	579.2	578.6	579.6	581.7	578.4	580.4	584.5	586.6	581.3	579.2	583.6	573.2 *	539.1 *	538.5 *	568.5 *	573.8 *	572 *	573.9 *	572.8 *	571.7 *	571.9 *	572.8 *	575.4 *	575 *	574 *	574.6 *	575.3 *	574.1 *	574.4 *	574 *	576.9 *	576.3 *	576.9 *	575.2 *	576.8 *	577 *	576.9 *	601.5 *
Elevaton of Water Level (measured on 10/22/2013)	--	--	--	575.8	--	568.5	--	563.9	--	567.5	562.2	550.5	--	580.1	--	580.6	--	581.0	--	581.3	--	580.7	--	580.5	580.4	--	579.9	--	--	--	--	--	578.9	--	579.8	--	579.7	--	579.6	--	575.7	--	576.3	--	580.7	--	580.7	--	581.0	--	580.8	--	580.3	--
Approximate Elevation of Water Seep(s) at Time of Test Pitting	(feet)	--	--	--	--	--	569	--	--	--	--	570	569	579	579	581	581	581	580	581.5	579	580	581	580	580	579	580	578.0 **	578.0 **	--	578.0 **	--	--	578	--	--	--	--	--	--	578	--	--	--	578	--	--	--	--	--	--	--	--	--
No. of Piles Visually Inspected	(piles)	--	--	--	--	--	6	3	--	--	--	4	2	2	12	2	2	2	2	2	24	13	8	11	2	24	15	--	--	--	--	--	--	15	--	--	--	--	--	--	15	--	--	--	4	3	--	--	--	--	--	3	--	--
No. of Piles Per Pier	(piles)	--	--	--	--	--	24	20	--	--	--	30	36	32	34	32	36	34	34	28	32	32	34	40	40	44	44	--	--	--	--	--	--	46	--	--	--	--	--	--	34	--	--	--	32	36	--	--	--	--	--	24	--	--
Exposed Pile - Visual Condition Rating ***	---	--	--	--	--	--	10	10	--	--	--	10	10	7	8	7	10	10	10	8	8	8	3	1	4	8	2	--	--	--	--	--	--	8	--	--	--	--	--	--	6	--	--	--	10	10	--	--	--	--	--	10	--	--
Average Section Loss Observed (visually rated)	(percent)	--	--	--	--	--	0 to 5	0 to 5	--	--	--	0 to 5	0 to 5	5 to 10	5 to 10	5 to 10	0 to 5	0 to 5	0 to 5	0 to 5	0 to 5	0 to 5	20 to 30	> 40	20 to 30	0 to 5	30 to 40	--	--	--	--	--	--	0 to 5	--	--	--	--	--	--	5 to 10	--	--	--	0 to 5	0 to 5	--	--	--	--	--	0 to 5	--	--
Tier 1 Criteria (A): H-Piles Embedded in Ash **and** Free Water Surface is Below Bottom of Footer, **and** fine grained soil w/ minimum resistivity < 300 ohm-cm near Pier, **and** Visual Condition of Exposed Pile Ranked 1 to 4	(yes/no)	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	yes	yes	yes	no	yes	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no
Tier 2 - Further Study Required: Criteria (B): Areas with Organics Encountered at Test Pits and/or Test Borings, **and** fine grained soil w/ minimum resistivity < 1,000 ohm-cm near Pier, **and** Active Microbes with > 1,000 Colonies/mL in Proximity to Pier, **and** Visual Condition of Exposed Pile Ranked 5 to 8	(yes/no)	--	--	--	--	--	no	--	--	--	--	no	no	yes	yes	yes	no	no	no	yes	yes	yes	no	no	no	yes	no	--	--	--	--	--	--	yes	--	--	--	--	--	--	yes	--	--	--	no	--	--	--	--	--	--	--	--	--
Tier 2 - Further Study Required: Criteria (C): Areas with **Organics** Encountered at Test Pits and/or Test Borings, **and** fine grained soil w/ minimum resistivity < 1,000 ohm-cm near Pier, **and** Active Microbes with > 1,000 Colonies/mL in Proximity to Pier, **and** Without Visual Condition Rating of Piling	(yes/no)	no	no	no	no	no	--	yes	yes	yes	no	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	no	yes	--	no	no	no	no	no	no	--	no	no	yes	--	yes	no	no	no	yes	no	yes	no	no
Tier 3 Criteria (D): **not** Included in Criteria A, B or C	(yes/no)	yes	yes	yes	yes	yes	yes	no	no	no	yes	yes	yes	no	no	no	yes	yes	yes	no	no	no	no	no	no	no	no	yes	yes	yes	yes	yes	no	no	yes	yes	yes	yes	yes	yes	no	yes	yes	no	yes	no	yes	yes	yes	no	yes	no	yes	yes
Foundation Setting		D	D	D	D	D	D	C	C	C	D	D	D	B	B	B	D	D	D	B	B	B	A	A	A	B	A	D	D	D	D	D	C	B	D	D	D	D	D	D	B	D	D	C	D	C	D	D	D	C	D	C	D	D

Notes:

* Bottom of footing elevation was estimated based on the as-built construction plans (circa 1970's/1980's), which were adjusted to accommodate a 1.1 foot elevation equality to match the current (2013) project datum [e.g. as-built EL 570.0 (circa 1980) = current EL 571.1 (2013)].

** River pool at EL 578.0 based on field survey conducted on 10/12/2013.

*** Visual Rating of Pile Condition: 1 = poor condition; 10 = fair condition.

Description		Less Corrosive	←		→	More Corrosive
Soil Resistivity	(ohm-cm)	> 1,000	701 to 1,000	301 to 700	101 to 300	≤ 100
Bacteria Concentration	(colonies/mL)	NR	1 to 10	10 to 100	1,000 to 10,000	> 100,000
Visual Rating of Pile Condition ***		--	9 to 10	6 to 8	1 to 4	--

Table 6 - Structural Capacity Pile Analysis - Summary of Results

CASE 1
Uniform Section Loss To All Piles, Structural Capacity Check, Pile Operating Rating Is Not Satisfied At One Pile; Geotechnical Capacity is OK By Default

	Pier 12	Pier 13	Pier 14	Pier 18	Pier 19	Pier 20	Pier 39
	No Redistribution	No Redistribution	No Redistribution	No Redistribution	No Redistribution	No Redistribution	No Redistribution
Total Pile Percent Uniform Section Loss Causing One Pile to Not Satisfy Operating Rating	45%	45%	45%	35%	40%	35%	40%

CASE 2
Uniform Section Loss To All Piles, Structural Capacity Check, Pile Operating Rating Is Not Satisfied For The Pile System After Load Redistribution; Verify Geotechnical Capacity Is OK

	Pier 12		Pier 13		Pier 14		Pier 18		Pier 19		Pier 20		Pier 39	
Pile Capacity Retained After Operating Rating Not Satisfied	50% Capacity	100% Capacity	50% Capacity	100% Capacity	50% Capacity	100% Capacity	50% Capacity	100% Capacity	50% Capacity	100% Capacity	50% Capacity	100% Capacity	50% Capacity	100% Capacity
Total Percent Uniform Section Loss Causing One Pile to Not Satisfy Operating Rating	45%	45%	45%	45%	45%	45%	35%	35%	40%	40%	35%	35%	40%	40%
Total Percent Uniform Section Loss Causing the Final Pile to Not Satisfy Operating Rating	25%	15%	10%	5%	10%	10%	10%	15%	2%	10%	15%	5%	5%	10%
Number of Times Pile Group is able to Redistribute after One Pile Does Not Satisfy Operating Rating	1	2	1	2	1	2	3	3	1	1	1	2	4	4
Number of Piles Allowed to Exceed Geotechnical Operating Rating Before Pile Group Operating Rating Not Satisfied	2	4	2	3	2	3	2	2	1	2	2	2	2	3
Number of Piles Allowed to Exceed Structural Operating Rating Before Pile Group Operating Rating Not Satisfied	3	5	3	6	3	4	3	3	3	3	2	4	4	4

CASE 3
Non-Uniform Section Loss, Local Structural Capacity Check, Pile Operating Rating For Local Flange Capacity Is Not Satisfied At One Pile; Geotechnical Capacity Is OK By Default

	Pier 12	Pier 13	Pier 14	Pier 18	Pier 19	Pier 20	Pier 39
	No Redistribution	No Redistribution	No Redistribution	No Redistribution	No Redistribution	No Redistribution	No Redistribution
Local Pile Flange Percent Section Loss Causing One Pile to Not Satisfy Flange Buckling Operating Rating	44%	44%	42%	34%	38%	37%	39%

CASE 4
Non-Uniform Section Loss, Local Structural Capacity Check, Pile Operating Rating For Local Flange Capacity Is Not Satisfied For The Pile System After Load Redistribution; Verify Geotechnical Capacity Is OK

	Pier 12		Pier 13		Pier 14		Pier 18		Pier 19		Pier 20		Pier 39	
Pile Capacity Retained After Operating Rating Not Satisfied	50% Capacity	100% Capacity	50% Capacity	100% Capacity	50% Capacity	100% Capacity	50% Capacity	100% Capacity	50% Capacity	100% Capacity	50% Capacity	100% Capacity	50% Capacity	100% Capacity
Local Pile Flange Percent Section Loss Causing One Pile to Not Satisfy Flange Buckling Operating Rating	44%	44%	44%	44%	42%	42%	34%	34%	38%	38%	37%	37%	39%	39%
Local Pile Flange Percent Section Loss Causing the Final Pile to Not Satisfy Flange Buckling Operating Rating	19%	19%	15%	11%	15%	16%	13%	20%	1%	15%	20%	6%	6%	16%
Number of Times Pile Group is able to Redistribute after One Pile Does Not Satisfy Flange Buckling Operating Rating	1	2	1	3	1	2	4	4	1	1	1	2	4	4
Number of Piles Allowed to Exceed Geotechnical Operating Rating Before Pile Group Operating Rating Not Satisfied	2	3	2	3	2	3	2	2	1	2	2	2	2	3
Number of Piles Allowed to Exceed Structural Operating Rating Before Pile Group Operating Rating Not Satisfied	3	4	2	5	3	4	3	3	3	3	2	4	4	4

Exhibits

Exhibits

Summary of Contents

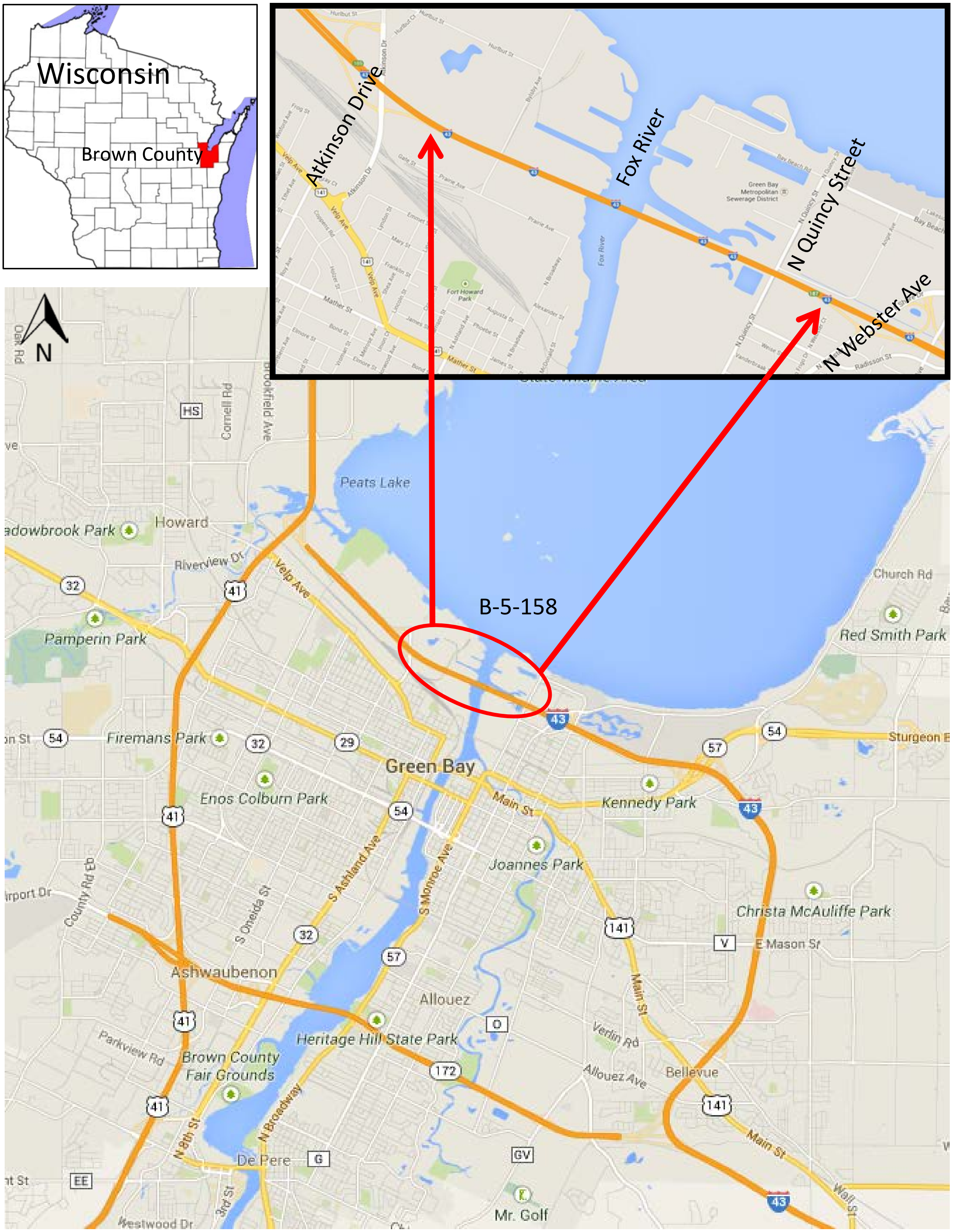
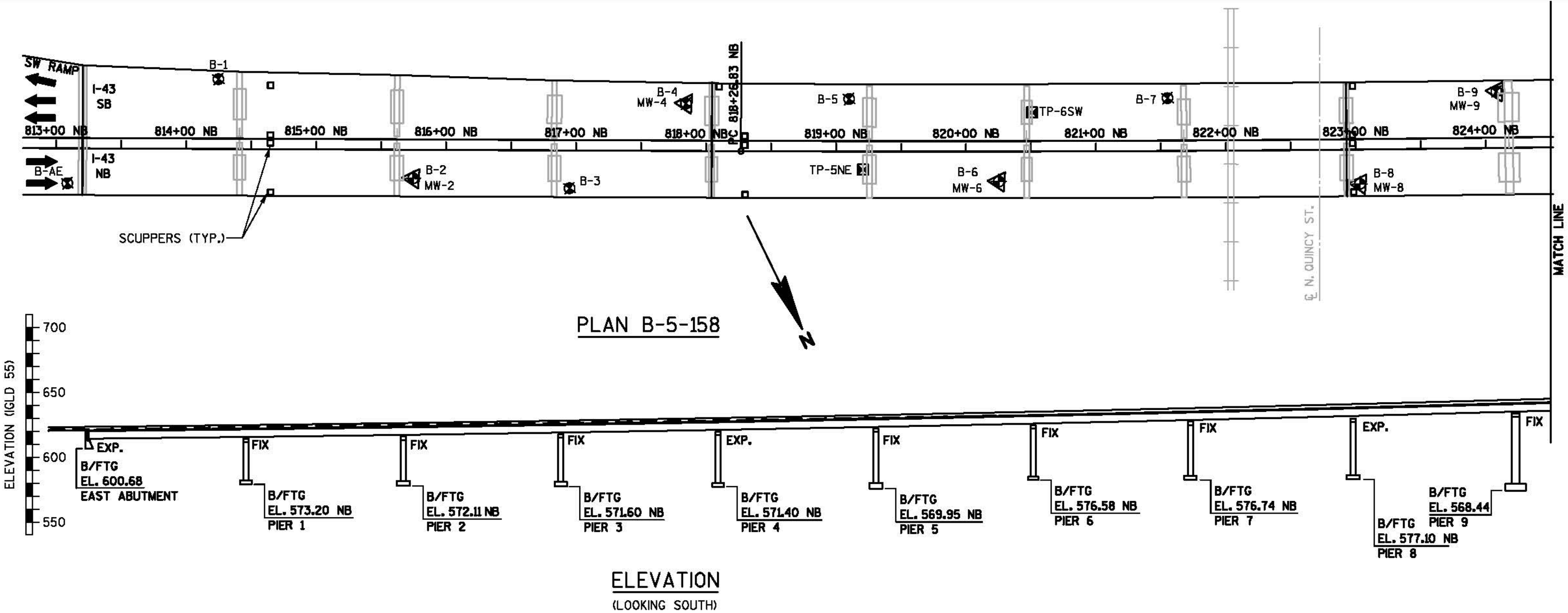


Exhibit 1: Project Vicinity Map

Exhibit 2 – Reach Layout



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8

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION

STRUCTURE B-5-158

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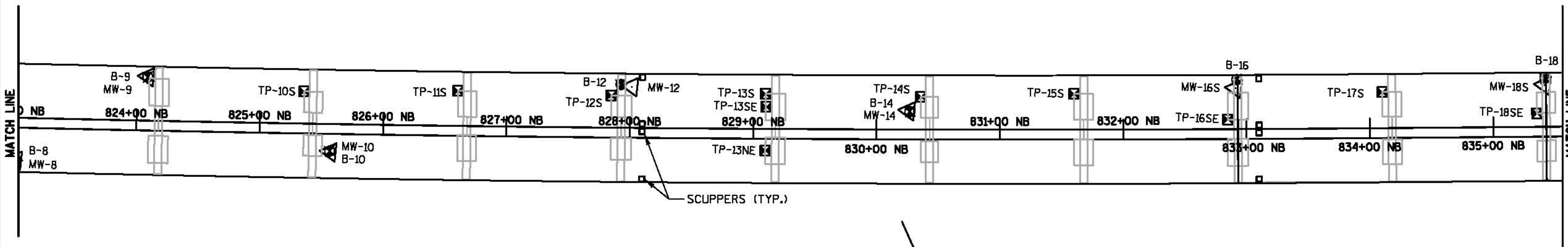
EXHIBIT 3

STATE PROJECT NUMBER



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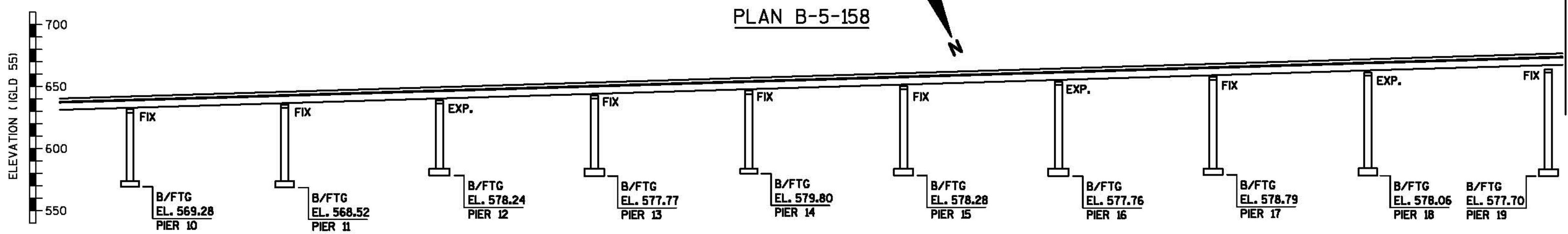

ELEVATION
(LOOKING SOUTH)

ALL ELEVATIONS TO BOTTOM OF FOOTING

8

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION

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EXHIBIT 3

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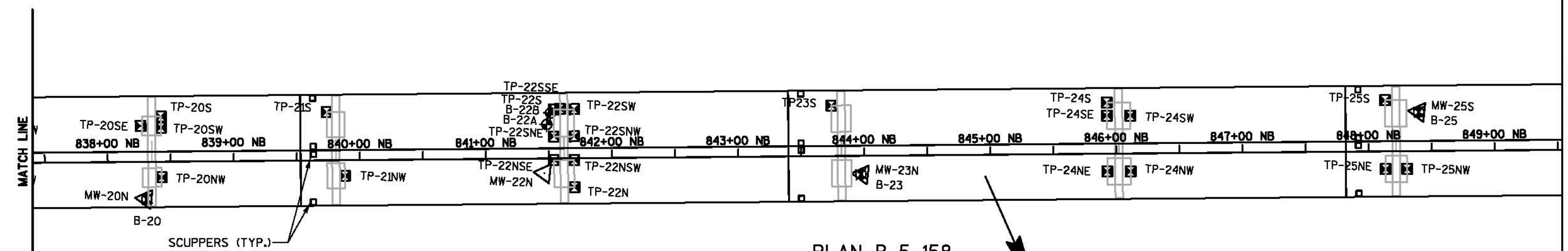

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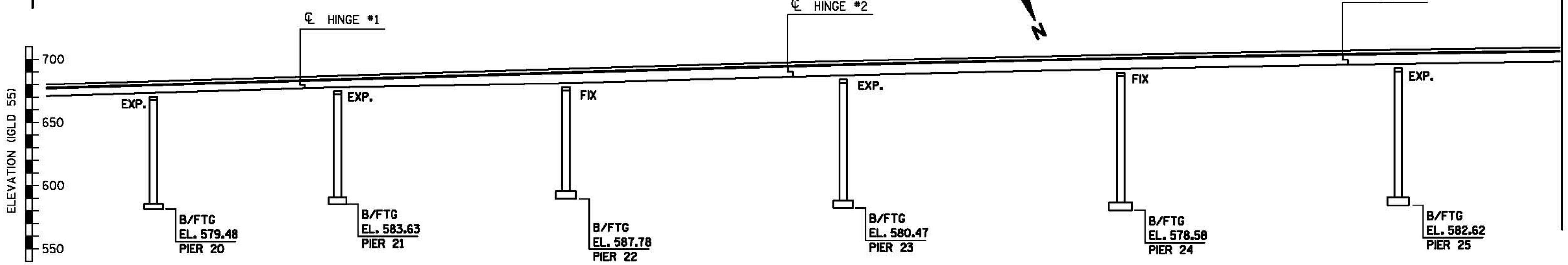

ELEVATION

(LOOKING SOUTH)

ALL ELEVATIONS TO BOTTOM OF FOOTING

8

NO.	DATE	REVISION	BY

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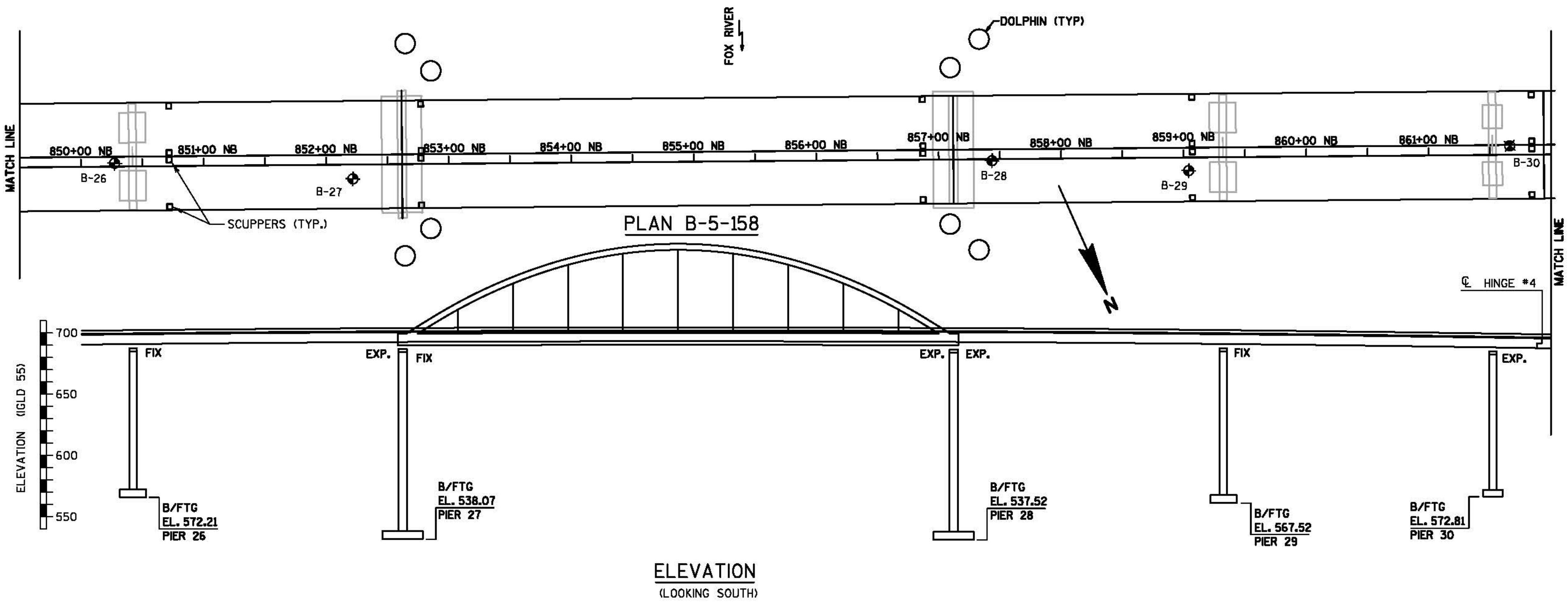

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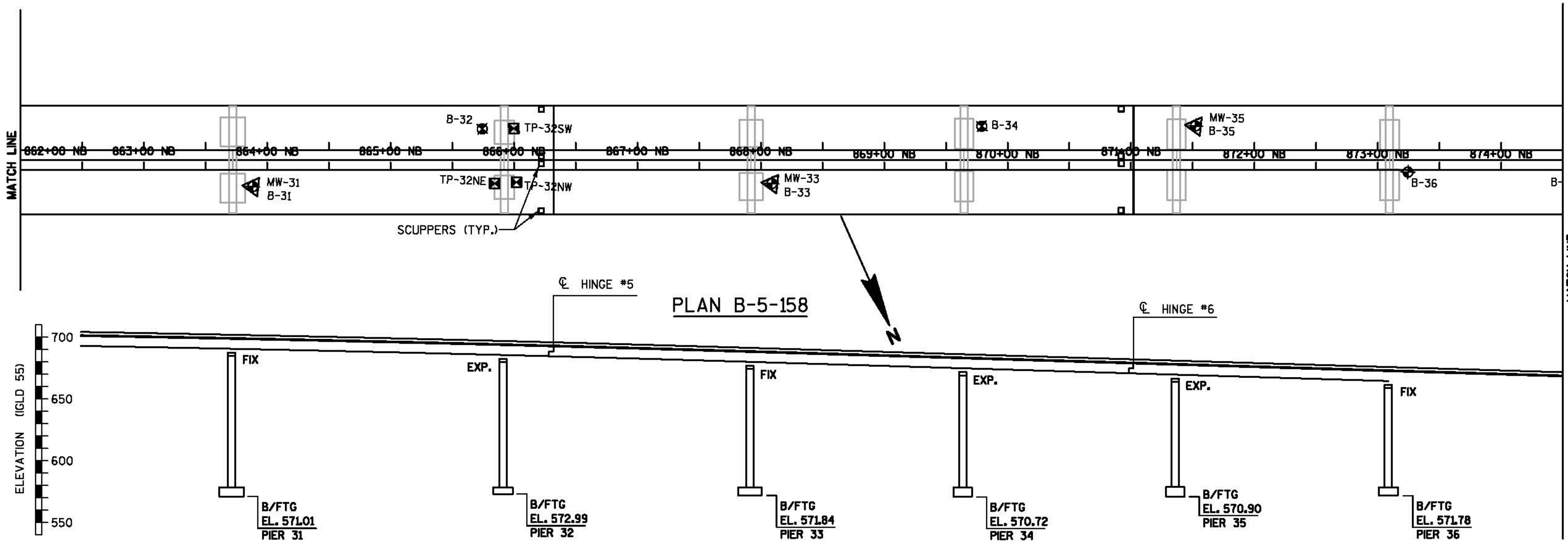

PLAN B-5-158

ELEVATION

(LOOKING SOUTH)

8

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STATE OF WISCONSIN
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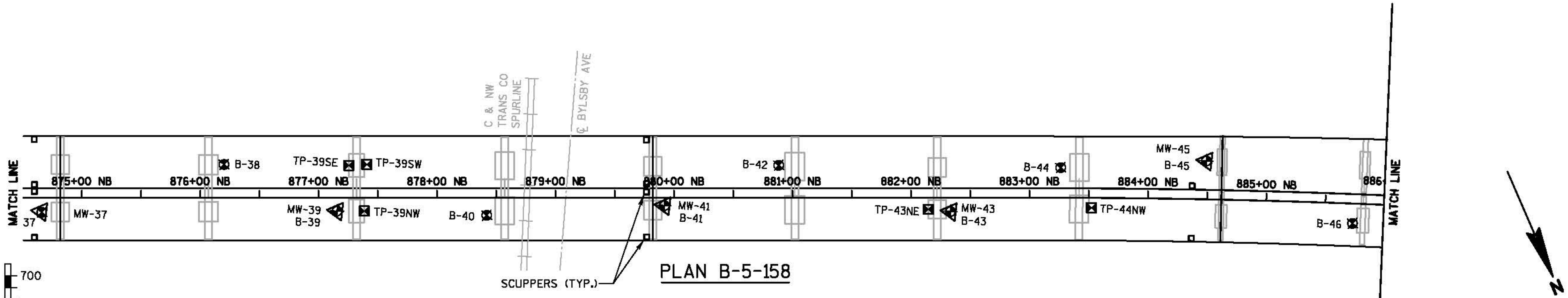
EXHIBIT 3



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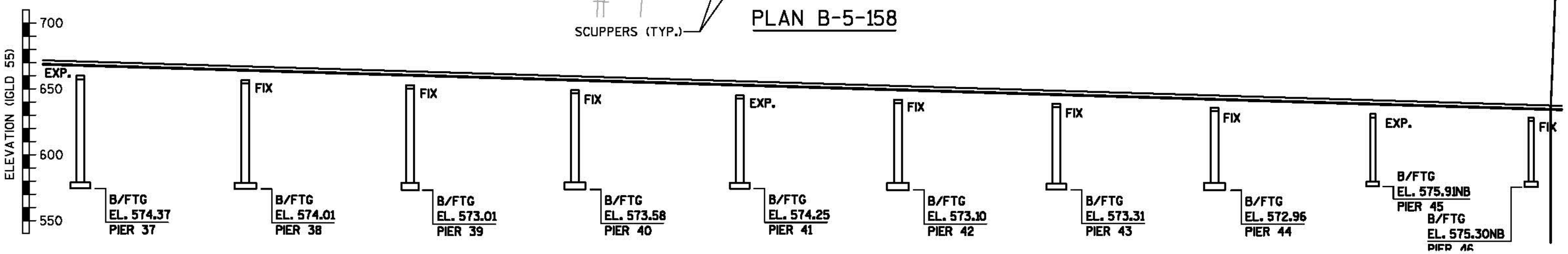

ELEVATION
(LOOKING SOUTH)

STATE PROJECT NUMBER

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION

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EXHIBIT 3

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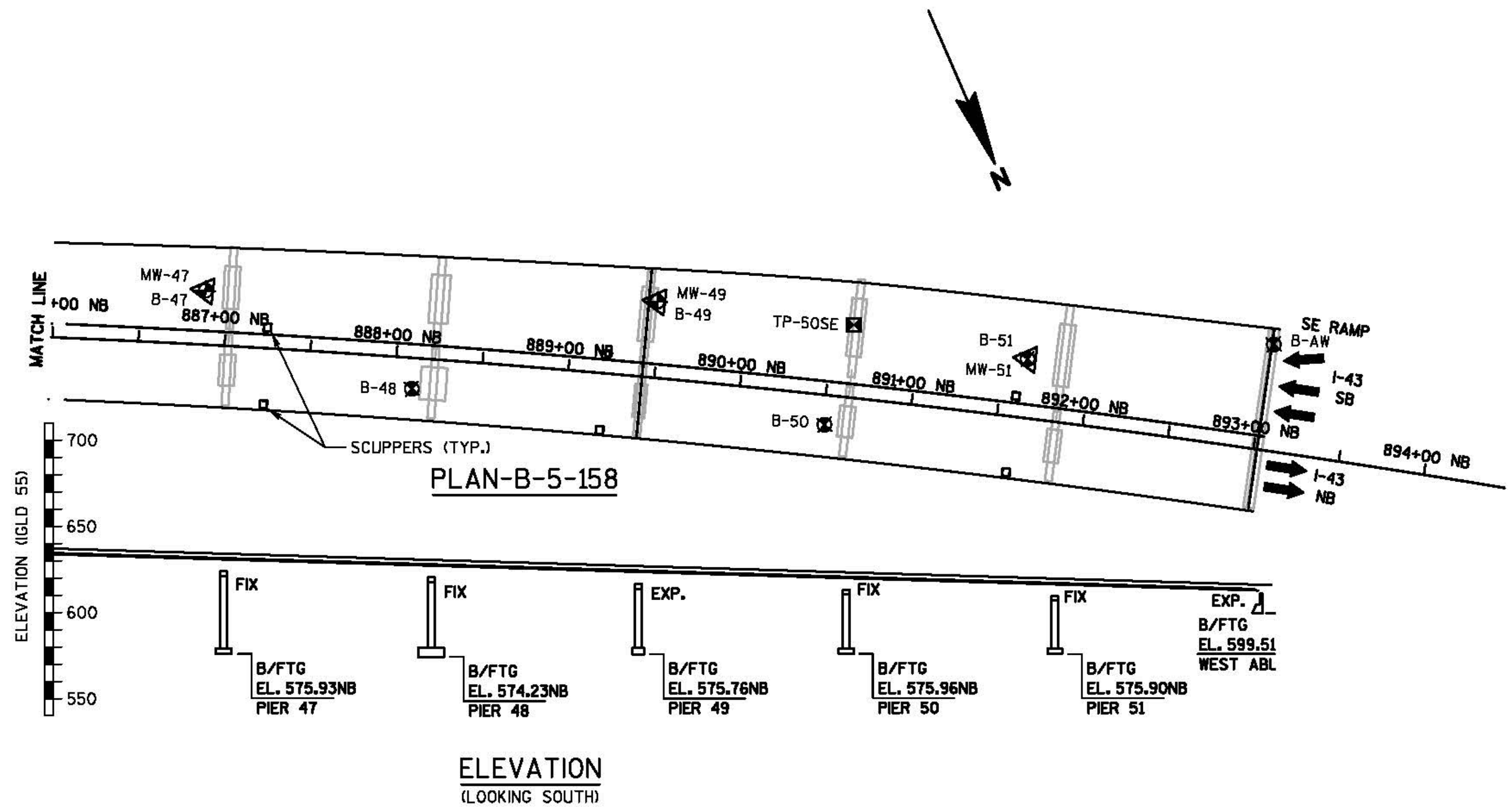
STATE PROJECT NUMBER



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NO.	DATE	REVISION	BY

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EXHIBIT 3

7 OF 7

Exhibit 4 – Historic Land Use Aerials

1938

Exhibit 4 – Historic Land Use Aerials

1960

Exhibit 4 – Historic Land Use Aerials

1978

Exhibit 4 – Historic Land Use Aerials

1992

Exhibit 4 – Historic Land Use Aerials

2005

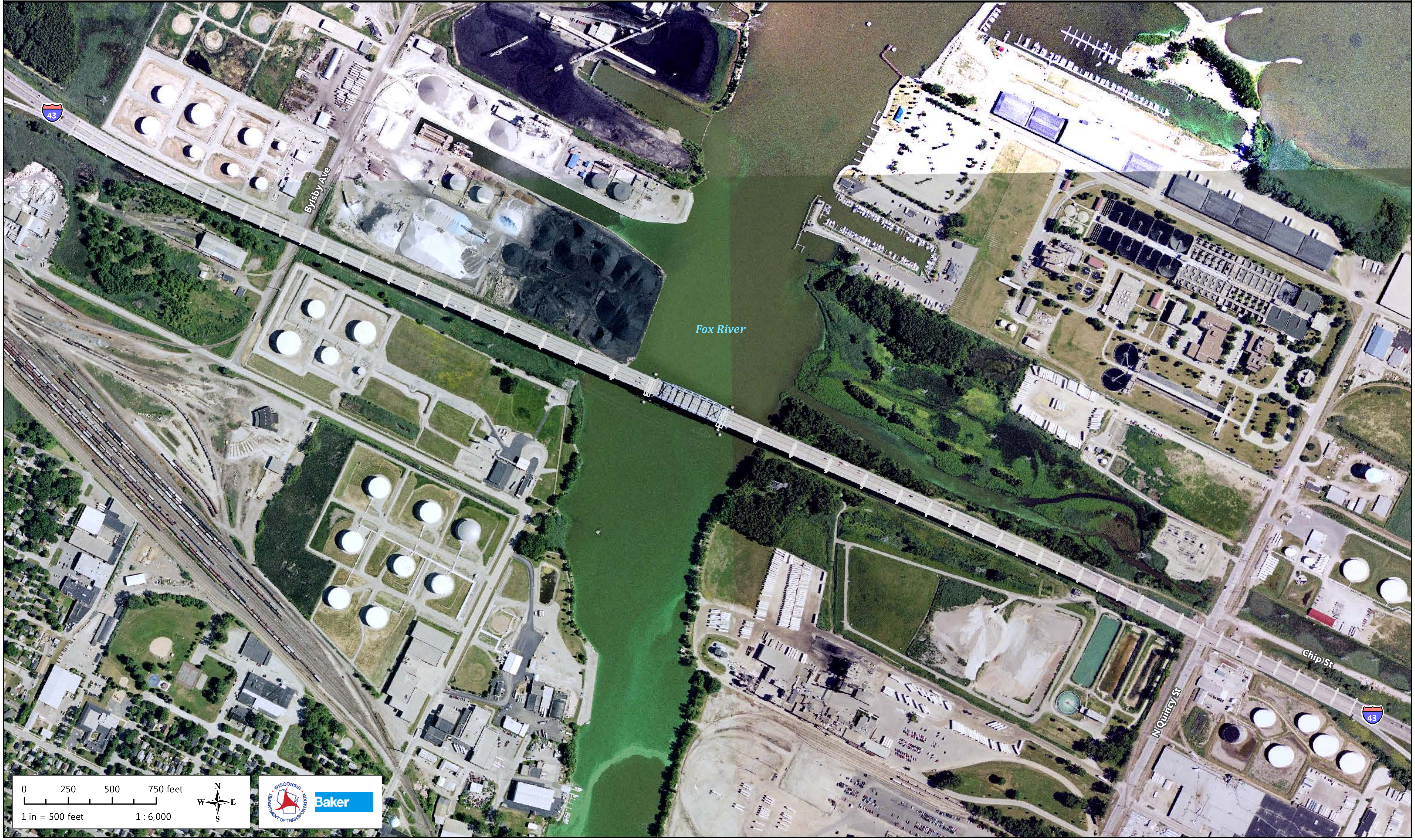


Exhibit 4 – Historic Land Use Aerials

2010

Exhibit 5 – Noteworthy Physical Features

LEGEND

Industrial Features

- Tower
- Electric Transmission Line(s)
- Petroleum Pipeline(s)
- Drainage Flow

Hydrographic Features

- Existing Water Body
- Abandoned Settling Basin

Contours

- 10' Index Contour

Bridge Alignment

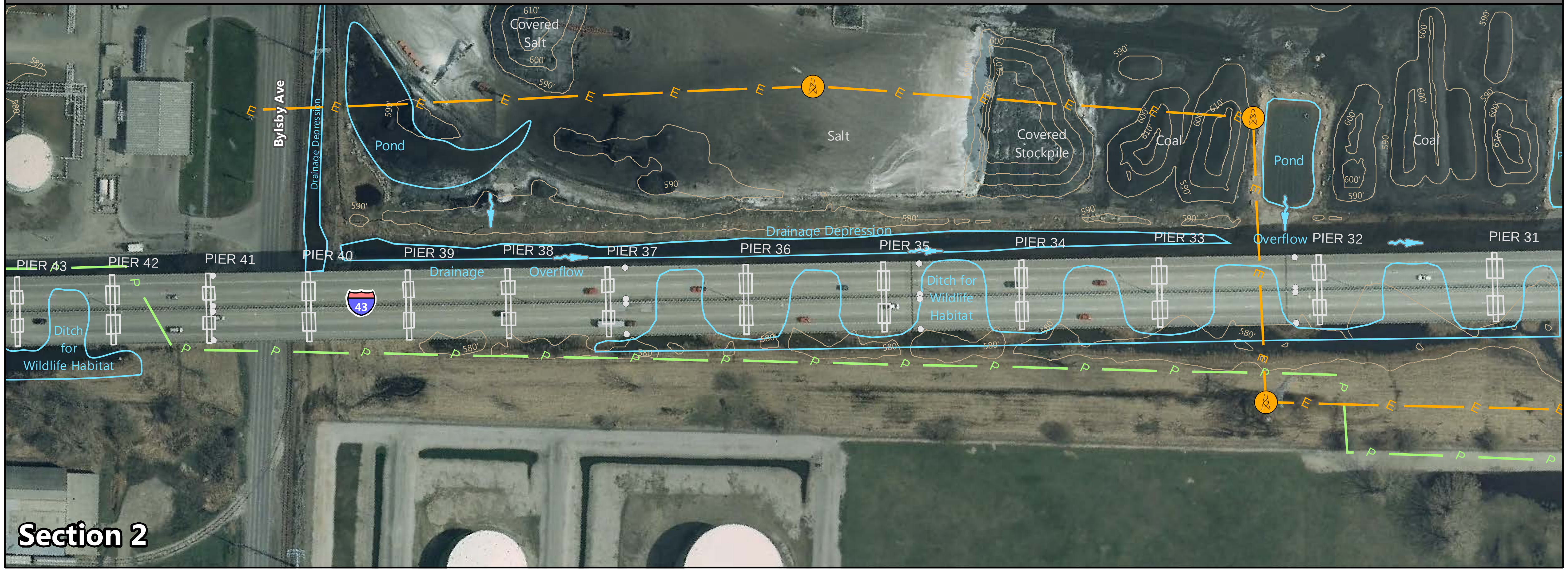
- Substructure
- Scupper

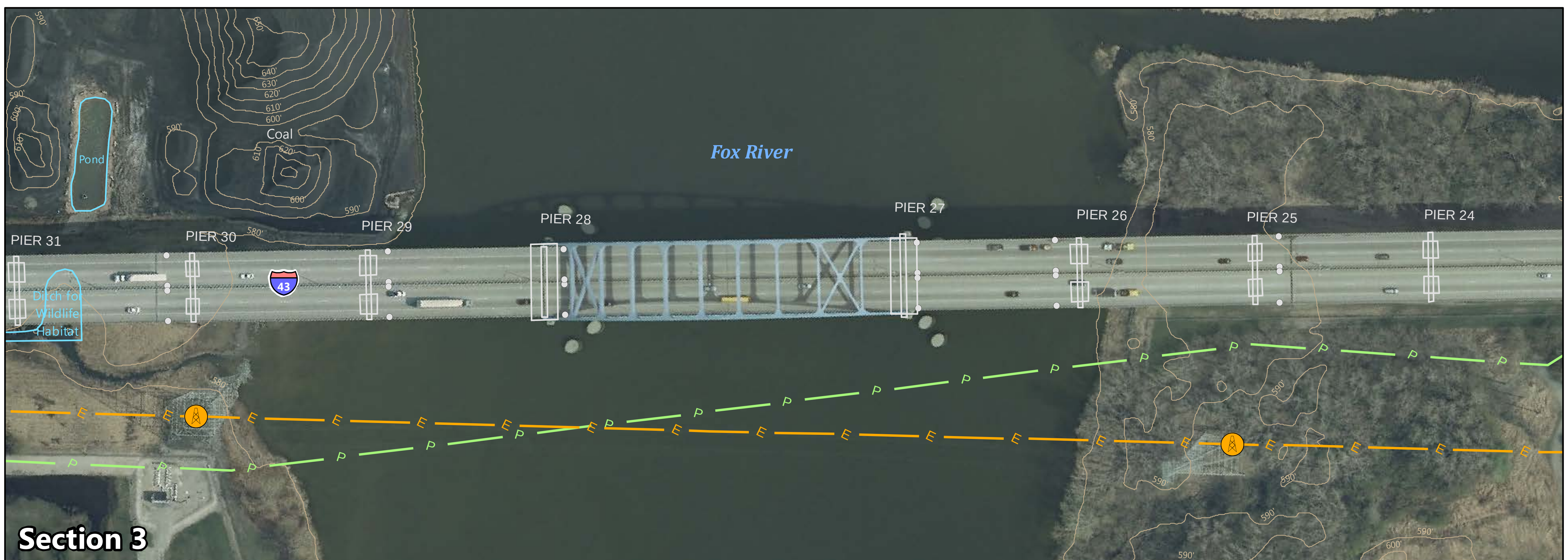
0 50 100 150 feet

1 in = 150 feet 1 : 1,800

N E S W

Exhibit 5 – Noteworthy Physical Features





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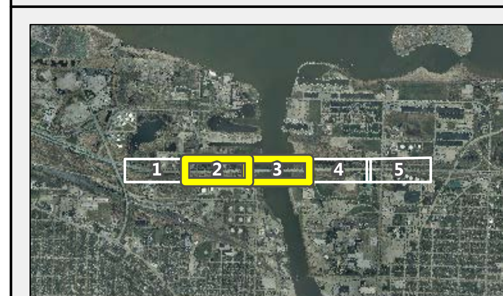

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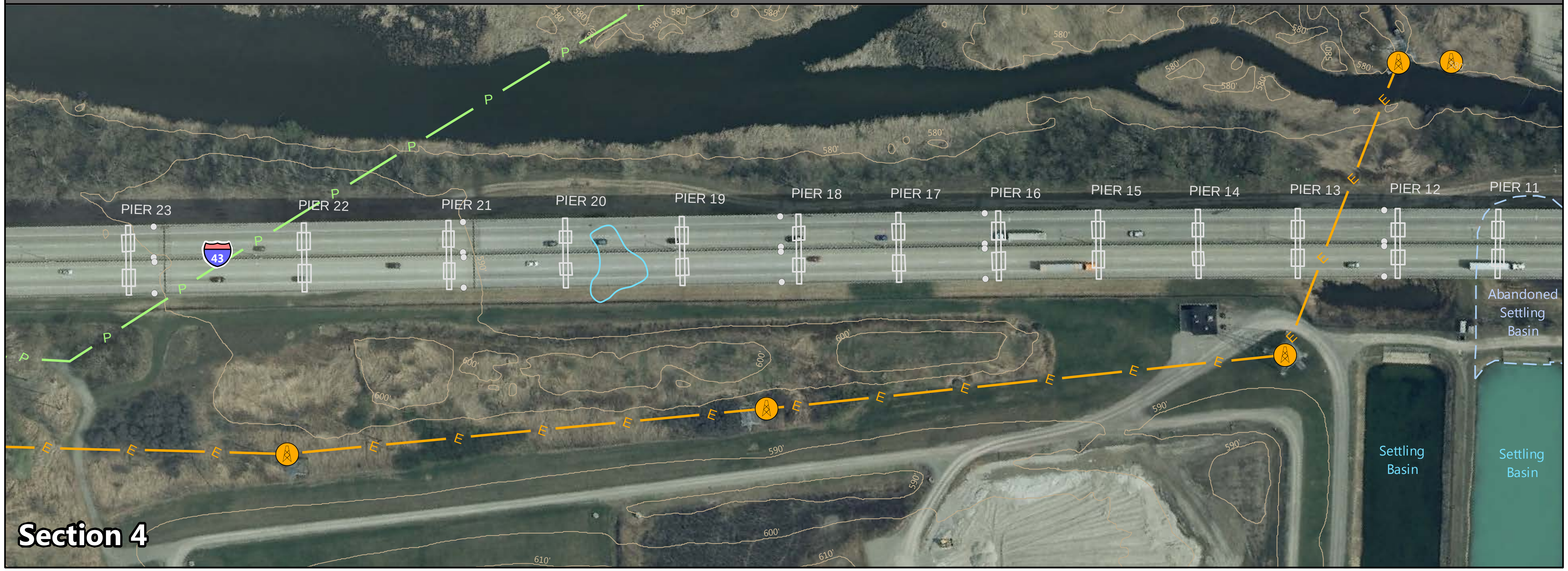

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Hydrographic Features

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Contours

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Bridge Alignment

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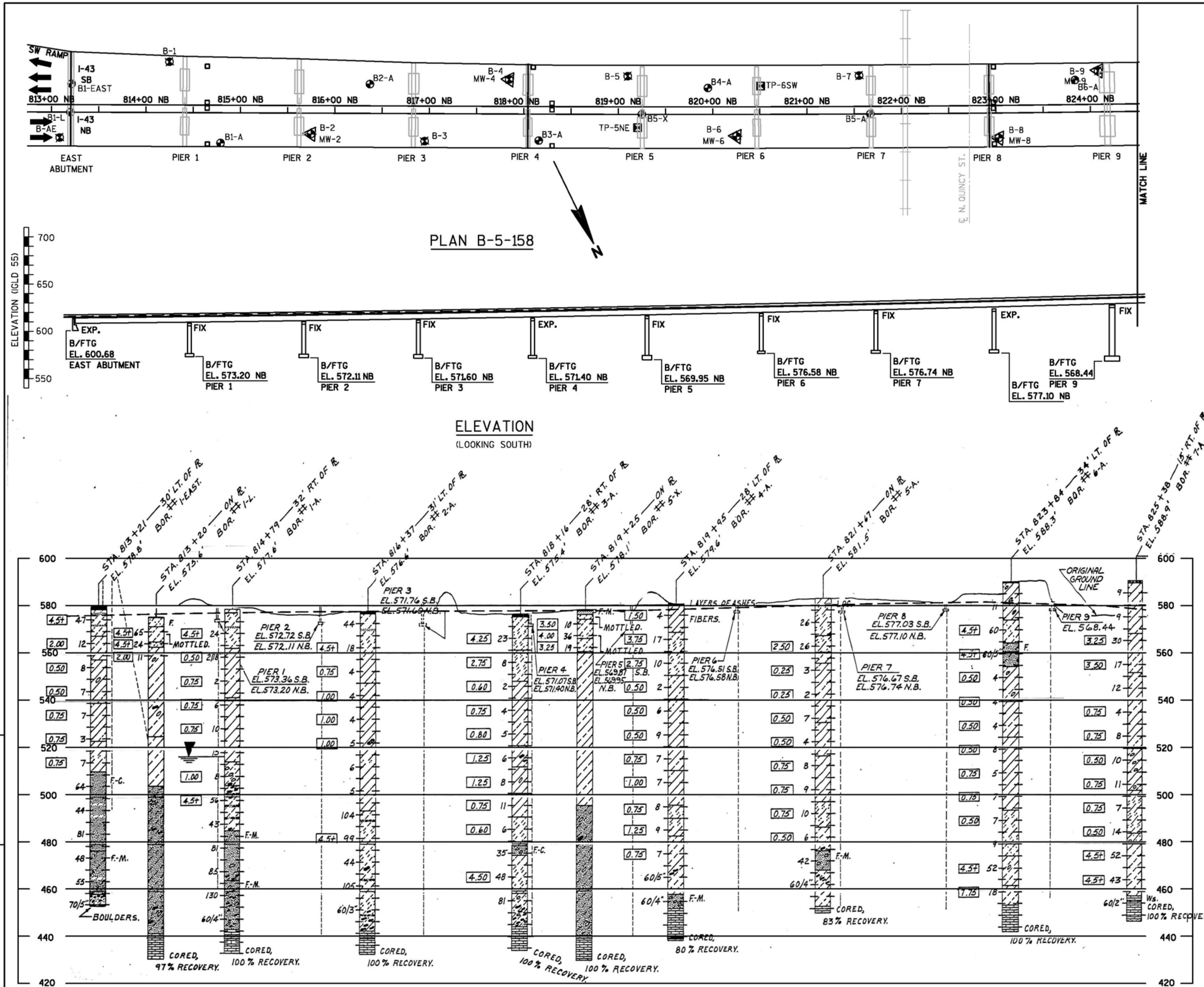
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

N E S W

Baker



STATE PROJECT NUMBER

ABBREVIATIONS

F — FINE　M — MEDIUM　C — COARSE
WS — WEATHERED　SO — SOUND

MATERIAL SYMBOLS

TOPSOIL　SILT　SANDSTONE
SAND　PEAT　LIMESTONE
GRAVEL　CLAY　IGNEOUS ROCK

LEGEND OF PROBING

PROBING NO.
STA.
ELEVATION
95/6=95 BLOWS FOR 6" PENETRATION
PROBING TAKEN WITH A 350# WT. FALLING 18" ON A 2" O.D. POINT.
7 AVERAGE BLOWS PER FOOT
REFUSAL 95/6

LEGEND OF BORING

BORING NO.
ELEV.　STA.
UNCONFINED STRENGTH 7.7
BLOWS PER FT. USING 140# WT. FALLING 30"
WASH SAMPLE
SHELBY TUBE — S.T.
GROUND WATER ELEVATION
NO GROUND WATER OBSERVED ABOVE THIS ELEVATION
SANDY GRAVEL
F. BOULDERS OR COBBLES
SAND
SILTY CLAY
SO LIMESTONE

UNLESS OTHERWISE SPECIFIED, THE BLOWS PER FOOT AT THE LOCATIONS INDICATED ARE BASED ON DRIVING A 2" O.D. X 1.4" I.D. SPLIT SPOON SAMPLER WITH A 140# HAMMER HAVING A FREE FALL OF 30". THE BLOW COUNT IS TAKEN IN UNDISTURBED SOIL IMMEDIATELY BELOW A CASED OR OPEN HOLE ELIMINATING SIDE FRICTION ON THE DRIVE PIPE.

SUBSURFACE EXPLORATION FOR FOUNDATION DESIGN AND BIDDERS INFORMATION

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8

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION

DRAWN BY　PLANS CK'D.

EXHIBIT 6

FILE=　SCALE =

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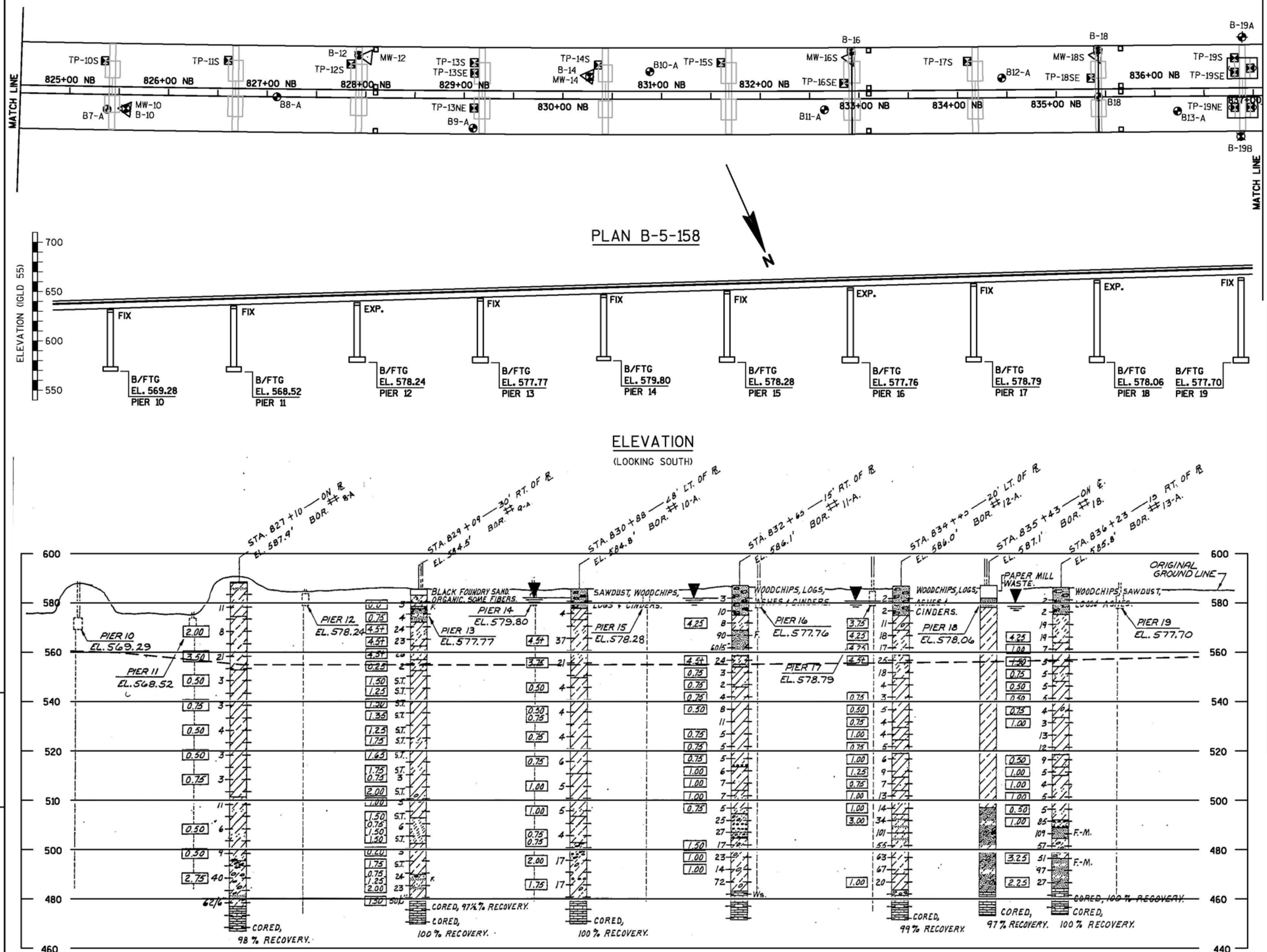
STATE PROJECT NUMBER



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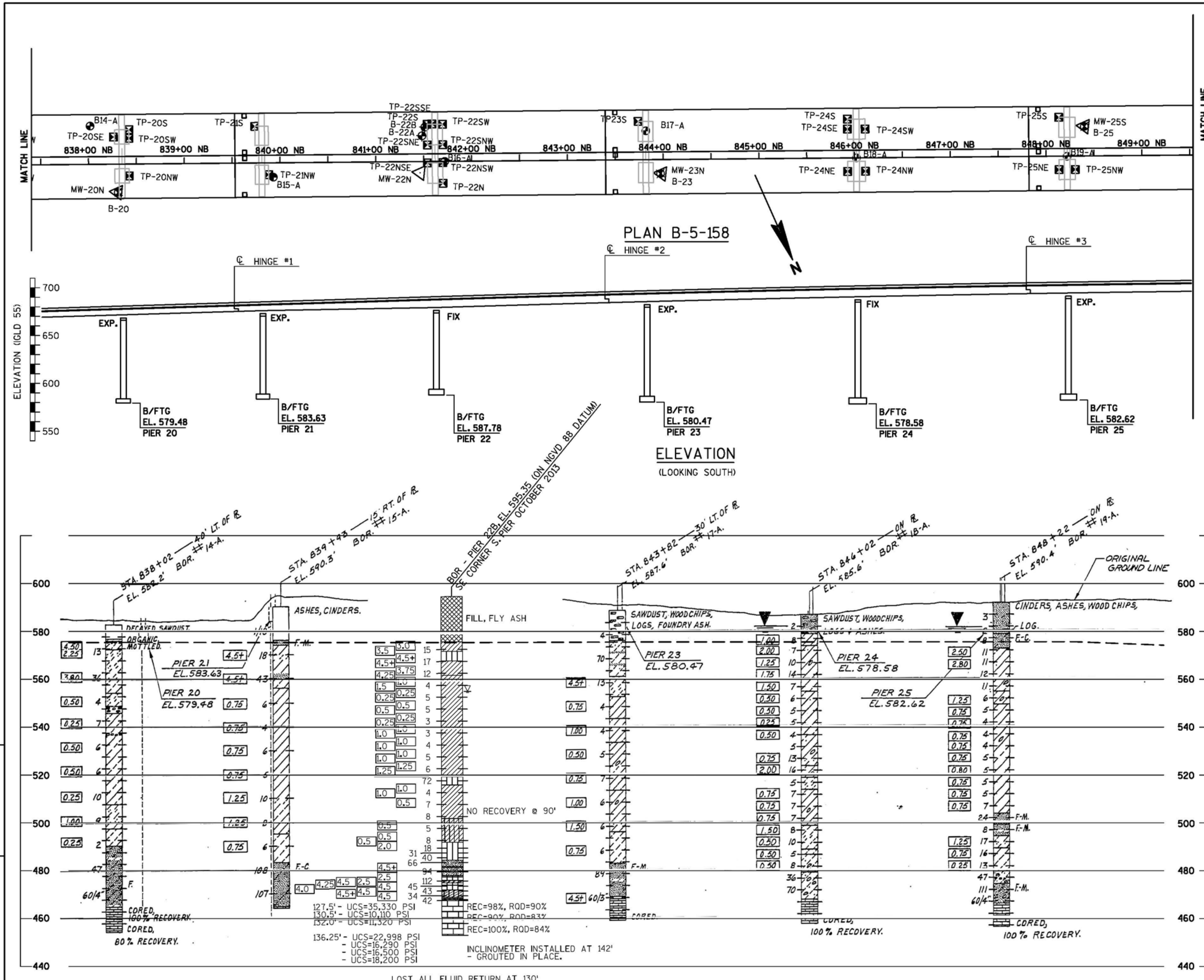

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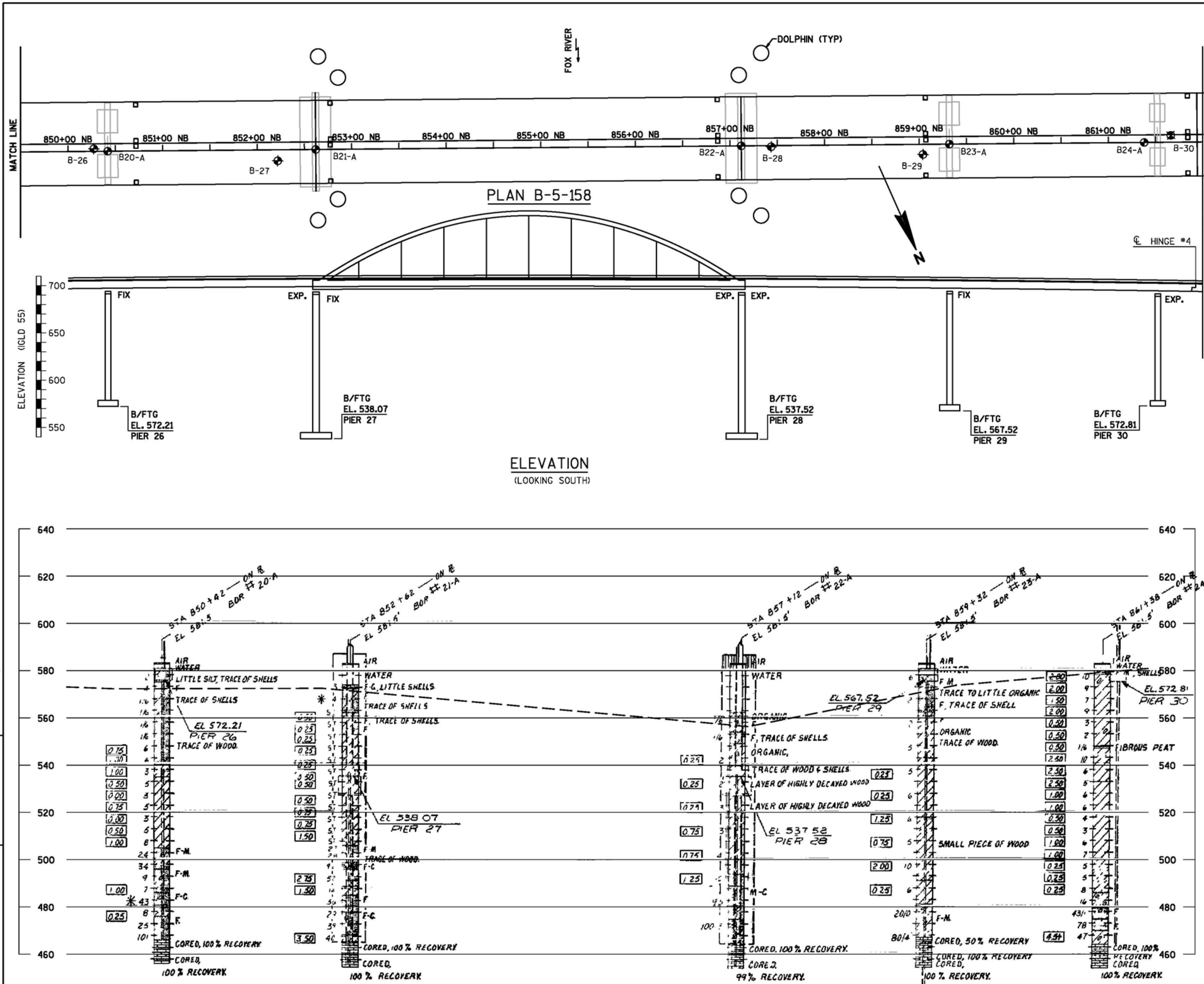

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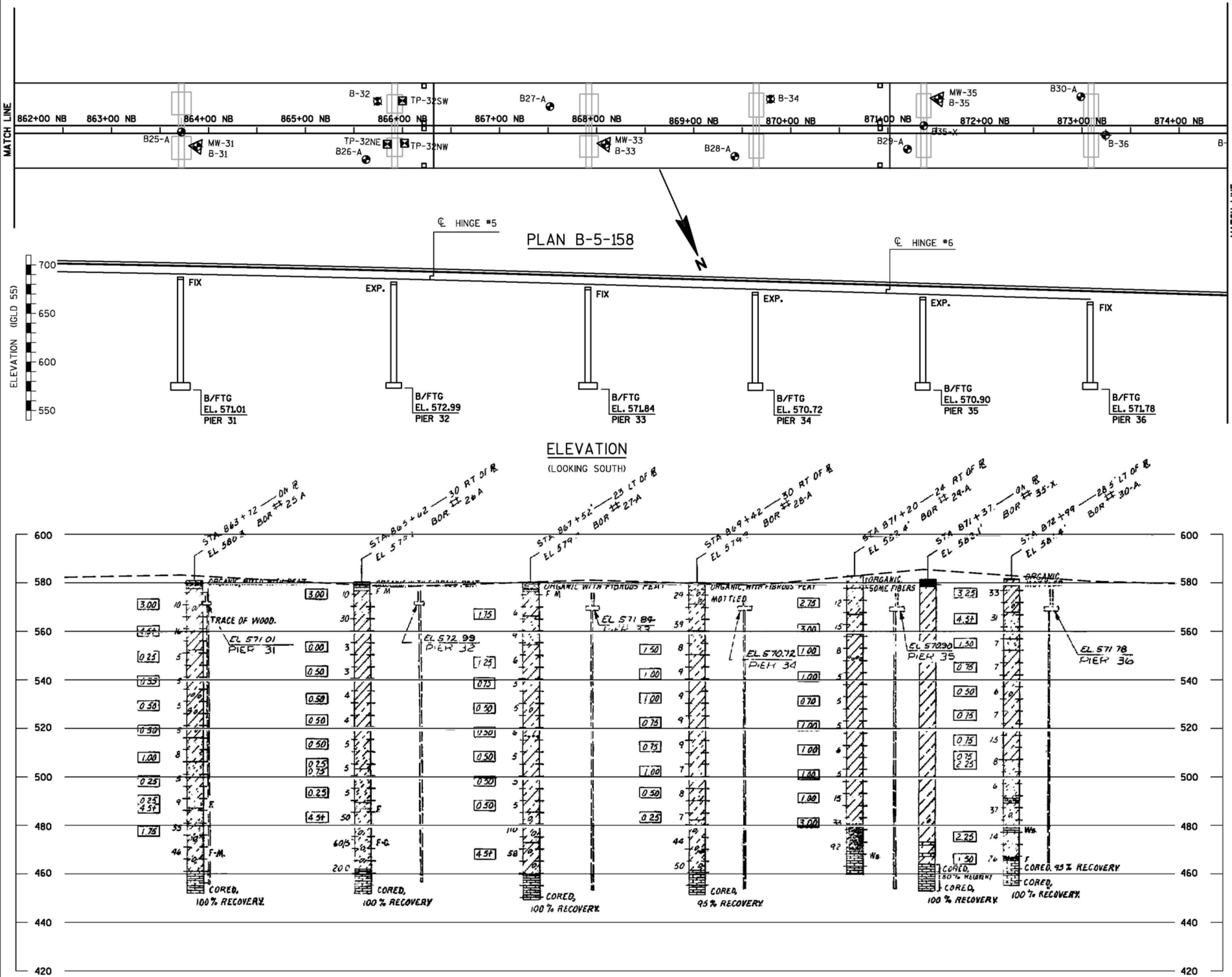

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EXHIBIT 6

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STATE PROJECT NUMBER

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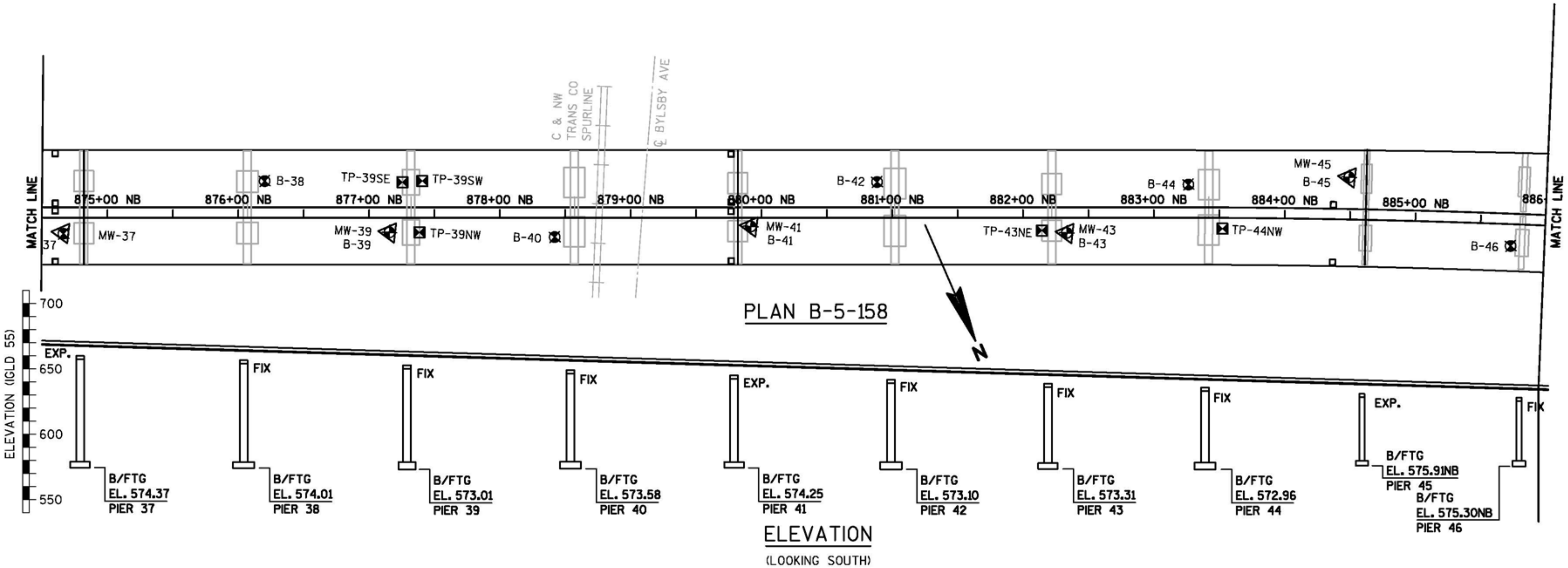

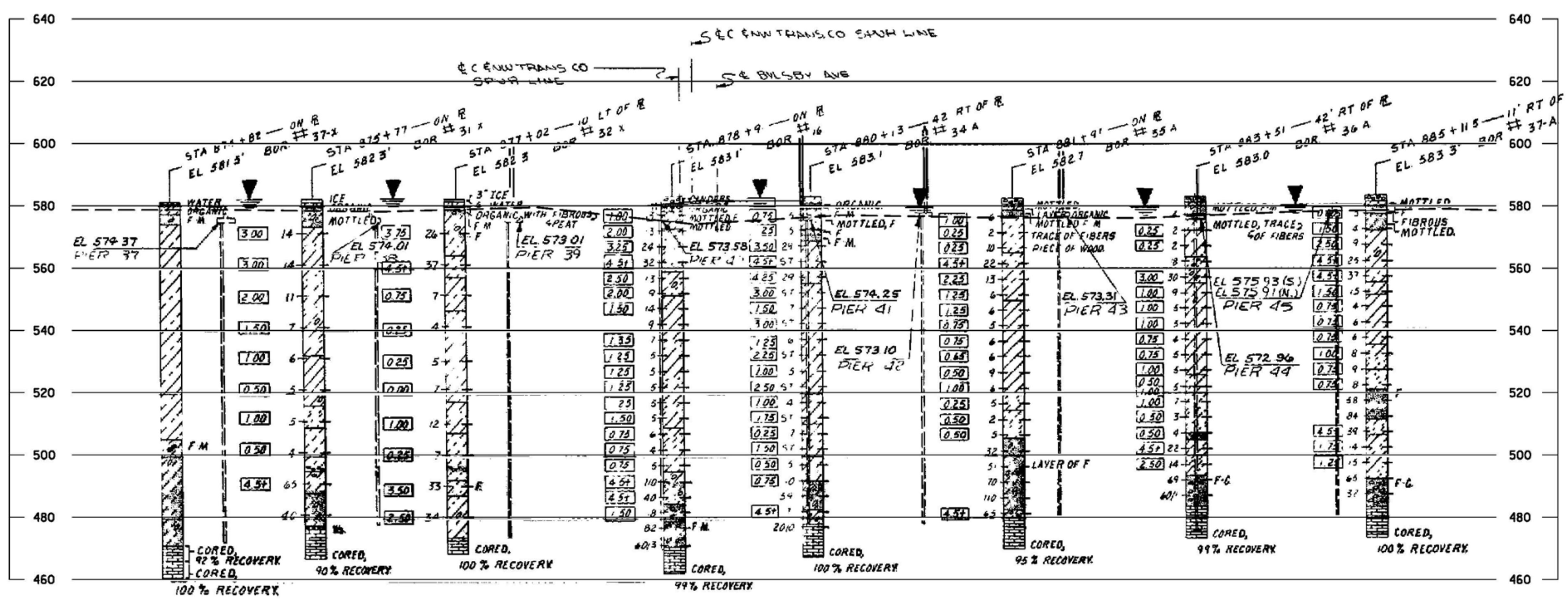

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EXHIBIT 6

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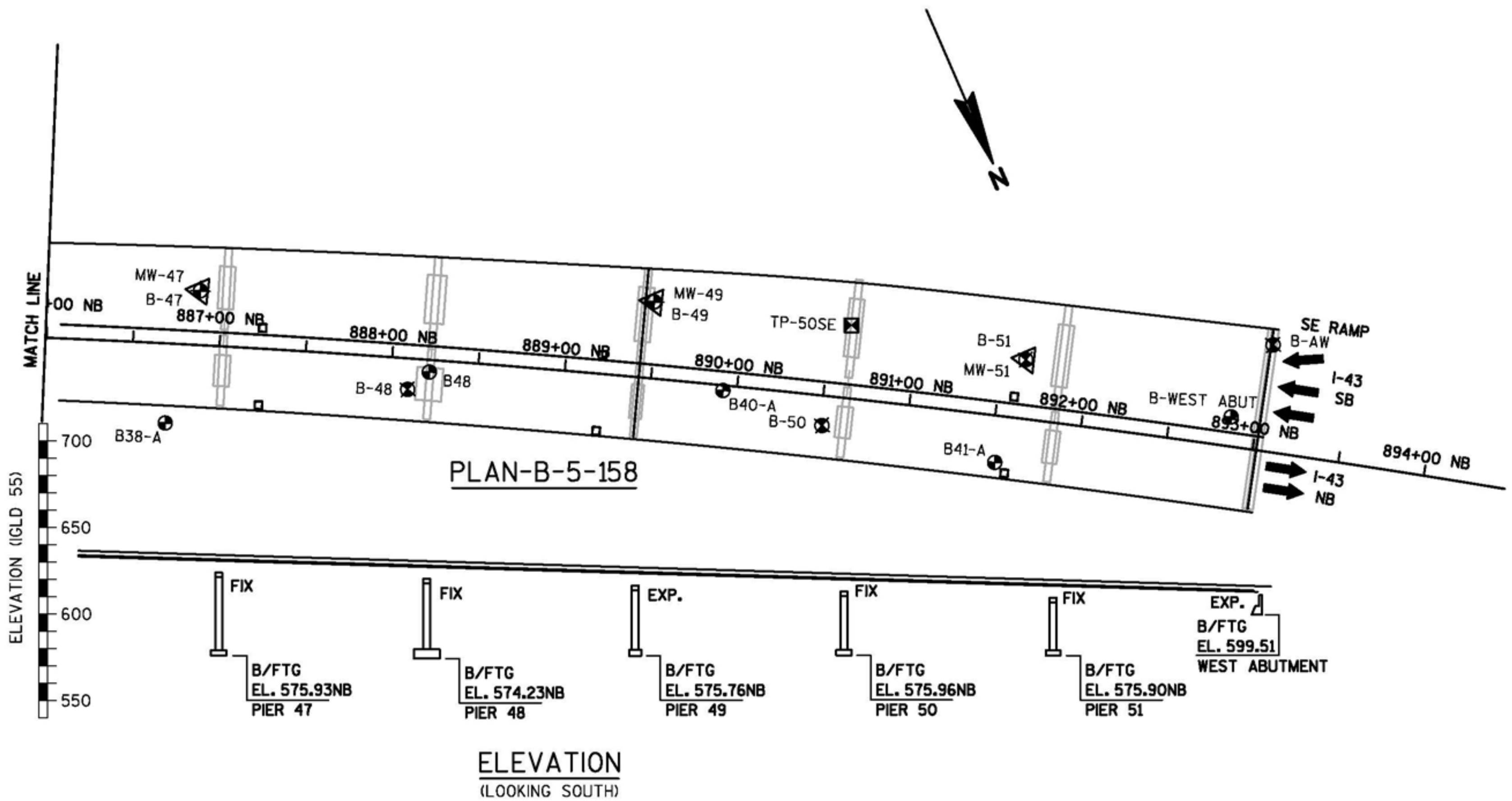
FILE= SCALE =



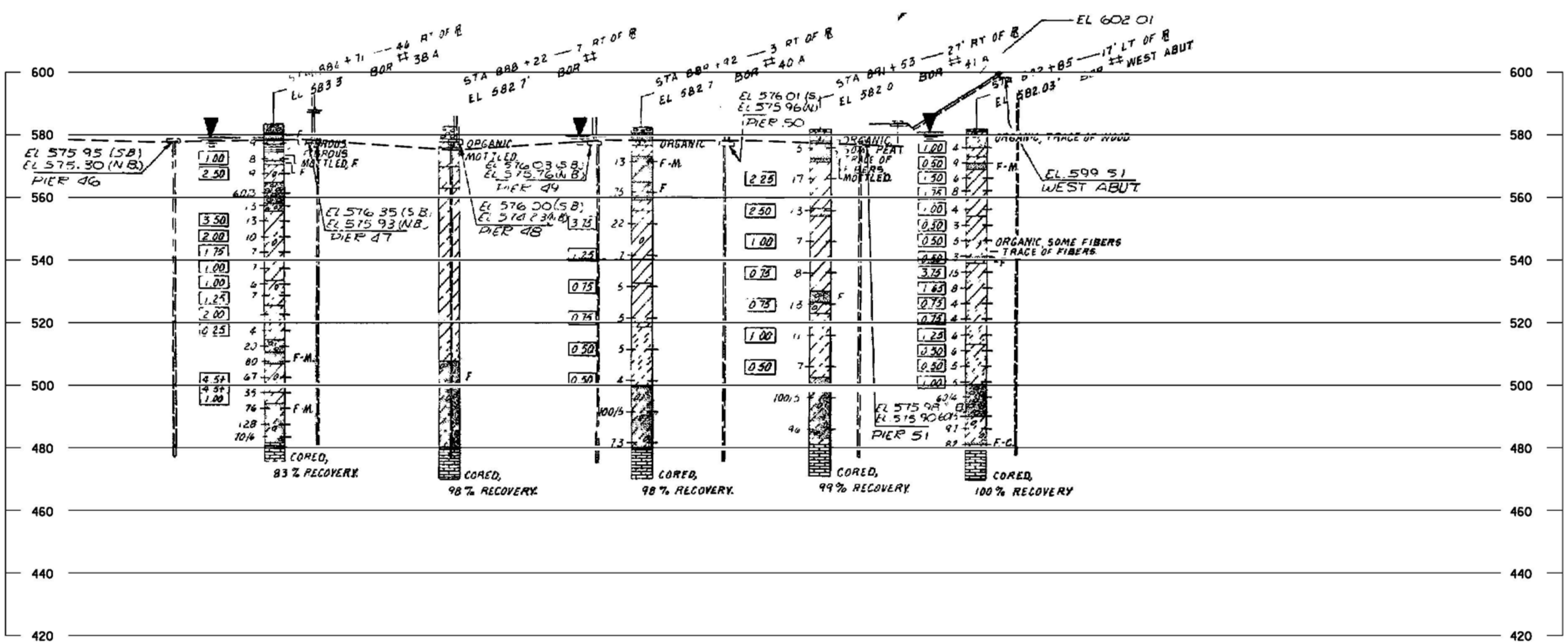


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EXHIBIT 6

7 OF 7

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FILE=
SCALE =

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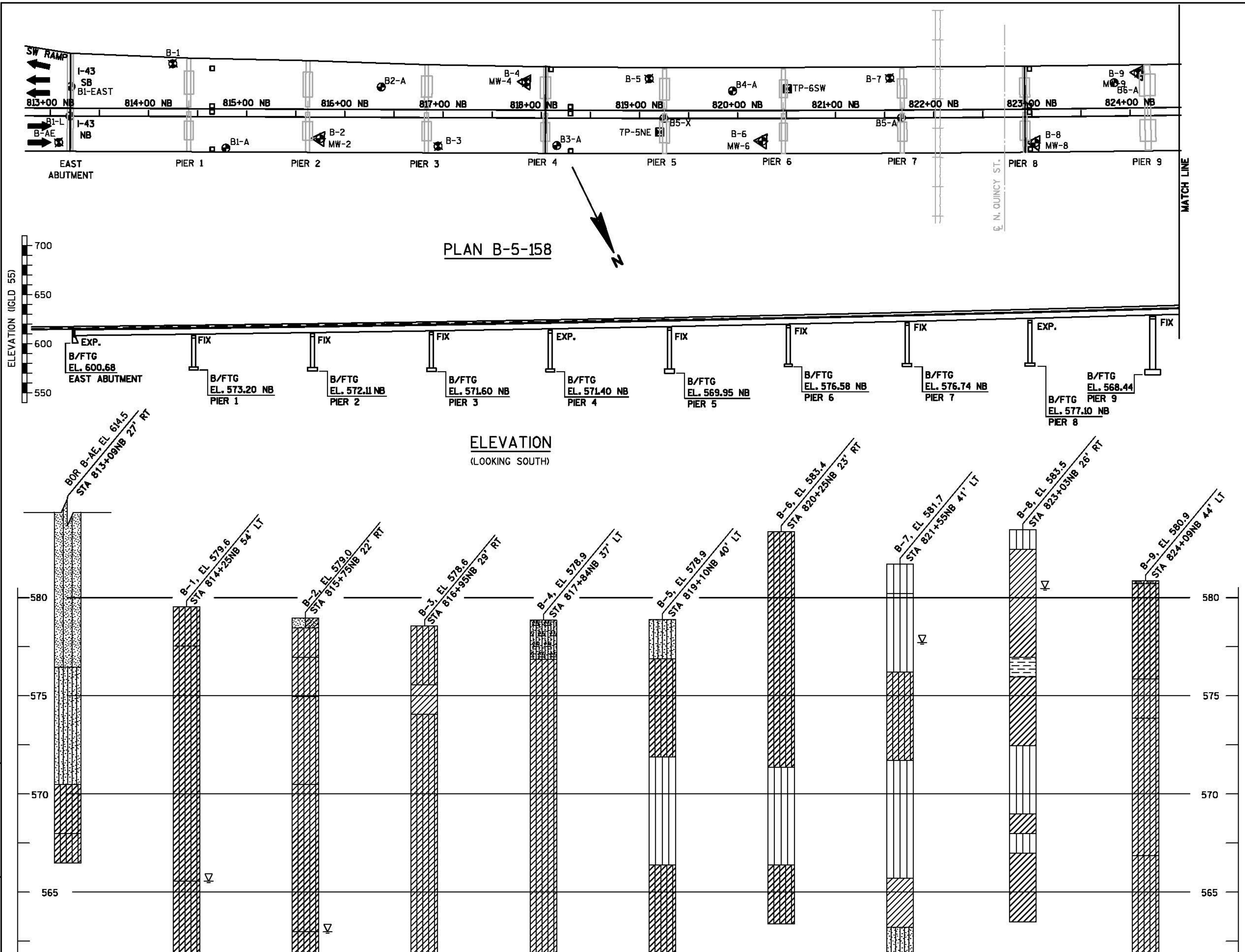

NOTES:

1. BOTTOM OF FOOTING (B/FTG) ELEVATIONS ARE BASED ON AS-BUILT PLANS (CIRCA 1970'S/1980'S), INTERNATIONAL GREAT LAKES DATUM 1955 (IGLD).
2. TEST PIT (TP-) AND BORINGS (B-) ELEVATIONS ARE BASED ON NGVD 88 DATUM (=IGLD + 1').

STATE PROJECT NUMBER

ABBREVIATIONS

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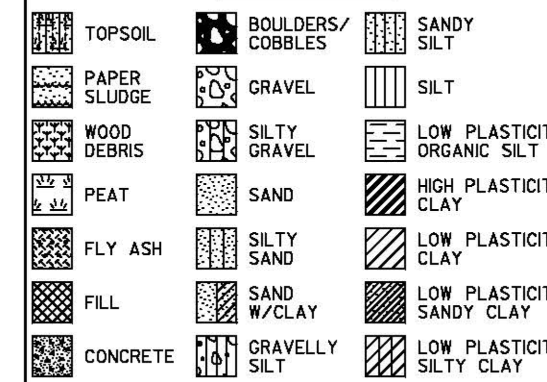
MATERIAL SYMBOLS



LEGEND OF BORING

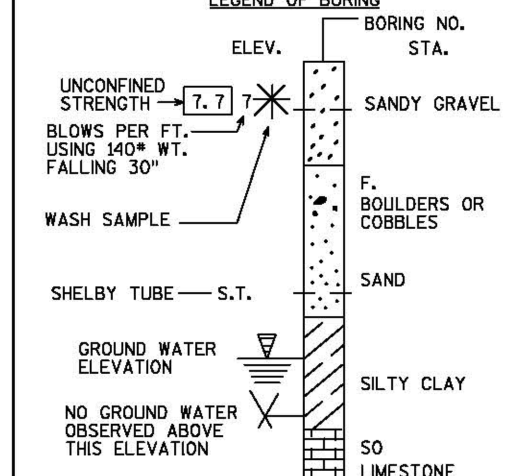


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EXHIBIT 7

1 OF 7

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FILE=
SCALE =

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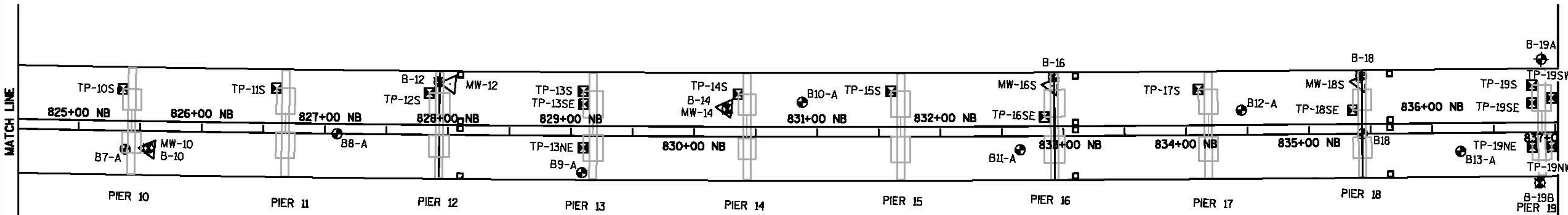
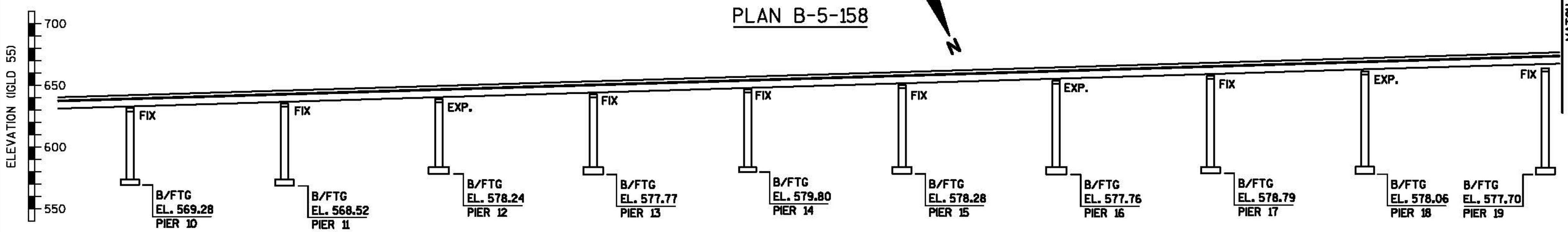
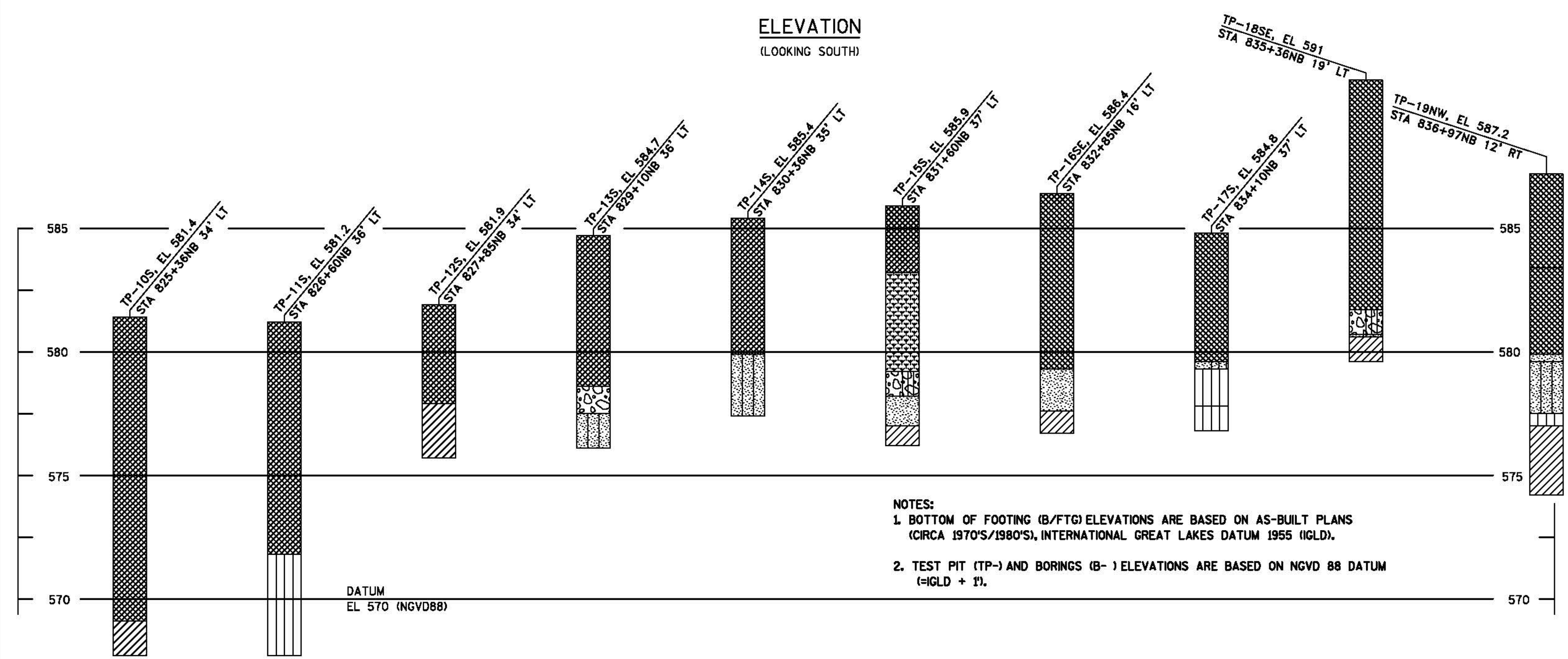


PLAN B-5-158



ELEVATION

(LOOKING SOUTH)



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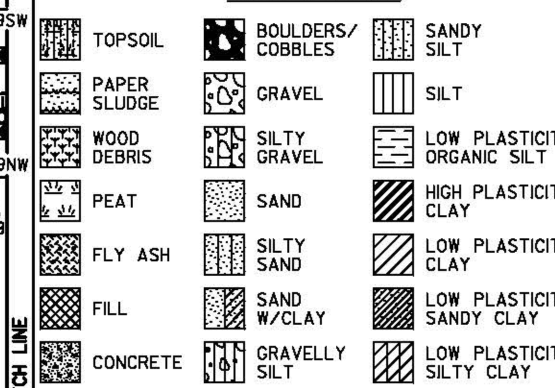
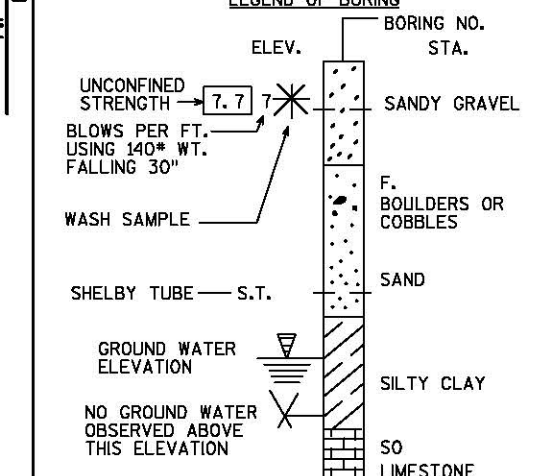
MATERIAL SYMBOLS



LEGEND OF BORING



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STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION

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EXHIBIT 7

2 OF 7

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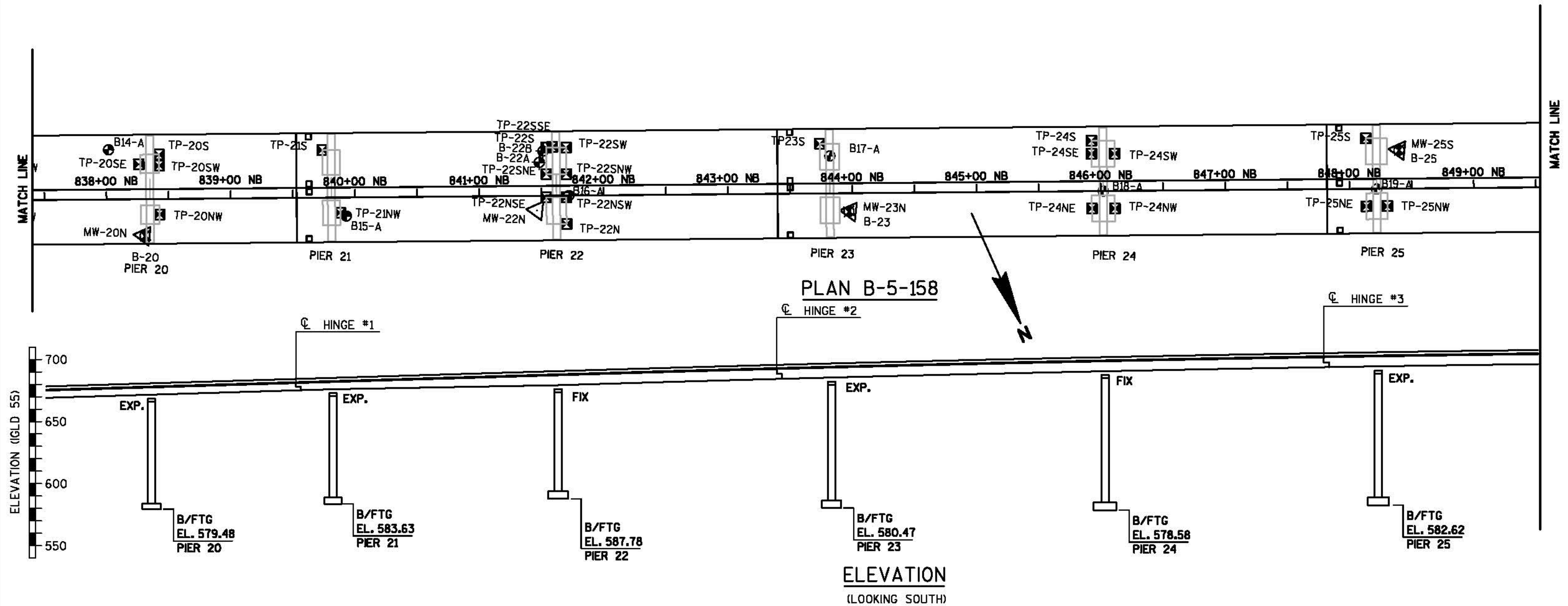

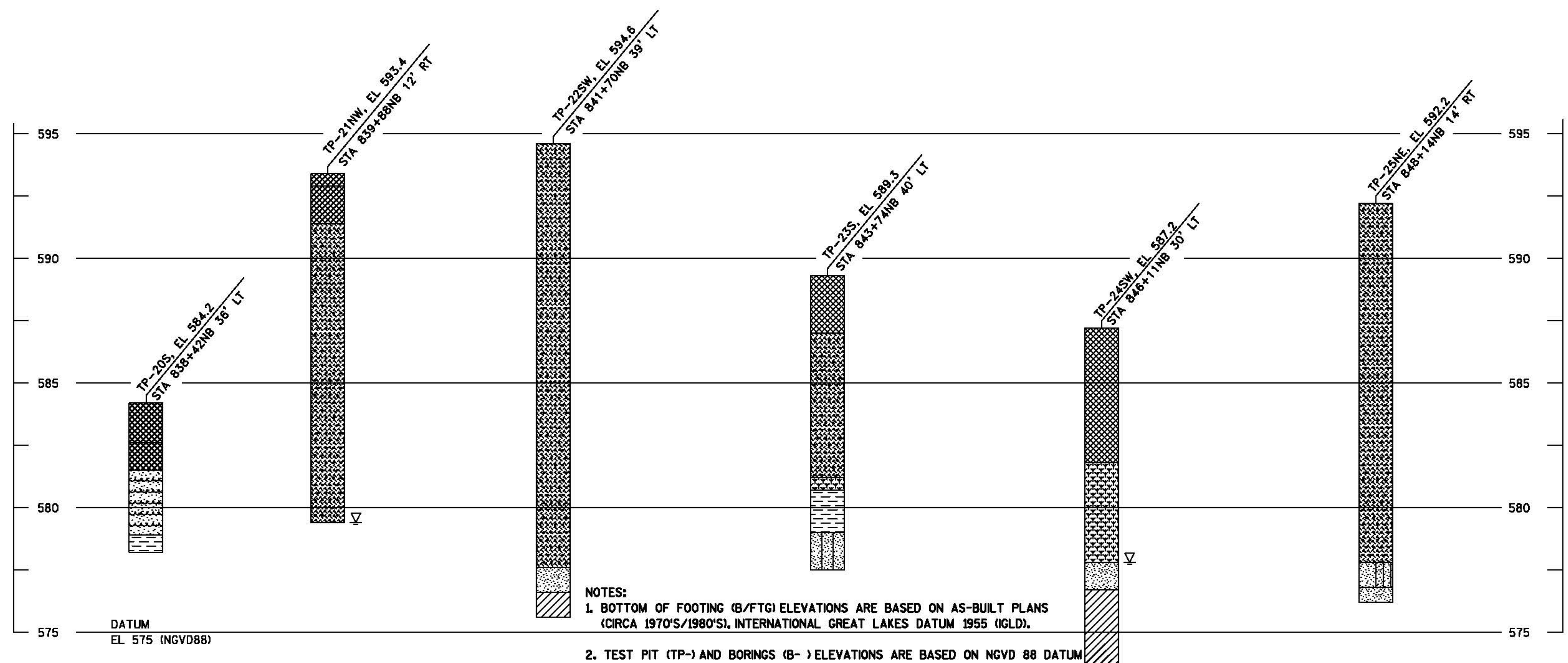

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MATERIAL SYMBOLS

TOPSOIL	BOULDERS/ COBBLES	SANDY SILT
PAPER SLUDGE	GRAVEL	SILT
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PEAT	SAND	HIGH PLASTICITY CLAY
FLY ASH	SILTY SAND	LOW PLASTICITY CLAY
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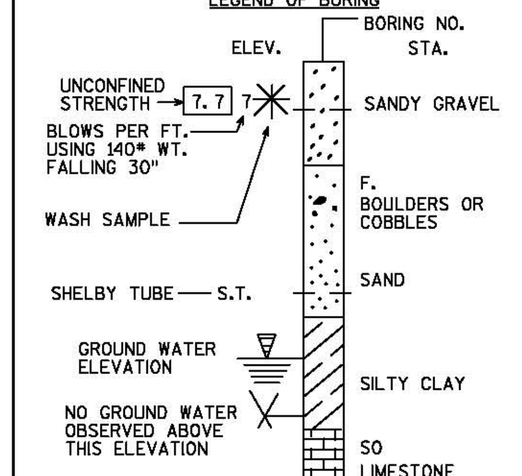
LEGEND OF BORING



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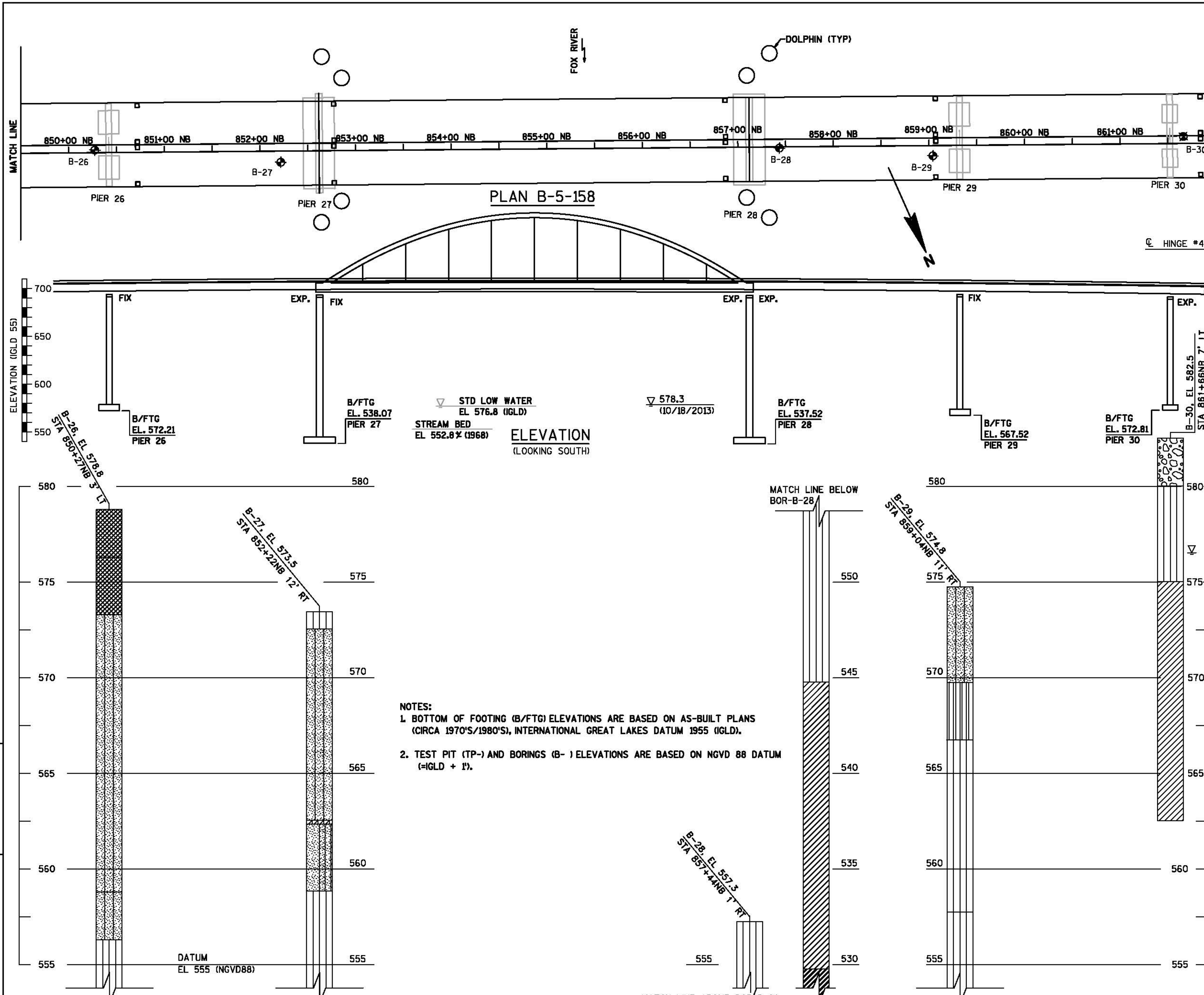

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ABBREVIATIONS

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LEGEND OF BORING

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EXHIBIT 7

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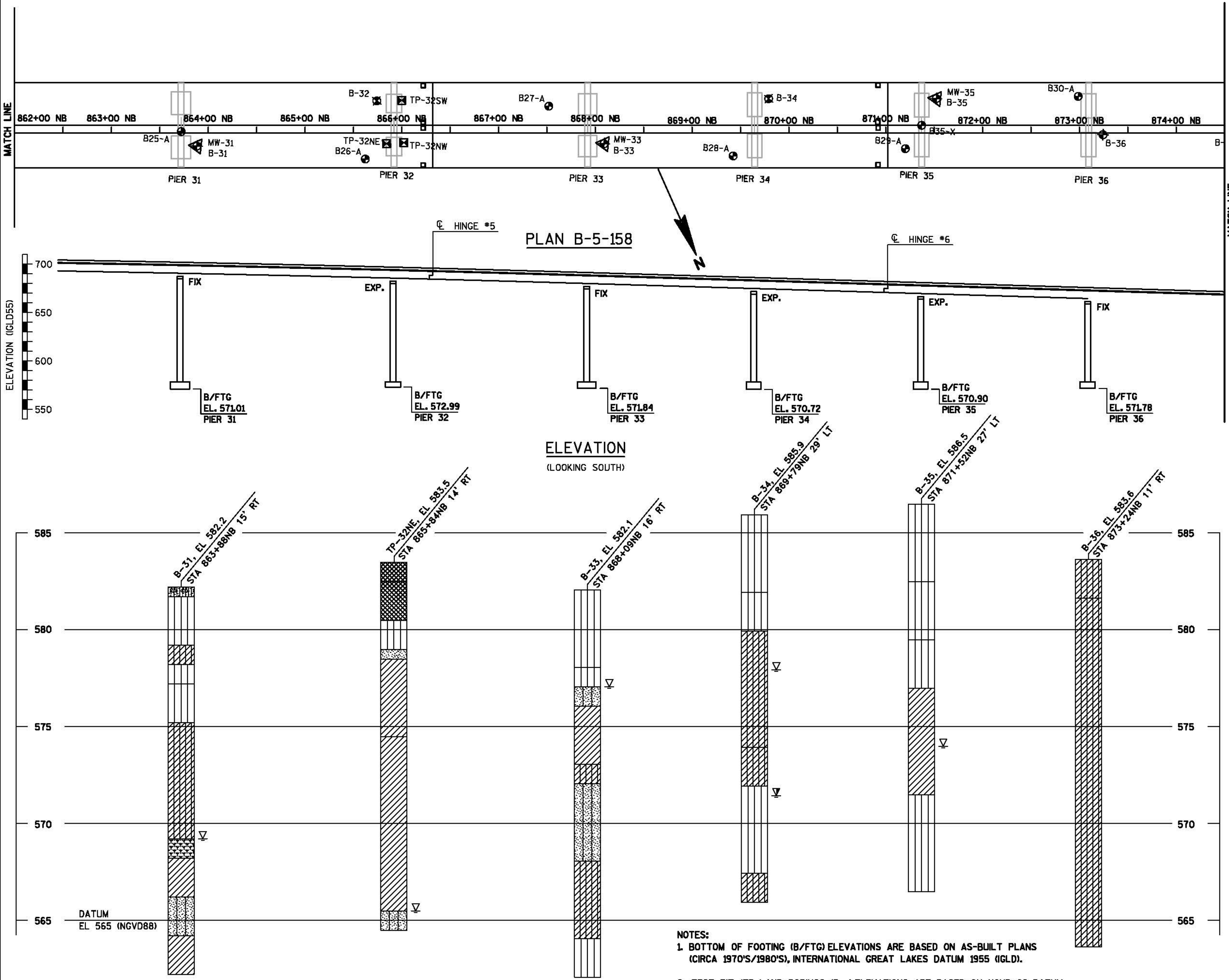

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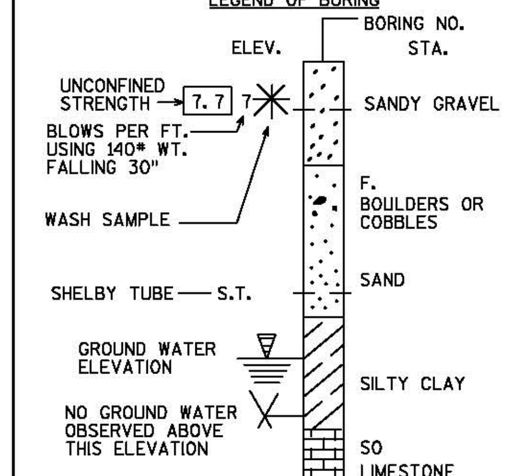

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EXHIBIT 7

5 OF 7

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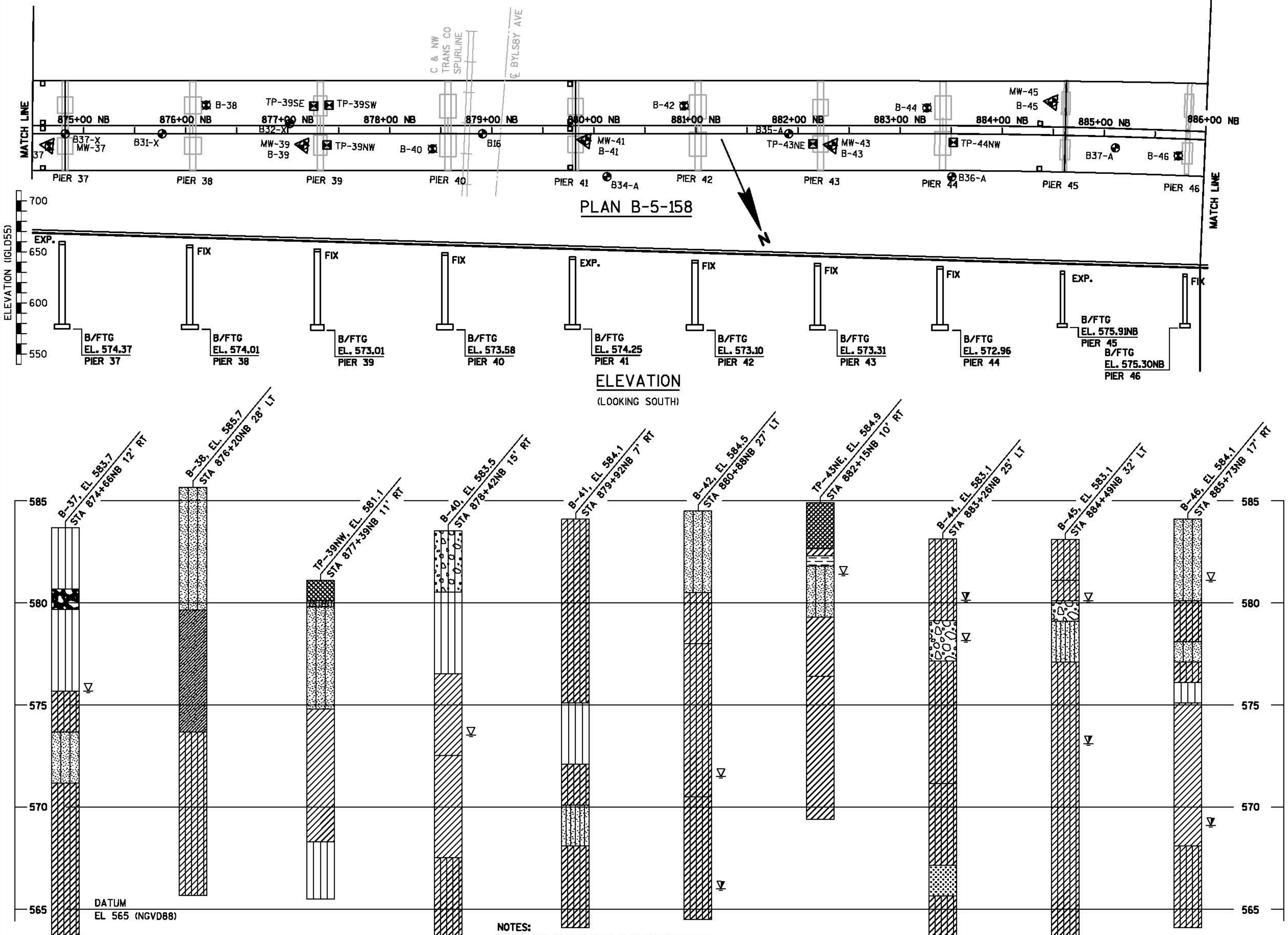

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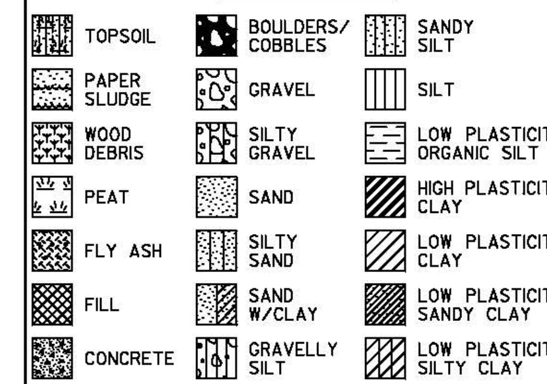
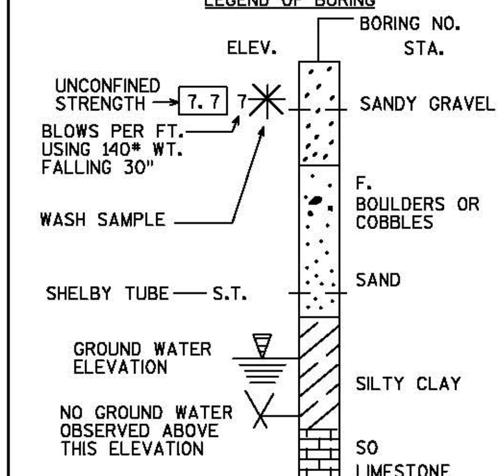


LEGEND OF BORING



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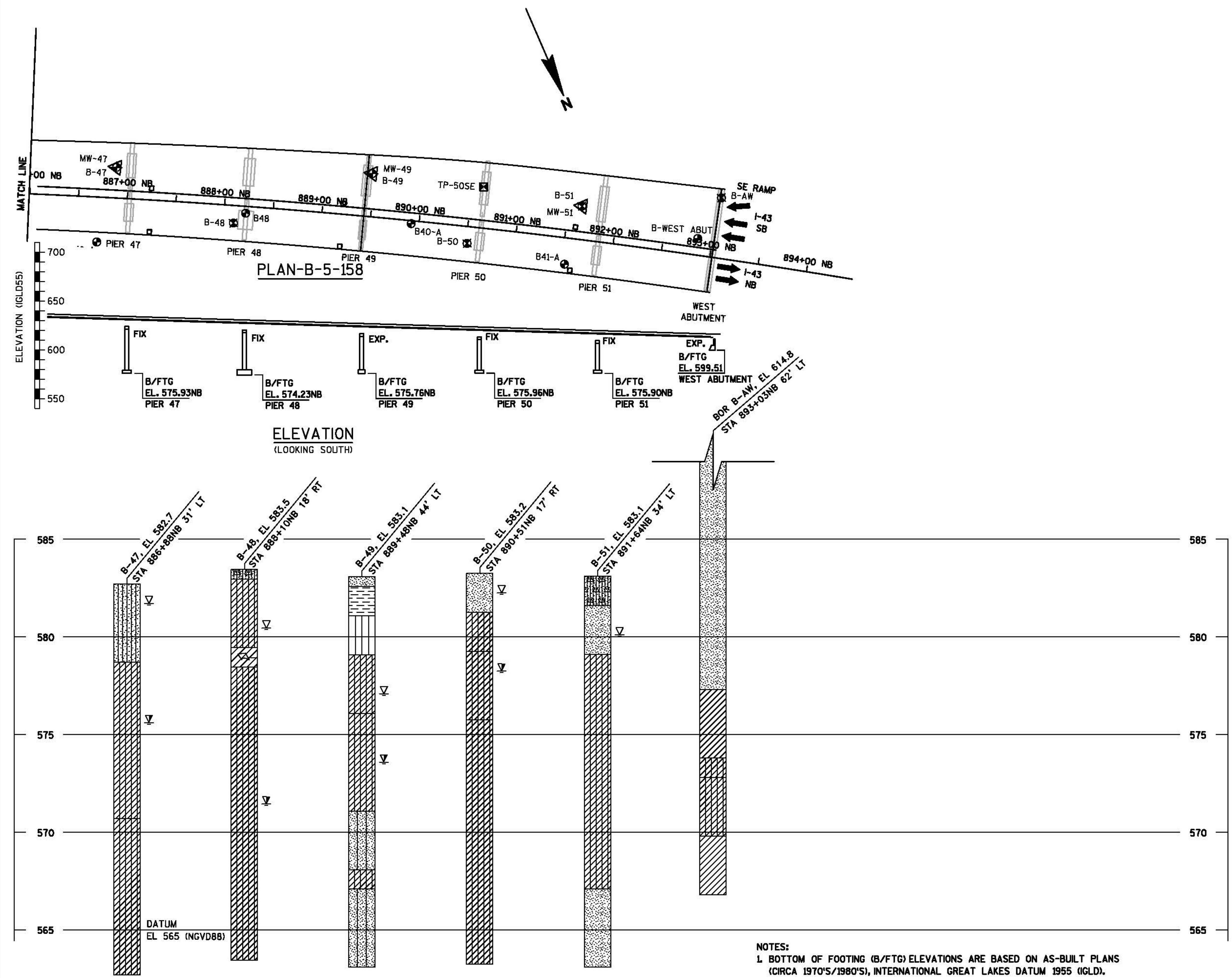

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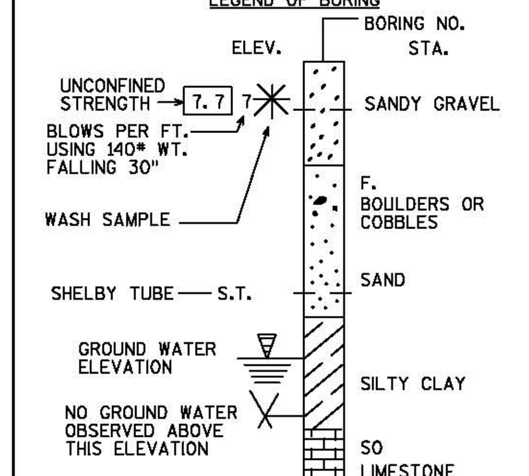

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EXHIBIT 7

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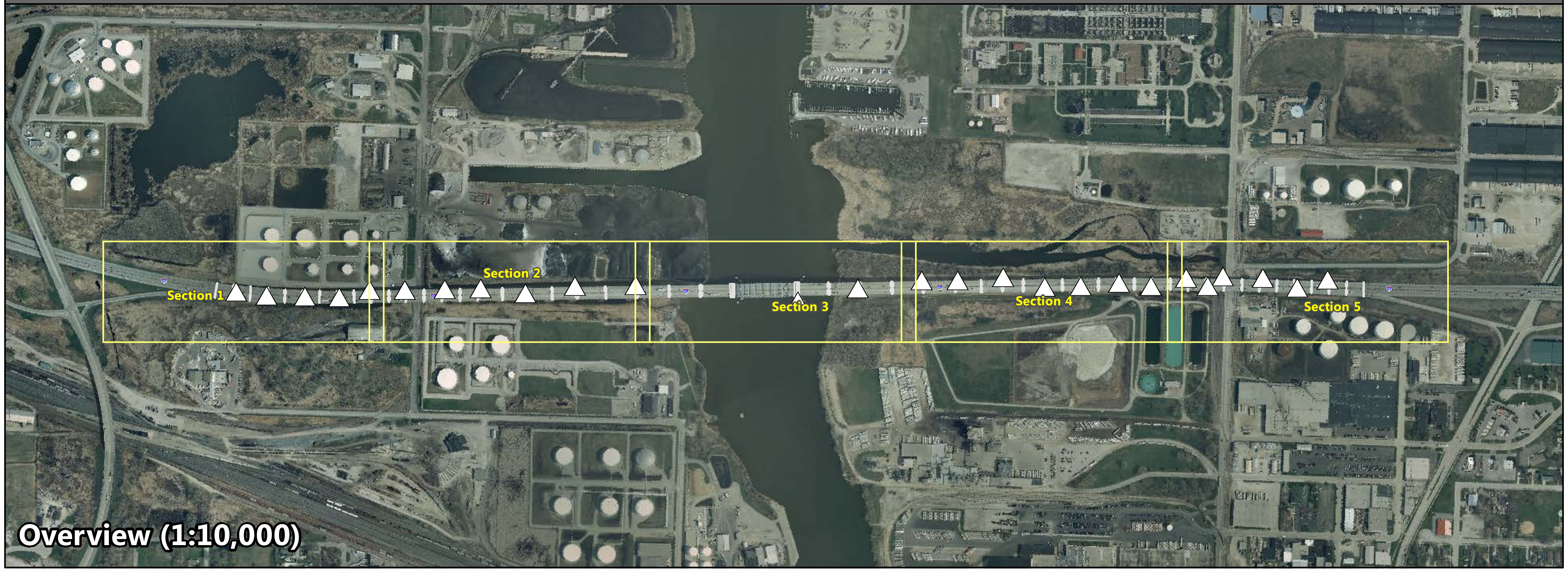
Exhibit 8 – Plot of Ground Water Elevations at Sampling Points



LEGEND

Ground Water Elevation

Ground Water Elevation*

Bridge Alignment

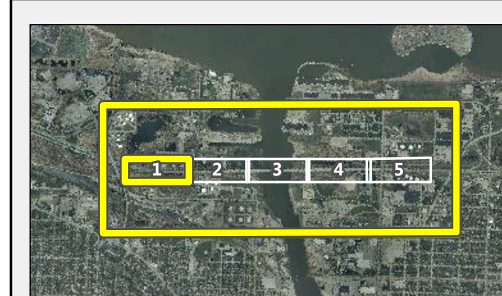
Substructure

Scupper

*The river pool elevation was measured on 10/18/2013. Ground water elevations were measured on 10/22/2013.

0 50 100 150 feet

1 in = 150 feet 1 : 1,800



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LEGEND

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△ Ground Water Elevation*

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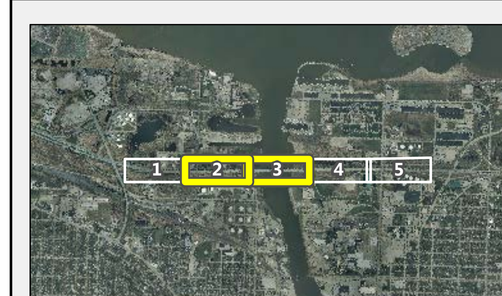

Baker

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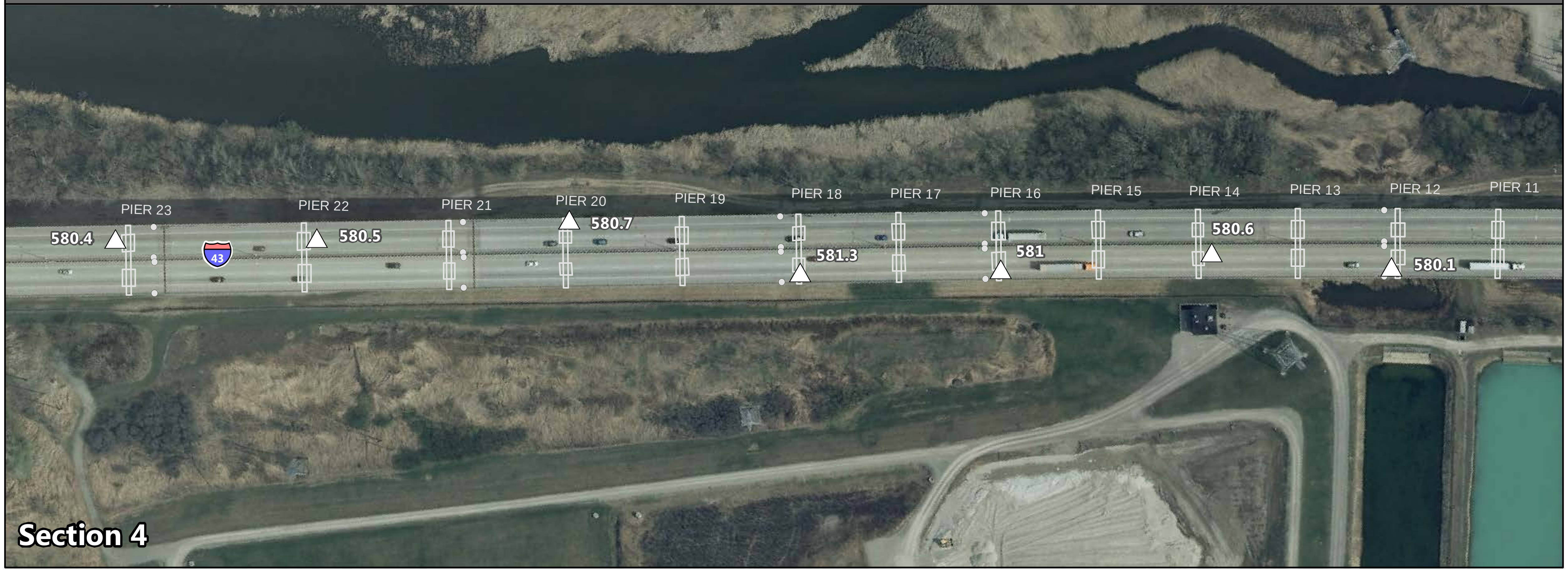

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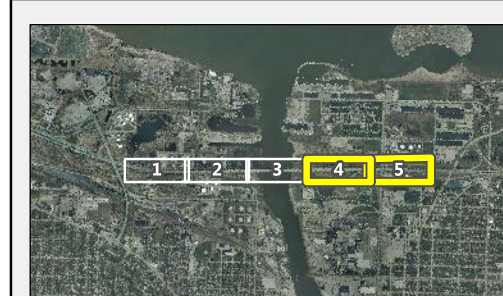
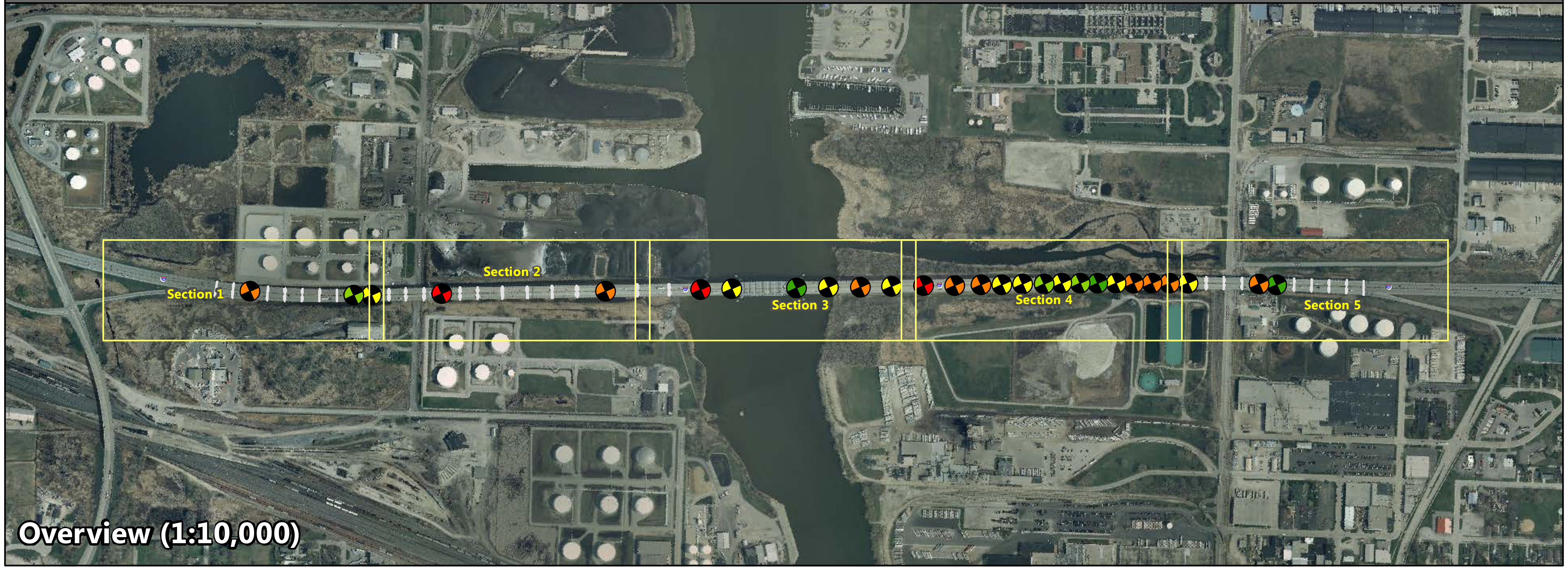
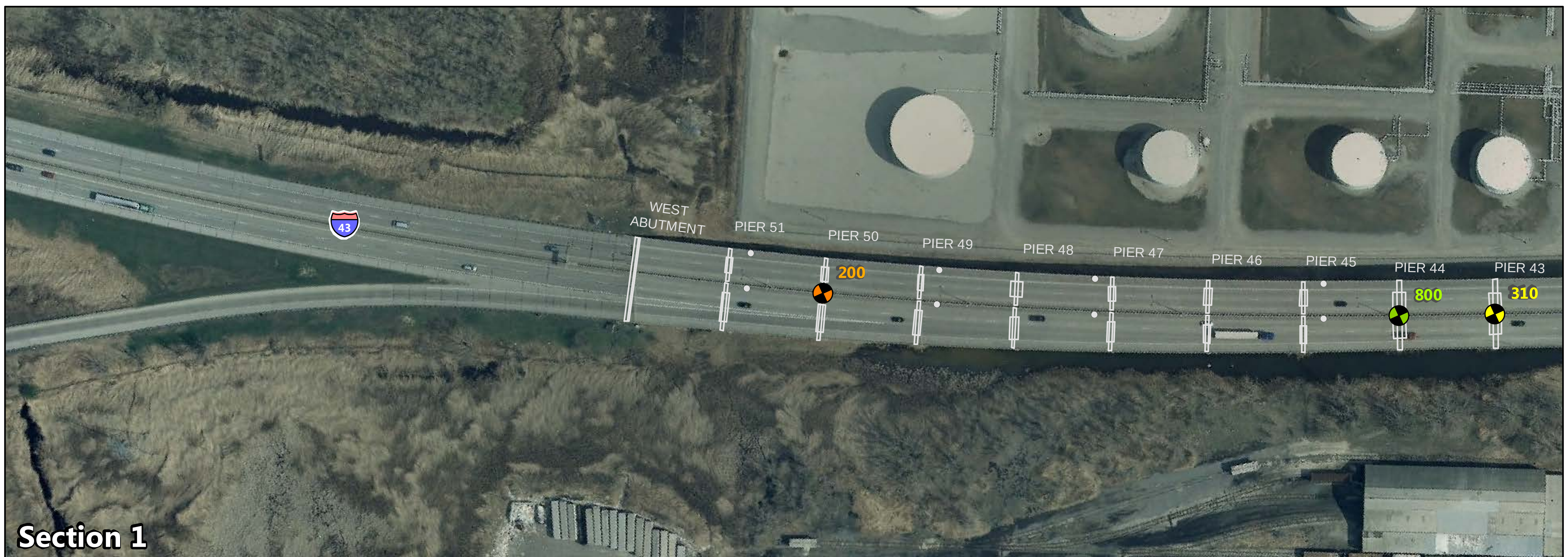


Baker

Exhibit 9 – Plot of Minimum Laboratory Soil Resistivity Measurements





LEGEND

Minimum Soil Resistivity (ohm-cm)

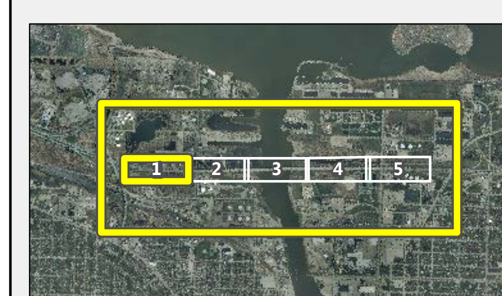
- <= 100
- 101 - 300
- 301 - 700
- 701 - 1000
- > 1000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800



Michael Baker INTERNATIONAL

Exhibit 9 – Plot of Minimum Laboratory Soil Resistivity Measurements

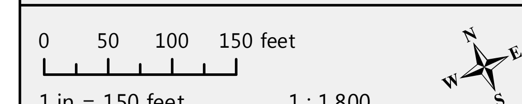
LEGEND

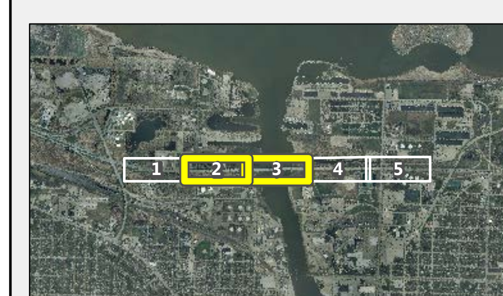
Minimum Soil Resistivity (ohm-cm)

- <= 100
- 101 - 300
- 301 - 700
- 701 - 1000
- > 1000

Bridge Alignment

- Substructure
- Scupper





Michael Baker INTERNATIONAL

Exhibit 9 – Plot of Minimum Laboratory Soil Resistivity Measurements

LEGEND

Minimum Soil Resistivity (ohm-cm)

- <= 100
- 101 - 300
- 301 - 700
- 701 - 1000
- > 1000

Bridge Alignment

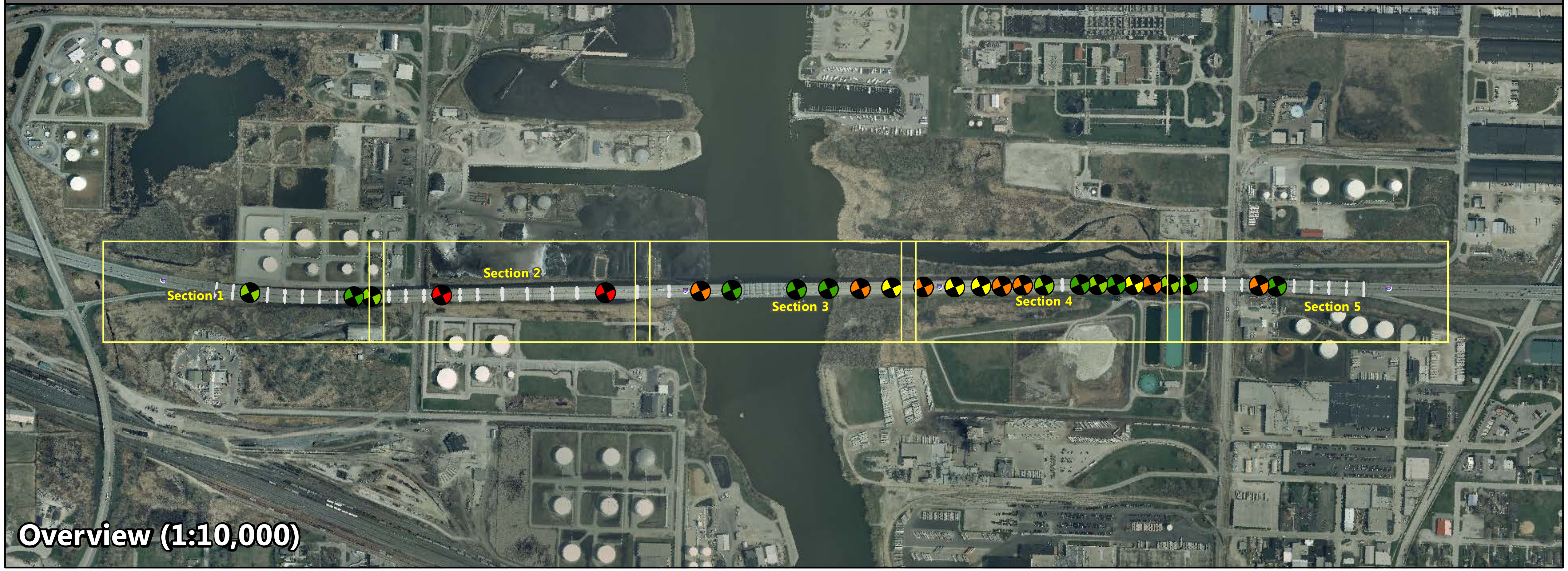
- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

Michael Baker INTERNATIONAL

Exhibit 10 – Plot of Maximum Soil Chloride Concentrations



LEGEND

Maximum Soil Chloride (mg/kg)

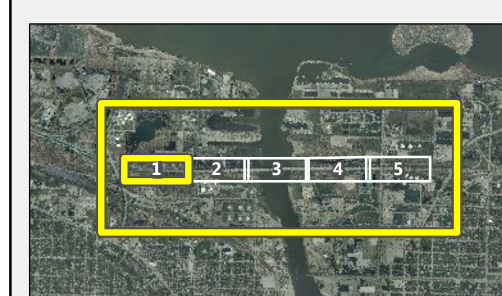
- <= 700
- 701 - 2000
- 2001 - 4000
- 4001 - 8000
- > 8000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800



Michael Baker INTERNATIONAL

Exhibit 10 – Plot of Maximum Soil Chloride Concentrations

LEGEND

Maximum Soil Chloride (mg/kg)

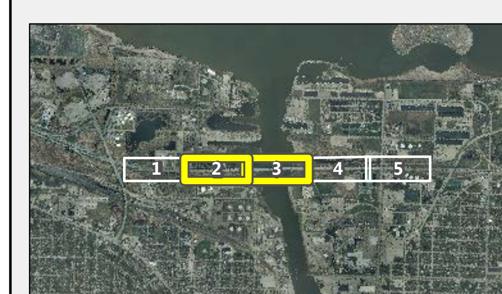
- <= 700
- 701 - 2000
- 2001 - 4000
- 4001 - 8000
- > 8000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800



Michael Baker INTERNATIONAL

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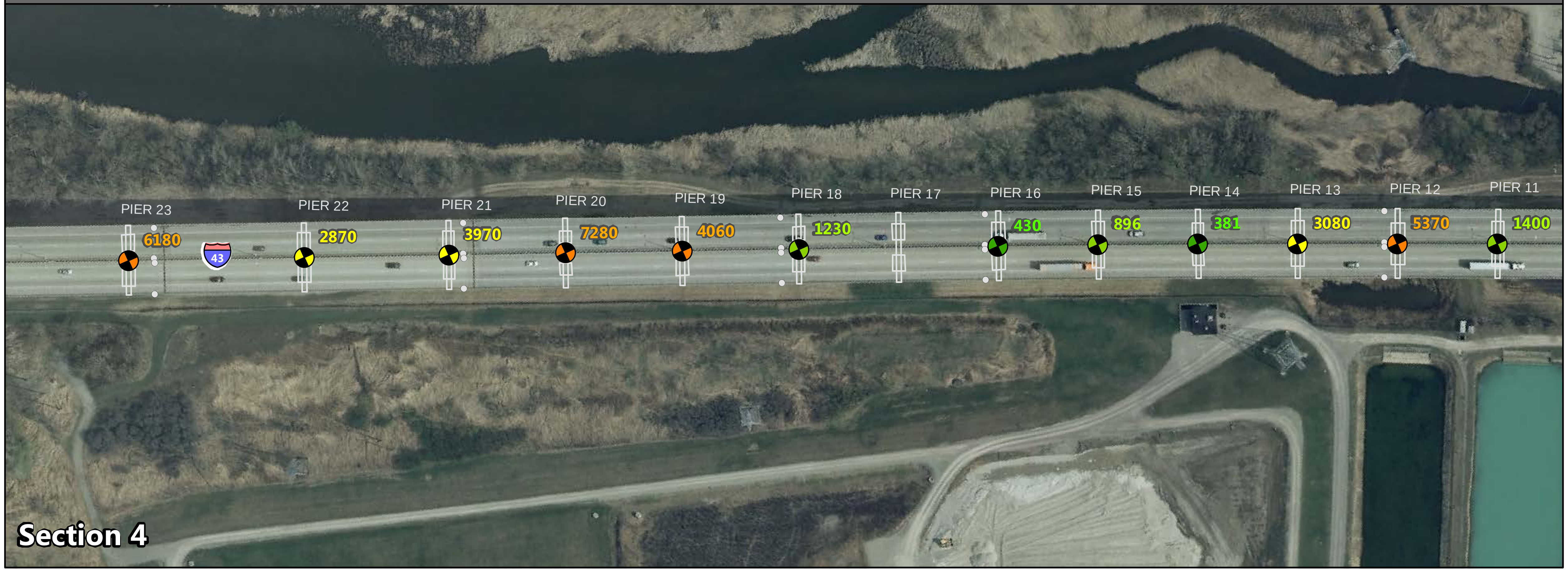


LEGEND

Maximum Soil Chloride (mg/kg)

- <= 700
- 701 - 2000
- 2001 - 4000
- 4001 - 8000
- > 8000

Bridge Alignment

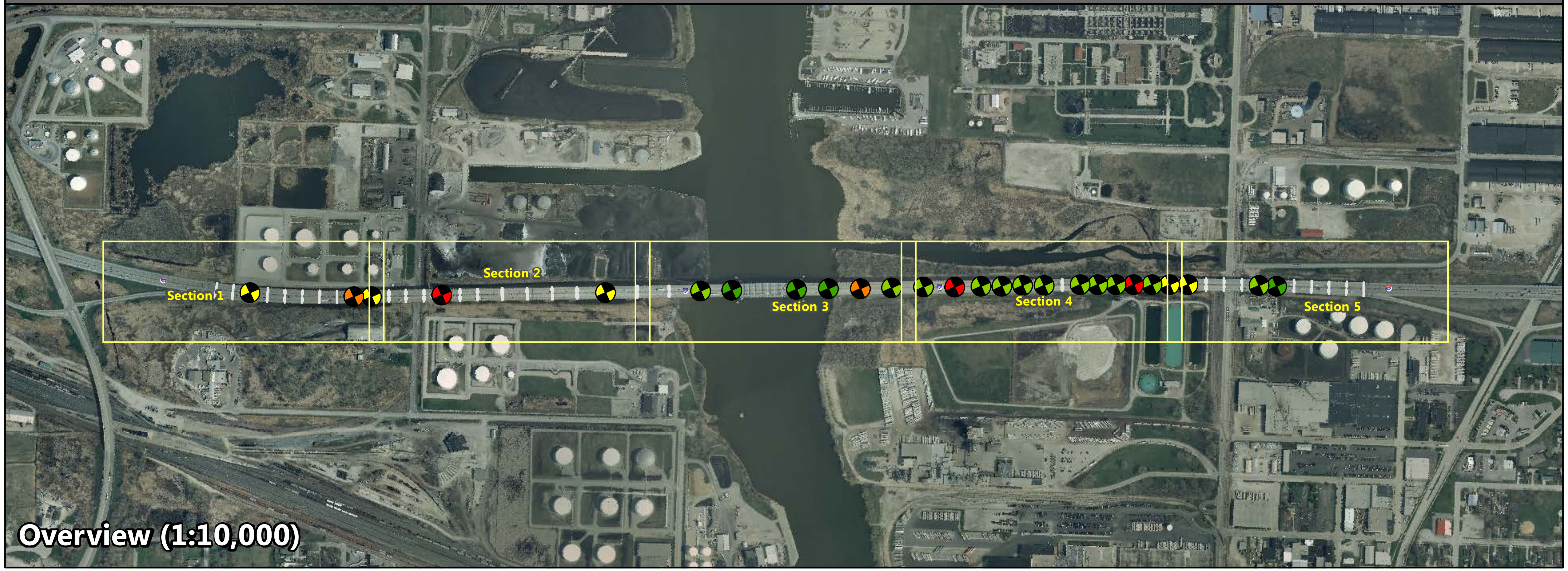
- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

Michael Baker INTERNATIONAL

Exhibit 11 – Plot of Maximum Soil Sulfate Concentrations



LEGEND

Maximum Soil Sulfate (mg/kg)

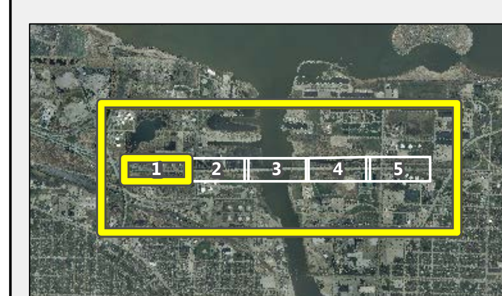
- <= 100
- 101 - 500
- 501 - 1000
- 1001 - 2000
- > 2000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800



Michael Baker INTERNATIONAL

Exhibit 11 – Plot of Maximum Soil Sulfate Concentrations

LEGEND

Maximum Soil Sulfate (mg/kg)

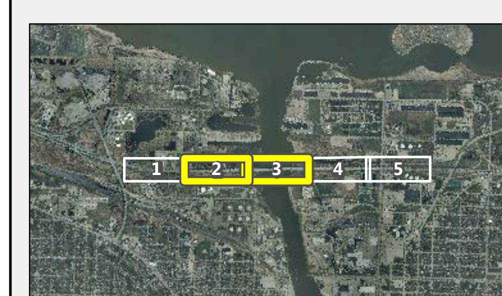
- <= 100
- 101 - 500
- 501 - 1000
- 1001 - 2000
- > 2000

Bridge Alignment

- Substructure
- Scupper

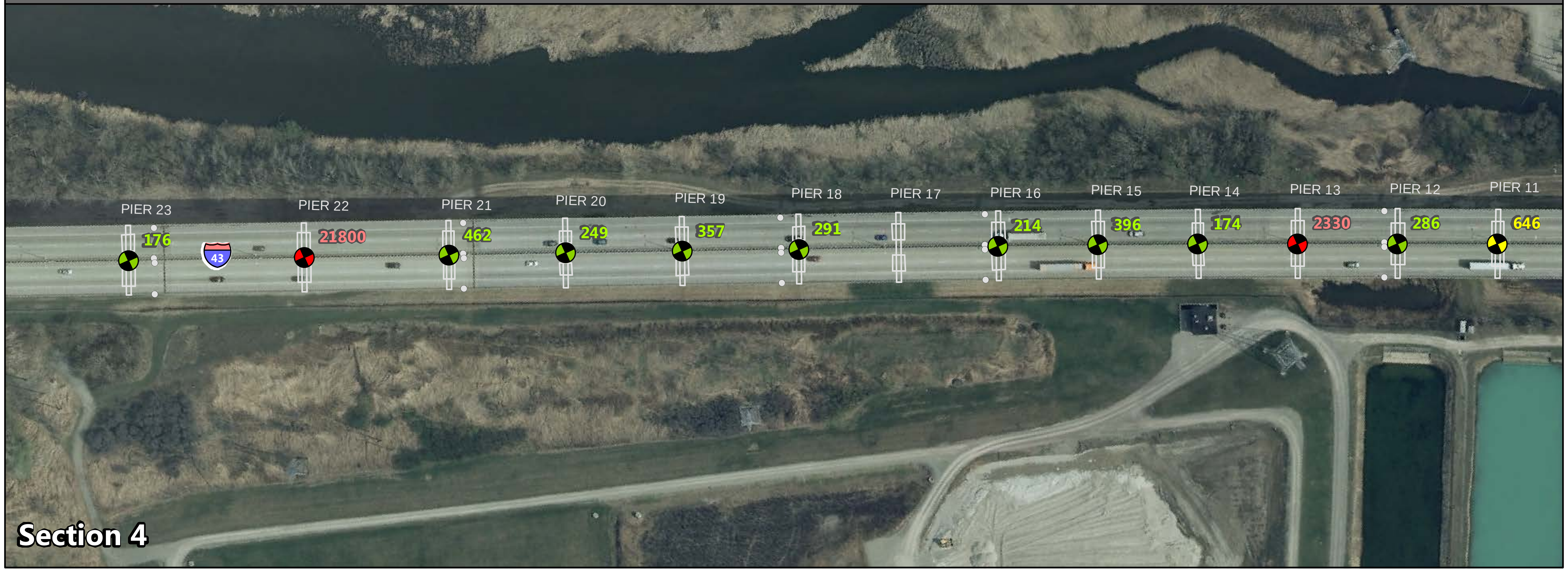
0 50 100 150 feet

1 in = 150 feet 1 : 1,800



Michael Baker INTERNATIONAL

Exhibit 11 – Plot of Maximum Soil Sulfate Concentrations



LEGEND

Maximum Soil Sulfate (mg/kg)

- <= 100
- 101 - 500
- 501 - 1000
- 1001 - 2000
- > 2000

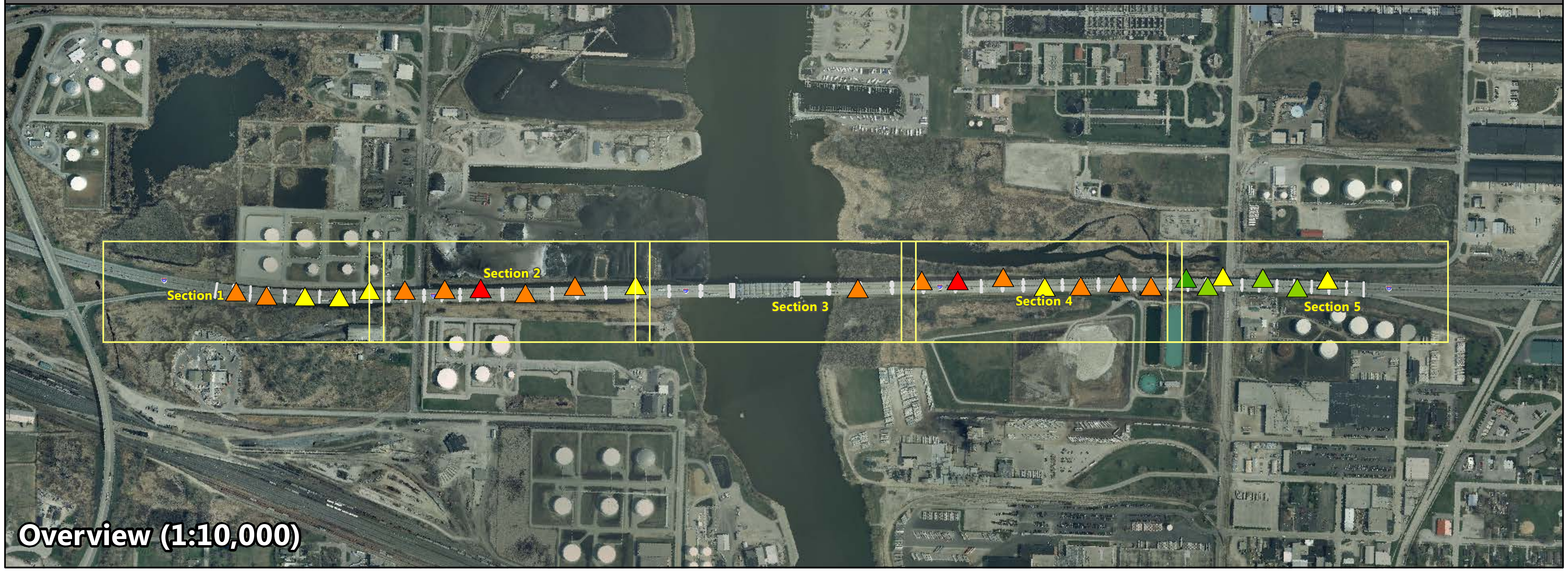
Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

Exhibit 12 – Plot of Ground Water pH at Sampling Points



LEGEND

Ground Water pH

- 6.0 - 6.5
- 6.6 - 7.0
- 7.1 - 7.5
- 7.6 - 8.0
- 8.0 - 8.5

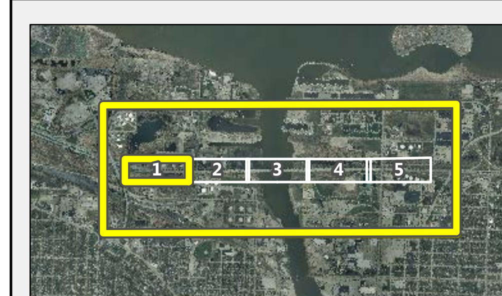
Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

N E S W



Baker

Exhibit 12 – Plot of Ground Water pH at Sampling Points

LEGEND

Ground Water pH

- 6.0 - 6.5
- 6.6 - 7.0
- 7.1 - 7.5
- 7.6 - 8.0
- 8.0 - 8.5

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

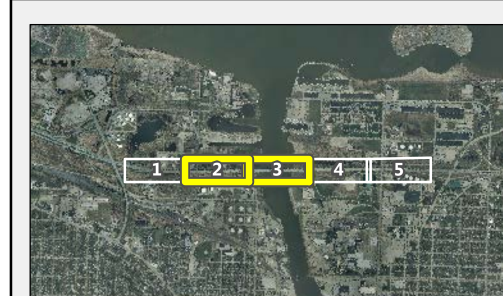
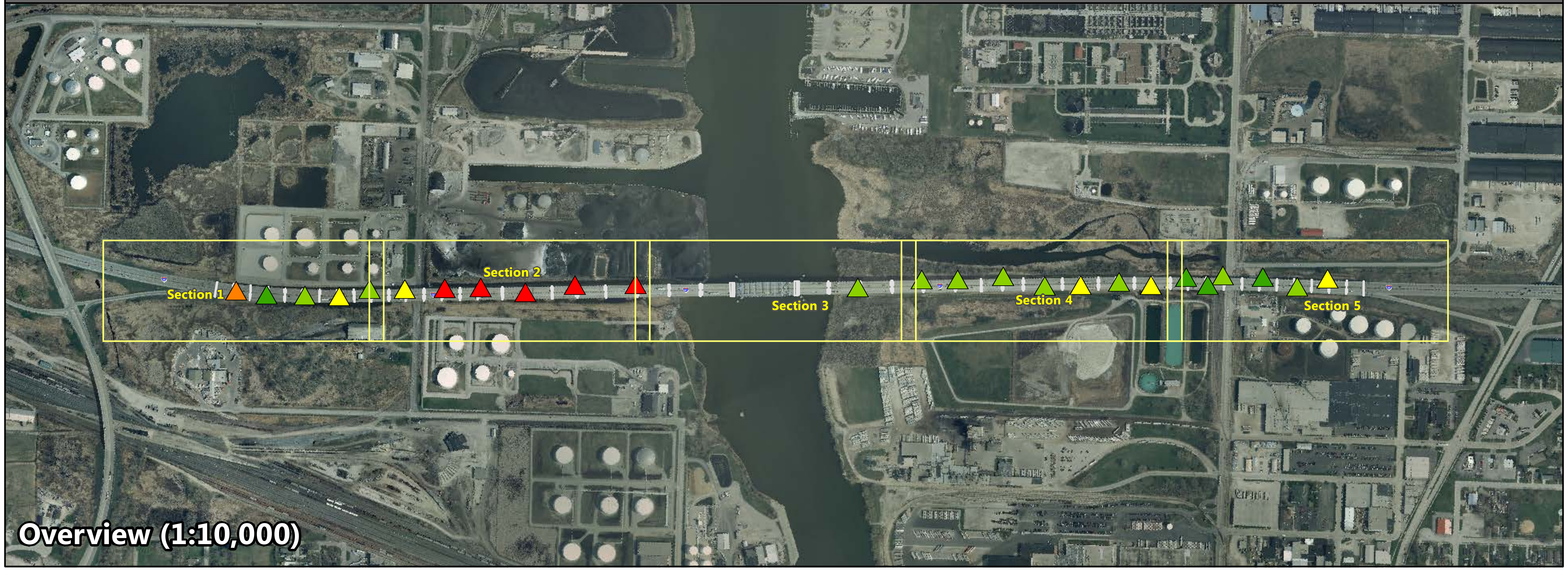


Exhibit 12 – Plot of Ground Water pH at Sampling Points

Exhibit 13 – Plot of Maximum Water Chloride Concentrations



LEGEND

Maximum Water Chloride (mg/L)

- <= 700
- 701 - 2000
- 2001 - 4000
- 4001 - 8000
- > 8000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

Exhibit 13 – Plot of Maximum Water Chloride Concentrations

LEGEND

Maximum Water Chloride (mg/L)

- <= 700
- 701 - 2000
- 2001 - 4000
- 4001 - 8000
- > 8000

Bridge Alignment

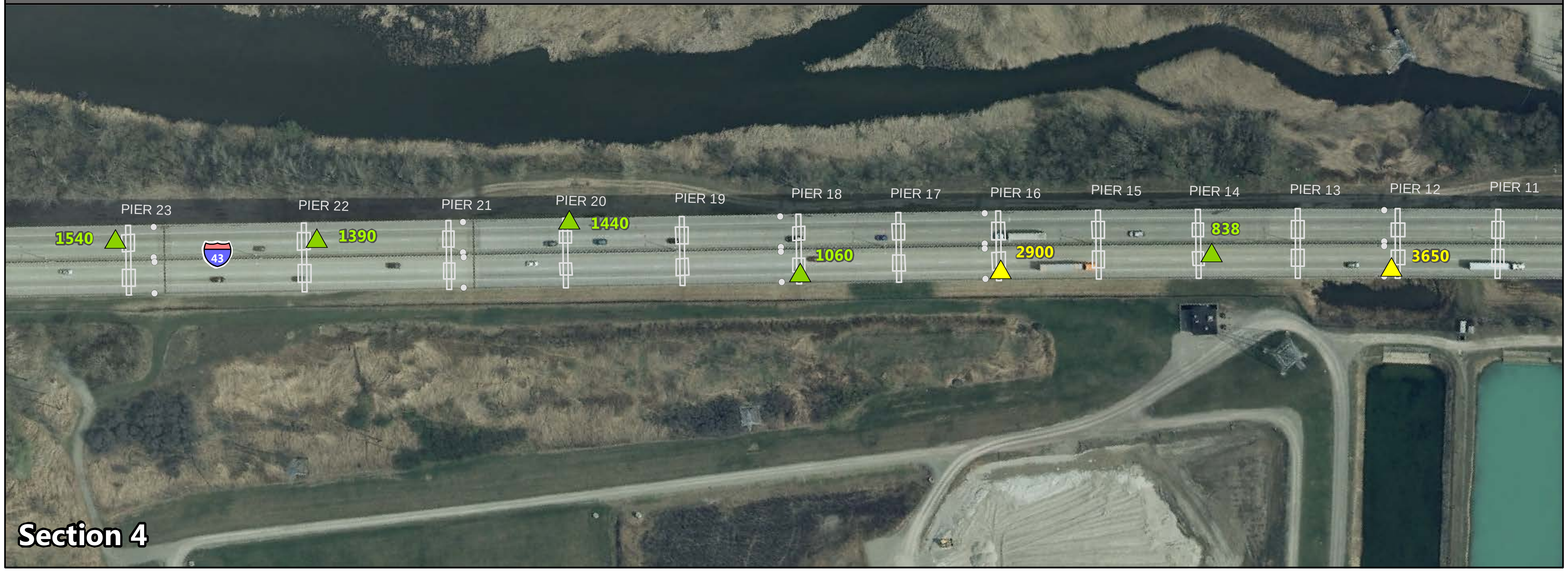
- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

Baker

Exhibit 13 – Plot of Maximum Water Chloride Concentrations



LEGEND

Maximum Water Chloride (mg/L)

- <= 700
- 701 - 2000
- 2001 - 4000
- 4001 - 8000
- > 8000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

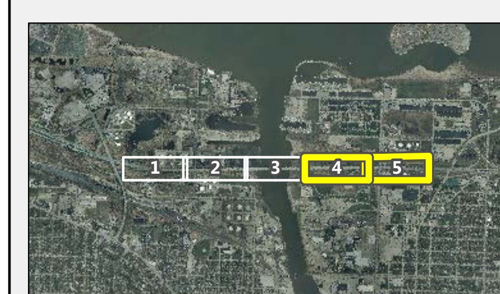
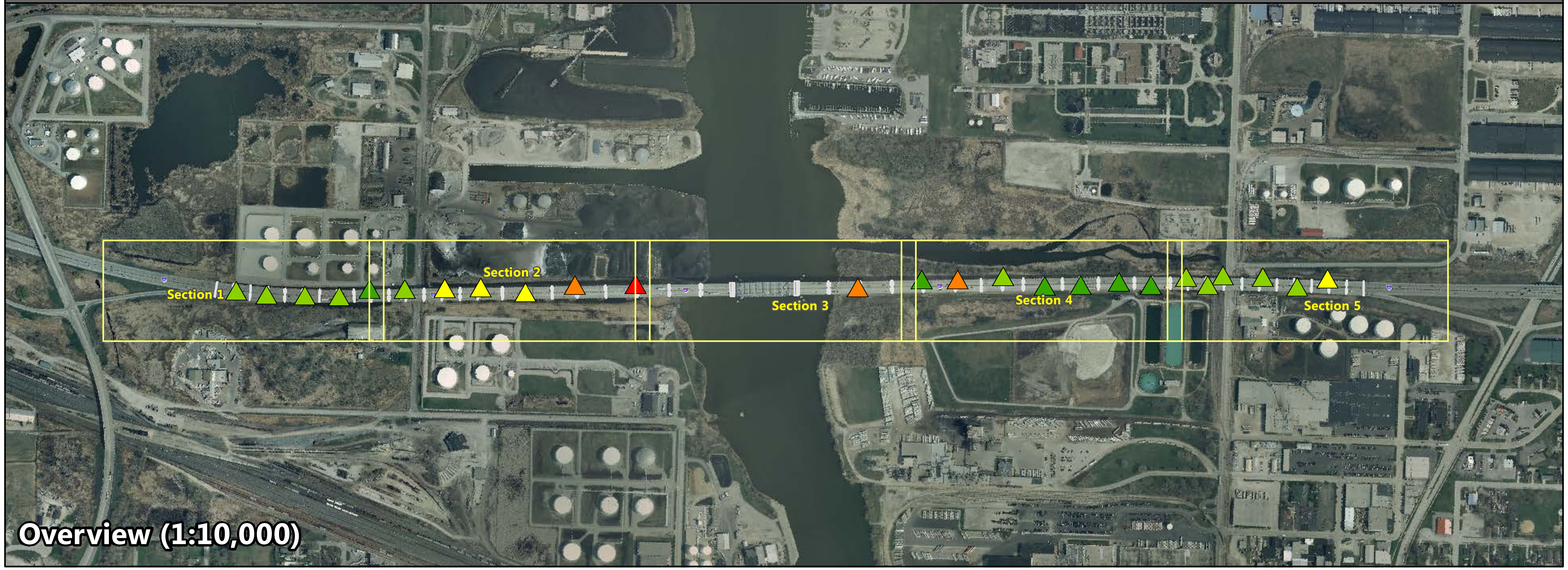


Exhibit 14 – Plot of Maximum Water Sulfate Concentrations



LEGEND

Maximum Water Sulfate (mg/L)

- <= 100
- 101 - 500
- 501 - 1000
- 1001 - 2000
- > 2000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

Baker

Exhibit 14 – Plot of Maximum Water Sulfate Concentrations

LEGEND

Maximum Water Sulfate (mg/L)

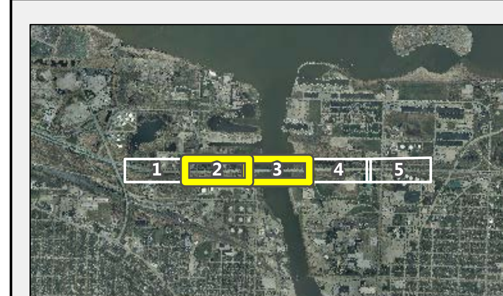
- <= 100
- 101 - 500
- 501 - 1000
- 1001 - 2000
- > 2000

Bridge Alignment

- Substructure
- Scupper

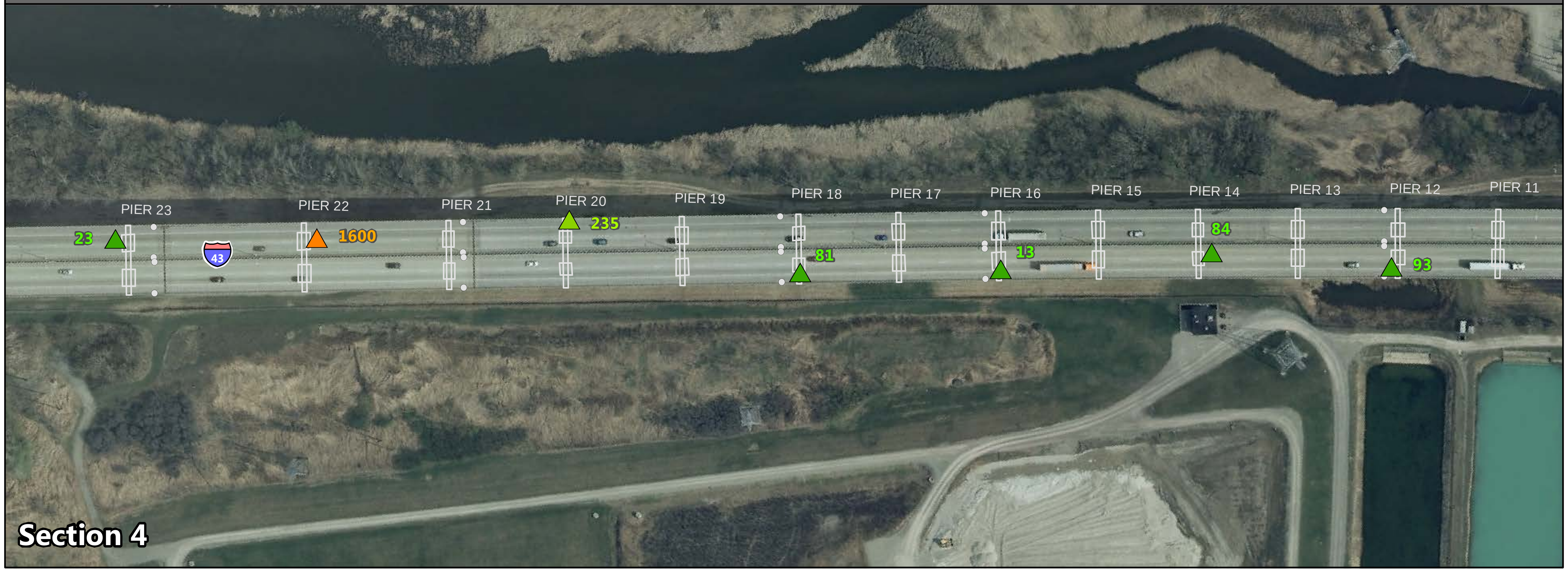
0 50 100 150 feet

1 in = 150 feet 1 : 1,800



Baker

Exhibit 14 – Plot of Maximum Water Sulfate Concentrations



LEGEND

Maximum Water Sulfate (mg/L)

- <= 100
- 101 - 500
- 501 - 1000
- 1001 - 2000
- > 2000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

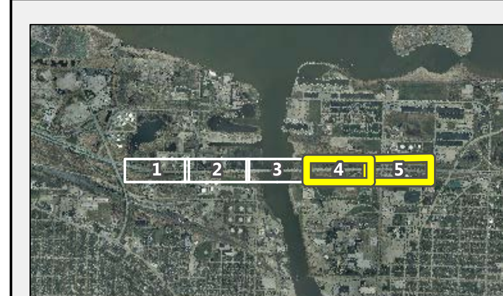
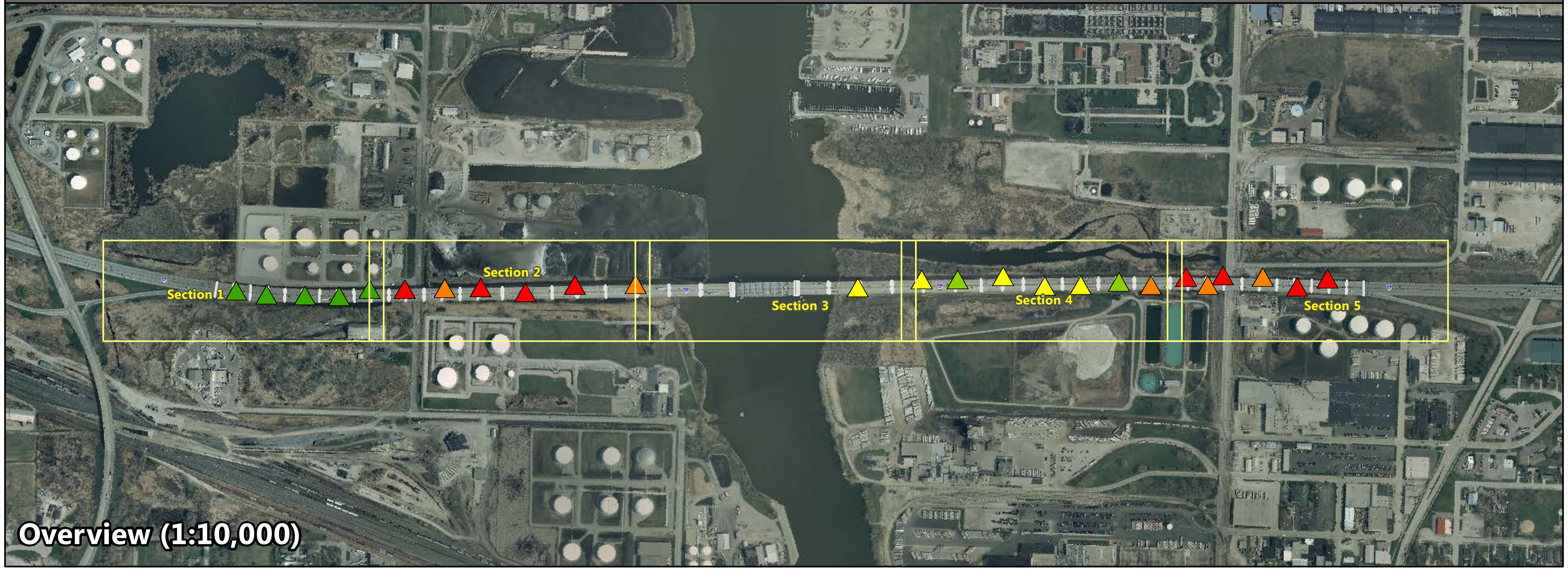


Exhibit 15 – Plot of Maximum Water Alkalinity



LEGEND

Max. Water Alkalinity (mg/L $CaCO_3$)

- <= 300
- 301 - 600
- 601 - 1000
- 1001 - 1500
- > 1500

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

Baker

Exhibit 15 – Plot of Maximum Water Alkalinity

LEGEND

Max. Water Alkalinity (mg/L $CaCO_3$)

- <= 300
- 301 - 600
- 601 - 1000
- 1001 - 1500
- > 1500

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

N E S W

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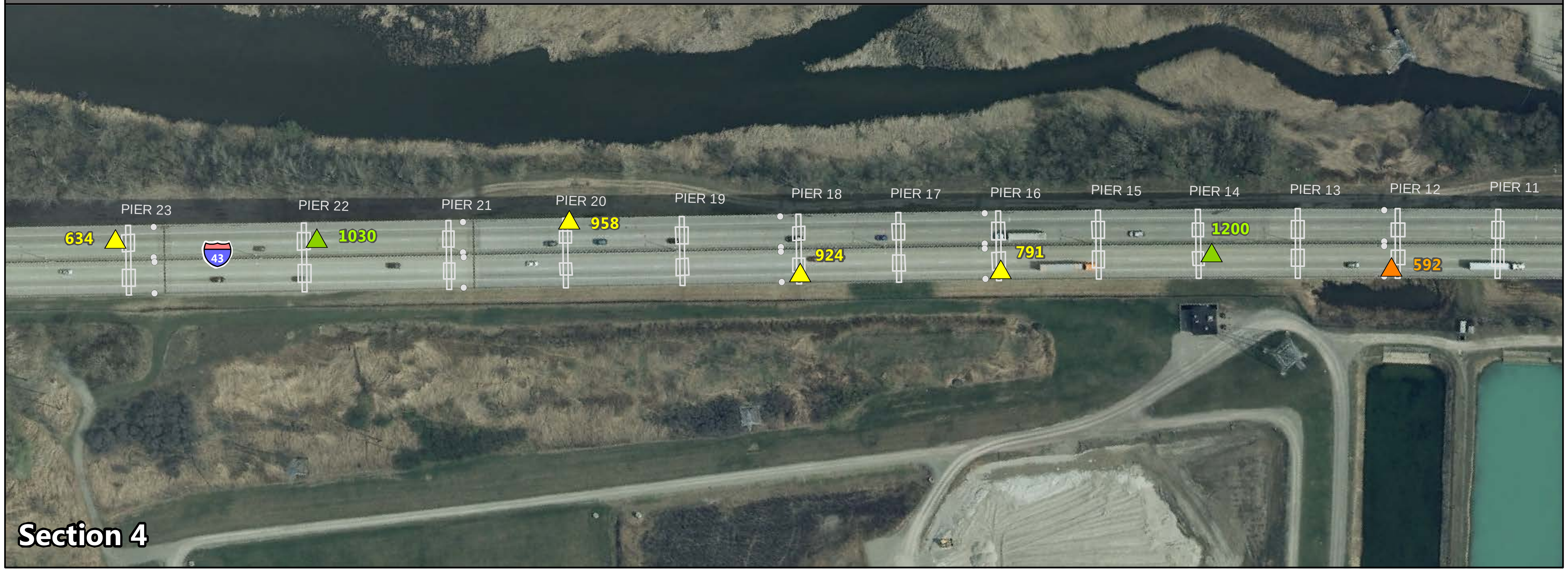
Exhibit 15 – Plot of Maximum Water Alkalinity



LEGEND

Max. Water Alkalinity (mg/L $CaCO_3$)

- <= 300
- 301 - 600
- 601 - 1000
- 1001 - 1500
- > 1500

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

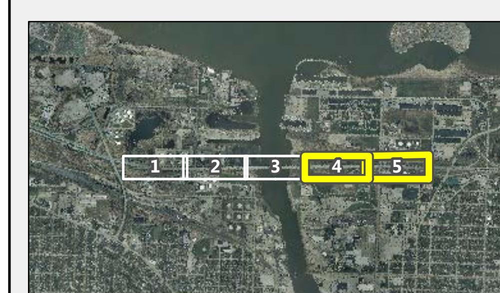
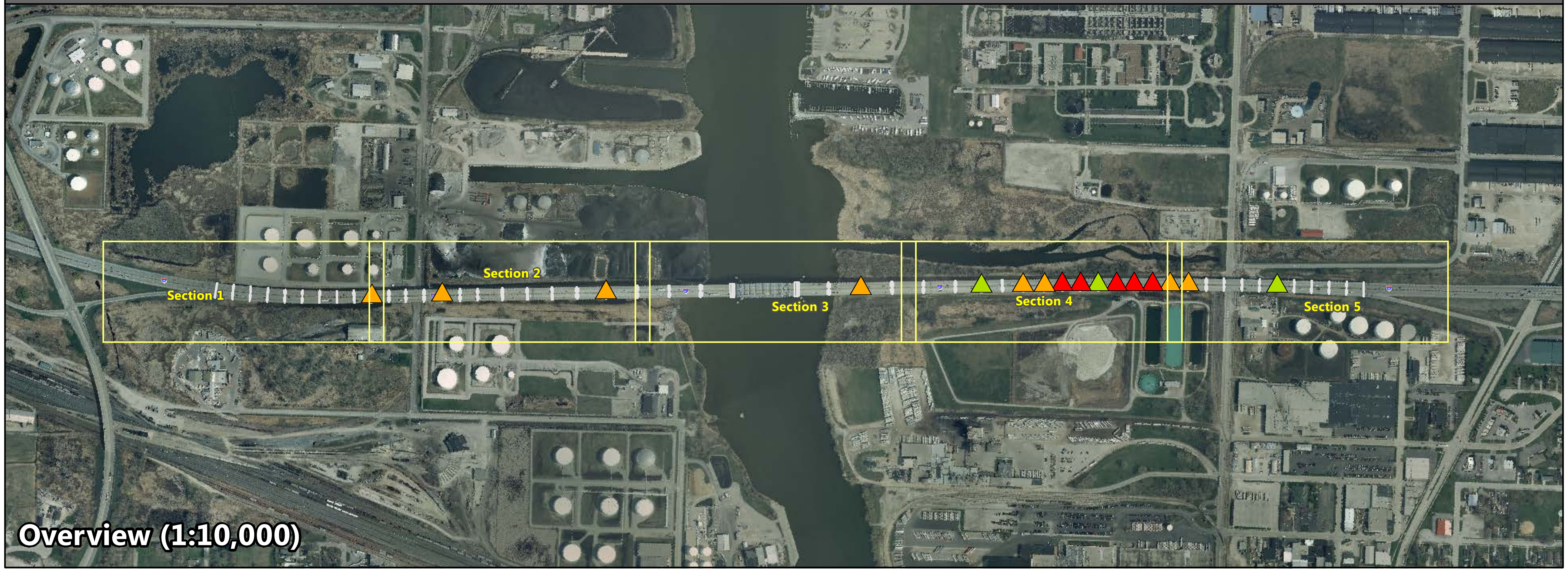


Exhibit 16 – Plot of Low Nutrient Bacteria



LEGEND

Low Nutrient Bacteria (colonies/ml)

- 10 to 100
- 1,000 to 10,000
- > 100,000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

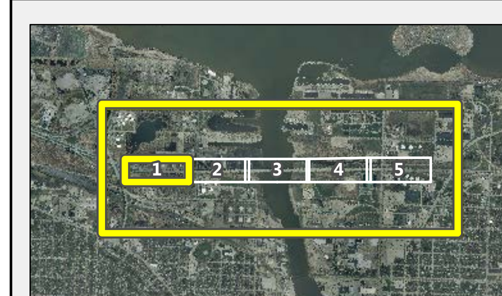


Exhibit 16 – Plot of Low Nutrient Bacteria

LEGEND

Low Nutrient Bacteria (colonies/ml)

- 10 to 100
- 1,000 to 10,000
- > 100,000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

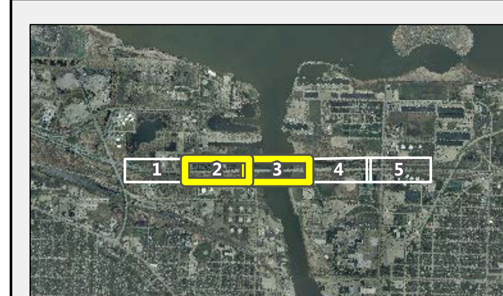
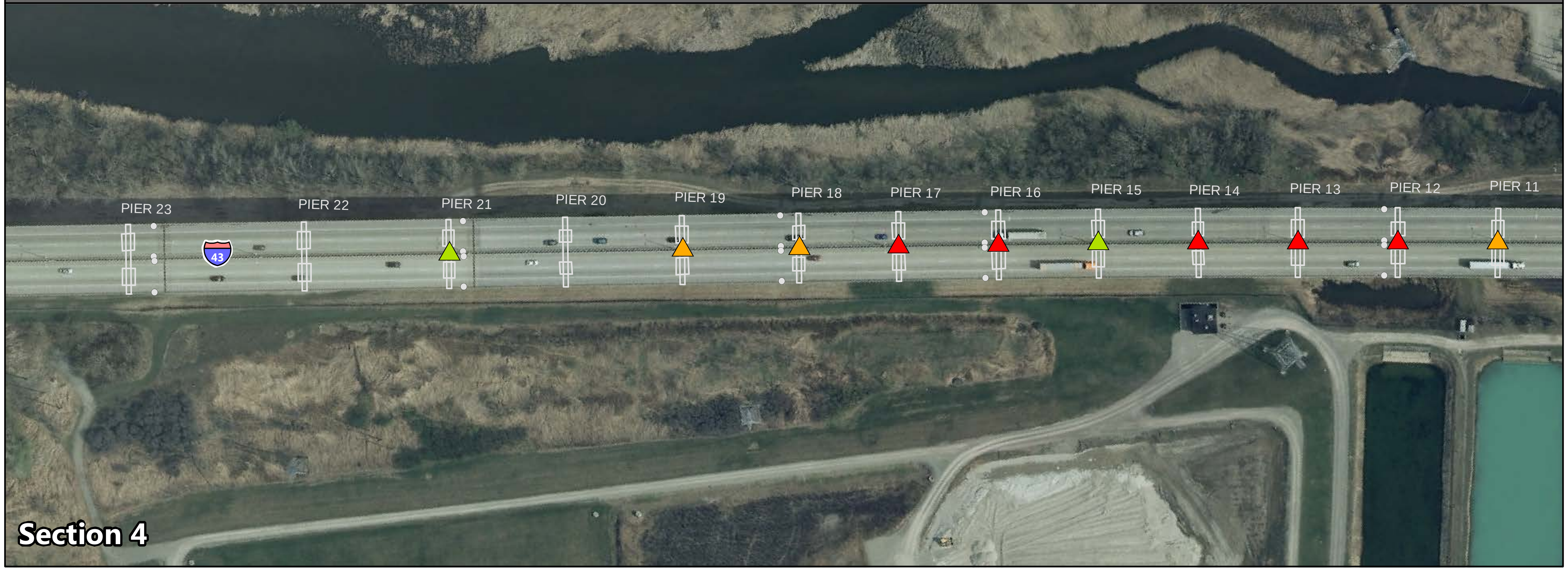


Exhibit 16 – Plot of Low Nutrient Bacteria



LEGEND

Low Nutrient Bacteria (colonies/ml)

- 10 to 100
- 1,000 to 10,000
- > 100,000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

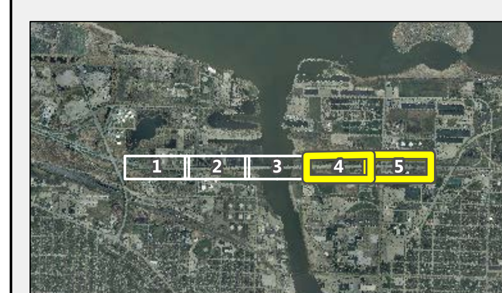
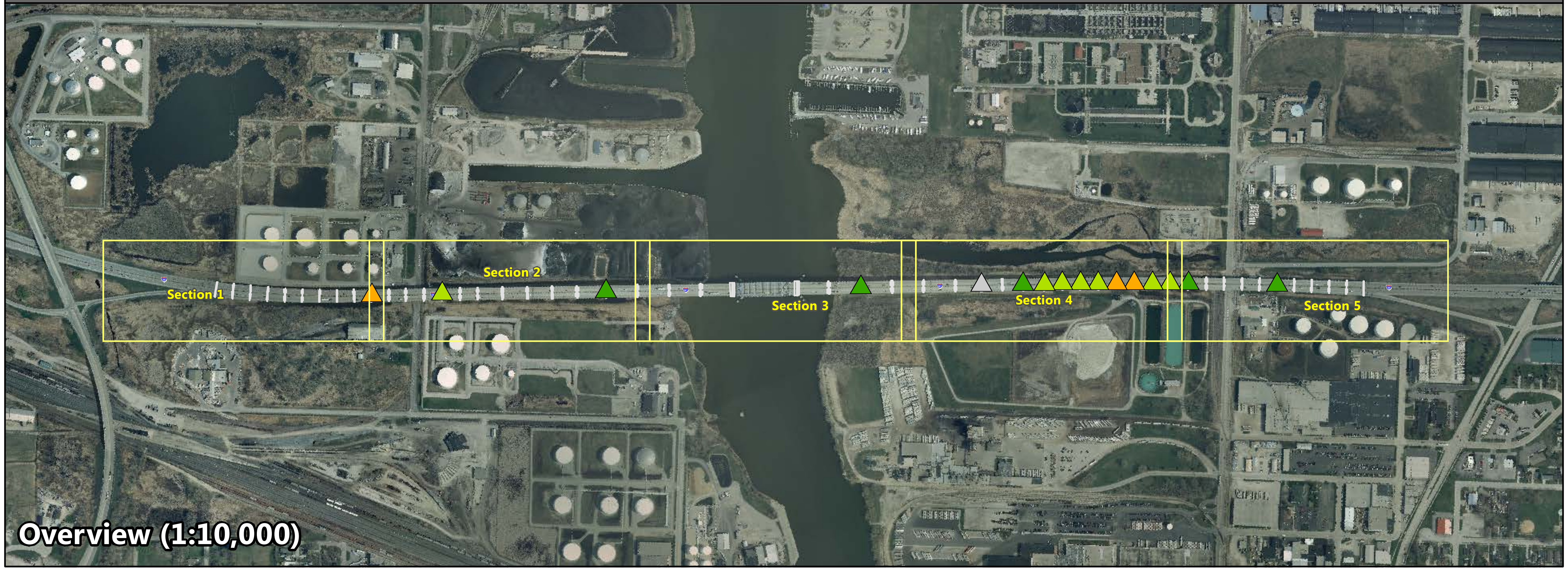
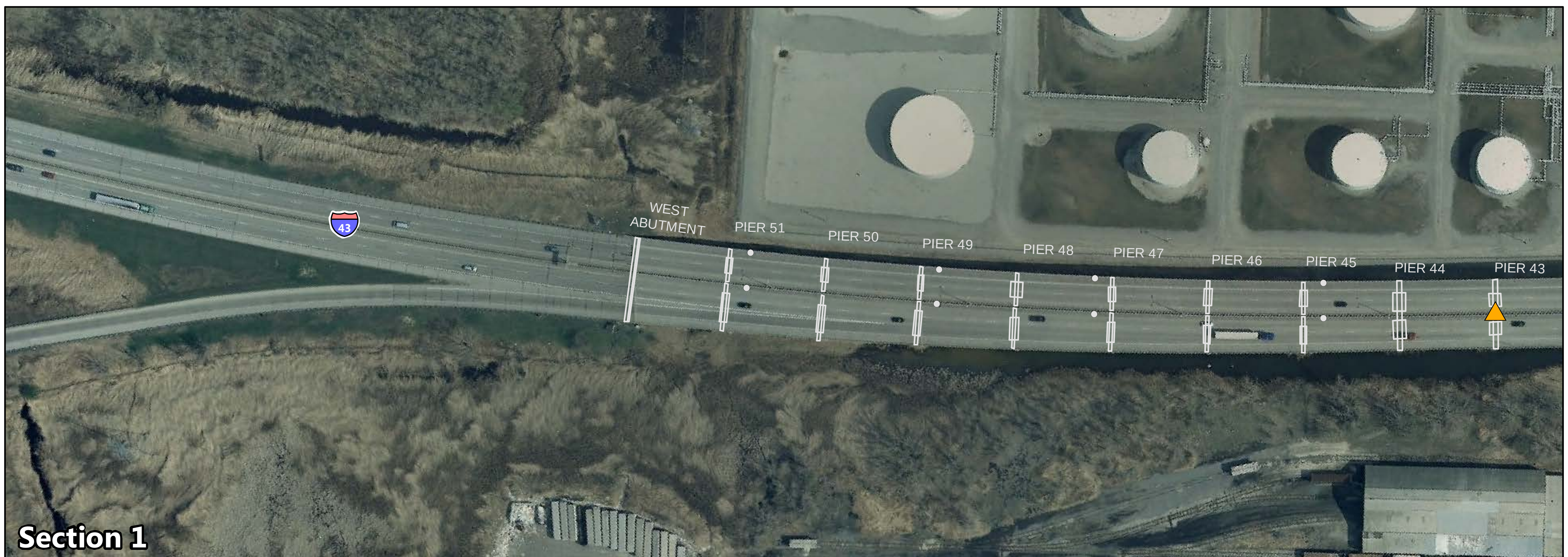


Exhibit 17 – Plot of Iron Reducing Bacteria





LEGEND

Iron Reducing Bacteria (colonies/ml)

- NR
- 1 to 10
- 10 to 100
- 1,000 to 10,000

Bridge Alignment

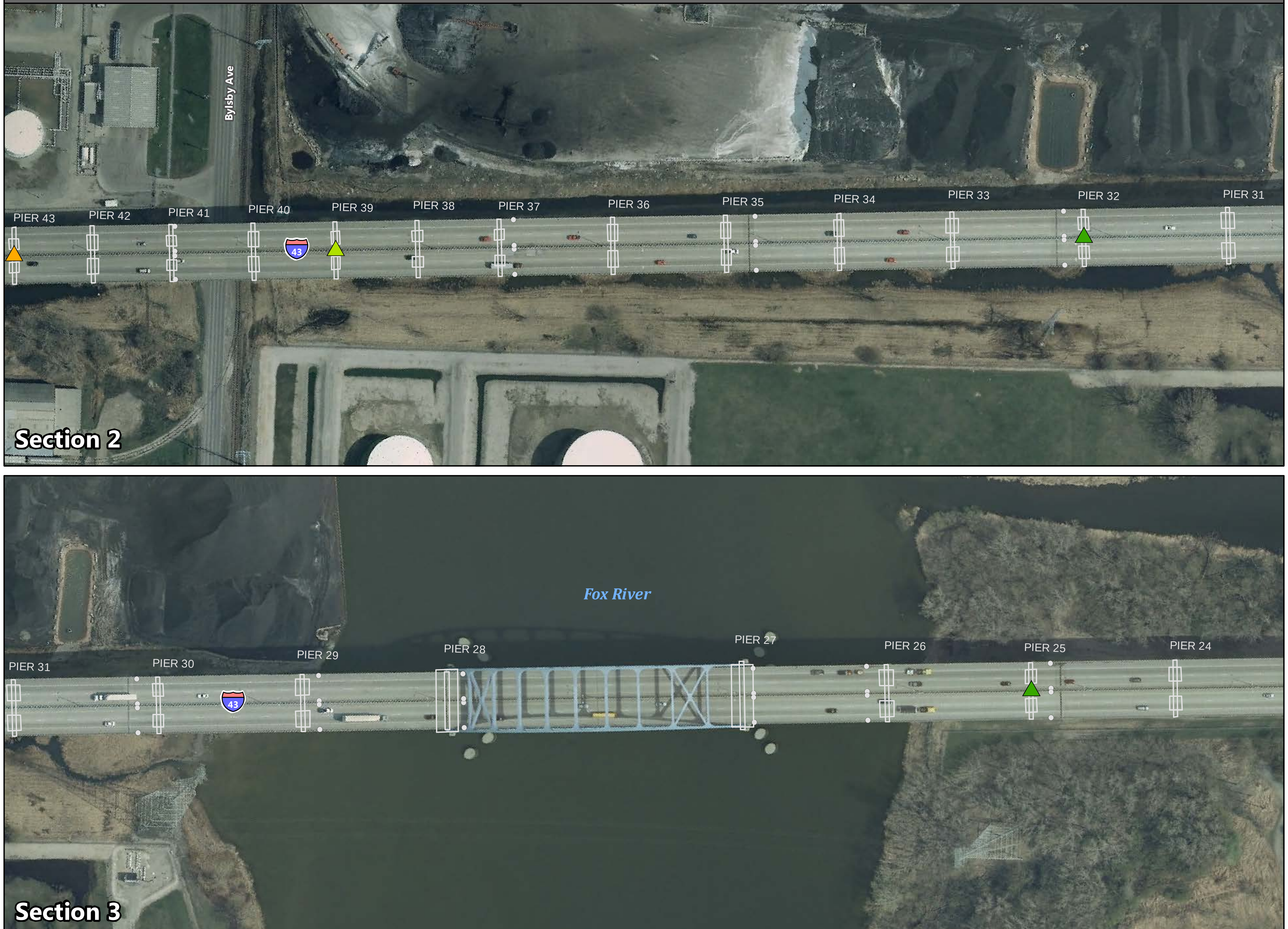
- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

Baker

Exhibit 17 – Plot of Iron Reducing Bacteria



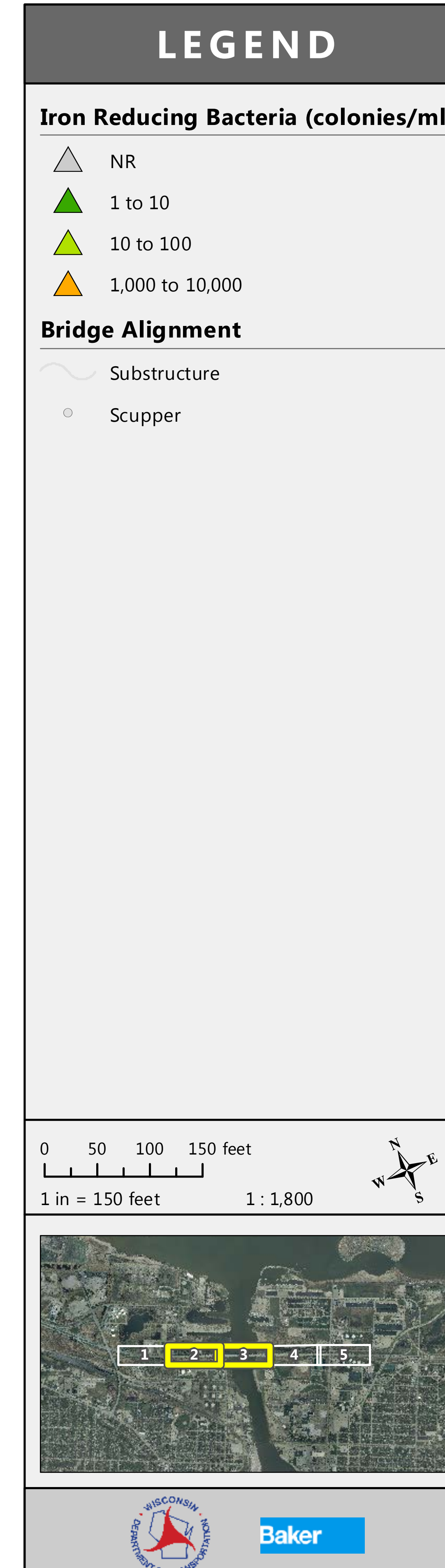


Exhibit 17 – Plot of Iron Reducing Bacteria

LEGEND

Iron Reducing Bacteria (colonies/ml)

- NR
- 1 to 10
- 10 to 100
- 1,000 to 10,000

Bridge Alignment

- Substructure
- Scupper

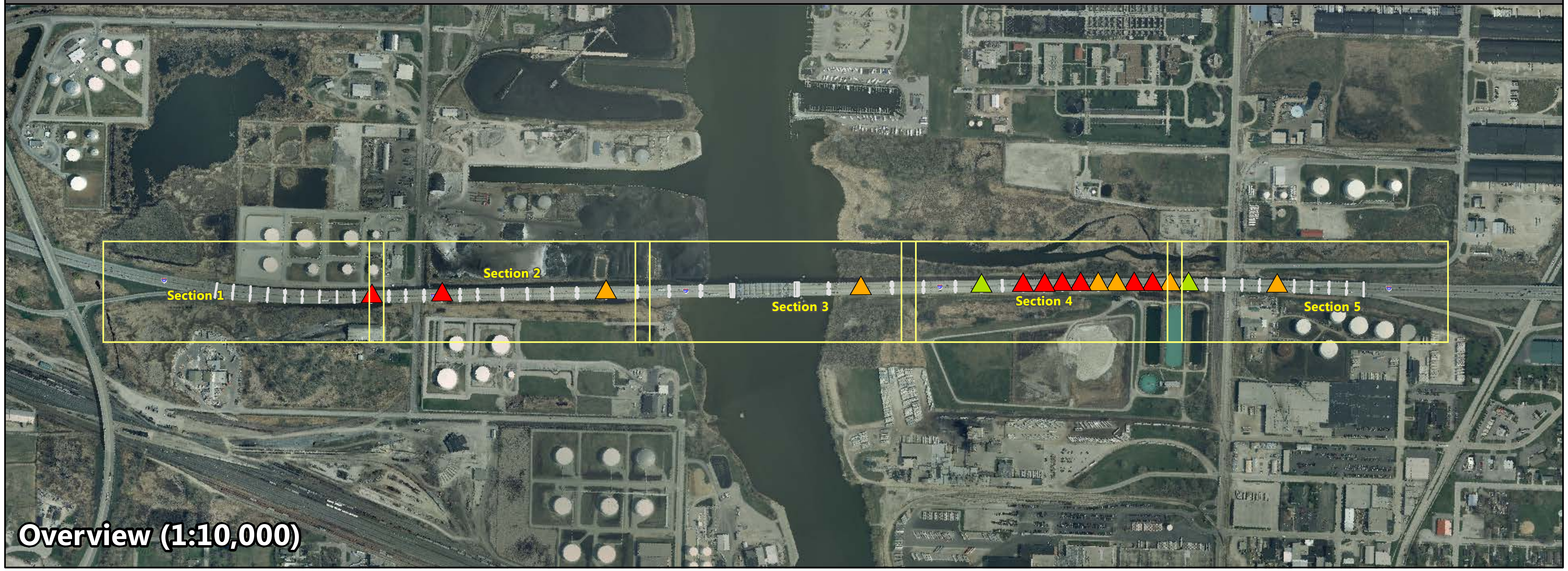
0 50 100 150 feet

1 in = 150 feet 1 : 1,800

N E S W

1 2 3 4 5

Exhibit 18 – Plot of Anaerobic Bacteria



LEGEND

Anaerobic Bacteria (colonies/ml)

- 10 to 100
- 1,000 to 10,000
- > 100,000

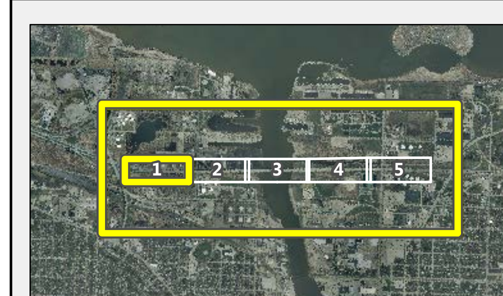
Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

N E S W



Baker

Exhibit 18 – Plot of Anaerobic Bacteria

LEGEND

Anaerobic Bacteria (colonies/ml)

- 10 to 100
- 1,000 to 10,000
- > 100,000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

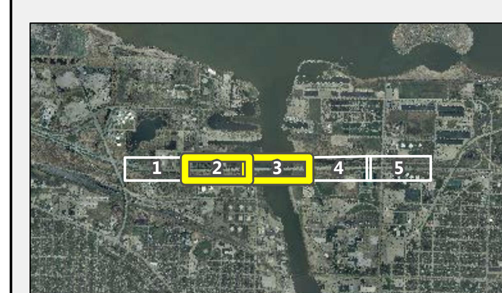
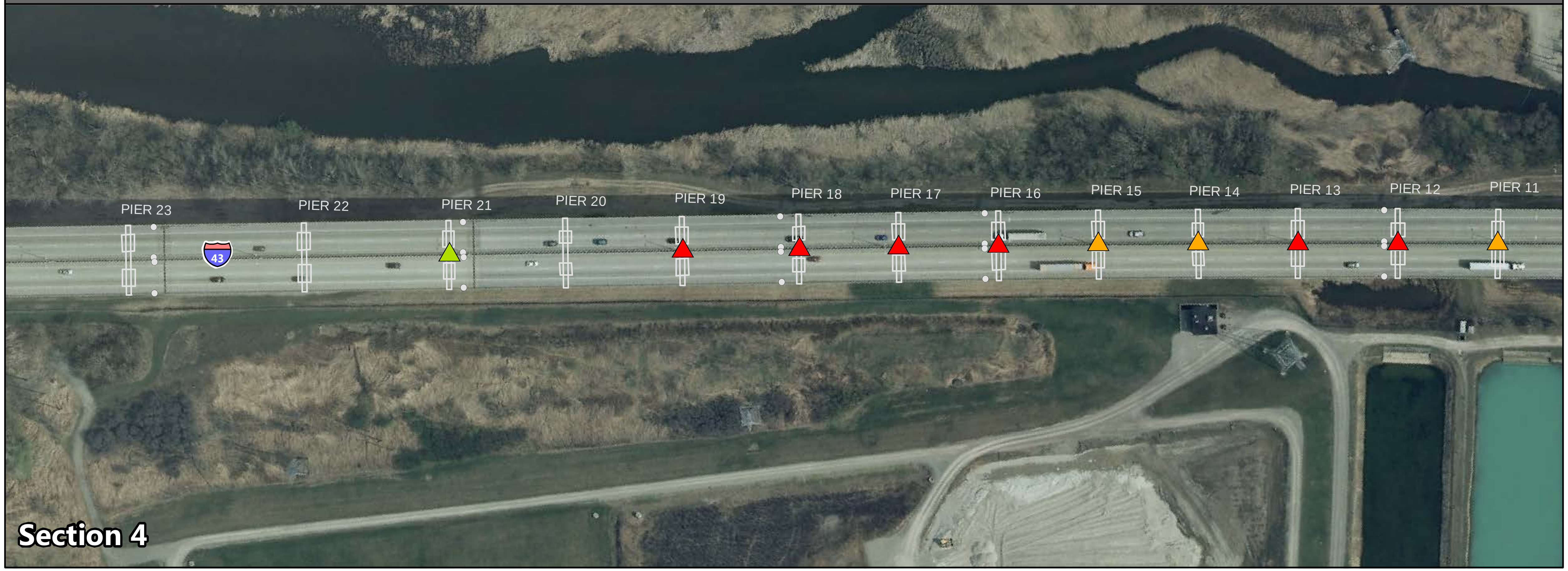


Exhibit 18 – Plot of Anaerobic Bacteria



LEGEND

Anaerobic Bacteria (colonies/ml)

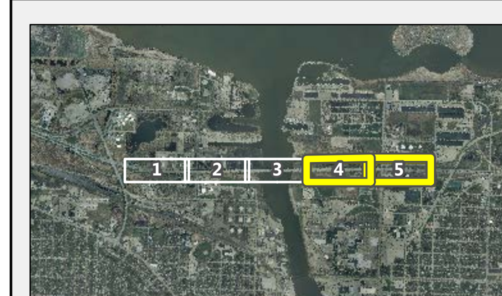
- 10 to 100
- 1,000 to 10,000
- > 100,000

Bridge Alignment

- Substructure
- Scupper

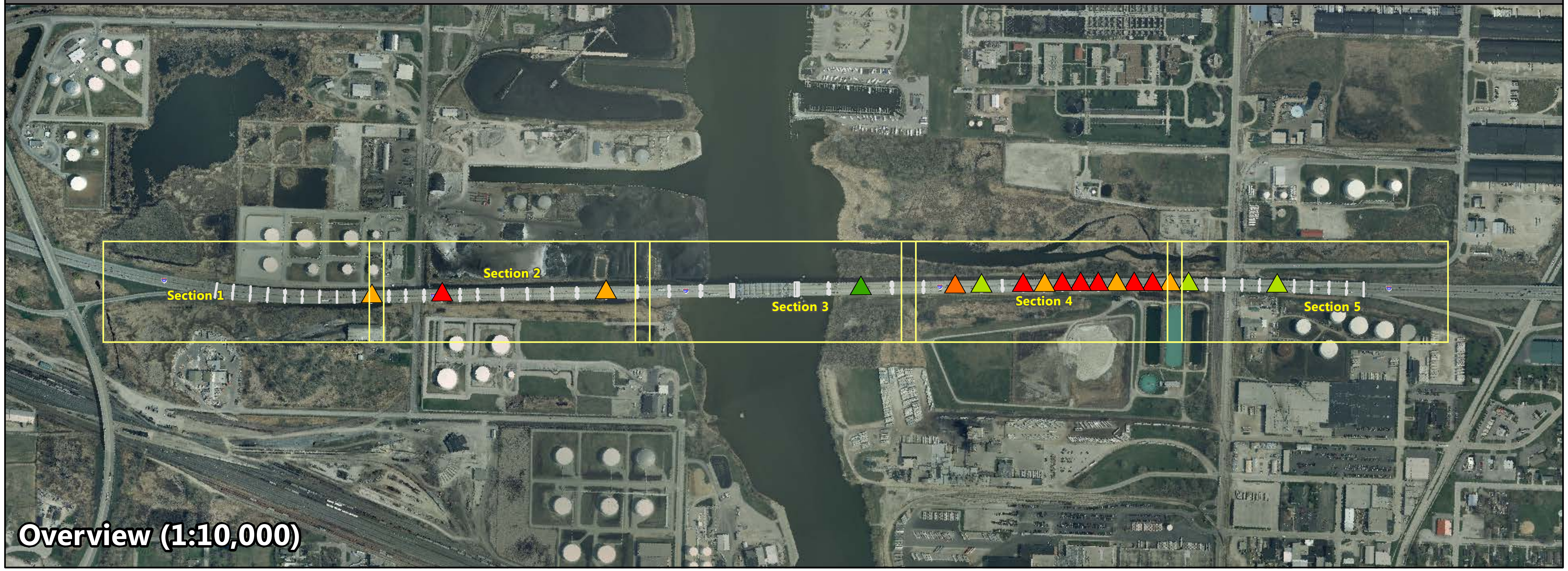
0 50 100 150 feet

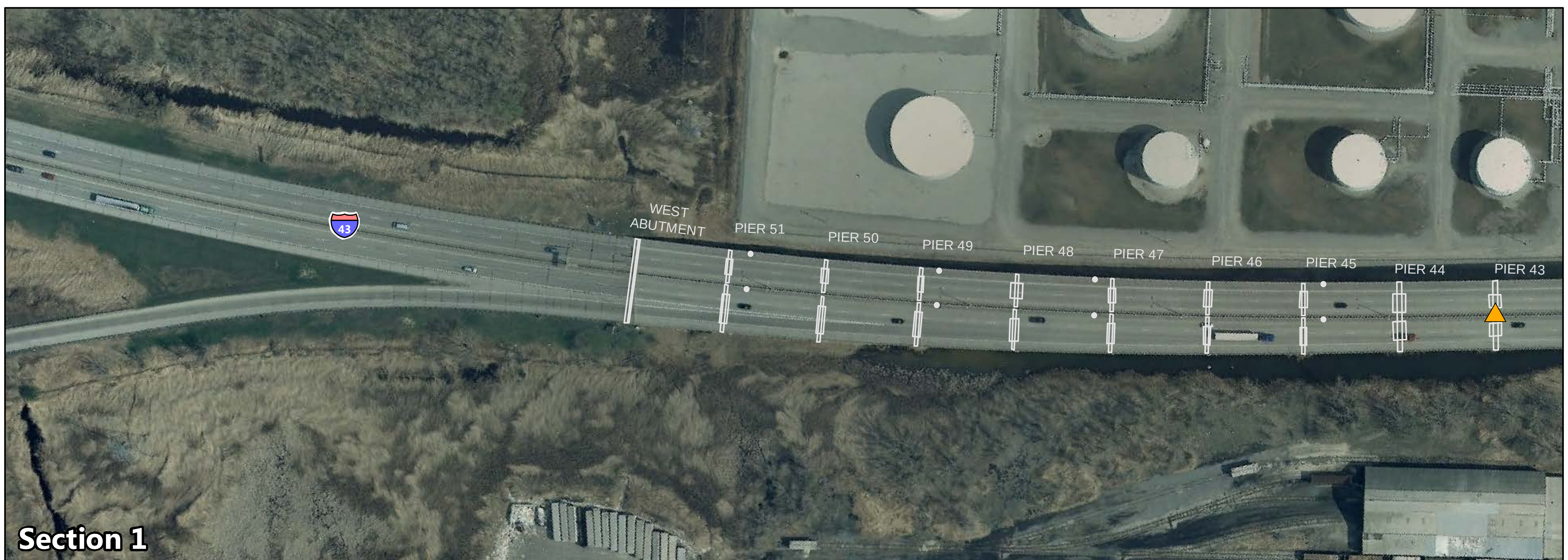
1 in = 150 feet 1 : 1,800



Baker

Exhibit 19 – Plot of Acid Producing Bacteria





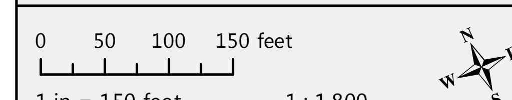
LEGEND

Acid Producing Bacteria (colonies/ml)

- 1 to 10
- 10 to 100
- 1,000 to 10,000
- 10,001 to 100,000
- >100,000

Bridge Alignment

- Substructure
- Scupper



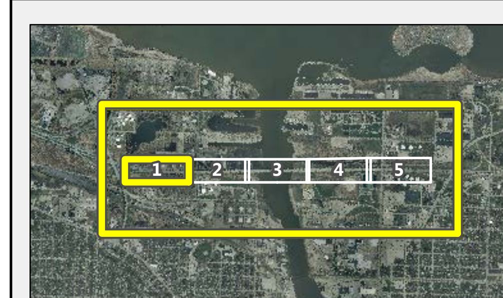


Exhibit 19 – Plot of Acid Producing Bacteria

LEGEND

Acid Producing Bacteria (colonies/ml)

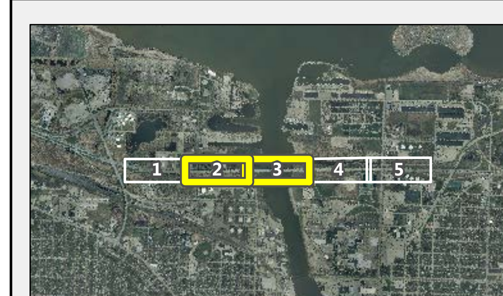
- 1 to 10
- 10 to 100
- 1,000 to 10,000
- 10,001 to 100,000
- >100,000

Bridge Alignment

- Substructure
- Scupper

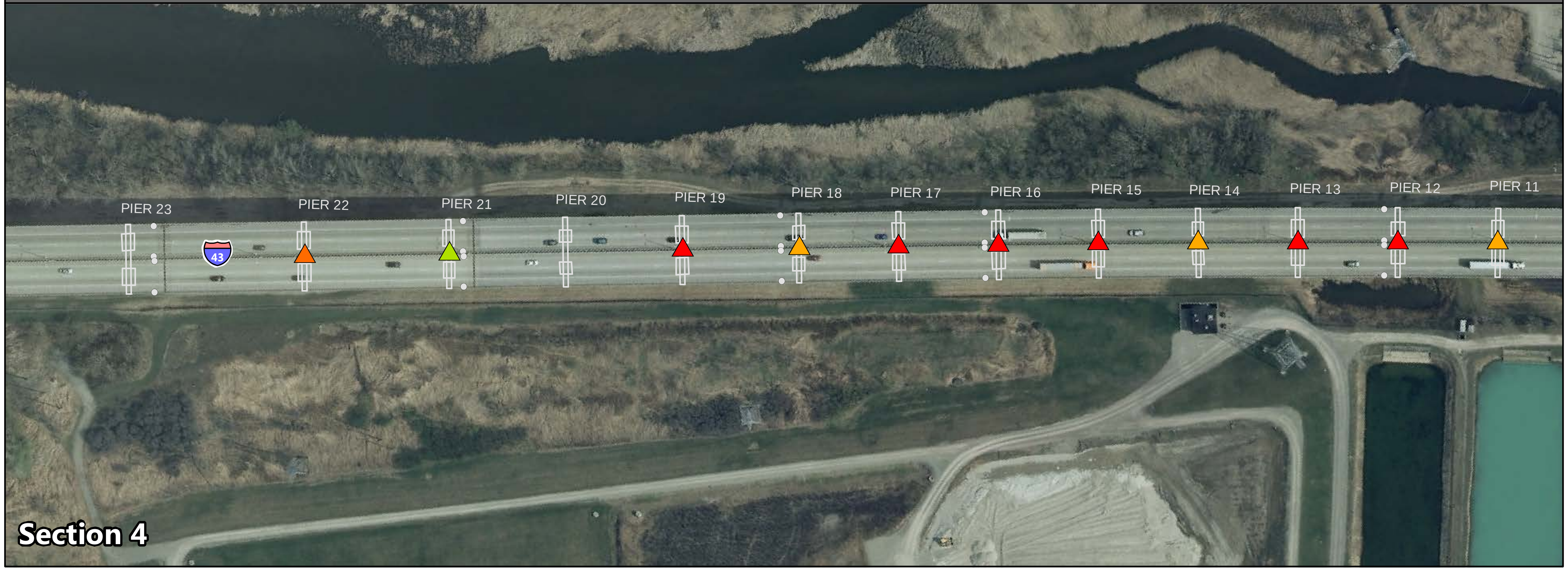
0 50 100 150 feet

1 in = 150 feet 1 : 1,800



Baker

Exhibit 19 – Plot of Acid Producing Bacteria



LEGEND

Acid Producing Bacteria (colonies/ml)

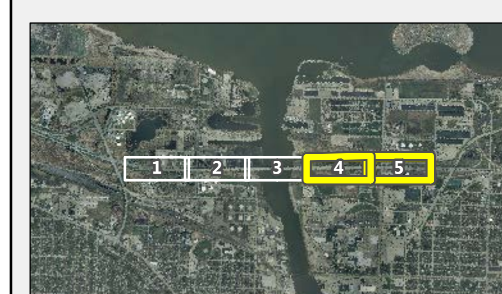
- 1 to 10
- 10 to 100
- 1,000 to 10,000
- 10,001 to 100,000
- >100,000

Bridge Alignment

- Substructure
- Scupper

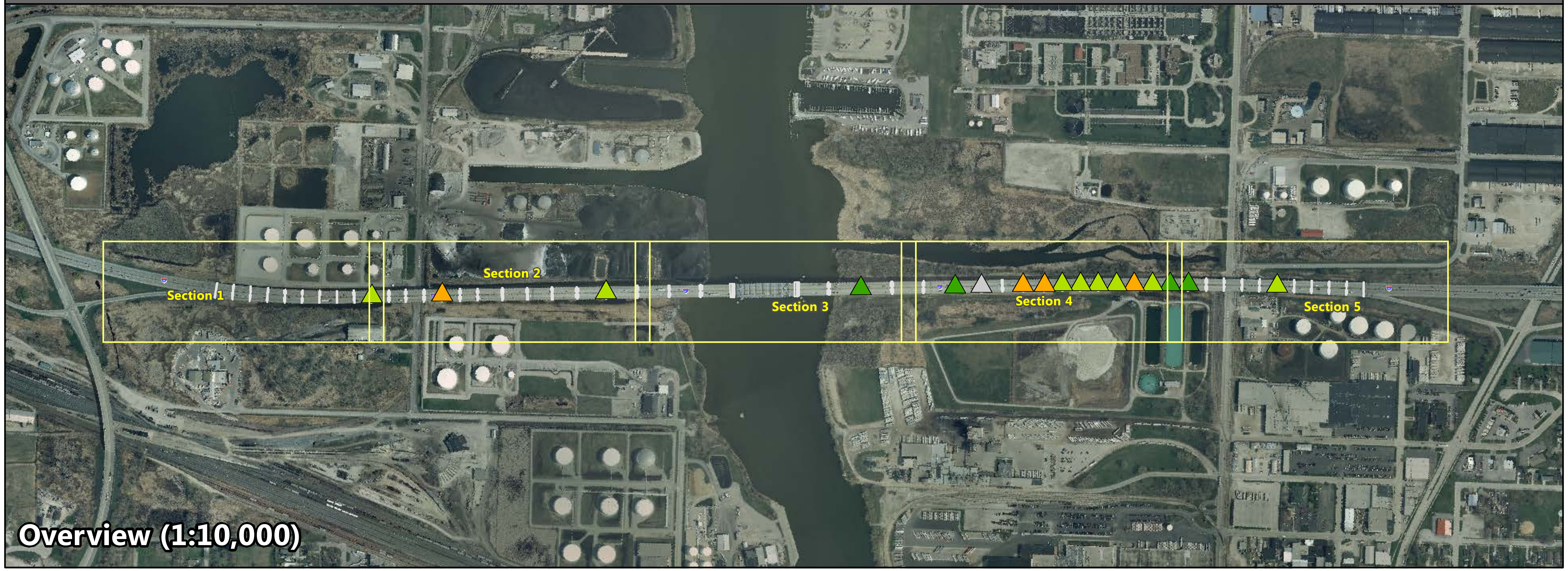
0 50 100 150 feet

1 in = 150 feet 1 : 1,800



Baker

Exhibit 20 – Plot of Sulfate Reducing Bacteria



LEGEND

Sulfate Reducing Bacteria (colonies/ml)

- NR
- 1 to 10
- 10 to 100
- 1,000 to 10,000

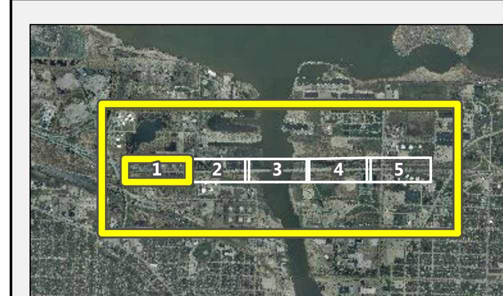
Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

N E S W



Baker

Exhibit 20 – Plot of Sulfate Reducing Bacteria

LEGEND

Sulfate Reducing Bacteria (colonies/ml)

- NR
- 1 to 10
- 10 to 100
- 1,000 to 10,000

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

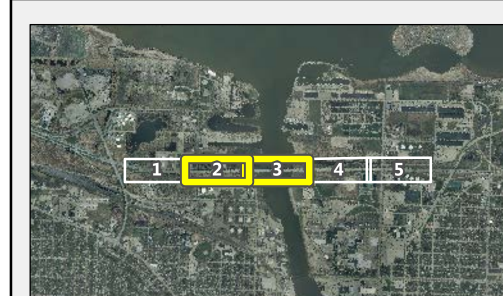
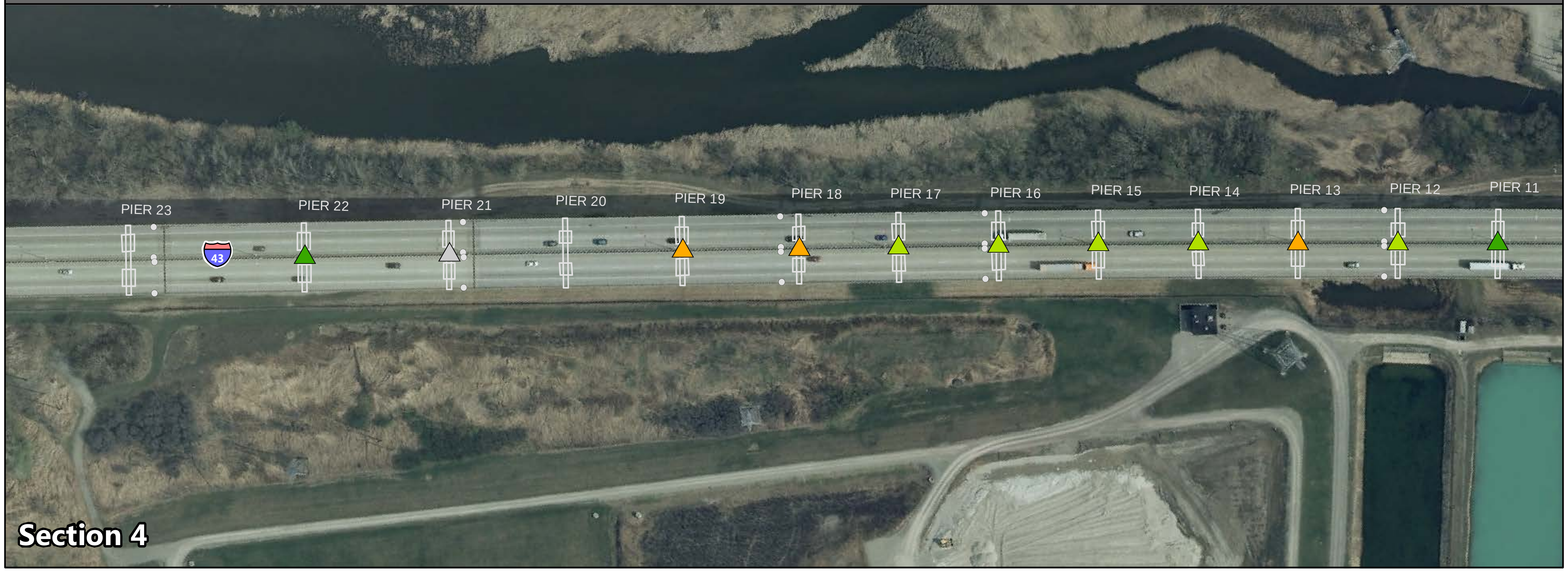


Exhibit 20 – Plot of Sulfate Reducing Bacteria



LEGEND

Sulfate Reducing Bacteria (colonies/ml)

- NR
- 1 to 10
- 10 to 100
- 1,000 to 10,000

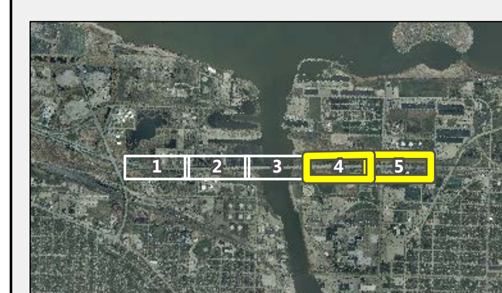
Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

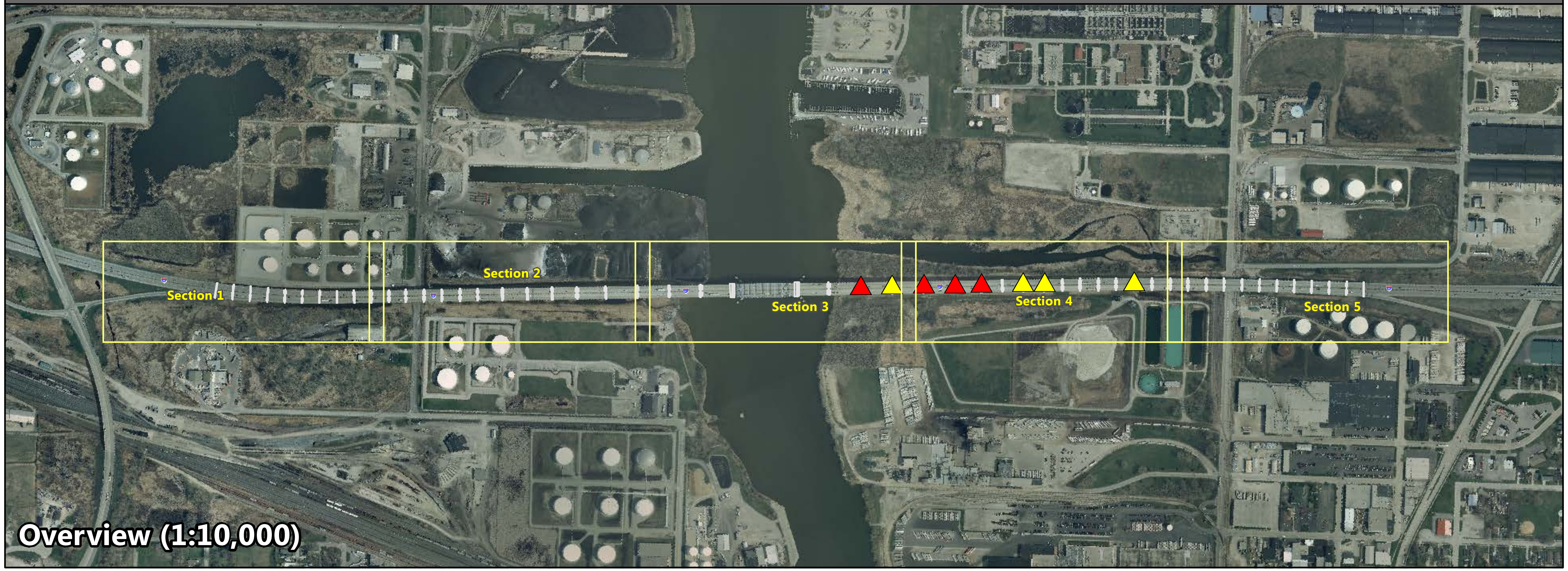
1 in = 150 feet 1 : 1,800

N E S W



Baker

Exhibit 21 – Plot of Ash Encountered



LEGEND

Ash Encountered

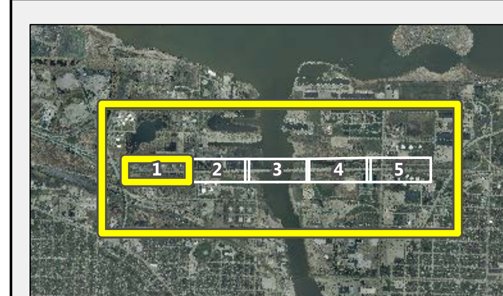
- With FWL* Above Bottom of Footer
- With FWL* Below Bottom of Footer

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800



Wisconsin Department of Transportation

Baker

Exhibit 21 – Plot of Ash Encountered

LEGEND

Ash Encountered

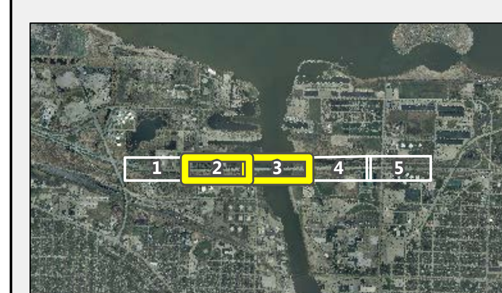
- With FWL* Above Bottom of Footer
- With FWL* Below Bottom of Footer

Bridge Alignment

- Substructure
- Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800



Baker

Exhibit 21 – Plot of Ash Encountered

LEGEND

Ash Encountered

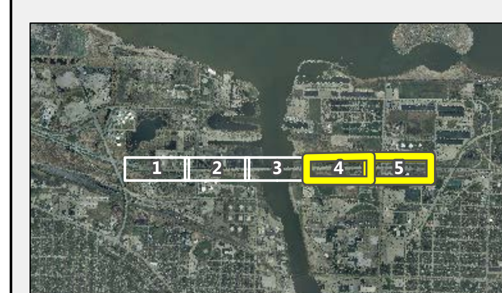
- With FWL* Above Bottom of Footer
- With FWL* Below Bottom of Footer

Bridge Alignment

- Substructure
- Scupper

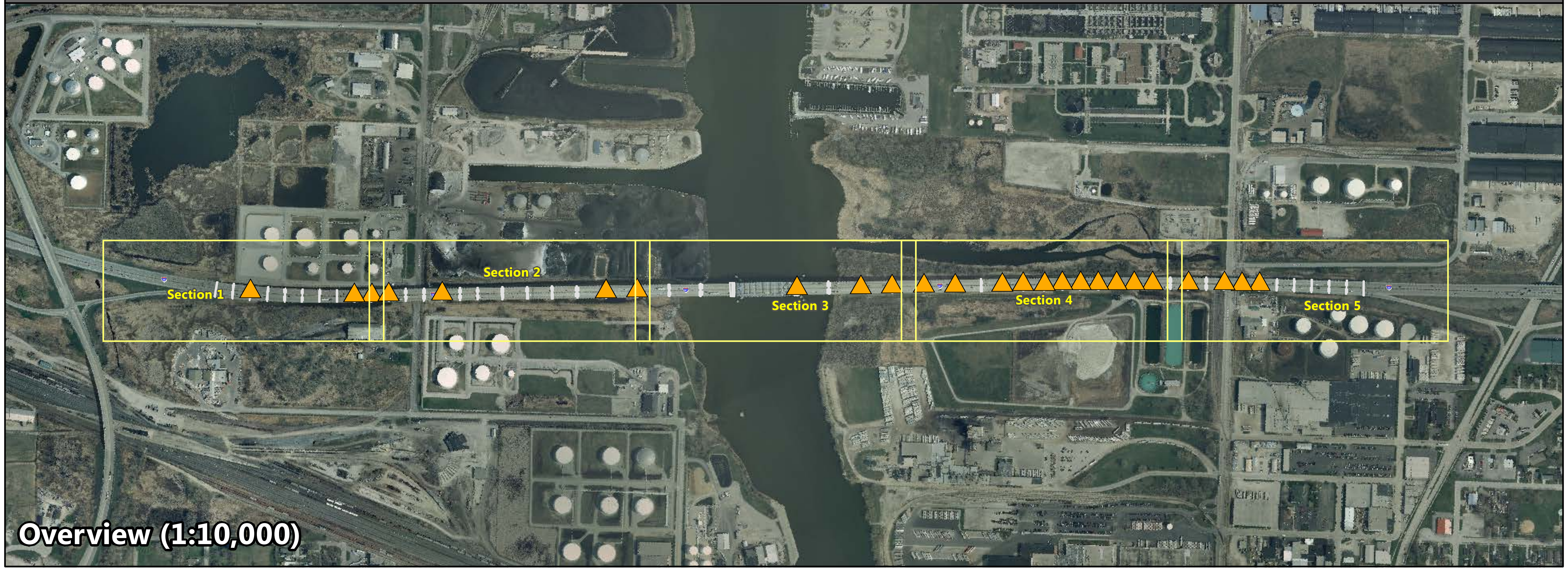
0 50 100 150 feet

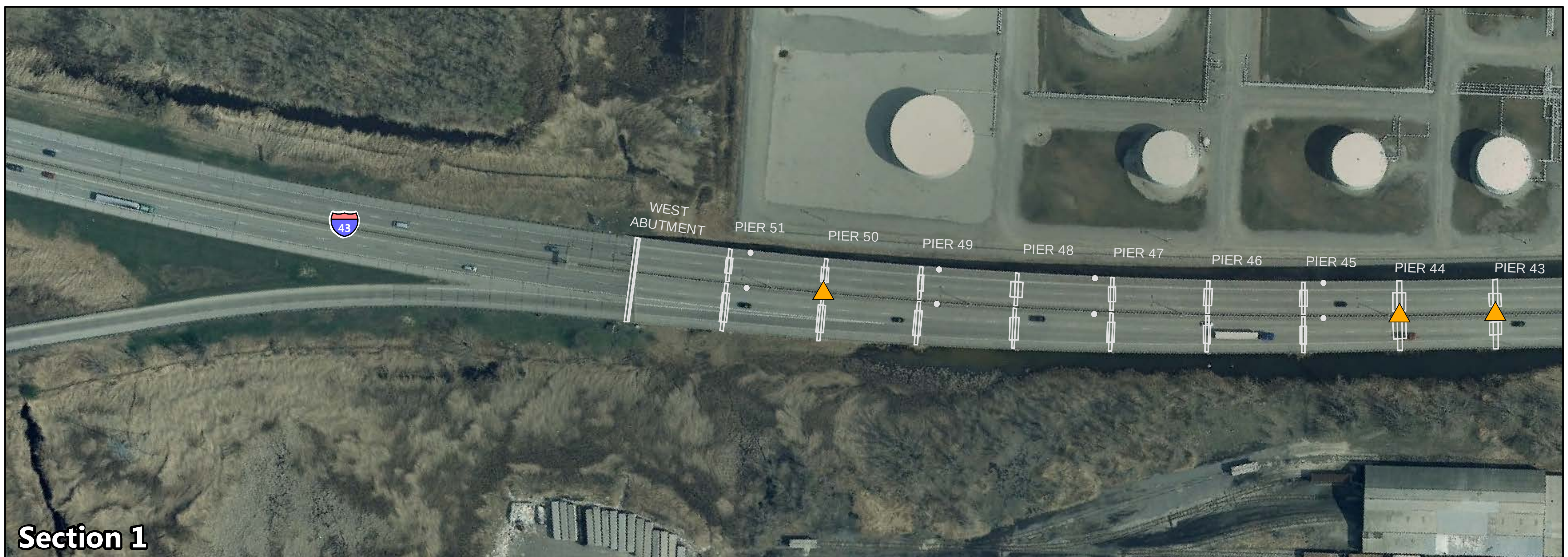
1 in = 150 feet 1 : 1,800



Baker

Exhibit 22 – Plot of Organics Encountered





LEGEND

Organics Encountered

Organics Encountered

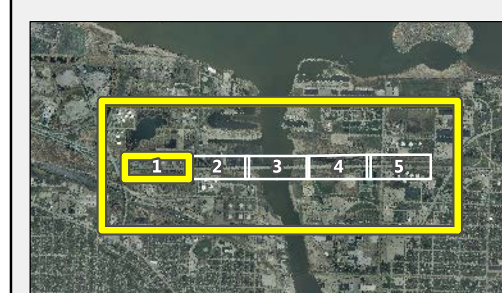
Bridge Alignment

Substructure

Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800



Baker

Exhibit 22 – Plot of Organics Encountered

LEGEND

Organics Encountered

Organics Encountered

Bridge Alignment

Substructure

Scupper

0 50 100 150 feet

1 in = 150 feet 1 : 1,800

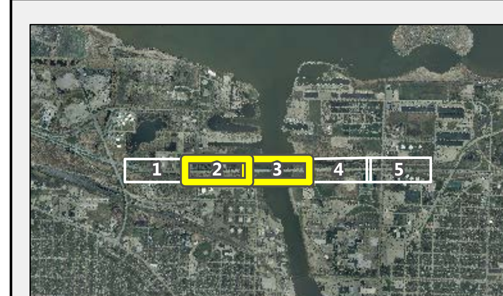
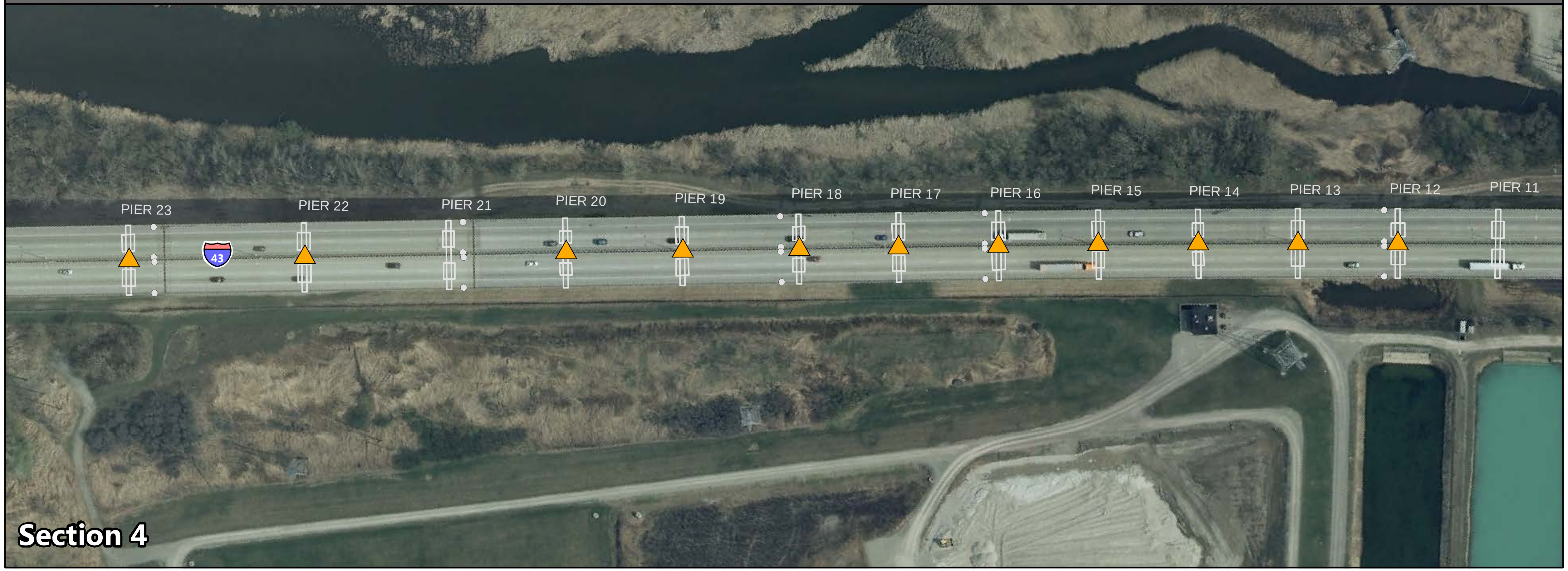


Exhibit 22 – Plot of Organics Encountered



LEGEND

Organics Encountered

Organics Encountered

Bridge Alignment

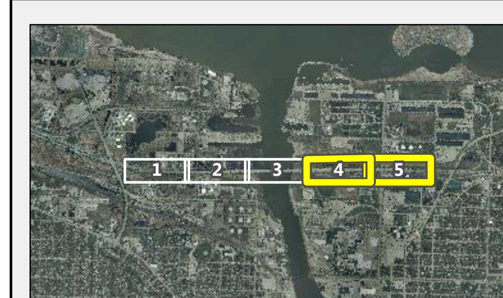
Substructure

Scupper

0 50 100 150 feet

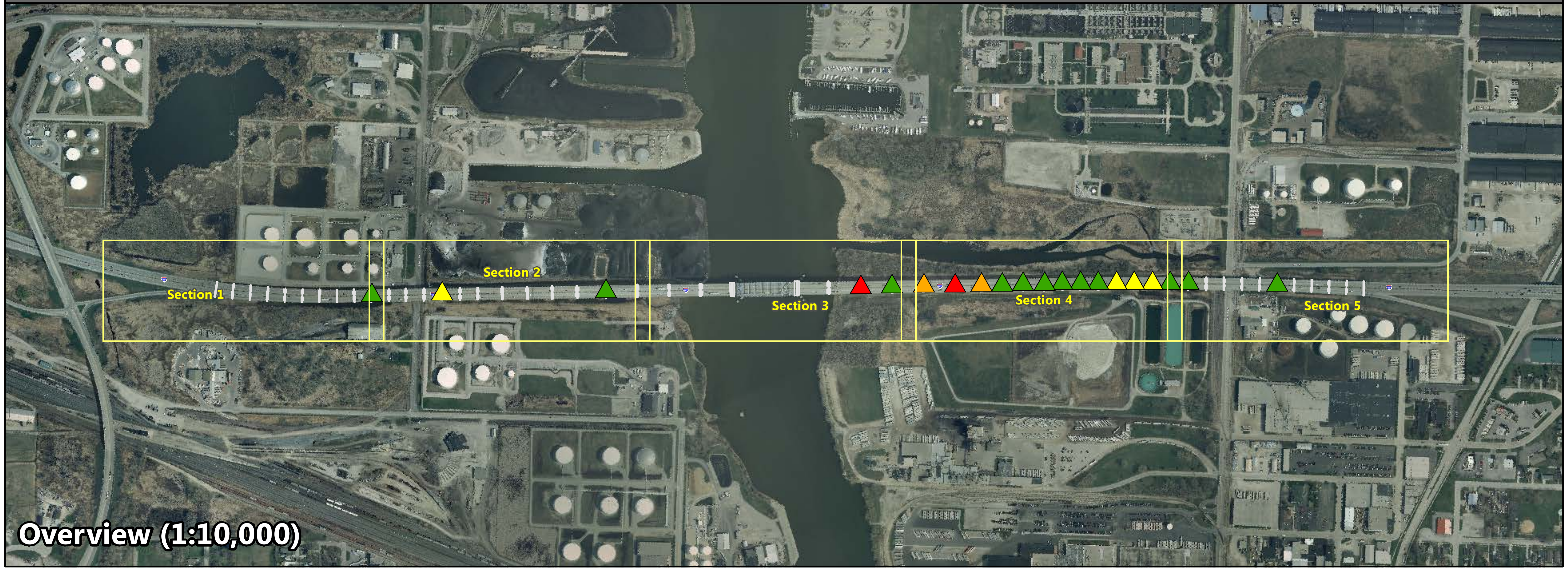
1 in = 150 feet 1 : 1,800

N E S W



Baker

Exhibit 23 – Plot of Average Section Loss Observed (Visually Rated)



LEGEND

Average Section Loss Observed*

- 0 - 5%
- 5 - 10%
- 20 - 30%
- > 30%

Bridge Alignment

- Substructure
- Scupper

*Visually rated

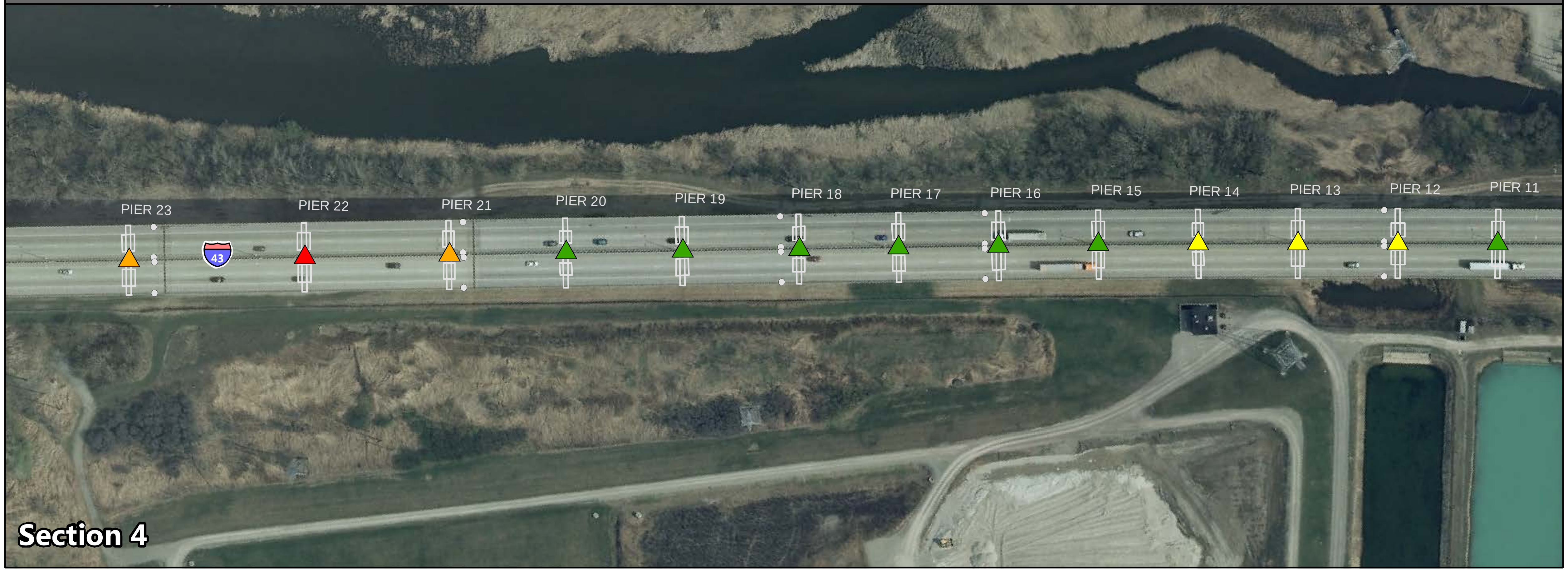
0 50 100 150 feet

1 in = 150 feet 1 : 1,800

Baker

Exhibit 23 – Plot of Average Section Loss Observed (Visually Rated)

Exhibit 23 – Plot of Average Section Loss Observed (Visually Rated)



LEGEND

Average Section Loss Observed*

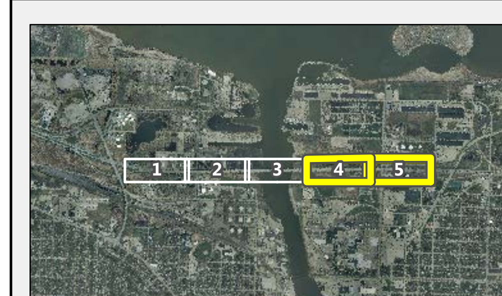
- 0 - 5%
- 5 - 10%
- 20 - 30%
- > 30%

Bridge Alignment

- Substructure
- Scupper

*Visually rated

0 50 100 150 feet
1 in = 150 feet 1 : 1,800



Baker