US Army Corps
of Engineers®
New York District

NEW YORK/NEW JERSEY HARBOR & TRIBUTARIES STUDY (HATS) – TO SUPPORT THE DESIGN AND COST OF EARLY ACTIONABLE ELEMENTS

East Riser Channel

Engineering and Design Sub Appendix B3
Feasibility Submission

MARCH 2026

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ACRONYMS AND ABBREVIATIONS

AASHTO LRFD	American Association of State Highway and Transportation Officials Load and Resistance Factor Design
AREMA MRE	American Railway Engineering and Maintenance-of-Way Association's Manual for Railway Engineering
ASD	Allowable Stress Design
B	Buoyancy
CDBG-DR	Community Development Block Grant – Disaster Recovery
CF	Centrifugal Force
cfs	Cubic Feet Per Second
CWA	Clean Water Act
CY	Cubic Yards
D/DC	Dead Load
DD	Downdrag
DW	Future Wearing Surface
D30	Particle Size Distribution D30
E	Earth Pressure
EH	Horizontal Earth Pressure
EIS	Environmental Impact Statement
ELF	Equivalent Lateral Force
EM	Engineering Manual
EQ	Seismic
ER	East Riser
EV	Vertical Earth Pressure
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FS	Factor of Safety
GIS	Geographical Information System
HUD	United States Department of Housing and Urban Development
H&H	Hydraulics & Hydrology
I/IM	Live Load Impact
LB	Left Bank
LF	Longitudinal Force/Linear Feet
LFD	Load Factor Design
L/LL	Live Load
LS	Live Load Surcharge
MHW	Mean High Water

MLW	Mean Low Water
MSE	Mechanically Stabilized Earth
MTPD	Meadowlands Transportation Planning District
MUTCD	Manual of Uniform Traffic Control Devices
NAVFAC	Navy Facilities Engineering Command
NCHRP	National Cooperative Highway Research Program
NJAC	New Jersey Administrative Code
NEPA	National Environmental Policy Act
NJDCA	New Jersey Department of Community Affairs
NJDEP	New Jersey Department of Environmental Protection
NJDOT	New Jersey Department of Transportation
NJDOT BSDM	New Jersey Department of Transportation Design Manual for Bridges and Structures
NJSEA	New Jersey Sports and Exposition Authority
NJ Transit	New Jersey Transit Corporation
NRCS	National Resources Conservation Service
NS	Norfolk Southern Railway
O&M	Operation and Maintenance
OHW	Ordinary High Water
OSHA	Occupational Safety and Health Administration
PCF	Pound per cubic foot
PSE&G	Public Service Electric and Gas Co.
PSF	Pound per square foot
RB	Right Bank
RBD/RBDM	Rebuild by Design Meadowlands
ROW	Right-of-Way
SPT	Standard Penetration Test
.	
TMP	Traffic Management Plan
USDA	United States Department of Agriculture
USEPA	United States Environmental Protection Agency
USACE	United States Army Corps of Engineers
WP	Water Pressure

1. INTRODUCTION

The Rebuild by Design (RBD) competition, initiated by the Hurricane Sandy Rebuilding Task Force in 2013, was created to develop solutions that would improve the physical, ecological, and economic resilience of regions affected by Hurricane Sandy. The RBD effort, funded in part through Community Development Block Grant – Disaster Recovery (CDBG-DR) funds allocated by the United States Department of Housing and Urban Development (HUD) under the Federal Sandy Supplemental legislation, supported multi-disciplinary teams to propose innovative regional and community-based projects promoting flood resiliency.

The RBD Meadowlands (RBDM) project, developed by the New Jersey Department of Environmental Protection (NJDEP) under the HUD grant, focuses on flood protection in the Hackensack River and Passaic River watersheds in northeastern New Jersey. Funding constraints and project priorities necessitate that implementation of proposed improvements be evaluated for consideration as actionable elements within the broader Hackensack Harbor and tributary study.

The subject of this feasibility sub-appendix is the East Riser (ER) Channel Improvements, located within the Boroughs of Moonachie and Carlstadt, New Jersey. The proposed channel improvements extend from the south face of the Moonachie Avenue Bridge over the ER and continue downstream in a generally southerly direction for approximately 4,150 feet, terminating near the ER Pump Station in Carlstadt, just north of Starke Road.

This appendix provides a condensed summary of the key project elements and engineering design previously developed by the State of New Jersey as part of the 100% detailed design effort. The information presented herein is intended to support feasibility-level evaluation and planning for potential implementation and does not represent final construction documentation. Detailed design criteria, engineering analyses, and final construction documentation are provided in the full 100% design package, which is not included in this public release.

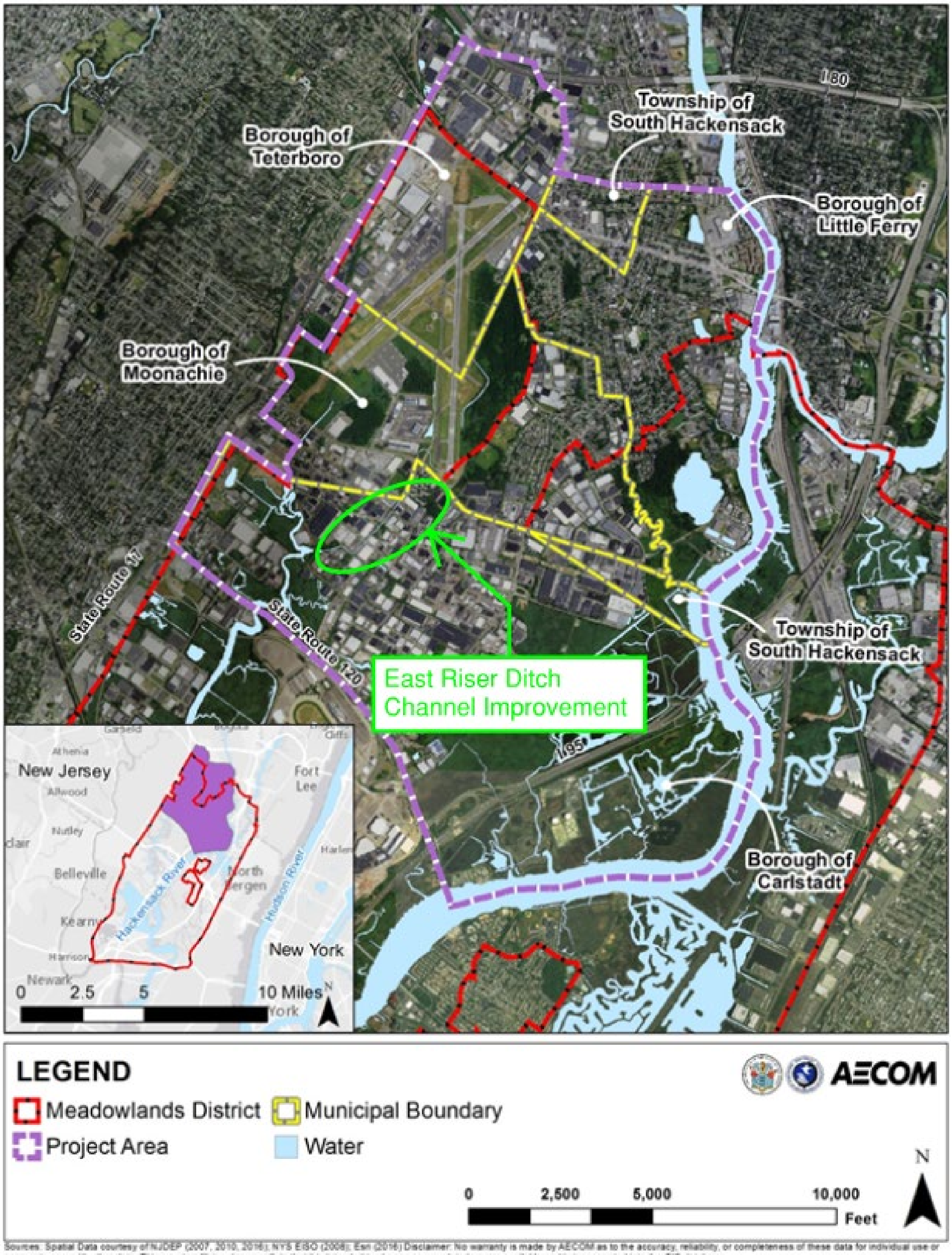


Figure 1-1. Project Area Map East Riser Channel Improvement

2. PROJECT DESCRIPTION

2.1. Project Information

Flooding in the ER basin can result from either coastal storm surges via Berry's Creek or rainfall runoff that drains to Berry's Creek but exceeds the capacity of the existing drainage system. The basin is partially protected from storm surges by existing high ground, constructed berms, tide gates, and pump stations. These existing features provide relief from tidal surges during high frequency storm events (e.g., a 20% to 50% Annual Exceedance Probability (AEP) coastal storm event), but for higher surge levels, large and low-lying portions of the inland area become inundated causing extensive property damages and risks to life-safety.

Because the area of East and West Riser are so flat and intensely developed, the current discharge exceeds the capacity of the bridges and channels. The channel capacity has also been limited by tidal backwater at the tide gate. That source of flooding will be mitigated by the construction of a 500 cfs pump station that will provide fluvial discharge even when tides or storm tides prevent discharge through the tide gates. NJDEP's RBDM study recommended the channel modifications of ER and the pump station as one project. The intent was for both the channel modifications and pump station to work as a system. NJDEP is implementing the pump station, and it is expected to be completed prior to end of construction for the ER Actionable Element. Without the channel modifications, the pump station would not provide flood risk management benefits at the intended level of performance. Consequently, it is likely that any reductions in water surface elevation provided by the pump station alone would be limited. To capture the true impact of the project, as agreed upon by USACE leadership and non-Federal sponsors and partners, the pump station and channel modifications are being treated as a system for the purposes of reporting project benefits. The with-project scenario includes both the channel modifications and the pump station.

Constraints on flow are related to limited hydraulic capacity at an undersized rail crossing, the undersized culvert at Commercial Ave, and a general lack of channel capacity. Upstream of Moonachie Ave (the upstream limit of the proposed channel improvements), extensive flooding occurs within Teterboro Airport. Much of that flooding is due to the lack of capacity at a long culvert on ER that runs underneath one of the primary runways. The result is extensive backwater flooding extending north on ER. A significant amount of the ER flow overtops the runway, with a portion of that flow eventually joining West Riser. The area between the two runways is subject to flooding from a combination of flows overtopping from both the East and West Riser.

Under future modelling scenarios, the frequency of inland inundation will increase as Relative Sea Level Change (RSLC) impacts the frequency and duration that discharge through the tide gates at East and West Riser are blocked by the tides and minor storm surge. Because the RBDM Project envisioned that improvements in the ER basin would ultimately be paired with Coastal Storm Risk Management (CSRM) project features, ER pump station and channel improvements were defined as the "Build Alternative" of a selected plan that included local Coastal Storm Risk Management measures. The fluvial flood risk management measures in for ER are considered a critical component of an overall plan since construction of the CSRM features would not be effective if the area was subject to continued fluvial flooding.

The Alternative 3 Build Plan includes measures to improve stormwater conveyance in the lower portions of the ER and Losen Slote, as well as open space and green infrastructure throughout the drainage areas of those streams. The improvements in the Alternative 3 Build Plan include increasing the channel conveyance capacity, ecological restoration of the channel banks, and installation of a 500 cfs pump station upstream of the ER tide gate. This document focuses on the ER channel improvements. The ER pump station and Energy Dissipater, Losen Slote pump station and force main, are addressed under separate contracts. Figure 2-1 provides a conceptual depiction of the ER project layout. Plan sheets at the 100% design level (100% Plans) are provided under separate cover.

The ER channel improvement portion of the Alternative 3 Build Plan includes 4,150-ft of dredging and channel excavation from the tide gate at Starke Road on the southern end of the project to the Moonachie Avenue bridge/culvert on the northern end of the project. These channel improvements are located within the boroughs of Moonachie and Carlstadt. The channel improvements are being constructed to increase flow conveyance capacity of the ER. In addition, through removal of accumulated sediments that have been impacted by contaminants from point and non-point sources, and incorporation of stormwater management into the design, the channel improvements will also improve water quality. Channel boundaries and adjacent areas falling within the riparian zone will be re-vegetated with native plant species consistent with that habitat type in the Project Area as shown in Figure 1-1. As identified in the Environmental Impact Statement, the ER area includes habitat types such as floodplain/riparian forest remnants, herbaceous/emergent and scrub/shrub deciduous. Example species from these habitats include Sweet pepperbush *(Clethra alnifolia),* Sweetgum *(Liquidambar styraciflua),* Black birch *(Betula lenta)*, and Seaside Goldenrod (*Solidago semervirens*). The vegetation will slow stormwater runoff, provide nutrient uptake and shade the stream, thus contributing to water quality improvements. In addition, to further improve water conveyance in the ER, two existing road bridge- culvert crossings and a railroad bridge will be removed and replaced to affect a larger flow area.

2.2. Feasibility Report Appendix Purpose

The 100% Feasibility Report Appendix provides a consolidated summary of design development completed for this phase of the project as developed in coordination with NJDEP.

The results contained in this report are based on the best available data at the time of publishing of this document, including field investigations, modeling efforts, and engineering evaluations.

2.3. Alternatives Assessment

The development of the East Riser plan considered a wide range of alternatives as documented in Chapter 7 of the Rebuild By Design Meadowlands Flood Protection Project, Final Feasibility Study, June 2021. Alternatives included both green and gray infrastructure for the area including Losen Slote, and the East and West Riser channels. For Losen Slote, the selected plan pump station and force main are currently under construction. It was determined that channel improvements and pump stations at West Riser would not be cost effective and were dropped from further consideration. Green infrastructure features were identified for several areas but are more suitable for implementation under other programs.

The East Riser channel, bridge, and pump station plan was identified as a potentially cost-effective plan to manage flood risks. The channel improvement limits were selected to reduce flood risks for the area downstream of the airport. The size and capacity of the channel improvements were limited due to nearby commercial and industrial development and the need to provide accessibility for maintenance. The larger plans were not considered implementable due to the availability of land. For the channel improvement to be effective during high tides that block the tide gates, a pump station is an essential feature for plan completeness. Because the channels and bridges result in higher peak flows reaching the tide gate, the pump station was sized to accommodate the increased flow without causing induced flooding. Given that spatial constraints governed the development of alternatives, evaluation of larger or smaller alternatives was not considered necessary as part of the NY NJ Harbor and Tributaries Study.

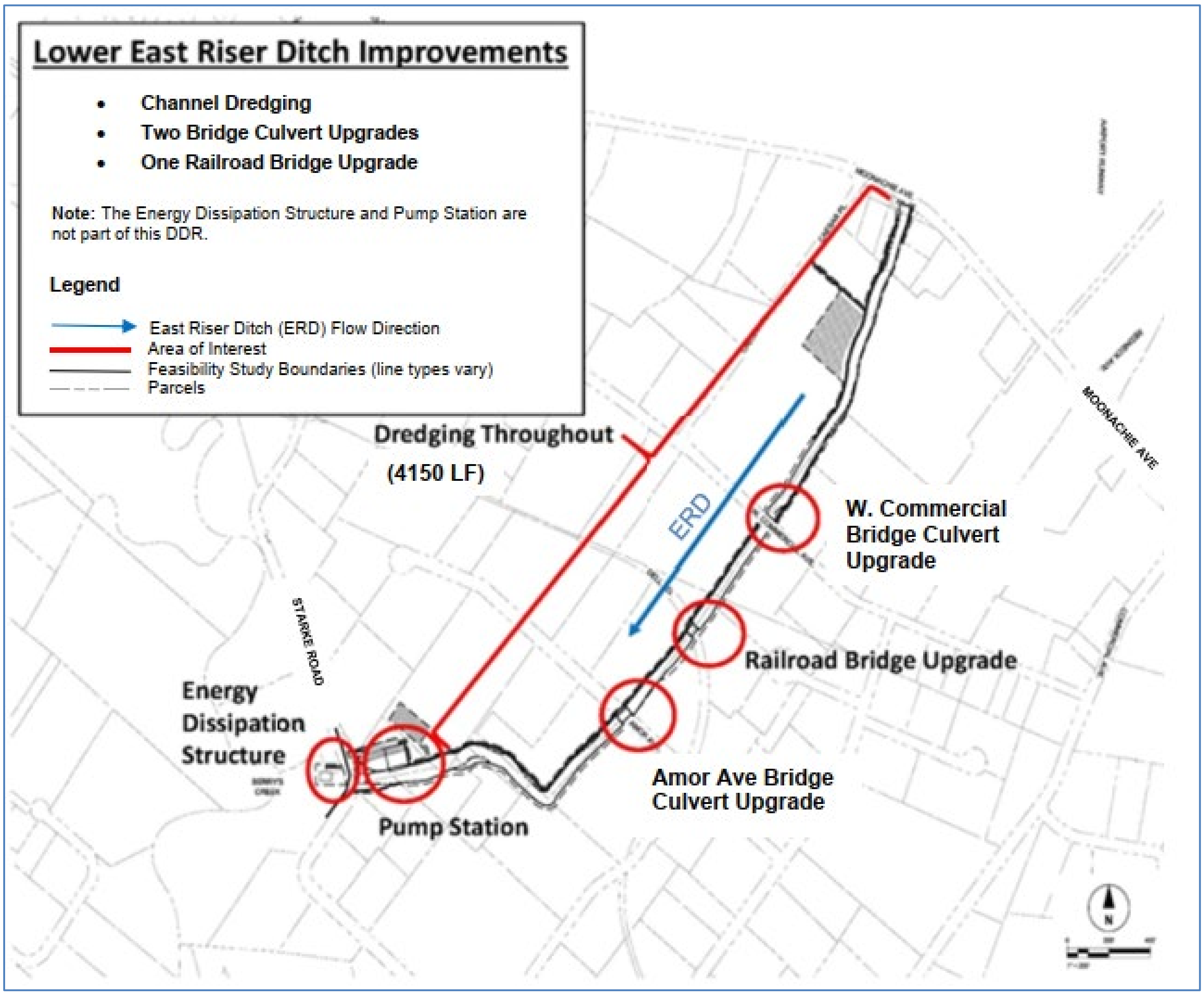


Figure 2-1. Project Site Map

3. REFERENCES

3.1. Project Reports & Design Related Documents

The following project reports and design related documents were utilized in developing this deliverable:

- Draft Feasibility Study for Rebuild by Design Meadowlands Flood Protection Project (NJDEP, April 2018).
- Draft East Riser Preliminary Design Report (AECOM/HDR, May 2019).
- Feasibility Assessment of Earth Levee, Double Sheet Pile Wall and Flood Wall New Meadowlands Flood Protection, Bergen County, New Jersey (AECOM, February 2017). This contains the subsurface investigation report developed during Feasibility.
- Final Environmental Impact Statement for the Rebuild by Design Meadowlands Flood Protection Project, October 2018.
- Record of Decision for the Rebuild By Design Meadowlands Flood Protection Project, December 2018.
- Draft Geotechnical Investigation Results Report, dated 11 October 2019, prepared by AECOM.
- Draft East Riser Topographic Survey, received by HDR 30 August 2019, prepared by Matrix New World for AECOM
- Materials Management Plan, dated June 2, 2020, prepared by HDR.
- Model Build, Calibration and Validation Report, dated September 2019, prepared by HDR.

3.2. Applicable Design Criteria

The following codes and standards are applicable to this project:

Hydrology & Hydraulics

- N.J.A.C 7:8 – Stormwater Management Rules
- N.J.A.C 7:13 – Flood Hazard Area Control Act Rules
- N.J.A.C. 7:20 Dam Safety Standards
- 44 CFR Parts 59 and 60 – NFIP Regulations
- Hackensack Meadowlands Floodplain Management Plan (NJSEA, 2016)
- “Hackensack Meadowlands District Regulations" N.J.A.C 19.3 et seq. Current through August 19, 2019
- Carlstadt Office of Emergency Management local floodplain development regulations
- Moonachie Code of Ordinances Flood Damage Prevention Chapter 19 Flood Damage Prevention
  - Sections 19-5 (Provisions for Flood Hazard Reduction)

- Section 19-6 (Stormwater Control)
- U.S. Army Corps of Engineers guidance
- ER-1110-2-1150 Engineering and Design of Civil Works
- EM 1110-2-1901 Hydraulic Design of Flood Control Channel
- EM 1110-2-2909 Conduits, Culverts and Pipes
- NJDOT Roadway Design Manual – Section 10 Drainage (2015)

Structural

- AASHTO LRFD *Bridge Design Specifications*, 8th Edition with Interim Revisions
- AREMA *Manual for Railway Engineering*, 2020 Edition
- NJDOT *Design Manual for Bridges and Structures*, 6th Edition
- NJ TRANSIT *General Requirement for Working Within the Right-of-way*
- NJ TRANSIT *MW-4 Manual for Construction, Maintenance, and Inspection of Track*, 2014
- Norfolk Southern *Public Projects Manual*, March 2021

Civil

- Channel Improvements
- The development of the proposed channel improvements involves property acquisition, potential permitting requirements impacting alignment of the channel, and mitigation of regulated areas along the stream. It also depends on coordination with utility owners and the following requirements related to the temporary or permanent removal, replacement or relocation of utilities. These utilities include sanitary sewer, overhead and underground electric and communications, natural gas, stormwater, potable water, etc. (Refer to Section 4.1.2 for more information related to design guidance associated with utilities).
- Roadway Design
- Asphalt Pavement Design Guide, 6th Edition (2019), Prepared by the New Jersey Asphalt Pavement Association in cooperation with New Jersey Department of Transportation.
- Traffic Management:
- Bergen County Code
- New Jersey Department of Transportation (NJDOT)
- New Jersey Sports and Exposition Authority (NJSEA), such as those required by the Meadowlands Transportation Planning District (MTPD) and detailed in the Hackensack Meadowlands District Regulations (N.J.A.C. 19.4-7.10 – Performance Standards, Traffic)
- Federal Highway Administration (FHWA) proposed road surface loading design requirements
- Manual of Uniform Traffic Control Devices (MUTCD).

- Site Controls:
  - Occupational Safety and Health Administration (OSHA) standards for the Construction Industry (29 CFR Part 1926), including regulations associated with construction noise, vibration, air quality (particulates and vehicle exhaust fumes), trenching and shoring, personnel safety and public safety
  - New Jersey Department of Environmental Protection (NJDEP) Soil Erosion and Sediment Controls, soil and material stockpiling requirements; New Jersey Administrative Code (NJAC) Chapter 7 Section 8, Stormwater Management Rules
  - The Standards for Soil Erosion and Sediment Control in New Jersey dated July 2017
  - Bergen County Soil Conservation District reviews and certifies Soil Erosion and Sediment Control Plans as mandated by the New Jersey Soil Erosion and Sediment Control Act, Chapter 251, P. L. 1975
  - NJSEA (N.J.A.C. 19:4) and/or Borough of Moonachie and Carlstadt, NJ requirements during construction activities associated with stormwater management, backfill material criteria, easement acquisitions
  - NJSEA (N.J.A.C. 19:4-7.3 Performance Standards: noise) and/or, Borough of Carlstadt and Moonachie municipal ordinances and codes associated with acceptable working hours, noise limits (Carlstadt Municipal Code 5-10 Regulation of Noise)
  - Work in proximity to Teterboro Airport – applicable regulations (e.g., landscaping near airports [attachment LS1]) as detained in the Tenant Construction Review Manual Port Authority of New York and New Jersey (December 2015).
- Materials Management:
  - United States Environmental Protection Agency (USEPA) for hazardous materials sampling, management, and disposal
- Federal Clean Water Act (CWA), specifically Sections 401 and 404, which address water quality and fill, respectively, as well as protection of biological and cultural resources.
- NJ Flood Hazard Area Control Act (N.J.A.C. 7:13), which regulates work in flood-prone areas, including the East Riser channel and adjacent riparian zone and require protection of biological resources. These regulations also place restrictions on storage of material in the flood hazard area.
- NJ Coastal Zone Management Rules (N.J.A.C. 7:7), demonstration of compliance with applicable Coastal Policies is necessary to obtain the CWA 401 Hackensack Meadowlands Water Quality Certificate for where the project is located. These rules require protection of biological and cultural resources and appropriate land use in the coastal zone.

Geotechnical

- AASHTO (2012). “AASHTO LRFD Bridge Design Specifications.” American Association of State Highway and Transportation Officials. 8th Edition, Published September 2017.
- APILE (2018), “APILE Version 2018.8.5-Offshore Technical Manual”, developed by Ensoft, Inc., 2018.

- AREMA (2017). “AREMA Manual for Railway Engineering,” American Railway Engineering and Maintenance of Way Association, dated 2017.
- ASCE (2017), “Minimum Design Loads for Buildings and Associated Criteria for Buildings and Other Structures”, (ASCE/SEI 7-16), prepared by the American Society of Civil Engineers, dated 2017
- Das (2006), “Principals of Geotechnical Engineering”, Braja M. Das, 6th Cengage Learning, 2006.
- Das (1997), Das, B.M. “Advanced Soil Mechanics, 2nd Edition”, 1997.
- EM 1110-2-1902, Slope Stability, 2003.
- EM 1110-2-2906, Design of Pile Foundations. 15 Jan 1991.
- EM 1110-2-6053, Earthquake Design and Evaluation of Concrete Hydraulic Structures, 2007.
- International Code Council, Inc. International Building Code, New Jersey Edition. 2015.
- JGAGE (2006), “Pile Spacing Effects on Lateral Pile Group Behavior”, prepared for The American Society of Civil Engineers Journal of Geotechnical and Geoenvironmental Engineering, dated October 2006.
- LPILE (2019), “LPILE v2019 User’s Manual”, developed by Ensoft, Inc., 2019.
- Meyerhof (1974), Meyerhof, G.G. “Ultimate Bearing Capacity of Footings on Sand Layer Overlying Clay,” Canadian Geotech. J., Vol. II, No. 2.
- National Engineering Handbook Part 654, Stream Restoration Design.
- NAVFAC (1986). Design Manual 7.1 and 7.2, Soil Mechanics and Foundation and Earth Structures. NJDOT (2016). “Design Manual for Bridges and Structures.” New Jersey Department of Transportation. Sixth Edition. Dated 2016.
- The Standards for Soil Erosion and Sediment Control in New Jersey dated July 2017.
- Tomlinson, M.J. (1987). “Foundation Design and Construction, Fifth Edition,” M.J. Tomlinson, Logman Scientific and Technical, London.
- Tomlinson, M.J. (1987). “Pile Design and Construction Practice,” Viewpoint Publication.
- USGS (2018), “Unified Hazard Tool”. United States Geological Survey, Earthquake Hazard Program. https://earthquake.usgs.gov/hazards/interactive/ (Accessed 6-14-2019).

Geomorphological

- USACE, 1994. Engineer Manual EM 1110-2-1601 Hydraulic Design of Flood Control Channels. US Army Corps of Engineers, Washington, DC.
- USDA NRCS, 2007. Part 654 National Engineering Handbook: Stream Restoration Design.

USDA NRCS Part 650, 1996. Engineering Field Handbook. Chapter 16: Streambank and Shoreline Protection

- Yochum, Steven E. 2018. Guidance for Stream Restoration. U.S. Department of Agriculture, Forest Service, National Stream & Aquatic Ecology Center, Technical Note TN-102.4. Fort Collins, CO.
- The Standards for Soil Erosion and Sediment Control in New Jersey, (7th Edition, January 2014, Revised July 2017).
- National Cooperative Highway Research Program Report 108: Tentative Design Procedure for Riprap-Lined Channels (1970). National Cooperative Highway Research Program Report 568: Riprap Design
- Criteria, Recommended Specifications, and Quality Control (2006). U.S. Department of Transportation Federal Highway Administration Hydraulic Engineering Circular 14: Hydraulic Design of Energy Dissipators for Culverts and Channels, 3rd ed. (July 2006).
- U.S. Department of Transportation Federal Highway Administration Hydraulic Engineering Circular 18: Evaluating Scour at Bridges, 5th ed. (April 2012).

Landscape Architecture and Urban Design

- FAA AC No 150/5200-33B Hazardous Wildlife Attractants on or Near Airports
- Teterboro Airport Wildlife Hazard Management Plan
- Port Authority of New York and New Jersey
- Bergen County Audubon Society –Native Plant List

Construction

- NJ Stormwater Management Rules, runoff quality standards (includes 50 ft offset from top of bank, Minimum 25 ft setback for new structures and limits on clearing of vegetation
- Soil Erosion and Sediment Control Plans (SESC)
- NJ Tidelands Law
- NJ Water Pollution Control Act
- NJ Site Remediation Reform Act (SRRA)
- Carlstadt Street Opening Permit and Traffic Permit
- Moonachie Street Opening Permit and Traffic Permit
- Bergen County Street Opening and Traffic Permit

3.3. Computer Programs

The following computer programs will be utilized to complete the engineering evaluations that support the project final design:

Hydrology and Hydraulics

- HEC-RAS 5.0.6
- HEC-HMS

Geotechnical

- Apile v2019
- GRLWEAP (2010)
- LPILE v2019
- Geostudio 2019- Slope/W
- Shoring Suite Version 8.12h created by CivilTech Software
- PYWall 2022

Geomorphology

- RIVERMorph 5.2

Structures

- LARSA 4D Version 9.07 r02
- RISA Foundation

4. DESIGN CRITERIA

4.1. Design Criteria – General

The following sections define the design criteria that will govern completion of the design of all project components. Design criteria for this project fall generally into five categories:

1. **Loading conditions**: those physical conditions – i.e., flows, weights, forces, moments, ground accelerations – that designed project components must withstand and maintain functionality throughout.
2. **Performance metrics**: quantities that reflect a project components response to loads – i.e., a stability factor of safety or a water surface elevation in a channel at a given flow rate.
3. **Configuration requirements**: system component requirements – i.e., pipeline release valves, sleeves for conduits, filter fabrics, structural reinforcement – and geometric requirements – i.e., spacing, offsets, slopes, and minimum cover.
4. **Material types**: material requirements – i.e., types of pipe material, concrete and steel, fill, and geotextiles.
5. **Operational controls**: Temporary activities during construction and permanent activities after construction that must conform to local, state, and federal requirements – i.e., erosion control measures during construction and operation and maintenance measures after construction.

Design criteria are presented below, by discipline, as they relate to design components. Each of the five categories listed above are not necessarily a part of each set of criteria. For example, the geotechnical design criteria include loading conditions, performance metrics, configuration requirements, and material types, but not operational controls. Demonstration that the project components are designed in a manner that satisfies the applicable design criteria is discussed in this report.

4.1.1. Civil – Channel Improvements

The ER channel improvements require evaluation of various factors to meet the project hydraulic conveyance and ecological restoration goals. The design recommendations contained in this document reflect the best available information regarding the following constraints. Refinements will be considered as new information becomes available.

Design constraints:

1. Available Easements/Real Estate
2. Operation and Maintenance Access Requirements
3. Allowable Channel Over Excavation
4. Offsets from Buildings
5. Bridge Scour Countermeasures
6. Constructability
7. Channel Bank Stability
8. Hydraulic Conveyance Capacity
9. Channel Ecological Restoration Requirements in EIS

Figure 4-1 illustrates the temporary easement, overcut, and side slope within a Typical Section. Required overcut and maximum side slopes were evaluated with geotechnical analyses during design, as described further in Section 4.1.4.

In addition to the factors listed above, there are various authorities and agencies with jurisdictions whose requirements will be accounted for in project design and construction. See Table 4-1.

Table 4-1. Authorities and Agencies with Jurisdiction

Authority	Agency
Traffic	New Jersey Dept. of Transportation (NJDOT)
	Bergen County
	NJSEA
	Borough of Carlstadt
	Borough of Moonachie
Materials Management	New Jersey Dept. of Environmental Protection (NJDEP)
	United States Environmental Protection Agency
Utility Conflict Identification & Management During Construction	PSE&G
	Veolia
	Communications (Verizon, T-Mobile, Zayo, AT&T, Charter, Comcast)
	Municipally owned sanitary and storm sewers
	New Jersey Sports and Exposition Authority
	NJDOT
	Bergen County
	NJ Transit
Flood Hazard Area and Riparian Zone	NJDEP
	Borough of Carlstadt
	Borough of Moonachie
	FEMA
	NJSEA
Stormwater Management	NJDEP
	NJSEA
	Bergen County Soil Conservation District
	Borough of Carlstadt
	Borough of Moonachie
Surface Water, Wetlands and Water Quality	USACE
	NJDEP
	NJSEA

1. Minimum and Maximum Overcut is the Available Space for Channel Over-Excavation to Construct the Engineered Stabilization Measures.
2. Refer to the 100% Plans (Sheets CT-601 through CT-607) for the Latest Typical Sections.

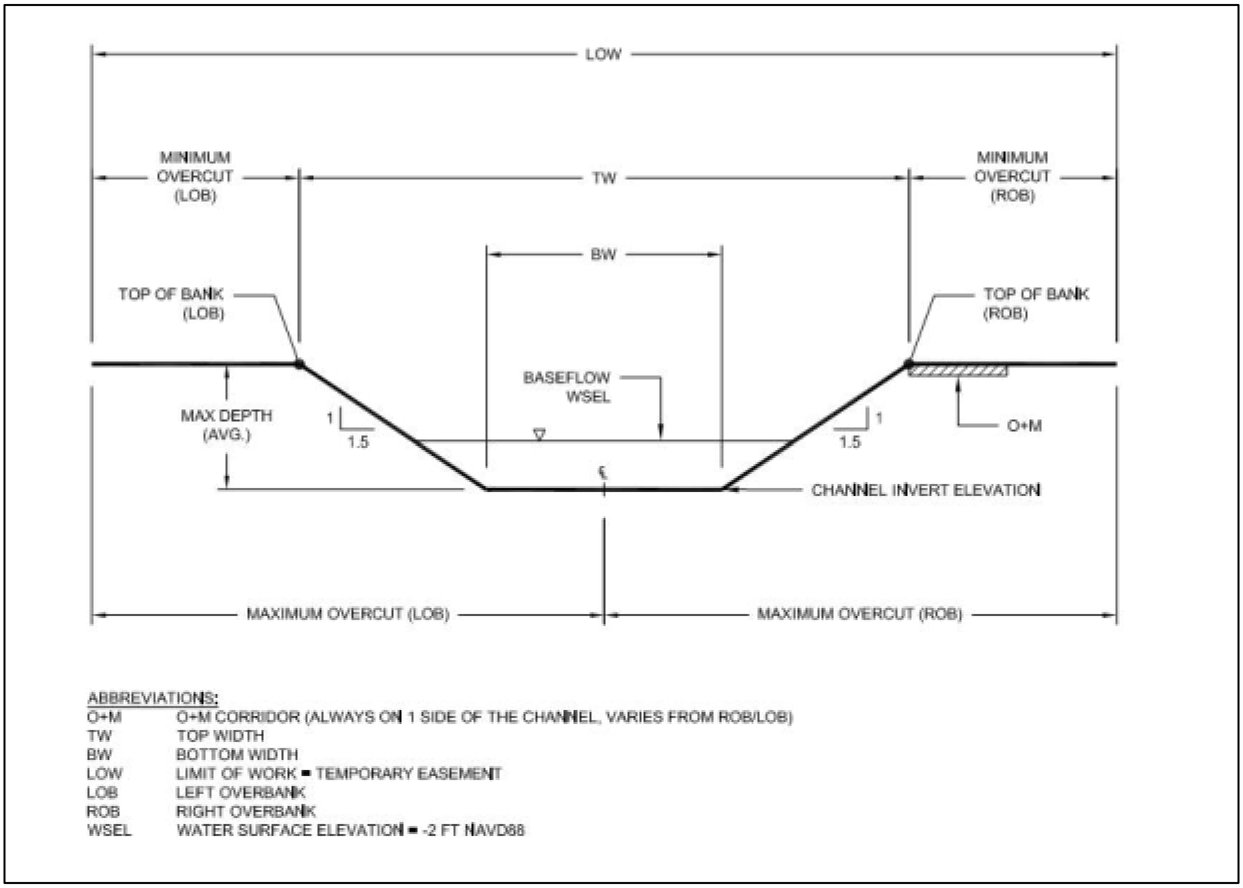


Figure 4-1. Typical Channel Section Looking Downstream

4.1.1.1. Road Work & Traffic Management

The removal of culverts at Amor Ave and West Commercial Ave and their replacement with bridges and related channel improvements are regulated by NJDOT, NJDEP, Bergen County, the Borough of Moonachie, the Borough of Carlstadt, and other affected municipalities. The permanent and temporary relocation of gas and electric to facilitate this work are regulated by PSE&G Gas and PSE&G Electric, respectively. Other utility tenants having overhead wires on the existing poles will coordinate with PSE&G Electric. Veolia will regulate relocation of the watermain. Carlstadt Sewerage Authority and Borough of Moonachie will regulate relocation of storm and sanitary sewers.

A portion of the project will be implemented within the public right-of-way (ROW). However, the majority will occur on private property. Temporary roadway crossings are not anticipated and, therefore, detours will be needed for the culvert removals and their respective bridge construction. The culvert 'remove and replacement' with bridge actions will require providing temporary access to businesses and potentially obtaining access agreements with private property owners during construction. In addition to the culvert removals and replacements with bridges, the movement of materials and equipment during construction will have the largest impact on access to private businesses on both sides of the channel. Flaggers, cones, and signs will be implemented to continue the flow of traffic to private businesses on either approach to the new bridge crossing the channel during construction.

The entire excavation area falls within the jurisdictional boundary of the Boroughs of Moonachie and Carlstadt. Moonachie has adopted the current Manual of Uniform Traffic Control Devices (MUTCD). Per Section 7-7 of the Borough's local ordinances, a pre-construction meeting must be held with the local police department. Road opening permits, review of traffic management plans, and detour routes will likely be required. Coordination with the municipalities and the NJSEA MDTP will occur in conjunction with, but be separate from, the 100% design. Pre-construction meetings will be held with the municipal police departments after the 100% submission, but prior to construction, to discuss police traffic director needs and requirements, road closures, and detour routes, among others. The Borough of Carlstadt has provided direction regarding design criteria for restoration at the Amor Ave crossing. Per their direction, the NJDOT Standard Specification for Road and Bridge Construction and NJDOT Standard Drawings should be complied with.

The construction schedule will be developed to permit work during the hours of 09:00 through 16:00 in accordance with Moonachie's ordinances unless otherwise approved. The same limitations will be assumed for working in Carlstadt. No construction materials, vehicles or equipment will be placed in the roadway or sidewalk within the Borough outside of the project limits, as shown on the 100% Plans. Preliminary road closure and detour plans have been developed for the two culverts remove and replacements with bridges and these plans will be further refined during subsequent phases of design in accordance with Moonachie, Carlstadt, and NJDOT's requirements. The Contractor will be required to develop an Implementation Plan for the Maintenance and Protection of Traffic.

Sections of existing roadways which are disturbed or removed during construction will be replaced in accordance with Borough and NJDOT requirements, as applicable. Temporary easements for truck hauling routes and proposed permanent easements for O&M access will be modified or designed with surfaces rated for heavy vehicle usage.

Traffic will be managed during all phases of construction, including material delivery, spoils haul-off, and channel excavation, dredging, and installation of the three remove/replace bridge/culvert structures. The channel improvements are expected to take approximately 2 to 3 years.

4.1.1.2. Roadway Geometry

The Amor Ave public ROW is 60-ft wide. The curb-to-curb width shall be 40-ft, including 12-ft travel lanes and 8-ft shoulders in the eastbound and westbound direction. A minimum of 4-ft to a maximum of 5-ft width sidewalks will be provided on both the north and south sides of the roadway such that the bridge parapet is set back 5-ft from the curb line. This will improve the clear zone of the roadway to approximately 13-ft for a low speed and low traffic roadway per AASHTO Roadside Design Guide. There are no existing sidewalks on either side of Amor Ave. There are existing driveways on the west side of the channel on Amor Ave. Due to channel widening, the west side driveways will be reduced in width from approximately 31-ft to a minimum of 20-ft on the north side of Amor Ave and from approximately 84-ft to 62-ft on the south side. The minimum 20-ft wide driveway on the north side will still accommodate a two-way driveway for the private property on the north side. The Operation and Maintenance (O&M) roadway will connect to driveways on the west side of the channel. The minimum 20-ft wide driveway will still accommodate a one-way maintenance truck access. The driveway on the north side of Amor Ave, west of the channel, will be replaced with HMA driveway Type B and the driveway on the south side will be replaced with HMA driveway Type A per NJDOT standard drawing CD-605-5. There is one driveway on the east side of the channel on the south side of Amor Ave that will not be reduced in width and will be partially replaced. This driveway will be replaced with HMA driveway Type B per NJDOT standard drawing CD-606-5. There is no O&M Roadway on the east side of the channel. Per the Borough of Carlstadt's direction, all work on the approach roadway to the new bridge, including but not limited to, the pavement, curb, sidewalk, and driveway material, is to be designed in accordance with NJDOT standard specs for road and bridge construction and NJDOT standard drawings. The area of the bridge deck overhang on the north side of the roadway beyond the 30-ft ROW from the centerline of the roadway will require a permanent easement for the watermain.

The West Commercial Ave public ROW is 50-ft wide. The curb-to-curb width shall be 40-ft, including 12-ft travel lanes and 8-ft shoulders in the eastbound and westbound direction. A minimum of 4-ft to a maximum of 5-ft sidewalks will be provided on both north and south sides of the roadway such that the bridge parapet is set back 5-ft from the curb line. This will improve the clear zone of the roadway to approximately 13-ft for a low speed and low traffic roadway per the AASHTO Roadside Design Guide. The new sidewalk on the bridge will cause the existing sidewalk on both sides of West Commercial Ave to be continuous. There are existing driveways on both sides of the channel on the north and south sides of West Commercial Ave. There are no reductions in width of existing driveways on the west side of the channel and driveways will be partially replaced within the limit of disturbance with HMA driveways per NJDOT standard drawing CD- 606-5 Type B. The O&M roadway will not have connections to the driveways on the west side of the channel. There are concrete driveways on the east side of the channel on both north and south sides of West Commercial Ave. They will be slightly reduced due to channel widening however an approximately 23-ft wide driveway can still be maintained for a two-way use by the private property on the south side and 30-ft wide driveway on the north side. The O&M roadway will only connect to the north side driveway. The concrete driveways will be replaced with concrete per NJDOT standard drawing CD-605-5 Type E. The Borough of Moonachie's road work requirements will be adhered to for the roadway pavement, curb, sidewalk, and driveway material. The NJDOT standard specs and standards will be used if no information is provided by the Borough. The area of bridge deck overhang on the north side of the roadway beyond the 25-ft ROW from the centerline of the roadway, to place the watermain, will require a permanent easement. The area of the bridge including the south parapet and behind the south parapet beyond the 25-ft ROW from the centerline of roadway, to mount the gas to the south parapet, will also require a permanent easement.

4.1.1.3. Staging

The construction limit, or limit of work, is defined as the horizontal extents of construction activities, including areas for staging material and equipment. Staging areas are necessary for construction and are part of the temporary project footprint. In accordance with the project scope of work construction staging areas will be required to be defined in a future "Site Logistics Plan" to be produced by the successful bidding Contractor. The following passage is cited from the Engineering Design Scope of Work:

> *"Due to urban, dense nature of the Project Area, any locations identified for staging, borrow, and stockpiling areas, as well as construction Contractor's trailers, support areas, decontamination facilities, restricted zones, construction entrances and temporary access roads may not be available or conditions may change between the time of bidding and the notice to proceed for construction. Therefore, instead of identifying areas or potential areas for these items, the specifications will include a section requiring the Contractor to*

develop and implement a Site Logistics Plan for review and approval prior to the start of heavy construction. The plan will not only cover staging, access and restricted areas, but will also include site security, safety and emergency…."

The future staging areas to be determined by the Contractor should be easily accessible, adaptable to the local traffic constraints, occupy substantial open space (field or parking lot), be located in the vicinity of the project and connect to a public road to allow construction vehicles to/from the site, and be adequately spaced along the project to avoid congestion, while limiting disturbance to nearby businesses, residences, and local traffic as much as possible. Material to be transported and managed includes both in-situ and imported material. It is assumed that all clean material and impacted material will be stored separately to avoid potential cross-contamination.

The staging areas to be selected by the Contractor shall be sized to adequately hold the equipment and material, of types and quantities necessary to ensure timely and safe construction activity. The duration of use of each of these staging areas will be left up to the Contractor to determine at the approval of the NJDEP as part of the future Site Logistics Plan. Locations may be evaluated by the Contractor to ensure the areas meet all the functional needs of the project, including jurisdictional and agency requirements.

4.1.1.4. Materials Management

The management of materials is closely tied to the staging of materials. Material will be moving in and out of the Project Area and temporarily stored within the staging areas. Control of material emissions and discharges will be consistent with the methods described Section 4.1.1.5. Excavated soils will be tested, handled and disposed of in a manner consistent with local, state, and federal regulations.

4.1.1.5. Site Access/Egress

The proposed footprints will be accessed during construction to build the project features and will also be accessed to provide periodic, long-term inspection and maintenance. Access to the temporary (construction) and permanent (operational) footprints will be from the following public roads: Grand Street/Starke Road, Amor Avenue, West Commercial Avenue, and Moonachie Avenue or others as deemed necessary during the Contractor's preparation of the Site Logistics Plan.

4.1.1.6. Project Site Real Estate

Most of the ER channel alignment crosses over privately owned lands with the few exceptions where it crosses public right-of ways (ROW) at the Starke Road, Amor Avenue, West Commercial Avenue, and Moonachie Avenue. It is understood that the channel portion within the Township of Moonachie may have existing easements, however due to the deepening and widening of the channel, any existing easements may need to be expanded to cover the full width of the improved channel and O&M roadway. As the channel passes through the Township of Carlstadt it is believed that no formal easements exist, it will be necessary to acquire easements for the improved channel and O&M roadway throughout the length of the project in Carlstadt.

There are four types of easements necessary to build the project: The Temporary Construction Easement, Permanent Operations and Maintenance Easement, Permanent Drainage Easement, and Access Area Easement. The easement widths vary along the length of the channel improvements and are shown on the 100% Plans as the extent of the construction limit / limit of work, the extent O&M access corridor, the extent of the overcut, and the extent of access over existing driveways, respectively, to accommodate the footprint of the improved channel, the service maintenance lane, and property access. NJDEP is responsible for acquiring all temporary construction and permanent project easements over these private lands along the length of the project. These efforts are ongoing during time of submittal of the 100% design package.

4.1.1.7. Other Site Controls

Other site controls include delineation and protection of environmental resources, including ecological and biological habitat, wetlands, wetland buffers, Flood Hazard Areas, Stream buffers, and Riparian Zone. Wetlands within the construction limits have been field delineated and wetland boundaries are shown on the existing conditions plans. In addition, a supplemental delineation of a portion of the LPS Industries parcel at 10 Caesar Place in Moonachie was conducted in July 2020. Details on this wetland will be included in the permit application; the wetland boundary is included on the plans. Wetland boundaries and all other regulated areas are shown on the permit plans. Additionally, the following will be addressed during design and construction:

- Erosion and sediment control Best Management Practices
- Noise and vibration control
- Air quality monitoring
- Protection of cultural and historical resources (if applicable)
- Contractor's extent of work areas (temporary easements)
- Protection of any nearby structures (stabilization measures for buildings)
- Utility conflicts, including relocation and protection measures covered in Section 4.1.2
- Coordination with local municipalities and public outreach activities
- Coordination with Teterboro Airport (Port Authority of NY & NJ) and construction activity restrictions, including equipment heights
- Construction will require coordination with various jurisdictions and agencies, as shown in Table 4-1

4.1.2. Civil – Utility Protect In-Pace, Modification, Relocation Design Criteria

The purpose of this section is to describe the design criteria that the utility protection and modifications should conform to. Utility relocation and removal and replacement will be coordinated with the owners, including their standard drawings and specifications. During project development, jurisdictional controlling design criteria will be adhered to such as state, county, municipal and other local regulations.

Sanitary Sewer:

- Carlstadt Sewerage Authority (CSA) Rules and Regulations Revised May 1998

- Pipe material shall be a 12-in ductile iron pipe per ANSI Standard A21.51 and joints shall be per ANSI standards A21.4. The trench excavation shall be according to Section I of CSA.

4.1.2.1. Sanitary Sewer at Amor Ave Crossing

Per the record drawings, there is a 12-in sanitary sewer line encased in a 24-in steel pipe casing running perpendicular to the channel alignment and crossing under the culvert. The Carlstadt Sewerage Authority design criteria and guidelines were followed for the design of the removal and replacement of the 12-in sewer in 24-in steel casing within the construction limit. A casing is provided for the section of sewer that crosses under the bridge and channel. The casing will be jacked under the channel. The casing design is pile supported and self-supporting between supports on each side of the bridge.

4.1.2.2. Storm Sewer

Existing storm sewers requiring reconstruction are sized and sloped the same as existing pipes. All storm sewers to be reconstructed within private property shall be constructed of ductile iron pipe. Pipes are not upsized as existing drainage areas to outfall pipes will be somewhat reduced due to reduction in impervious areas in some locations resulting from channel widening.

Outfalls were designed per the New Jersey Administrative Code, New Jersey Sports and Exposition Authority Regulations, and New Jersey Standards for Soil Erosion and Sediment Control as noted herein.

New Jersey Administrative Code: Where a drainage system discharges to grade or other body of water, a concrete headwall with wing-walls and a rip-rap apron pad, or other as approved by NJMC shall be constructed. The apron pad and/or scour hole design shall consider the tailwater elevation to be equal to the mean low water (MLW) elevation if tidally influenced. Where maximum velocities exceed the allowable velocities for soil stability, as determined in the Standards for Soil Erosion and Sediment Control, scour control is provided.

New Jersey Sports and Exposition Authority: Any comments or design requirements will be addressed as needed. New Jersey Standards for Soil Erosion and Sediment Control: The thickness of the riprap lining at the outlet meets the requirements of this standard. The allowable velocity shall be checked with the appropriate table in section 12 of this standard.

Borough of Carlstadt: Any comments or design requirements will be addressed as needed.

Borough of Moonachie: Any comments or design requirements will be addressed as needed.

Stormwater structures (inlets, manholes, wingwalls, and headwalls) were designed per the New Jersey Department of Transportation Standard Specifications and Details.

4.1.2.3. Water

The Veolia Water New Jersey design criteria were applied for the temporary and permanent work listed herein.

Water Main at Amor Ave Crossing: Per the record drawings, there is an 8-in water main running perpendicular to the channel alignment and crossing over the existing culvert. The Veolia Water design criteria and guidelines have been incorporated into the design per the comments received for the water main relocation design**.** Veolia submitted their comments after review of 95% design.

Water Main at West Commercial Ave Crossing: Per the record drawings, there is an 8-in water main running perpendicular to the channel alignment and crossing under the existing culvert. The Veolia Water design criteria and guidelines have been incorporated into the design per the comments received for the water main relocation design. Veolia submitted their comments after review of 95% design.Veolia has requested upsizing the watermain on the bridge to 12” in diameter for the future growth. The 12” diameter watermain has been proposed at West Commercial Ave bridge crossing over the channel.

4.1.2.4. Gas

Per the record drawings there is a 6” diameter plastic buried gas line on the south side of both Amor Ave and West Commercial Ave crossing above the existing culvert. Since a new bridge is proposed to replace the existing culverts, the existing gas lines shall be relocated from the buried condition to a 6” diameter steel gas mounted on the outside of the new bridge parapet. PSE&G reviewed the 95% design and submitted two Map Index for the temporary and permanent gas at Amor Ave titled 2D-49 and for West Commercial Ave titled 2C-50. Both Index maps have been adhered to for the relocation design of both temporary and permanent gas at Amor Ave and West Commercial Ave.

4.1.2.5. Electric

The PSE&G Standard Drawings and Specifications will be utilized for the suggested relocated overhead electric facilities. PSE&G coordination for relocation shall be done by the Contractor, with input by the design team if necessary. The permanent and temporary pole relocation design is on-going with PSE&G. The Design Team has yet to receive any direction or guidance as which entity will relocate utility poles and overhead wires and the schedule for this work has yet to be determined. Drawings showing suggested relocation of poles and temporary poles during construction have been forwarded to PSE&G for their review and required revisions are still in progress. All other private utility companies having an overhead utility on the relocated poles will be contacted to acquire their standard drawings and specifications during the design development. See Fig 5-37 cross section for the suggested new overhead and utility pole relocation. Other PSE&G tenants such as Communication and Cable shall coordinate their relocation with PSE&G. Responsible parties will be determined after meeting with PSE&G.

4.1.2.6. Telecommunications

Verizon and AT&T are tenants of PSE&G and their overhead wire relocation shall be coordinated with PSE&G.

4.1.2.7. Cable

Charter, Comcast, Zayo, and JoeMax Telecomm are tenants of PSE&G and their overhead wire relocation shall be coordinated with PSE&G

4.1.3. Geotechnical

Geotechnical design criteria apply to the slope stability of channel improvements along the ER, and to shallow and deep foundations supporting the upgraded roadway bridge and railroad bridge structures. The applicable design criteria for each are summarized in the following sections.

The subsurface exploration for the project consisted of advancing twelve (12) Standard Penetration Test (SPT) sample borings, four of which included bedrock coring. Overburden sampling included obtaining both disturbed SPT and “undisturbed” thin-walled samples. The subsurface exploration was executed by Craig Test Boring Company, Inc. At the completion of the subsurface exploration, a laboratory testing program was developed and executed to determine material properties for both the overburden soils and bedrock. The laboratory testing program consisted of tests to determine Atterberg Limits, grain size, consolidation properties, and material strengths. Both triaxial and unconfined compressive strength testing were performed to estimate the soil and bedrock strength, which was used to support the engineering evaluations. The complete scope and results of the subsurface investigation and laboratory program for the ER is detailed in Section 5.10.2 of this report. Engineering analyses were performed and the results compared to design criteria outlined herein to evaluate the suitability of the design.

The results of the subsurface exploration and laboratory testing program are summarized in a separate Geotechnical Investigation Results Report dated January 17, 2020 (See Attachment B-3C). The results from this report were utilized to develop subsurface profiles and develop strength properties of the foundation soils and bedrock. Engineering evaluations, including slope stability, deep foundation design, and retaining wall design, were conducted as part of this phase of the project and informed the proposed design. All geotechnical data and results of geotechnical evaluations are included within Geotechnical Design Attachment B-3C.

4.1.3.1. East Riser Embankment Slope Stability

Slope stability analyses of potential deep-seated failures were performed using the computer program SLOPE/W (GeoStudio, 2019). The program uses limit equilibrium techniques to search for the location of the critical failure surface that produces the minimum factor of safety (FS). Specifically, the Spencer method of analysis for circular arc surfaces was selected for the analysis. The slip surfaces are then optimized to determine the minimum FS for each loading condition. The optimization combines block surfaces and circular arc shapes to estimate the critical slip surface.

Evaluation of the factor of safety for each loading case is based on the criteria presented in EM 1110-2- 1902, Slope Stability, (USACE, 2003) and the general state of the practice. The minimum required factors of safety for various loading conditions are provided in Table 4-2.

Table 4-2. Minimum Required Factors of Safety (FS)

Design Condition	Loading Condition	Basis for Shear Strength	Minimum FS [a]
1	End of Construction	Total Stress	1.3
2	Rapid Drawdown	Effective Stress	1.2
3	Long-Term Steady State	Effective Stress	1.5
4	Pseudostatic (Seismic)	Total Stress	1.1

[a] Based on recommendations provided in USACE EM 1110-2-1902 (USACE, 2003).

Design shear strength and soil parameters for the project were developed based on laboratory testing and SPT data completed at the site, as well as the results of past strength testing of similar soils, published correlations, and our general experience. Total stress strength parameters (short-term) were estimated using the results of triaxial tests in conjunction with SPT correlations. SPT correlations can be used to estimate cohesion (total) in fine-grained soils and friction angles (total/effective) for coarse grained soils from the SPT "N" value. Effective stress strength parameters (long-term) were developed based on correlations with Atterberg limits, as well as utilizing results of past strength testing of similar soils and our experience in this area.

Considerations for the influences on groundwater on the proposed design and construction, which includes dewatering, design water elevations, and estimated groundwater elevations, are provided in Section 5.10 Geotechnical Design. In general, design groundwater levels consisted of three different water elevations – ordinary high water, ordinary low water, and top of channel. The design groundwater level for Design Conditions 1 and 3 in Table 4-2 included both the ordinary low and high water levels. For Design Condition 2, the design groundwater level considered a scenario where the water level is drawn down from the top of the channel (representing a flooded condition) to the ordinary low water level.

4.1.3.2. Seismic Design Considerations

The American Society of Civil Engineers (ASCE) Standard SEI7-16, Minimum Design Loads for Buildings and Other Structures provided guidelines for determining the seismic hazard. The seismic hazard was characterized by the acceleration response spectrum and the site factors associated with the relevant site classification. Based on these guidelines and ASCE 7-16 Table 20.3-1, the site is assumed to be classified as a Site Class E for seismic design unless specified otherwise in the Project Geotechnical Engineering Report.

The peak ground accelerations for each return period are provided in Table 4-3.

Table 4-3. Peak Ground Accelerations

Return Period (years)	Peak Ground Acceleration (%g)
475	0.05
975	0.09
2475	0.18

A peak ground acceleration of 0.09 (%g), corresponding to the 975-year return period, was considered during the seismic slope stability evaluation based on guidance presented in USACE ER 1110-2 1806 related to maximum design earthquake (MDE). Results of the seismic slope stability analysis are provided in subsequent sections of this report.

4.1.3.3. Deep Foundations

Engineering analyses for deep foundations deriving resistance from the underlying soils was performed for the proposed deep foundation structures supporting the roadway bridges and railroad bridge. Deep foundation design followed the procedures and recommendations outlined in NJDOT, "Design Manual for Bridges and Structures." Allowable Stress Design (ASD) methodology was utilized in the analyses and development of the axial resistances of the piles for the bridge. Axial resistance was developed utilizing a factor of safety of 2.5 within the soil column and based on soil parameters derived from the results of the subsurface investigation and published correlations between Standard Penetration Test (SPT) data and shear strengths. A subsurface profile was developed for each bridge location, based on the specific conditions encountered at sample borings drilled in the vicinity of the bridge. As discussed in Section 4.1.4.1, soil parameters were estimated using the results of triaxial tests in conjunction with SPT correlations. AASHTO LRFD Bridge Design Specifications methodology was utilized in the design of bridge foundations, if needed, resistance factors will be utilized from AASHTO Table 10.5.5.2.3-1. The computer program APILE (2018) was utilized in the estimation of the axial resistance of the pile foundations

The foundations are selected based on the results of the Project Geotechnical Engineering Report. See Section 5.10 for details about the design of the proposed deep foundation elements. Vibrations generated from the installation of deep foundation elements and the potential impact on neighboring structures is discussed in Section 5.10.7.

Geomorphological

The following sections discuss aspects of geomorphology and design criteria that apply to the project components.

- Channel substrate will be placed below the Ordinary High Water (OHW) elevation. The OHW elevation was determined for final design from data collected as part of the InfoWorks calibration and validation effort. The channel substrate will consist of the channel bottom material and the riprap side slopes. The top of the wetland shelf substrate will be placed equal to or above the OHW elevation. The wetland shelf substrate will be suitable planting media; the specific material is detailed in Specification 32 91 13 Topsoiling and Finished Grading.
- The channel riprap side slope material will be placed on the channel banks below the OHW elevation, at a slope not steeper than 1.5H:1V, and no wetland or landscape plantings will be installed below this elevation. Following placement of the riprap side slope and toe material a gravel mix with gradation the same as the channel bottom material will be placed and spread over the riprap area to fill the voids of the riprap.
- The design D50 (median particle diameter) substrate size was calculated following the dimensional critical shear stress approach in NEH654, Chapter 11 (Rosgen Geomorphic Channel Design) and velocity-based approach in USACE Hydraulic Design of Flood Control Channels (EM-1110-2- 1601) (1994). The critical shear stress was calculated for 27 representative cross section locations for the 500-year return period event (0.2% Annual Exceedance Probability; AEP). This event was chosen at it maximizes the hydraulic depth and boundary shear stress for the proposed design. Critical shear stress was calculated for each of the proposed typical channel cross-sections, at bends, and hydraulic structures. The critical shear stress was used to calculate the maximum entrained particle size using both the Rosgen, and Leopold, Wolman, and Miller power trendlines in NEH654 Chapter 11, to develop the estimated range of movable particle size. A factor of safety of 1.1 was be applied to the estimated maximum entrained particle size to guide the selection of the gradation of the placed channel substrate (channel bottom and riprap side slopes). The D30 channel bottom and riprap side slope substrate size was calculated at 16 representative cross section locations for the 0.2% AEP event using procedures in EM-1110-2-1601. The D50 value was calculated from the calculated D30 using a multiplier of 1.2 and the results were compared to the movable particle sizes calculated with the critical shear stress analysis to assess hydraulic stability of the proposed channel substrate. Once the design D50 was selected then the design gradation was calculated using the riprap gradation multipliers found in the Standards for Soil

Erosion and Sediment Control in New Jersey (2014). The calculated gradations were further modified after consideration of the channel side slopes using guidance and standard riprap gradations in NCHRP Report 568 (2006).

- The minimum thickness of the placed substrate is designed to be the greater of the D100 or 1.5 times the D50 of the design substrate gradation. An underwater placement factor of 1.5 and a minimum factor of safety of 1.1 were applied to determine the minimum thickness of the placed channel substrate. The ER channel bottom design was developed based on information prepared during the prior ER design phases, with objectives including:
    - Maintaining channel conveyance that was established during Feasibility,
    - Establishing a stable channel bed that will not degrade (incise),
    - Allowing for future operation and maintenance dredging,
    - Providing aquatic habitat functions, and
    - Providing opportunities for improved water quality.
- The additional design components of fluvial geomorphologic design, including channel alignment, longitudinal bed profile, and channel cross sectional dimensions were evaluated based on site conditions and industry standard design guidance.

4.1.4. Structural

The structural design criteria are divided into four sections corresponding to the bridge replacements under the roadways and the railroad bridge replacement, in addition to the retaining wall design and building foundation check. The two bridge sections – one for roadway bridges and the other for the railroad bridge – are necessary due to the different design live loads, design methodology, governing design standards, owner preference, and geometric constraints of roadway supporting structures versus railroad supporting structures.

4.1.4.1. Building Stability

The structural design criteria for the building stability analyses were ASCE 7-16 and IBC 2018 using the software, RISA Foundation to de sign assumed strip footings using estimated loads from images to determine whether the building stability would be maintained during construction or not. There were assumptions made to determine how the dead loads, live loads, snow loads, etc. would impact the stability of the selected buildings. The buildings were chosen based on assumed proximity to the channel and proposed line or work and based on which buildings' footings would be impacted the most. The proposed work of over excavation and regrading is critical to this analysis as to check whether the work will undermine the capacity of the soil under the assumed building footings.

4.1.4.2. Combined Wall Systems

The current and previous design of retaining walls (both sheet piles and combined wall systems) was based on IBC 2018 and USACE EM 1110-2-2504 Design of Sheet Pile Walls. Evaluations performed through 100% Design were based on the referenced guidance and were executed using the Shoring Suite Version 8.16g software developed by CivilTech. Design performed through this Phase determined the required wall embedment, section properties, and estimated deflections for “S-Case” soil strengths, controlled by drained (effective) shear strengths. S-Case (Slow Test - Serviceability) conditions are present long after construction when excess pore pressures have dissipated and represent the normal or serviceability conditions. After the 95% Design, the basis of design was reevaluated and determined to be inappropriate for the subsurface conditions and anticipated loading for the wall sections. In the Q-Case (Quick test - Quantitative Risk Case) analysis for retaining wall design, the design considers extreme loads such as seismic forces, surcharge loads, hydrostatic pressure, and live loads, along with key soil properties including cohesion, friction angle, liquefaction susceptibility, pore pressure, and bearing capacity, to ensure the wall's stability and safety under the most severe conditions.

Each retaining wall section was reevaluated for “Q-Case” soil strengths, controlled by undrained (total) shear strengths. Q-Case conditions typically are present immediately after construction and result from the development of excess pore water pressures. Excess pore pressure can occur when soils are loaded quickly, not allowing for pore pressures to drain, like during a flood event or if a soil mass is subject to a surcharge load. Undrained conditions are expected at the site, given the presence of a thick, very soft clay stratum and the anticipated loads. The reevaluation performed for 100% Design indicated the Q-Case soil strengths govern the design, resulting in a more robust retaining structure consisting of a combined wall system.

The updated analysis was performed using the PYWall 2022 program developed by Ensoft, Inc. As discussed in Section 4.1.4.1, soil parameters were estimated using the results of triaxial tests in conjunction with SPT correlations and the results of past strength testing of similar soils and engineering knowledge. The combined wall systems sections analyzed are based on NUCOR/Skyline standard sections. The maximum tolerable top-of-wall deflection is considered as 1-inch, unless noted otherwise herein. Intermediary sheet piles shall be driven to bedrock, while steel king piles are socketed into competent bedrock at lengths specified herein. Further discussion on the results of the analyses performed for sheet piles-king pile and the combined walls are provided in Subsection 5.7.

4.1.4.3. Roadway Bridge Replacement

The ER improvement project will include replacement of two culverts under existing roadways with new bridges. The structural design of the bridges will be governed by AASHTO *LRFD Bridge Design Specifications*, 8th edition (AASHTO LRFD) as modified by the NJDOT *Design Manual for Bridges and Structures, 6th edition* (NJDOT BSDM). Key criteria are summarized below.

All applicable load combinations from AASHTO LRFD Table 3.4.1-1 will be investigated. Detailed descriptions of each load type in the table are in Chapter 3 of AASHTO LRFD and in the NJDOT BSDM.

4.1.4.4. Railroad Bridge Replacement

The ER improvement project will include replacement of one railroad bridge carrying two tracks of the Seeman’s Lead industrial spur. The spur originates at Milepost 9.7 of New Jersey Transit’s (NJ TRANSIT) Pascack Valley Line. NJ TRANSIT owns the spur and associated infrastructure including the rail bridges over the ER. Norfolk Southern Railway (NS) operates and maintains the spur, serving local industrial customers. The structural design of the railroad bridge is governed by American Railway

Engineering and Maintenance-of-Way Association's *Manual for Railway Engineering*, 2020 edition (AREMA MRE). Chapter 8 of AREMA MRE governs design of concrete elements and foundations; Chapter 15 governs design of steel elements. Concrete elements follow Load Factor Design (LFD) methodology while steel elements and foundations follow Service Load Design (ASD) methodology as detailed in the AREMA MRE. Guidelines from NJ TRANSIT and NS manuals listed in the References section of this chapter also apply. Key criteria are summarized below.

4.1.4.4.1. Design Loads

All applicable Group Loadings from AREMA MRE Table 8-2-5 for concrete elements and Load Combinations from AREMA MRE Table 15-1-10 have been investigated. Detailed descriptions of each load type in the table are in Chapters 8 and 15 of the AREMA MRE

For impact on transverse members such as floor beams, see AREMA MRE 15-1.3.4.2.3. was included in accordance with AREMA MRE 15-1.3.5d.

Rocking effect

As the track design speed at the bridge location is low (10 mph), the bridge design took advantage of a reduction to vertical live load impact allowed by AREMA for speeds less than 60 mph. See AREMA MRE Chapter 15 part 7.3.2.3.a.

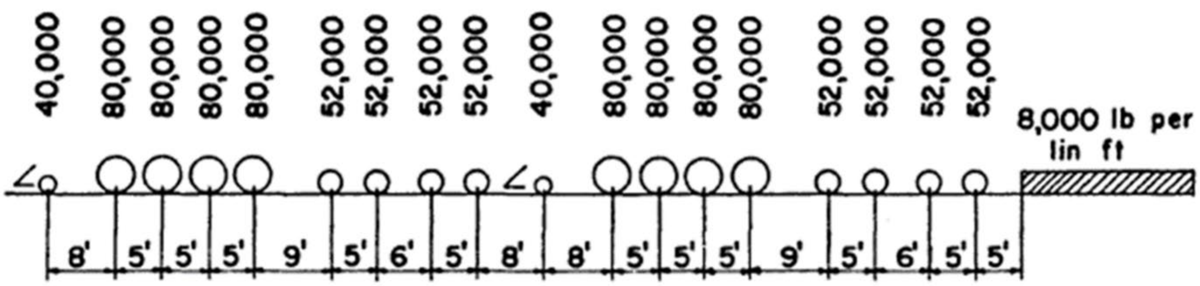


Figure 4-2. Cooper E-80 Live Load

4.1.5. Landscape Architecture and Urban Design

Landscape architecture and urban design seeks to support flood reduction, improved water quality, and improved channel ecology. Additionally, public realm opportunities are considered where appropriate to provide improved visual experience and recreation. The landscape architecture and urban design plans respond to constrained site conditions and proposed channel design grading and embankment strategies to meet these goals. Additionally, disturbance to riparian zone vegetation and wetlands will be minimized, and temporarily disturbed areas will be re-vegetated with appropriate native vegetation. The locations of existing wetlands are shown on the existing conditions plan sheets in the design set and are also illustrated on the permit plans included with the permit application that will be submitted to NJDEP DLRP and USACE.

The ER site conditions at the edges of the channel vary along its length throughout the Project Area and are affected by the encroachment of adjacent light industrial parcels. The relationship between existing parking lot and facility operations, existing building footprints, and the need for a long-term operations and maintenance corridor influence the proposed planting limits. See

Figure 4-3 below for existing development related site conditions, such as existing buildings and impervious surfaces. The proposed planting plans will address all non-hardscape areas within the limits of disturbance. The planting seeks to improve ecology and views of the channel from road crossings and adjacent properties.

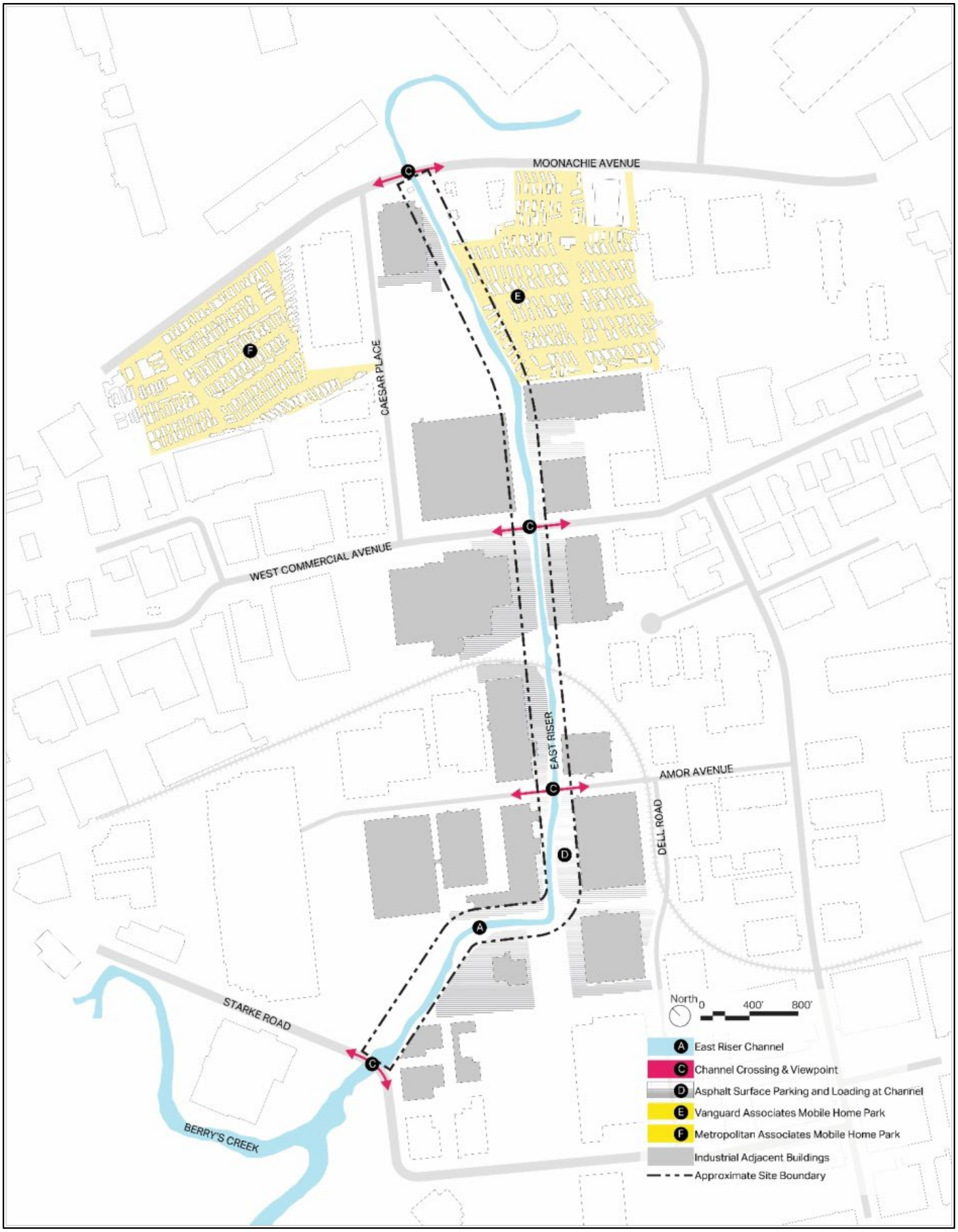


Figure 4-3. Existing Landscape Constraints and Conditions

4.1.5.1. Landscape Performance

The ER is made up of three components: bottom of channel, embankment, and riparian zone. Each of these three elements contributes to the project goals of stormwater conveyance and ecological restoration, and each plays a unique role in providing an overall holistic ecological uplift to the channel. See Figure 4-4 below for a conceptual illustration of each zone; in the image shown, there is an existing paved area within the riparian zone. The landscape design is primarily driven by planting species selection and design location.

The design criteria include suitability to wet conditions, salinity, frequency of inundation, sun and shade tolerance, growing zone, native and non-invasive categorization, and ability to provide habitat for wildlife. The landscape design is influenced by the channel engineering design, and plant species selection will support the hydraulic and ecological goals of the project. Where feasible given constraints of existing infrastructure and channel stability design, the landscape restoration comprises re-vegetation and habitat restoration within two zones of the channel, the embankment zone, and the riparian zone of the channel. The planting strategy in the embankment zone is determined by conveyance requirements and embankment slope stabilization methods. In the riparian zone outside of the top of bank, the planting strategy responds to micro-climactic conditions, existing plant communities, and operations and maintenance considerations. Figure 4-5 illustrates an idealistic holistic ecological approach; this approach will be incorporated where design constraints and the infringement of existing development allow.

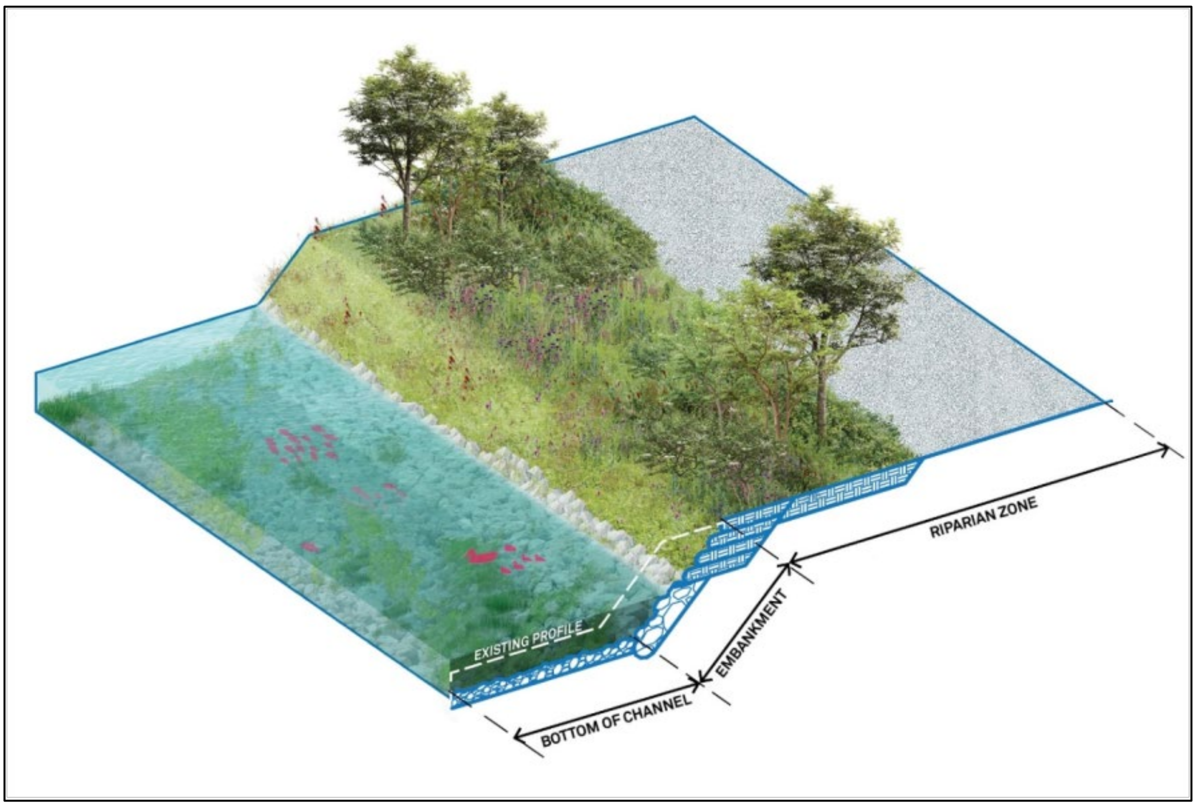


Figure 4-4: Conceptual Channel Components

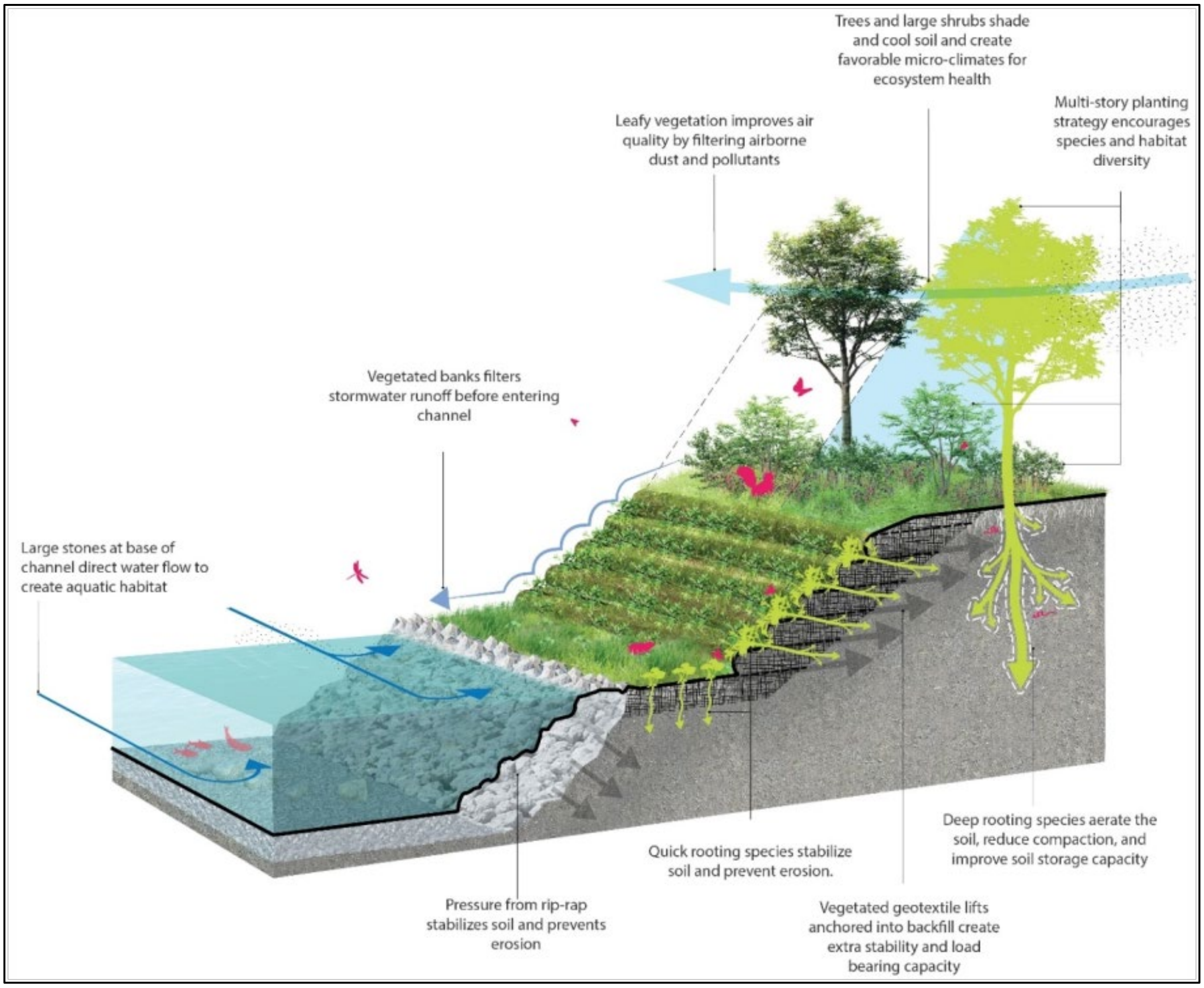


Figure 4-5. East Riser Ecological Restoration[1]

4.1.5.2. Public Realm

Consistent with the larger RBDM project goals, the team seeks to provide co-benefits and public realm improvements where appropriate. For the ER Channel Improvements, water quality improvements and ecological enhancements are the primary opportunities for co-benefits. Water quality improvements would be achieved through removal of accumulated sediments that have been impacted by various levels of contaminants from both historic contamination and anthropogenic sources. Additional recreation benefits are considered where feasible. The public realm opportunities occur within the riparian zone of the channel improvements, above the top of bank.

[1] This figure illustrates the maximum extent of ecological restoration that was considered in early design phases. This approach has been revised to factor in channel stability/design considerations and space constraints due to existing development. Refer to the Landscape Plans (LP Sheets) included in the 100% design drawings for final planting areas/ecological restoration along East Riser.

5. SUMMARY OF DESIGN

The following sections provide a summary of the project design components, and engineering evaluations completed through the 100% design phase. The approach to finalizing the design as it relates to the project design milestones is also provided where possible.

5.1. Description of Proposed Design

The project includes 4,150 LF of channel improvements, which consists of excavation and dredging and three upgrades to existing stream crossing structures. The estimated volume of initial excavation is 61,606 cubic yards (CY) and channel maintenance is estimated to require dredging and removal of 8,000 CY of sediment every five years. The geolift and reinforced slopes both have a ‘finish layer,’ as shown on the typical sections in the 100% plans, with a minimum of 2-ft thickness beneath the operation and maintenance corridor and a 1.5-ft thickness for banks not used for operation and maintenance. The finish layer provides protective cover over the top reinforced lift so that the stabilizing geogrids within are not damaged. This layer consists of either asphalt pavement or vegetated soil. Section and material details are included in the 100% Plans and Specifications. This layer is graded to drain towards the channel at a 1 to 3% slope to promote positive drainage to the channels. Combination wall caps are set at grade to allow drainage to the channel.

Three types of channel barriers were incorporated into the design for all reaches except those having only in-channel improvements (LB Sta 20+50 – Sta 25+60 and LB Sta 32+00 – Sta 41+15.) Jersey barriers found in the pavement section are used for existing parking lots (Type 1 barrier). Boulders (2-ft x 2 ft x 2f) spaced at 25 feet are used where the operation and maintenance corridor falls on a vegetated soil finish layer (Type 2 barrier). And signage is used where there is a vegetated soil finish layer that does not coincide with the operation and maintenance corridor (Type 3 barrier).

Refer to the 100% plans for a summary of the ER channel upgrades. There are fourteen (14) typical sections on the 100% Plans, which represent the proposed channel sections varying along each reach. Table 1 in the 100% Plans, ER Schedule of Improvements, provides a comprehensive summary of the entire channel length of 4150 LF. All channel reaches were over-excavated, beyond the depth required to improved channel conveyance. This will allow clean material to be placed to bring the channel to final grade. In general, the channel section types consist of different combinations of lightly vegetated reinforced lifts, riprap, channel bottom material, combination walls, and wetland shelves.

Two sections, developed in response to real estate concerns, are located along sides of the channel that are sensitive to disturbing existing business operations (LB Sta 20+50 – Sta 25+60) and homeowners (LB Sta 32+00 – Sta 41+15).

Typical section #8 will be constructed along the Loomis Property (LB Sta 20+50 – Sta 25+60), along the left bank, to minimize disturbance to business operations. Geotechnical slope stability evaluations completed showed that additional measures were required to keep the Loomis bank stable during and after construction. Those measures include leaving a support buttress of existing material along the toe of the bank, installing and embedded riprap toe adjacent to, and on the buttress.

A combination wall will be installed along the right bank. Typical sections #11, 12, 13, and 14 will be constructed adjacent to the Vanguard Mobile Home Park (LB Sta 32+00 – Sta 41+15) to avoid impacting manufactured homes in that area. Geotechnical slope stability evaluations showed that additional measures were required to keep the manufactured homes bank stable during and after construction as channel excavation was too close to the existing boundary. The final design leaves a 10-20 ft buttress of existing material in place adjacent to the property boundary. On the opposite side of the channel, there is open space with more real estate available to provide ecological restoration while improving hydraulic conveyance. The reach includes a reinforced bank with a wetland shelf downstream of the Caesar Place culvert and a reinforced slope and combination wall upstream of it.

There are two other special typical sections downstream near the ER Pump Station. Due to site constraints, combined wall (king-sheet pile) was selected to provide hydraulic conveyance and slope stability. These sections, Typical Section #1 and Typical Section #1A, consist of the ER Pump Station intake structure and combination wall, selected based on the limited space, steep existing slopes, and the results of geotechnical slope stability modeling in this location. The combination sheet pile wall is discussed in detail in Section 5.10.5.

The typical sections are included in the 100% Plans and Tables 2 and 3, provide detailed channel geometry and offset information of both banks. The channel top widths vary from approximately 25 to 70 feet, moving from up- to downstream. The bottom widths vary from approximately 15 to 45 ft over the same reach. Channel depth from top of bank varies from 6 to 12 feet, from up- to downstream. Three segments, totaling 800 LF along the right bank include 6-ft wide wetland shelves set at elevation 0.0 ft (OHW). These areas will be filled with suitable material for planting media and vegetated with native plant species. Refer to Specification 32 91 13 – Topsoiling and Finished Grading for details on the material that will be placed in the wetland shelves.

The geolift and reinforced slopes embankment upgrades require an overcut width of 11-ft from the landward channel hinge to satisfy the various stability loading conditions at the appropriate factor of safety. This offset, plus additional footage for an operation and maintenance corridor, will be preserved as permanent easements. The civil plan/profile sheets (CP sheets) provide the extents of the temporary and permanent easements.

The geolift and reinforced slopes both have a ‘finish layer,’ as shown on the typical sections in the 100% plans, with a minimum of 2-ft thickness beneath the operation and maintenance corridor and a 1.5-ft thickness for banks not used for operation and maintenance. The finish layer provides protective cover over the top reinforced lift so that the stabilizing geogrids within are not damaged. This layer consists of either asphalt pavement or vegetated soil. Section and material details are included in the 100% Plans and Specifications. This layer is graded to drain towards the channel at a 1 to 3% slope to promote positive drainage to the channels. Combination wall caps are set at grade to allow drainage to the channel.

Three types of channel barriers were incorporated into the design for all reaches except those having only in-channel improvements (LB Sta 20+50 – Sta 25+60 and LB Sta 32+00 – Sta 41+15.) Jersey barriers founded in the pavement section are used for existing parking lots (Type 1 barrier). Boulders (2-ft x 2 ft x 2f) spaced at 25 feet are used where the operation and maintenance corridor falls on a vegetated soil finish layer (Type 2 barrier). And signage is used where there is a vegetated soil finish layer that does not coincide with the operation and maintenance corridor (Type 3 barrier).

Refer to the 100% plans for a summary of the ER channel upgrades. Plan, profile and representative cross-section plan sheets showing the channel geometry along the alignment are included in the 100% plan sheets under separate cover. Final embankment stability results are included in Section 5.6.

A hydrology and hydraulics analysis was previously completed for the ER Channel Improvements as part of prior design efforts. For the purposes of this feasibility appendix, a new H&H analysis was performed to verify design assumptions; however, the results of the updated analysis are included in Attachments B-3A (Hydrology) and B-3B (Hydraulics).

5.2. Site Layout and Controls

5.2.1. O&M Corridor

Establishment of a 10-foot-wide O&M corridor will allow for the continued, long-term maintenance of the channel improvements. The O&M corridor needs to extend along the entire reach of the channel improvements and varies along the left and right overbanks, with access to public roads. The O&M corridor will consist of a combination of new asphalt and a vegetated soil surface, to decrease stormwater runoff into the channel. In locations where existing paved parking lots or driveways fall within the extents of the proposed O&M corridor, the asphalt pavement will be replaced in kind to reduce disruptions to the function and appearance of the property. Along the O&M corridor, a barrier will be installed at certain locations to promote safety and site security. Details about the new barrier, including the exact limits, type, and size, are included in the 100% plans and discussed in Section 5.1. Design drawings developed for the NJ Rebuild by Design Meadowlands (RBDM) are provided as Attachment B-3D. This will require coordination with affected property owners. The type of maintenance equipment will be identified during the final design phase, as part of the O&M plan. Slope stability has been evaluated, given the extra weight from equipment along the channel banks. Input from property owners and the acquisition of property and easements will also impact the final width and location of the corridor. Where the corridor is a vegetated surface the O&M authority will access using mats to minimize the potential for rutting and damage to the reinforced embankments.

5.2.2. Easements and Real Estate

The project will require easements from private landowners. Due to a lack of existing channel easements, most of the improvements fall on private property. Temporary easements will allow for construction access and permanent easements will allow for construction of the channel widening, stabilization measures, and to secure O&M access on private property in future built conditions to maintain the improvements. An additional access easement will be obtained to allow property owners to maintain access through existing driveways.

Where possible, structures will be protected and maintained during construction. As noted on the 100% Plans, relocation locations and final boundaries will be revised after consultation with property owners has been completed. As of March 2024, the temporary and permanent easements shown on the September 2023 100% plans have been confirmed to have been documented by the project Surveyor, Matrix, and are being finalized with NJDEP.

5.2.3. Protection of Built and Natural Environments

In addition to easements, several physical measures will be installed to protect the natural and manmade environments during construction. Soil erosion controls will be applied in accordance with NJDEP, municipal and soil conservation district requirements along the perimeter of the construction limits to avoid release of sediment offsite. Visual warnings such as orange safety fencing or jersey barriers will also be installed surrounding all road openings during construction to mitigate interaction with the public, per OSHA and NJDOT construction regulations. This boundary is also defined in the 100 plans as the "construction limit of work." Existing buildings and other structures are avoided where possible. There were some exceptions requiring temporary and/or permanent relocation of structures, as shown on the plans.

Existing chain link and other types of fencing, curbs, and guard rails will be replaced in kind.

An existing dilapidated wooden bar screen immediately upstream of the existing tide gate at Starke Road, as shown on the 100 Plans, will be removed. In its existing state, it has no positive function and collects debris in the channel, reducing flow capacity. The removal will benefit the channel improvements and pump station. A photo of the debris screen is shown in Figure 5-1.

Figure 5-1. Existing Bar Screen in ER

5.2.4. Construction Access

Staging areas will be established by the contractor, as explained in Section 4.1.1.2. Equipment to be staged includes track-hoes, bulldozers, front-end loaders, and dump trucks. Material to be exported and staged includes demolition and clearing debris, soil and dewatering water, asphalt, and miscellaneous concrete, steel, and railroad material (piles, ballast, ties, etc.). Clean material to be imported and staged includes soils and aggregate, rock riprap, hot mix asphalt, pipe, precast concrete inlets, ready – mix concrete and grout, reinforcement steel, structural steel and piling products, other fabricated materials, geotextiles, planting materials, and trees.

In accordance with the project scope of work construction staging areas will be defined in a future "Site Logistics Plan" to be produced by the successful bidding Contractor. The future Site Logistics Plan will address the following:

- Access to public roadways,
- Placement of construction trailers,
- Connection to local electrical power source,
- Installation of a portable diesel fuel tank,
- Storage of material,
- Compliance with site specific stormwater pollution prevention plan, and
- Revegetation/restoration of all disturbed areas.

5.2.5. Management of Traffic

5.2.5.1. Plan Purpose

Construction of the planned bridge replacements for the ER will require work to be performed within the right of way for two roadways in two adjacent municipalities. In Carlstadt, Amor Avenue will be affected between Dell Road and the Albee/Camera Repair property immediately west of the ER. It is a two (2) lane local roadway with on-street parking, serving a predominantly industrial/commercial area for less than a quarter of a mile. In Moonachie, W. Commercial Avenue will be affected between Caesar Place and Gotham Parkway. This roadway is a two (2) lane arterial roadway with no on-street parking in the immediate area, also serving a predominantly industrial/commercial area. The construction process will require a designated staging area, truck loading/unloading zones, and at least partial closure of the local road. However, a full closure of W. Commercial Avenue at the channel is the most probable construction scenario.

A full closure of Amor Avenue at the channel is proposed. A full closure of W. Commercial Avenue is proposed for the area mentioned above. To maintain regional and local mobility along with local land access as best as possible, a Traffic Management Plan (TMP) is required. The TMP has been developed according to the guidelines set forth in the most recent edition of the Manual on Uniform Traffic Control Devices (MUTCD).

Along West Commercial Avenue, there are public roads where traffic can be diverted and still allow drivers to reach their intended destination. However, the surrounding streetscape and roadway circulation is different at Amor Avenue. The westernmost side connects to private property (specifically, the truck loading area for 350 Starke Road). To accelerate construction along Amor Avenue, a temporary traffic easement will be requested to facilitate traffic access and circulation through private property between Amor Avenue and Starke Road.

5.2.5.2. Construction Staging

The premise for construction staging is that construction along Amor Avenue and W. Commercial Avenue will occur separately. Both Amor Avenue and W. Commercial Avenue are proposed for full closures with detour mitigations.

5.2.5.2.1. Amor Avenue Staging

Amor Avenue staging will require a full roadway closure at the ER. As a result, the proposed TMP would temporarily remove all on-street parking (both sides) in the immediate area for the full duration of construction. Further, vehicular traffic will have to be re-routed to the south in the immediate construction area. Drivers seeking to access Amor Avenue west of the bridge will need to utilize Starke Road (eastbound and westbound), travel north through the 350 Starke Road private driveway, and travel eastbound along Amor Avenue.

Drivers seeking access to Amor Avenue east of the bridge have more options, depending on their origin. If they are coming from the south, drivers may utilize Starke Road (eastbound and westbound), turn onto northbound Dell Road, and turn left or right onto eastbound Amor Avenue. Drivers from the north may utilize southbound Gotham Parkway and turn right onto eastbound Amor Avenue. The detour is approximately 0.70 miles to 1.12 miles, depending upon direction of travel. It is shown in Figure 5-2.

Figure 5-2. Detour Route for Amor Avenue

5.2.5.2.2. West Commercial Avenue Staging

W. Commercial Avenue staging will require a full roadway closure at the ER. As a result, vehicular traffic will have to be re-routed to the north and south in the immediate construction area. For those drivers seeking to travel eastbound on W. Commercial Avenue, they will need to turn left onto Caesar Place, right onto Moonachie Avenue, and then another right to return to W. Commercial Avenue. This detour is approximately one mile. For those drivers seeking to travel westbound on W. Commercial Avenue, they will need to turn left onto Gotham Parkway, right onto Starke Road/Grand Street, followed by a right onto W. Commercial Avenue. This detour is approximately 1.5 miles and is depicted in Figure 5-3 below.

Figure 5-3. Detour Route for W. Commercial Avenue

It should be noted that there are NJ Transit bus stops in both directions along W. Commercial Avenue. In the eastbound (southeast) direction, NJ Transit Bus Routes 76, 144, 703, and 772 have a designated stop immediately after the intersection with W. Commercial Avenue and Caesar Place. In the westbound (northwest) direction, NJ Transit Bus Route 76, 144, and 772 have a designated stop immediately prior to the physical Riser Channel. Discussions with NJ Transit are in progress to make the agency aware of interim modifications to bus routes along W. Commercial Avenue during the full closure.

NJ Transit representatives were contacted to inform them of proposed W. Commercial Avenue full closure due to construction that would result in the need for temporary suspended bus stop service along Caesar Place. Based upon a field visit and various conference calls to coordinate relocated bus stops and traffic control devices at said locations, a set of design plans was developed to visually depict the proposed bus stop relocations and traffic control devices provided. The relocated bus stops were proposed along the eastbound and westbound sides Moonachie Avenue, between 150 feet and 180 feet west of Caesar Place. The full TMP package, including the NJ Transit bus detour route drawings, was presented to Bergen County and Moonachie Borough representatives for review and approval. At a meeting to discuss the plans and proposed bus stop relocation in detail, the Borough and County representatives rejected any notion of proposed bus stop relocations, primarily due to Moonachie Avenue being a high-volume roadway where vehicular travel speeds may be higher than the posted speed limits. Their position was that this temporary relocation may present safety concerns to all road users, from pedestrians attempting to cross Moonachie Avenue to typical drivers not expecting pedestrians at the bus stop locations, thus resulting in graver life-safety concerns. Instead, County representatives advocated that existing bus stops at Moonachie Avenue & Charles Lindbergh Boulevard (in front of Teterboro Airport) serve as the temporary bus stop locations for the Caesar Place bus stops until W. Commercial Avenue at the ER is reopened to vehicular traffic. This recommendation was brought to the attention of NJ Transit representatives, who conceded.

5.2.6. Borrow Material

Locations of proposed borrow areas and other construction material sources (i.e., nature and source of any proposed fill if not virgin [quarry] product, particularly recycled concrete aggregate) that are within close proximity to the project area were selected during Feasibility. These supplies are also listed within the NJDOT database of approved Coarse Aggregate Suppliers, indicating that they can likely conform to anticipated quality control standards to be specified for this Proposed Project. Their proximity and quality standards make them likely suppliers of quarry products or recycled concrete aggregates for the Proposed Project. Final selection of material suppliers would be contingent on the proposed design and be made by the Contractor and would be dependent on total cost of materials delivered to the Project Area, as well as on the ability of the supplier to meet the contractor’s delivery schedule. It is possible that a contractor could select a more distant supplier depending on those circumstances.

5.2.7. Disposal Facilities

Facilities were identified during Feasibility based on proximity to the site and assumed levels of contamination of the exported material. Exported material includes but is not limited to asphalt, concrete, mixed metal, soil, water, trees, brush, and other stripped, cleared, and grubbed material. Soil and sediment quality data collected during feasibility and design will be made available to the Contractor. The final location of the disposal facilities will be defined in the Contractor’s Site Logistics Plan.

5.3. Channel Improvement Quantities

5.3.1. Channel Bottom Material

Using the available draft geotechnical data and detailed modeling results, the channel bottom material selected is a relatively small substrate. This is based on modeled velocities, hydraulic depth, and shear stresses and knowledge of existing conditions channel protection measures. The designed channel bottom material is a coarse gravel, D50 = 1 inch with a thickness of 2 feet, for sections of improvement reach as shown on the design grading plans. This sizing was based on the design guidelines in the USACE Engineer Manual EM 1110-2-1601 Hydraulic Design of Flood Control Channels (1994), the Standards for Soil Erosion and Sediment Control in New Jersey (2014), and the National Cooperative Highway Research Program Report 568: Riprap

Design Criteria, Recommended Specifications, and Quality Control (2006). Figures showing the short and long-term channel surcharge limits are provided at the end of this sub-appendix

5.3.2. Overexcavation and Fill

It was decided during a project team meeting with MIMAC and USFWS in February 2021 that 2-ft of overexcavation is sufficient throughout the extents of the channel improvements where there is ecological exposure, which includes the channel bottom and wetland shelf, to account for potential non-hazardous soil material. The overcut associated with construction of the geolifts is sufficient and the riprap will not need the 1ft barrier. Clean fill will be used to match the proposed dredge template, as shown on the 100% Plans. Material placed in the wetland shelf locations will be suitable planting media that will hold soil moisture and facilitate plant establishment and growth. The specific material is detailed in Specification 32 91 13 Topsoiling and Finished Grading.

5.3.3. Geolifts

The Geolift material requirements were derived from a product called GreenLoxx® Mechanically Stabilized Earth (MSE) vegetated living retaining wall (GroSoxx®), produced by the manufacturer Filtrexx®. This product allows for a steep side slope, including the proposed 1 (H): 5.7 (V), as shown on the 100% Plans. Details were provided by the manufacturer to inform the cost estimate of the geotextile material and freight costs needed for the improvements and the volume for the amount of fill was calculated to be 5,842 CY. This material was quantified based on the dimensions matching the hydraulic model and as represented on the Typical Sections shown on the 100% Plans.

5.3.4. Riprap for Channel Side Slopes

During the final design for each project location, the proposed scour countermeasures—including riprap median stone size (D50), layer thickness, and areal extent—were reviewed for consistency with the previously developed design and evaluated in accordance with FHWA HEC-23 guidance. No changes to the riprap design were required.

Riprap is proposed along the channel side slopes and toe, yielding a median stone size (D50) of 6 inches and a minimum thickness of 12 inches. This sizing is based on established design guidance presented in the USACE Engineer Manual EM 1110-2-1601, Hydraulic Design of Flood Control Channels (1994), and the National Cooperative Highway Research Program Report 568, Riprap Design Criteria, Recommended Specifications, and Quality Control (2006).

Riprap beneath the ordinary high water (OHW) elevation will be placed on channel side slopes graded at 1.5(H):1.0(V), as shown on the Typical Sections. Additional riprap protection is assumed at structure abutments, extending 15 feet upstream and downstream along the side slopes. Following placement of the riprap along the side slopes and toe, a gravel mix with gradation consistent with the channel bottom material will be placed to fill voids within the riprap layer.

5.3.4.1. Geotechnical Data

The Geotechnical Report dated January 17, 2020 was used to estimate the D50 at each location.

5.3.4.2. Scour Calculations and Results

For each location, the potential scour depths were calculated following the FHWA HEC 18 and HEC-23 guidelines.

5.3.4.3. Rip Rap Design

A future analysis of the riprap design may be needed to identify minor changes due to changes in the hydrology and hydraulic (H&H) design.

5.3.4.3.1. Amor Ave

- Riprap Thickness= 3.0ft, D50 = 10”
- Riprap Top Width = 5.0 ft
- Riprap Bottom Width = 11.0 ft

5.3.4.3.2. Railroad Bridge

- Minimum Rock riprap Thickness = 12” = 1.0ft per scour calculations
- Use Class I Riprap with D50=6.0”, D100=12”

5.3.4.3.3. West Commercial Ave

- Riprap D50 = 8”
- Riprap Top Width = 5.0 ft
- Riprap Bottom Width = 20.0 ft

5.4. Utilities

This section identifies the major existing utilities within the construction limit and describes the course of action needed to protect, remove, temporarily relocate, permanently relocate and replace affected utilities such that continuous service is maintained throughout construction. Utility locations were determined from existing records and in person meetings with each utility owner ER flow is conveyed under bridges at Amor Ave and West Commercial Ave, and under a railroad bridge. Both culverts under Amor Ave and West Commercial Ave will be upgraded to bridges, and the existing railroad bridge will be widened to accommodate the proposed channel improvements. The utilities embedded within, and anchored to, the channel and the noted structures will be impacted.

Figure 5-4 shows the channel typical section annotated with the major utility conflicts along the alignment, most importantly utility poles and overhead wires. Figure 5-5 through Figure 5-11 depicts new location of major utilities at new bridges at Amor Ave and West Commercial Ave replacing existing culverts including watermain, gas line, storm drains and sanitary line crossing under Amor Ave. The records for the utilities attached to the railroad bridge will be obtained from the owner during the design development. A utility conflict inventory has been updated for final design and is included in Appendix A.

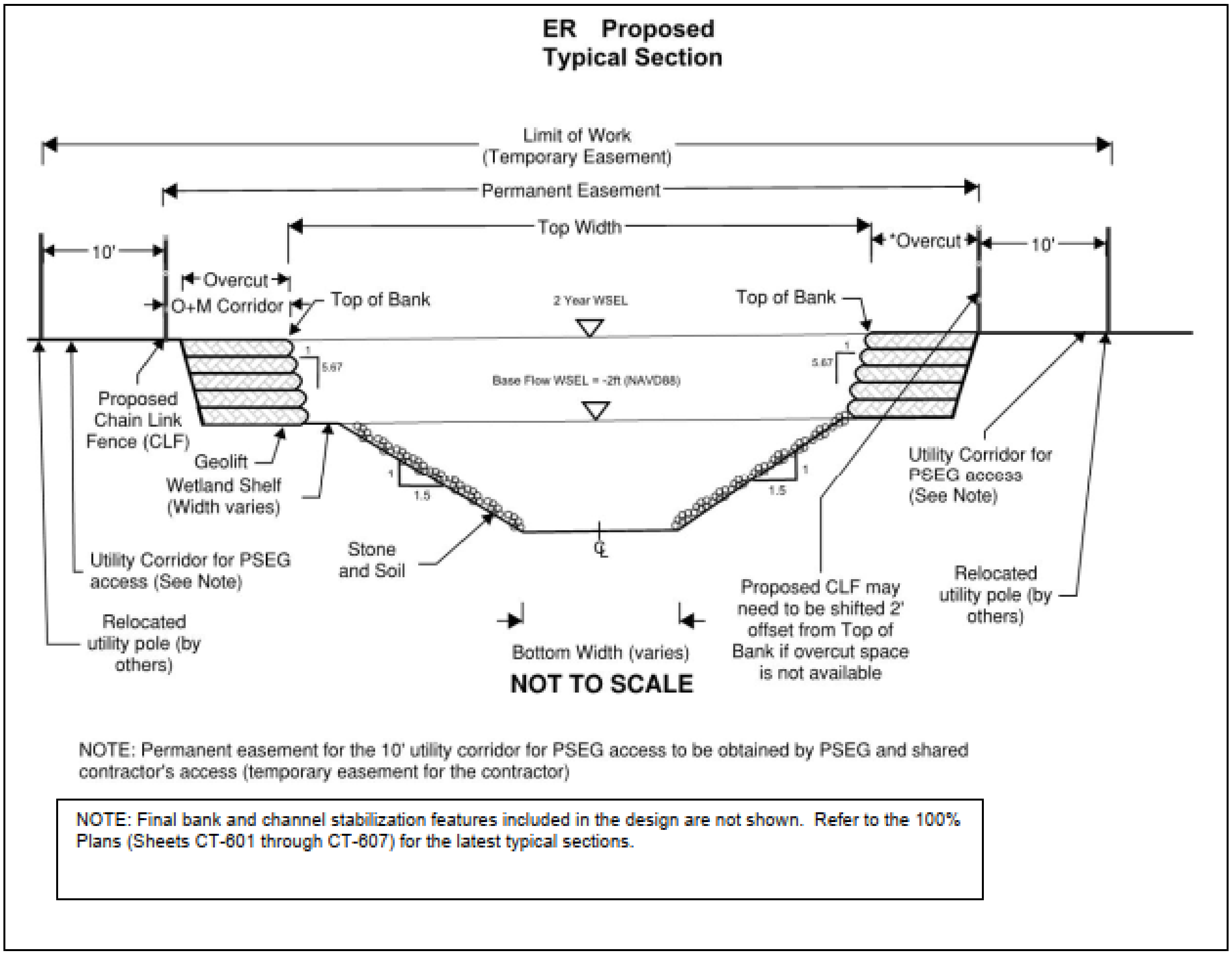


Figure 5-4. East Riser - Proposed Typical Section

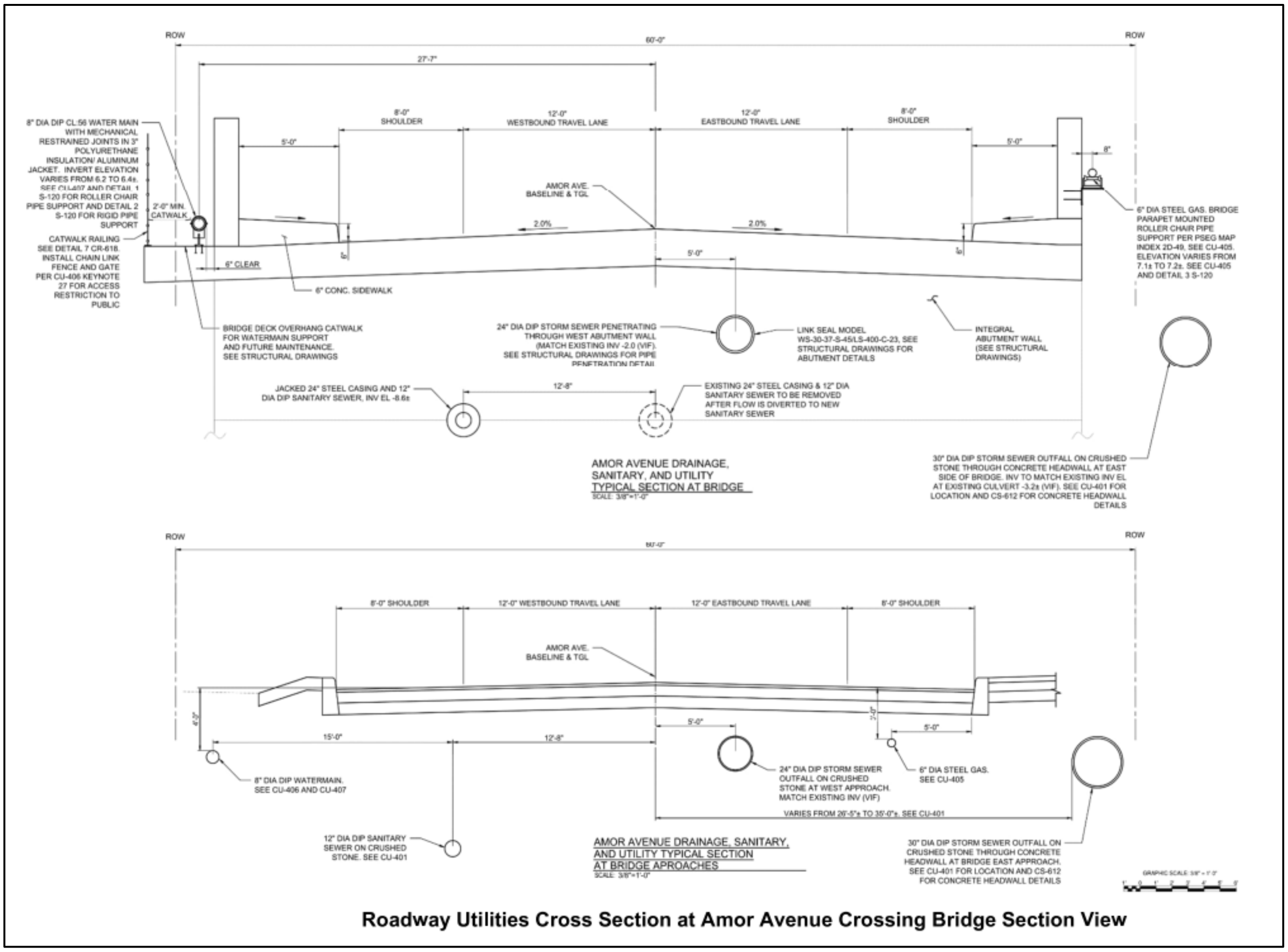


Figure 5-5. Roadway Utilities Cross Section at Amor Avenue Crossing Bridge Section View

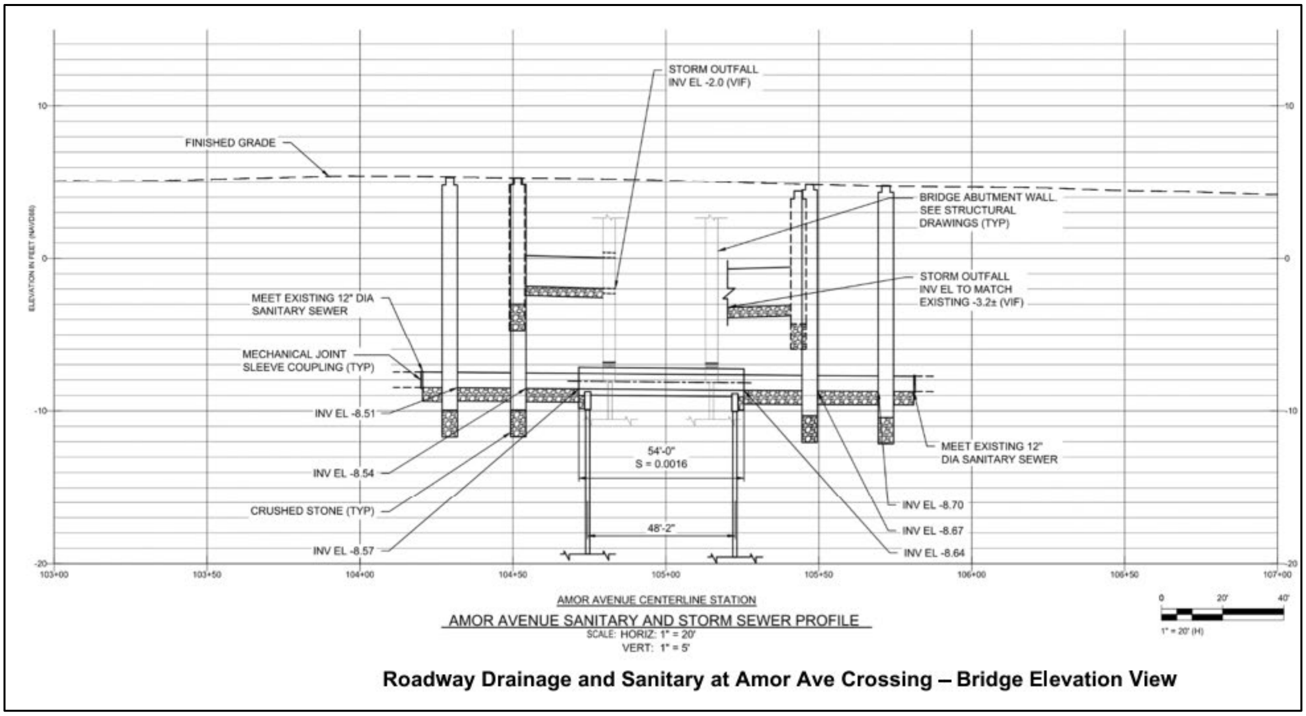


Figure 5-6. Roadway Drainage and Sanitary at Amor Avenue Crossing - Bridge Elevation View

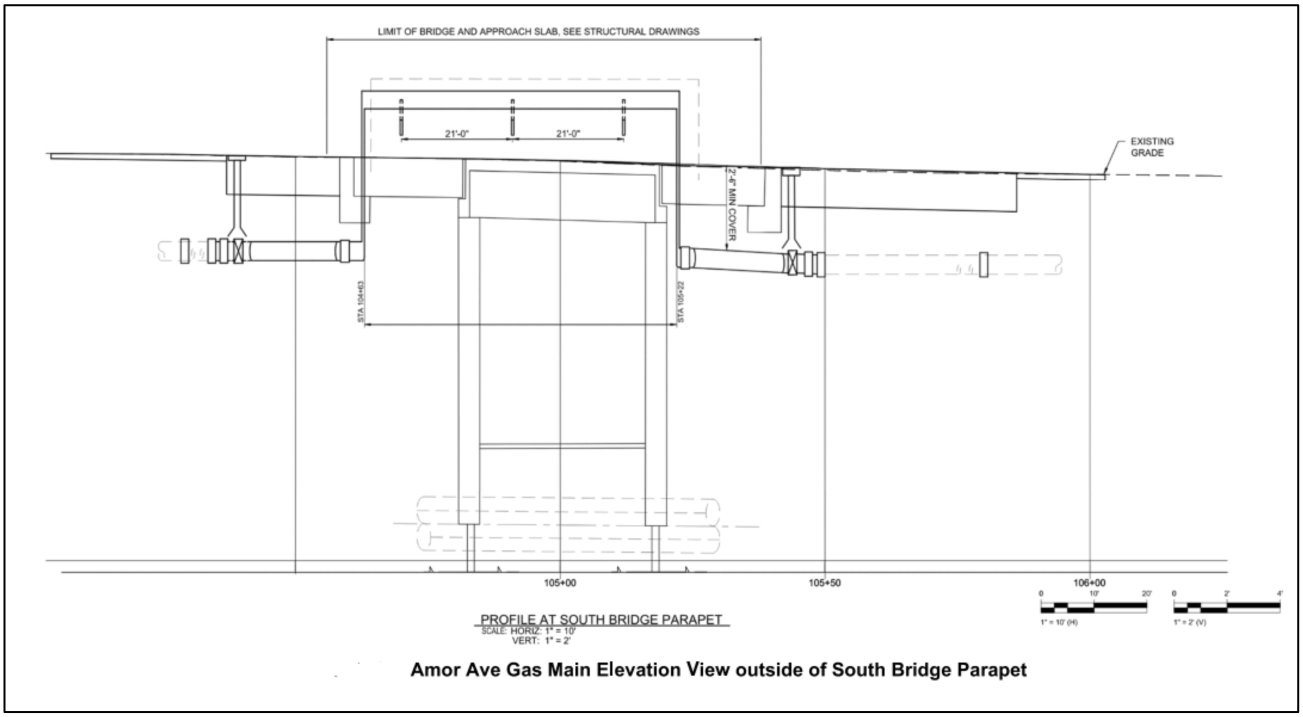


Figure 5-7. Amor Ave Gas Main Elevation View outside of South Bridge Parapet

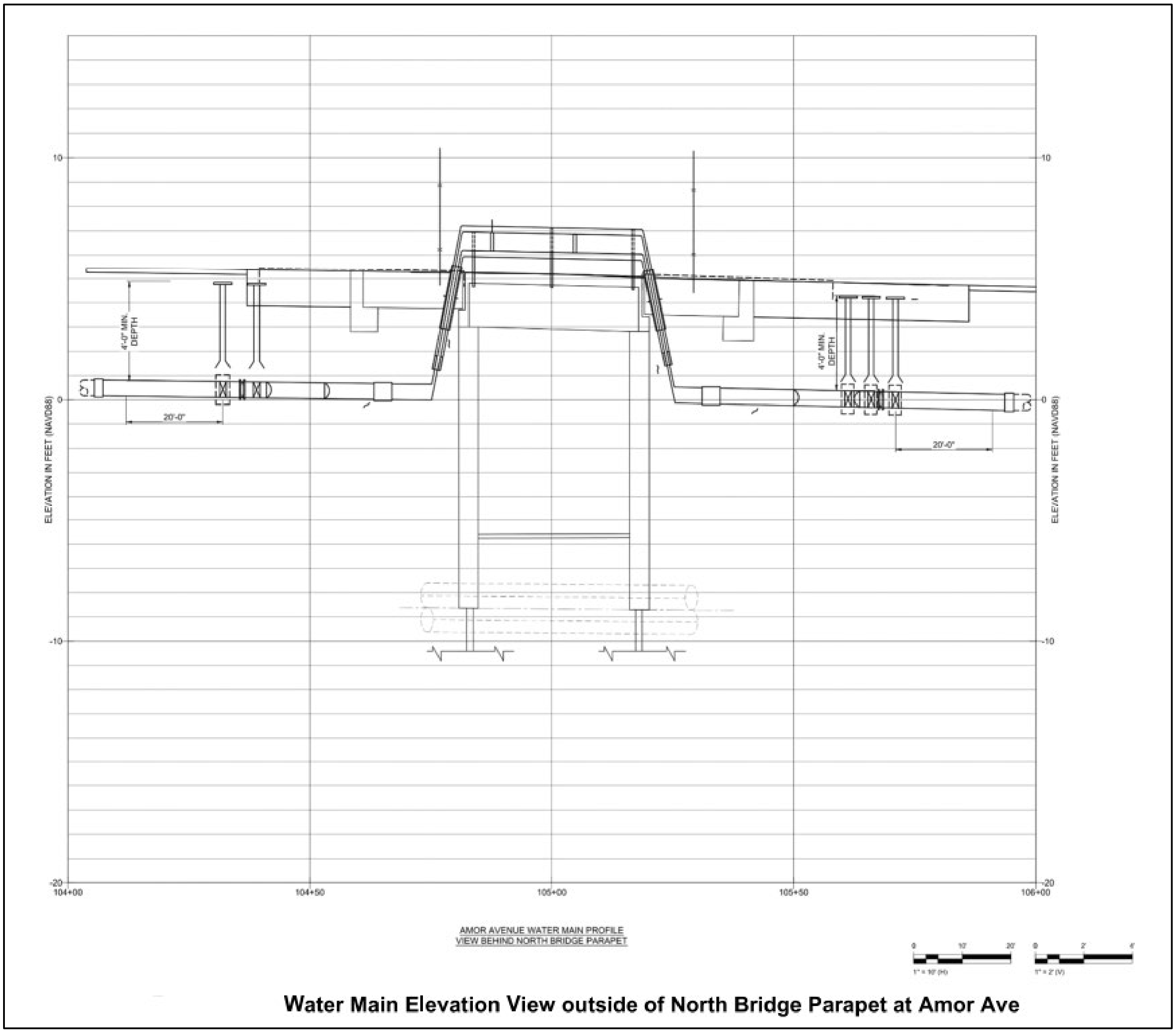
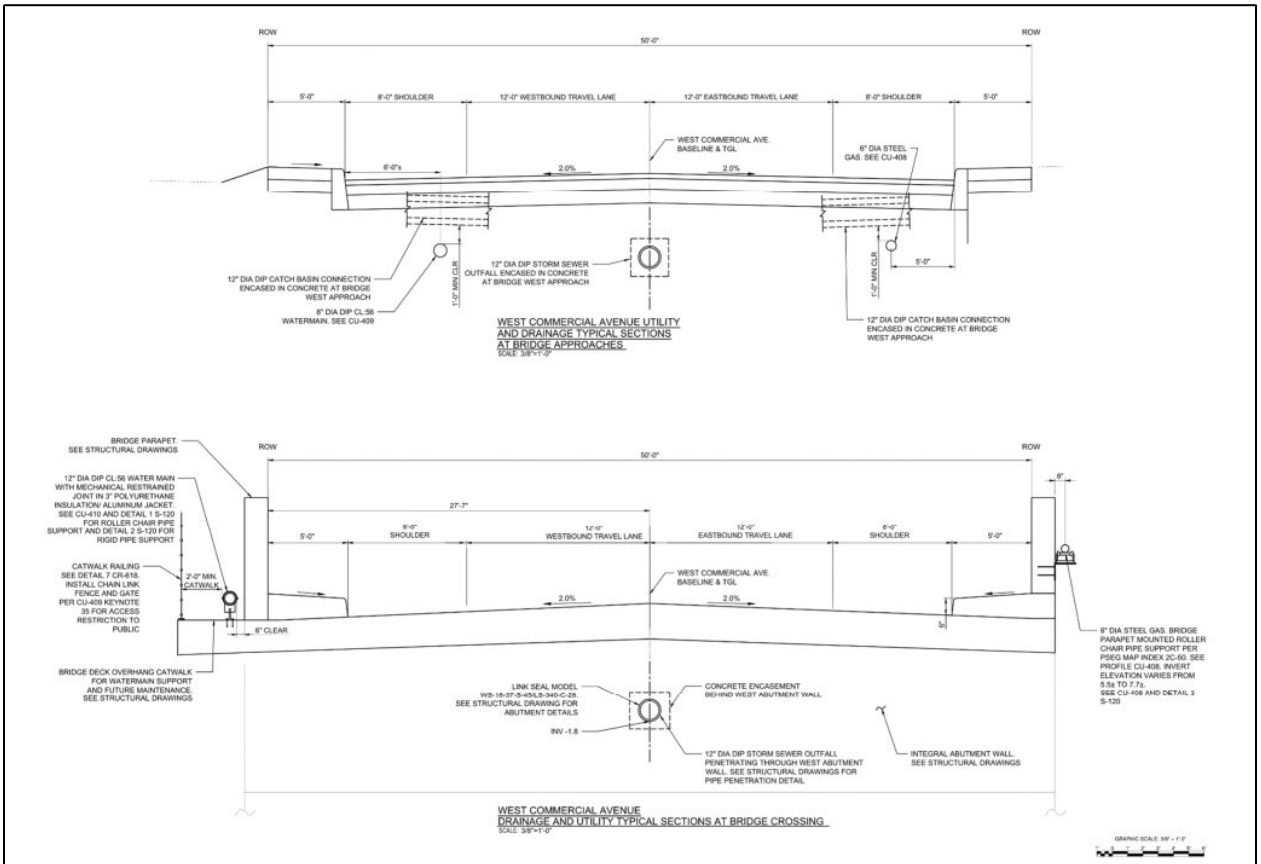


Figure 5-8. Water Main Elevation View outside of North Bridge Parapet at Amor Avenue



Roadway Utilities at West Commercial Ave Crossing – Bridge Section View

Figure 5-9. Roadway Utilities at West Commercial Avenue Crossing - Bridge Section View

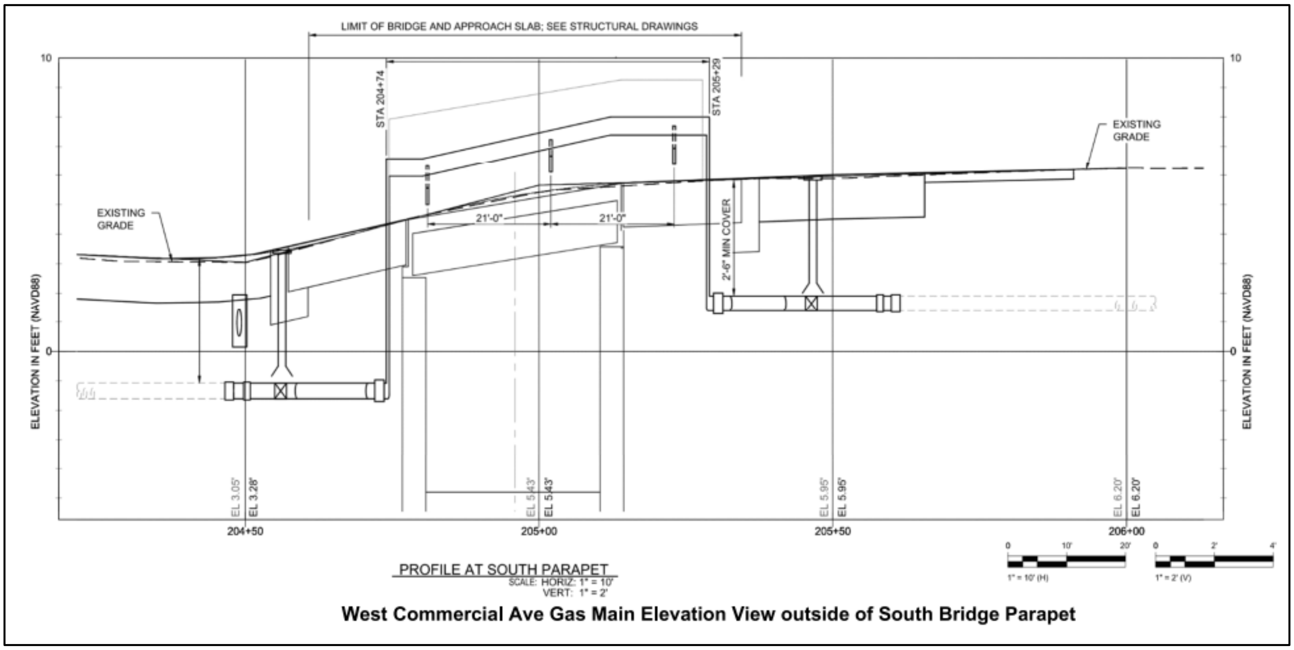


Figure 5-10. West Commercial Avenue Gas Main Elevation View outside of South Bridge Parapet

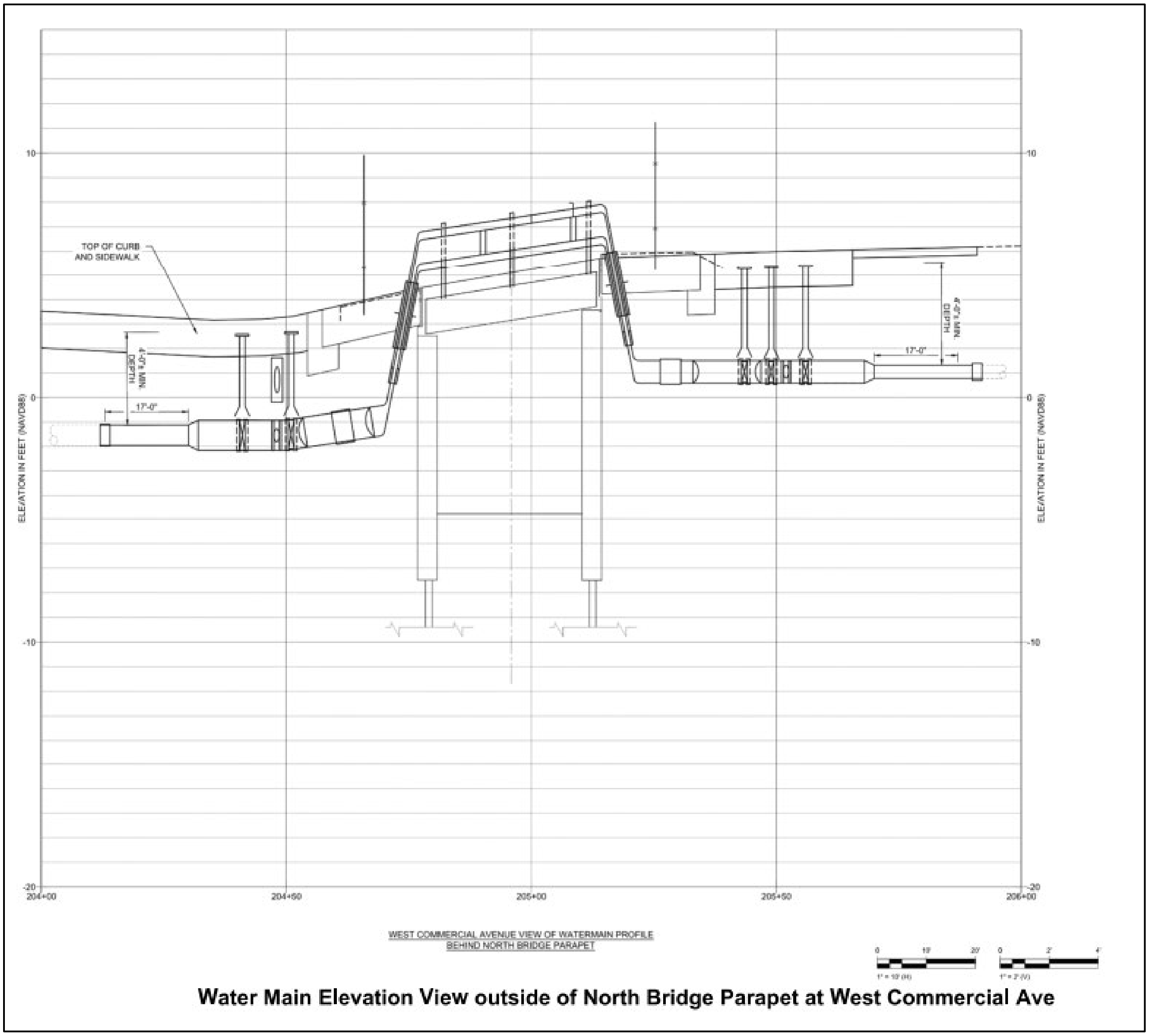


Figure 5-11. Water Main Elevation outside of North Bridge Parapet at West Commercial Avenue

5.4.1. Sanitary Sewer

5.4.1.1. Amor Avenue Sanitary Sewer Relocation

The existing 12-in sanitary sewer in 24-in steel casing shall be removed and replaced within the construction disturbance due to the unknown existing condition of the pipe as shown in Figure 5-5, cross section and Figure 5-6, elevation views. A new 24-in steel casing (with 12-in DIP carrier pipe) shall be jacked under the existing culvert at a 12’-8” lateral offset from the existing sewer (12-in sanitary in 24-in casing) on the north side of the street. Sewer flow shall be diverted from the existing to the new sewer prior to removing the existing line. The new sanitary sewer’s casing is designed to accommodate the final dead load and uplift condition as a free span underground structure supported on piles. The new sanitary sewer shall be maintained and protected during bridge construction. There is no sanitary sewer under West Commercial Ave within the project limits. Carlstadt Sewerage Authority and Borough of Moonachie will regulate, review, and approve relocation of sanitary sewers.

5.4.1.2. Amor Avenue Storm Sewer Outfall Modifications

Amor Ave: the existing 24-in storm sewer on the west side of the channel will be removed and cut back and replaced with a new 24-in DIP storm sewer and a new manhole on the existing storm connecting to the west abutment wall, discharging to the channel. The 30-in storm sewer on the east side will be removed and relocated with a new 30-in DIP storm sewer with a new manhole on the existing storm sewer connecting to a concrete headwall on the south side of Amor Ave bridge discharging to a riprap apron on the channel east embankment. See Figure 5-5, cross section and Figure 5-6, elevation views.

5.4.1.3. West Commercial Avenue New Storm Sewer Outfall

West Commercial Ave: A new 12-in DIP storm sewer will be installed in the center of the roadway to drain the runoff from the new catch basins at the low points on the west side of the channel discharging to the channel through the west abutment wall. See Figure 5-9 cross section view.

5.4.2. Water

5.4.2.1. Amor Avenue Water Main Relocation

Water Main at Amor Ave Crossing: The existing 8-in water main shall be removed and replaced with a new 8-in insulated water main supported on the new bridge (bridge mounted water main installation) on the north side of Amor Ave due to the lack of cover over the new bridge within the street bed. The new watermain shall be connected to the 8” existing CIP on the north side of Amor Ave using mechanical joint sleeve coupling. During bridge construction a temporary 8-in watermain mounted on a temporary bridge is required to provide a continuous water service to the properties on the west side of the channel after the existing buried watermain will be removed**.** It is anticipated that the temporary bridge-mounted watermain will be in service for approximately 12 months until the permanent Amor Ave bridge is completed and the permanent watermain is installed on the permanent bridge. See Figure 5-8 elevation view. The bridge- mounted watermain will be insulated and covered with an aluminum jacket. The Amor Ave watermain will be heat traced as the watermain dead ends at the end of Amor Ave and use of water is limited by the users during winter months. The Amor Ave watermain design has been reviewed by Veolia and their comments have been addressed in the 100% design.

5.4.2.2. West Commercial Avenue Water Main Relocation

The existing 8-in diameter buried watermain under the roadway and crossing under the channel shall be removed and replaced with a new 12-in diameter DIP insulated watermain mounted on the bridge deck overhang connecting to the existing CIP on the north side of Amor Ave using mechanical joint sleeve couplings. Upsizing of the water main to 12” has been requested by Veolia during their review of 95% design. A temporary 8-in DIP shall be installed on a temporary bridge crossing the channel on the north side of West Commercial Ave to service private users on the west side of West Commercial Ave after the existing buried watermain will be removed. It is anticipated that the temporary bridge-mounted watermain to be in service for approximately 12 months until the permanent West Commercial Ave bridge is completed and the permanent watermain is installed on the permanent bridge. See Figure 5-11 elevation view.

W. Commercial Ave watermain design will be reviewed by Veolia to assure compliance with their standard.

5.4.3. Gas

Gas record drawings have been obtained through PSE&G. The current ground survey has located the actual alignment of the gas components within the construction limit. From the available information, gas line sizes vary from 6-in to 8-in diameter and are run perpendicular to the channel. At both Amor Ave and West Commercial Ave, a 6-in diameter gas line crosses over the existing roadway culvert. The 6- in gas line at trunk crossings will require temporary relocation during bridge replacement and will be installed at the final position after the new bridge will be installed, see Figure 5-7 and Figure 5-10 elevation views at both Amor Ave and West Commercial Ave bridge replacements. PSE&G gas relocation coordination shall be done by the contractor for the dead and live work that will be done PSE&G. The actual temporary relocation and permanent replacement work will be done by the contractor during and after bridge construction respectively in coordination with PSE&G and the design team. The details of gas relocation and responsible parties for this work will be determined after meeting with PSE&G.

5.4.3.1. Amor Avenue Gas Main Relocation

PSE&G reviewed the 95% design and submitted their proposed temporary and permanent gas layout including fittings and valves per Map Index 2D-49. The new 6-in diameter steel gas will be mounted on the outside of south bridge parapet using roller chair pipe supports as shown on Map Index 2D-49. During bridge construction a temporary buried 4” plastic gas pipe and a temporary 4” steel gas pipe mounted on a temporary bridge crossing the channel on the south side of Amor Ave to provide continuous service to the abutters on the west side will be installed as shown on Map Index 2D-49. It is anticipated that the temporary bridge mounted gas to be in service for approximately 12 months until the permanent Amor Ave bridge is completed, and the permanent gas is installed on the permanent bridge. See Figure 5-7 elevation view.

5.4.3.2. West Commercial Avenue Gas Main Relocation

PSE&G reviewed the 95% design and submitted their proposed temporary and permanent gas layout including fittings and valves per Map Index 2C-50. The new 6-in diameter steel gas will be mounted on the outside of south bridge parapet using roller chair pipe supports as shown on Map Index 2C-50. During bridge construction a temporary buried 4” plastic gas pipe and a temporary 4” steel gas pipe mounted on the temporary bridge will be installed per Map Index 2C-50. The temporary 4” gas pipe crossing the channel on the south side of West Commercial Ave will provide continuous service to the abutters on the west side. It is anticipated that the temporary bridge mounted gas to be in service for approximately 12 months until the permanent West Commercial Ave bridge is completed, and the permanent gas is installed on the permanent bridge See Figure 5-10 elevation view.

5.4.4. Electric

Several light poles for private businesses will require relocation by the contractor to accommodate the channel widening. The location and design of these are shown on the plans. All utility pole relocation and associated overhead wire work will be done by PSE&G.

5.4.5. Utility Poles and Overhead Electrical, Communication and Cable TV Utilities

Utility poles, in general, are located along the channel. The utility overhead lines run parallel, oblique, or perpendicular to the channel. Utility poles also run along Amor Ave and West Commercial Ave. The utility poles and overhead lines in conflict with the channel widening will need to be permanently relocated (see Figure 5-5 cross section view). The final location of the utility poles needs to provide a minimum of 10-ft clearance for the PSE&G O& M access corridor, where planned, along the channel. During bridge removal and replacement activities at Amor Ave and W. Commercial Ave, the utility poles, and overhead lines in conflict with the construction work will need to be permanently relocated. The poles in conflict with permanent features such as O & M access road curb cuts will need to be permanently relocated. Low headroom construction equipment must be used to perform the channel widening to protect overhead lines in areas of conflict with construction operation. The utility pole and overhead line relocation shown on plans and typical sections will require to be approved by PSE&G per their direction. Overhead utility relocation coordination with PSE&G will be done by the contractor with input from the design team. The permanent pole relocation work along the channel will be done by PSE&G prior to construction. The interim pole relocation work at Amor Ave and West Commercial Ave and permanent replacement will be done by PSE&G in coordination with the contractor and the design team. Other PSE&G tenants such as Communication and Cable shall coordinate their relocation with PSE&G. The utility pole relocation design is in progress and will be finalized once PSE&G comments are addressed. The following assumptions were made during the electrical design:

1. Temporary electrical and communications work is to be completed before the culvert construction begins.
2. Permanent electrical and communications work to be completed after the culvert construction and temporary utilities work are completed and before channel construction begins.
3. All underground electrical utilities construction to be located within the temporary easement.
4. Overhead electrical utility poles and wiring along Amor Ave, West Commercial Ave, and Moonachie Ave span several blocks and consequently are located outside of the LOW.

Permanent easements for permanent electrical work performed by utility will be required per utility's design.

5.4.6. Telecommunications

Telecommunication tenants of PSE&G will coordinate with PSE&G for relocation of their lines.

5.4.7. Cable

Cable tenants of PSE&G will coordinate with PSE&G for relocation of their lines.

5.4.8. Storm Sewer outfalls along the channel

There are 24 known outfalls along the proposed ER project area that will be impacted by this design. There are three general outcomes for these outfalls.

Existing outfall is demolished to the limits of bank excavation, where a new storm structure is constructed, and the outfall reconstructed with a ductile iron pipe. A cast-in-place concrete headwall will be provided for all outfalls with a 12-inch diameter and larger.

Existing outfall is demolished to the limits of bank excavation, where bank reconstruction is not scheduled, a cast-in-place concrete headwall will be provided for outfalls 12-in diameter and larger. All storm sewers placed in reconstructed banks will be encased in controlled low strength material (CLSM).

At bridge abutment walls or steel bulkheads, the existing storm sewer will be demolished to the limits of the wall/bulkhead construction. Openings in the walls will be as per details in the Contract Drawings. Specific stormwater utility work in the Amor Avenue and West Commercial Avenue rights of way is described in the subsequent sections.

There are also some outfalls requiring unique approaches. Specifically, at 50 Amor Avenue six (6) roof leader outfalls to the existing channel will be truncated at the base of the building and have splash pads placed to minimize scouring impacts to the site. The existing and proposed ground will still promote positive drainage to the channel. Debris was observed at the stormwater outfall at Sta. 31+71. The outfall area will be regraded to allow for positive drainage to the channel, and the outfall will be maintained and protected during construction.

5.5. Existing Building Stability Analysis

Multiple buildings will be evaluated to verify the proposed channel work does not adversely affect the existing building foundation system. The selected buildings were chosen based on their proximity to the channel and certain sections were chosen to show how close the assumed building strip footings are to the work, them being: Sta. 26+90, Sta. 27+25, Sta. 30+50, and Sta. 31+70. The buildings' loads were calculated using a load takeoff based on images taken at the site of each selected building. There were several assumptions made regarding materials and sizes of steel due to the lack of as-built drawings for each building. Therefore, a conservative analysis was used and all the loads including, dead load, live loads, snow loads, and more were taken conservatively. Then a strip footing was assumed along the wall that was closest to the channel and using the loads calculated a footing was designed in RISA Foundation. The size the program determined was then checked against the over excavation boundaries and determined whether the construction would impact the stability of the buildings. Any deviations to assumed foundation conditions resulting from that investigation will be addressed prior to advertisement of the affected areas for construction bidding and award.

5.6. Combination Wall

A permanent combination wall (king-sheet pile) system is needed along the channel at five locations. The wall heights and tip depths of the combination wall piles vary and are summarized in a schedule in the 100% Plans. The total length of combination wall along required along the channel is 1090 LF.

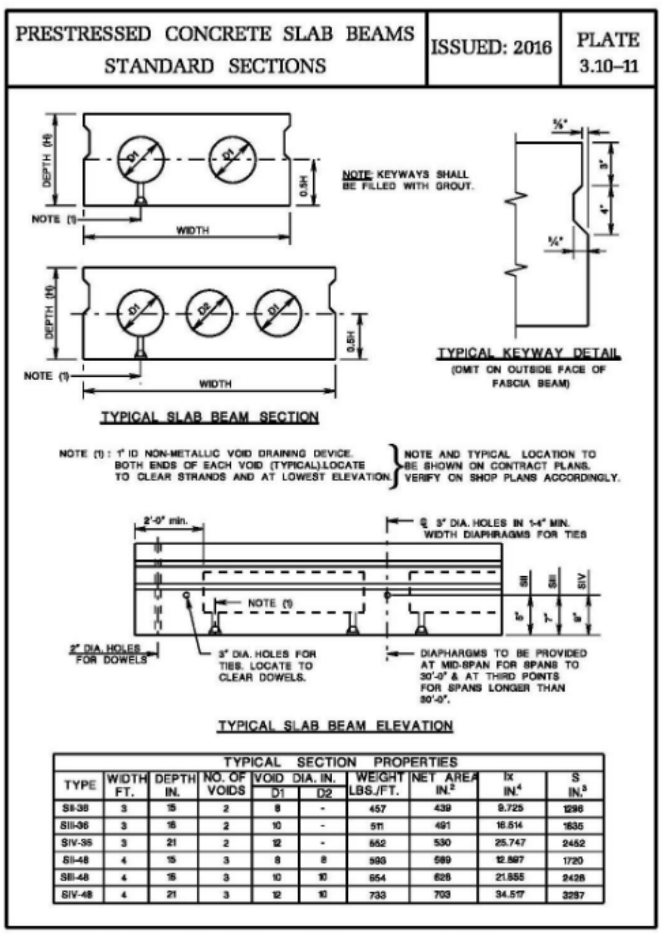
A cast in place curb barrier will be required to be installed on the top of the permanent combination wall at the locations shown on the Plans. The curb is to provide a barrier between the parking area for trucks and the improvements for ER. The barrier will be designed for the HS-20 truck loading and the reaction will need to be carried by the combination wall system.

5.7. Roadway Culvert Replacements

The two existing culverts beneath Amor Ave and West Commercial Ave along the ER will be removed and replaced to produce greater conveyance capacity in the ER channel, as demonstrated during feasibility. The structure sizes selected and modeled during feasibility factored in the channel dimensions upstream and downstream of each structure and the existing roadway elevations.

Due to cofferdam constructability considerations and to optimize hydraulic capacity, the originally proposed replacement roadway culvert structures were revised to replacement bridge structures, as reflected in the NJDEP-developed design documented for this phase of the project. The bridge structures for both Amor Ave and West Commercial Ave consist of the following:

1. A single span superstructure composed of 14 AASHTO Type SIV-48 prestressed concrete box girders (shown below in Figure 5-12) and cast in place concrete deck and approaches slabs.



TYPE	WIDTH FT.	DEPTH IN.	NO. OF VOIDS	VOID DIA. IN. D1	VOID DIA. IN. D2	WEIGHT LBS./FT.	NET AREA IN.²	Ix IN.⁴	S IN.³
SII-36	3	15	2	8	-	457	439	9.725	1296
SIII-36	3	18	2	10	-	511	491	16.514	1835
SIV-36	3	21	2	12	-	562	530	25.747	2452
SII-48	4	15	3	8	8	593	569	12.897	1720
SIII-48	4	18	3	10	10	654	628	21.855	2428
SIV-48	4	21	3	12	10	733	703	34.517	3287

NJDOT Design Manual for Bridges and Structures – 6th. Edition, 2016

Figure 5-12. NJDOT Bridges and Structures Design Manual Plate 3.10-11

2. An integral abutment substructure with wingwalls supported by nine (9) HP14x87 steel H-piles per abutment driven to bedrock.

3. The bridge deck includes sidewalks in both directions and utility support outside of the roadway profile. Refer to Section 5.5 for additional information.

5.8. Railroad Bridge Replacement

The existing railroad bridge carrying two Seeman's Lead tracks over ER will be removed and replaced to produce greater conveyance capacity in the ER channel. The design criteria governing the replacement design are summarized in Section 4.1.6.4 of this document. Recommendations for the bridge replacement design based on work performed to this point are outlined below.

To obtain approval to access the railroad right of way and construct a new railroad bridge over the ER, the bridge owner or the owner's representative will need to review and approve the new railroad bridge design. The Seeman's lead tracks and existing railroad bridge are owned by NJ Transit and leased to and operated by Norfolk Southern Railway. NJ Transit has elected to defer approval of the proposed railroad bridge to Norfolk Southern Railway, who will act as NJ Transit's representative. Norfolk Southern will review the proposed railroad bridge under their agreement with NJDEP.

5.8.1. Structure Type

The existing bridge is approximately 30 feet long (measured along the bridge superstructure). A longer replacement bridge will be needed to provide an adequate hydraulic opening. Different structure types were investigated to determine the recommended replacement alternative. The project team met with NJ TRANSIT engineering staff on April 10, 2019, to discuss preferred bridge replacement types. In the meeting, the possibility of leaving the existing bridge in place and installing an additional culvert in a new supplementary channel was discussed. Due to the complexity and cost of constructing a new culvert under existing tracks in the constrained project area, NJ Transit prefers installation of a new structure at the existing bridge location. Per NJ Transit direction, only ballasted decks were considered. If structure depth becomes a limiting issue, the project team will coordinate with NJ Transit and Norfolk Southern to discuss a path forward.

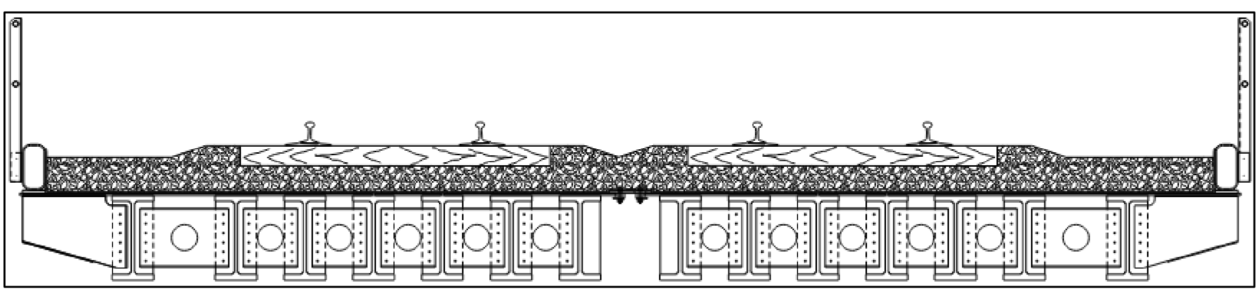


Figure 5-13. Typical Section of Ballast Deck Rolled Steel Beam Superstructure

Figure 5-13 shows the recommended superstructure replacement alternative. The span will be supported by new precast concrete abutments and driven steel H-Piles. Allowable pile loads were determined in coordination with the geotechnical team. Given the required span length to cross the new channel and the estimated existing track and design flood water surface elevations, this alternative will provide the least disruption during construction and meet NJT and NS project requirements.

5.8.2. Replacement Alternative Development

Geometry requirements of the bridge opening were determined by hydraulic analysis and consideration for the relative locations of the existing and proposed abutments. Structure depth (top of rail to low chord) was determined by structural analysis. Preliminary engineering was performed assuming no track profile modifications are allowed, in order to eliminate impacts to adjacent properties modifications to switches (turnouts) to the west and the Amor Avenue at-grade crossing to the east. Based on the required span length for the new hydraulic opening and the assumed existing top of rail elevation from previous survey, the proposed low chord will be approximately seven inches below the 100-year WSE for the proposed conditions.

The 65% design was developed by the following steps:

1. Base preferred alternative selected based on the initial discussion with NJT and NSL Ballast deck, single-span rolled beam bridge, supported by concrete adjustments and driven steel H-Piles.
2. The bridge plan geometry was laid out using the required bridge opening from hydraulic modeling. The new channel width combined with the bridge skew results in a bearing-to-bearing span length of 71"-4".
3. Proposed beams were sized using design criteria in Section 4.6.1.2 of the Detailed Design Report. The beam size for the 30% and 65% designs was W40x503.
4. 65% plans, quantities, and notes were developed based on the beam size determined in number 3 above.

During 95% design, alternative geometry was developed and the structural design finalized using the following steps:

1. Receive detailed survey including existing top of rail elevations.
2. Regress existing track profiles across the bridge.
3. Confirm low chord elevation based on actual track profile data generated in number 2 above.
4. Compare low chord elevation to proposed flood event elevations. As proposed low chord for the 65% geometry was below the 100-year WSE, the proposed superstructure was revised to minimize structure depth and decrease the 100-year WSE elevation distance above low chord. Beam size for the 95% design was W27x301.
5. Proposed bridge was also revised to utilize square spans and abutments instead of requiring skewed construction with longer spans. Span length bearing-to-bearing for 95% was 53'-4".

If rail profile modifications became necessary, it would likely be very difficult to avoid major impacts such as the turnouts to the west and the grade crossing to the east. The existing top of rail profile will be verified by survey, but record drawings show it as a crest curve over the bridge with 1% approach slopes. It is unlikely that NJ Transit and NS will accept grades higher than 1% on a freight spur. This would require any increase in the top of rail elevation to run out much further than if higher grades were allowed. Vertical curves are not allowed in the limits of turnouts, further complicating any profile modifications. See Figure 5-14.

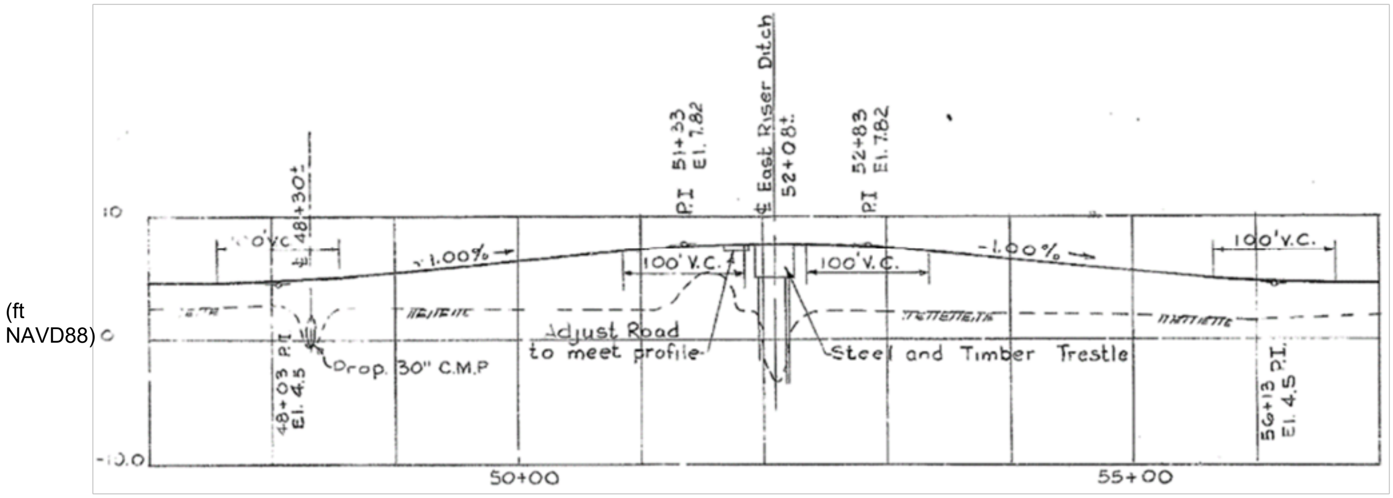


Figure 5-14. Top of Rail Profile across ER from Record Drawings

Based on the 2019 survey, the base of rail elevation across the ER bridge is 6.28 referenced to the modern NAVD88 datum. All elevations for the design of the new bridge across ER utilize the NAVD88 datum. The record drawings of the existing ER railroad bridge were used for information on the existing bridge configuration only. Elevations shown on the record drawings are referenced to an outdated elevation datum from the mid-1900s and, as a result, do not directly correlate with the modern survey. Any elevations on the existing bridge that need to be included in the design drawings for the new bridge were computed from 2019 survey points in the NAVD88 datum.

The following miscellaneous geometry criteria were used for sizing and the design of the replacement bridge:

- 136 RE running rail
- Timber ties
- 9” minimum ballast depth below ties
- ½” minimum steel deck thickness
- Track Spacing – Match existing and verify the proposed structure could support the load of a future track spacing of 14’ centered on the bridge
- Clearances – per Figure 5-15 below and per the NS Public Projects Manual underpass grade separation criteria. Increase horizontal dimensions of clearance diagram 1½” per degree of track curvature.

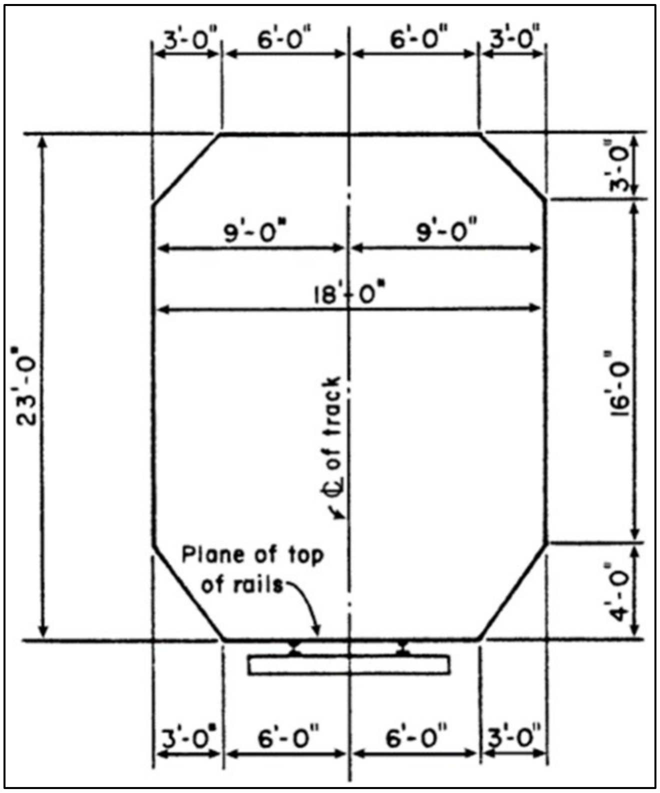


Figure 5-15. Minimum Clearances for Tangent Track

5.9. Geotechnical Design

The geotechnical considerations for the ER project components are informed by the results of subsurface investigations conducted in January 2020. Survey data collected in November 2019 were used to develop cross sections for slope stability evaluations. Three sections of channel slope stability geometries were analyzed and the results are summarized in Section 5.10.5.1. Deep foundation design was completed for the road crossing bridges and the NJT rail bridge. Selective stability evaluations are reported in the following sections.

Based on the geotechnical investigation results documented in the Rebuild by Design – Meadowlands Flood Protection Project Geotechnical Investigation Results Report (January 2020), provided in Attachment B-3C: Geotechnical Design, subsurface conditions indicate low bearing capacity and a high potential for settlement. As a result, deep foundations extending to bedrock are anticipated to be required to support proposed structures. These structures include the New Jersey Transit (NJT) rail bridge along Amor Avenue and roadway crossing bridges.

The geotechnical investigation report includes depths to bedrock at each evaluated location, generalized soil profiles, and soil parameters derived from laboratory testing, published correlations, and established reference values. This information is used to inform the foundation type considerations for the ER Project.

5.9.1. Site Topography and Geologic Conditions

The project site is within the Boroughs of Moonachie and Carlstadt, in the southern portion of Bergen County, New Jersey. Topography in the project site area can be described as primarily flat with elevations ranging from 2.1 to 11.4 feet above mean sea level (MSL, NAVD88 = MSL – 0.18 feet). The southern portion of Bergen County is relatively low and flat, making it susceptible to flooding. The bedrock consists of siliciclastic sedimentary rocks of Lower Jurassic and Upper Triassic age, mapped as Sandy Mudstone and Sandstone, both units of the Passaic Formation.

A custom soil resource report obtained from the United States Department of Agriculture (USDA), Natural Resources Conservation Service (NCRS) indicates the surficial soil deposits within approximately 36 inches of ground surface are urban land complex soils (fills). They are primarily sandy soils placed during construction and development of the area.

Overlying the bedrock, the native soils consist of lake-bottom sediments and minor estuarine, alluvial and till deposits deposited during the Quaternary period.

According to the Geotechnical Design (Attachment B-3C), the sandy mudstone is reddish-brown to brownish-red, massive, silty to sandy mudstone and siltstone, which are bioturbated, ripple cross-laminated and interbedded with lenticular sandstone. Sandstone is interbedded grayish-red to brownish-red, medium- to fine-grained, medium- to thickbedded sandstone and brownish- to purplish-red coarse-grained siltstone; this unit is planar to ripple crosslaminated, fissile, locally calcareous, containing desiccation cracks and root casts (Drake et al., 1996).

5.9.2. Subsurface Explorations

The subsurface exploration for the project consisted of drilling 12 Standard Penetration Test (SPT) sample borings, designated herein as Boring BA-01, BA-02, BA-03, BA-04, BA-05, BA-06, BB-01, BB-02, BB-03, BC-01, BC-02, and BC-04. Borings were advanced along the alignment of the channel and at pertinent structures along the alignment. Rock core samples were collected within Borings BA-01, BA-03, BA-06, BB-01, BC-02, and BC-04. The typed boring logs and boring location map are included in Attachment B-3C.

The boring locations and elevations were determined by surveying conducted by Tectonic Inc. Table 5-1 provides a summary of the estimated latitude, longitude, elevations, and pertinent depths of the borings at the project site in ft Above Mean Sea Level (AMSL), NAVD88 = MSL – 0.18 feet.

Table 5-1. Boring Summary

Boring ID	Northing	Easting	Surface Elevation (feet AMSL)	Depth to Groundwater Table[1] (feet)	Depth to Refusal[2] (feet)	Depth to Bedrock[3] (feet)	Total Depth of Boring (feet)	Bottom of Boring Elevation (feet AMSL)
BA-01	728416	610040	3.7			90.0	100.0	-96.3
BA-02 (OW)	728424	610161	6.2	7.3	80.0		80.0	-73.8
BA-03	728441	610257	11.4			80.0	90.0	-78.6
BA-04	728453	610310	4.4		70.0		70.0	-65.6
BA-05	728398	610319	6.9		75.5		75.5	-68.1
BA-06	728444	610387	4.1			75.0	85.0	-80.9
BB-01	728632	611421	5.0			55.0	65.0	-60.0
BB-02 (OW)	728604	611465	4.8	6.9	50.0		50.0	-45.2
BB-03	728225	611210	2.1		55.0		55.0	-52.9

Boring ID	Northing	Easting	Surface Elevation (feet AMSL)	Depth to Groundwater Table[1] (feet)	Depth to Refusal[2] (feet)	Depth to Bedrock[3] (feet)	Total Depth of Boring (feet)	Bottom of Boring Elevation (feet AMSL)
BC-01	729475	612146	4.0		45.8		45.8	-41.8
BC-02 (OW)	729419	612222	5.7	5.9		45.0	55.0	-49.3
BC-04 (OW)	729033	611763	6.3	7.6		55.0	65.0	-58.7

[1] Groundwater only measured in borings that included observation wells
[2] Refusal depth defined as reaching spoon refusal of greater than 100 blows in less than 3” of advancement
[3] Depth to bedrock indicates the depth at which rock coring began, rock coring was performed for a total of 10’

The presence of groundwater could not be recorded during drilling operations due to the mud rotary drilling method. A total of four observation wells were installed for the borings along the ER channel. Observation wells were installed in Borings BA-02, BB-02, BC-02, and BC-04. Drilling and sampling techniques were accomplished generally in accordance with the American Society for Testing and Materials (ASTM) procedures.

5.9.2.1. Laboratory Testing and Results

Field boring logs were reviewed to estimate the depth and thickness of the soil strata. A laboratory testing program was developed to evaluate the engineering properties of the recovered samples and to substantiate the soil classifications determined in the field. The following laboratory tests were performed on selected soil and rock samples: Moisture Content, Atterberg Limits (plasticity), Grain Size (Sieve) Analysis, Acid Producing Soil Testing, Consolidation Testing, Unconsolidated-Undrained Triaxial Testing, Unconfined Compression Testing of rock, and Splitting (Brazilian) Tensile Testing of rock. Laboratory tests were conducted in accordance with the American Society of Testing and Materials (ASTM) test procedures applicable at the time of testing. Laboratory test results are presented in Attachment B-3C.

5.9.2.1.1. Undisturbed Soil Samples

UU Triaxial Testing

Unconsolidated-Undrained (UU) triaxial compressive strength testing was performed on seven cohesive soil samples to provide information for estimating total-stress shear strength parameters. The compressive strength value for the thin-walled samples tested ranged from 440 pounds per square foot (psf) to 1,400 psf, with an average of 809 psf. The compressive strength can be utilized to estimate the cohesion of a fine-grained soil. Table 5-2 summarizes the results of the compressive strength testing performed on the undisturbed soil specimens.

Table 5-2. Summary of UU Triaxial Tests on Soil

Boring ID	Northing	Easting	Sample Depth Interval, (feet)	Test Type(a)	Dry Unit Weight (pcf)	Moisture Content (%)	Unconfined Compressive Strength (psf)	Undrained Shear Strength (psf)
BA-02	728424	610161	22.0 - 24.0	UU	71.6	50.2	1,400	700
BA-04	728453	610310	52.0 - 54.0	UU	92.0	30.8	960	480
BA-05	728398	610319	37.0 - 39.0	UU	72.0	50.7	440	220

Boring ID	Northing	Easting	Sample Depth Interval, (feet)	Test Type[(a)]	Dry Unit Weight (pcf)	Moisture Content (%)	Unconfined Compressive Strength (psf)	Undrained Shear Strength (psf)
BA-06	728444	610387	27.0 - 29.0	UU	76.3	45.3	560	280
BB-02	728604	611465	25.0 -27.0	UU	75.2	47.1	780	390
BC-01	729475	612146	30.0 - 32.0	UU	72.6	49.1	600	300
BC-04	729033	611763	22.0 - 24.0	UU	77.8	40.6	920	460

(a) UU represents Unconsolidated-Undrained triaxial compressive strength testing

One-Dimensional Consolidation Testing

One-dimensional consolidation testing was also performed on three thin-wall samples collected at varying depths. The results of the consolidation tests can be utilized to determine consolidation settlements of compressible soil strata. Table 5-3 summarizes the results of the one-dimensional consolidation testing performed on the undisturbed soil specimens.

Table 5-3. Summary of One-Dimensional Consolidation Tests

Boring ID	Sample Depth Interval, (feet)	USCS Class	Void Ratio, eo	Recompress, Index, Cr	Compress, Index, Cc	Preconsol, Pressure, Pc (tsf)	Over-consolidation Ratio[1], OCR
BA-01	32.0 - 34.0	CL	1.235	0.13	0.34	0.6	1.0
BA-03	42.0 - 44.0	CL	1.014	0.25	0.69	1.8	1.1
BC-04	22.0 - 24.0	CL	1.038	0.23	0.65	1.8	2.0

OCR = Pc / Pc, where Pc is the vertical effective stress. A unit weight of 120 pcf was assumed in the estimation of Pc.
Note: For Boring ID BA-01, the over-consolidation ratio (OCR) value was changed from the 0.5 value provided in the design document prepared by NJDEP to a value of 1.0, based on comments from USACE.

5.9.2.1.2. Summary of Rock Core Testing

Unconfined Compressive Strength Testing

One uniaxial compressive strength test was performed on a rock core sample from Boring BA-06, with compressive strength values testing at 6,011 pounds per square inch (psi). Table 5-4 summarizes the results of the uniaxial compressive strength testing performed on the intact rock core sample.

Table 5-4. Summary of Uniaxial Compressive Strength of Intact Rock

Boring ID	Northing	Easting	Sample Depth (feet)	Uniaxial Compressive Strength (psi)
BA-06	728416	610040	75.0 — 80.0	6,011

Indirect Tensile (Brazilian) Testing

Indirect Tensile (Brazilian) Testing was performed on 3 rock core samples with tensile strength values testing at 1,045 pounds per square inch (psi) to 1,284 psi, with an average of 1,149 psi. Table 5-5 summarizes the results of the Indirect Tensile testing performed on intact rock core samples.

Table 5-5. Summary of Brazilian Tests

Boring ID	Northing	Easting	Sample Depth (feet)	Tensile Strength (psi)
BA-3	728441	610257	80.0 - 85.0	1,118
BA-6	728444	610387	75.0 - 80.0	1,284
BC-4	729033	611763	55.0 - 60.0	1,045

5.9.3. Subsurface Conditions

The drilling and sampling operations performed at the project site indicate the subsurface materials along the channel alignment and structures consist of soil deposits ranging from 45.8 feet to 90.0 feet in thickness. In general, the subsurface materials observed during drilling operations consisted of granular fill material overlying silty clay, fat clay, silt, silty sand, and organic silt, which overlies decomposed rock extending to mudstone bedrock. Drilling operations in the area suggest the following stratification:

Fill Material (SM, CL-ML, SP) was encountered at the existing ground surface and extended to a depth ranging from 5.0 feet to 13.5 feet below existing ground surface. The material was described as brown, very dense to loose, medium to fine grained sand, trace to some silt, little asphalt, little brick and concrete, little wood, and trace roots. Laboratory testing was not performed extensively on this layer. Testing indicated this layer to be non-plastic. The average uncorrected field blow counts and in-situ moisture content for this soil layer was 15 blows per foot (bpf) and 19.8 percent. The SPT blow counts indicate a medium stiff density.

Organic Clay, Silt, Organic Silt, Clay, and Silt (OL, ML, CL) was encountered at depths ranging from 5.0 feet to 6.0 feet and extended to a depth of 13.5 feet to 18.0 feet below the existing ground surface within Boring BA-02 and BA-04. The material was described as gray to brown, soft to stiff, organic silt, clay and silt, and silt. Laboratory testing indicates this material ranges from non-plastic to medium plastic. The average uncorrected field blow counts and in-situ moisture content for this soil was 7 bpf and 32.5 percent. The SPT blow counts indicate medium stiff consistency.

Clay and Silt, Fat Clay, Elastic Silt, Silt (CL, CH, MH, ML) was encountered at depths ranging from 5.0 feet to 13.5 feet and extended to depths ranging from 33.5 feet to 73.0 feet below existing ground surface. The material was described as brown to gray, clay and silt, trace fine sand; brown to gray, silty fat clay; gray, clayey silt; and gray clayey silt. Laboratory testing indicates this material is a medium plastic material with an average plasticity index of approximately 16 percent. The average uncorrected field blow counts and in-situ moisture content for this soil layer was 2 bpf and 38.3 percent. The SPT blow counts indicate a very soft consistency.

Silty Sand, Clayey Sand, Silty Clay (SM, SC, CL) was encountered at depths ranging from 44.0 feet to 73.0 feet and extended to a depth ranging from 50.0 feet to 80.0 feet below existing ground surface. The material was described as brown to red, dense, fine to coarse sand, some silt, some gravel, some clay; and brown to red, hard, silty clay, some gravel, trace sand. The average uncorrected field blow counts for this soil layer was 40 bpf. Laboratory testing indicates this material is non-plastic. The SPT blow counts indicate a dense relative density.

Clayey Gravel, Poorly Graded Gravel, Clayey Silty Sand (GC, GP, GC-GM, SC-SM) was encountered at depths ranging from 38.5 feet to 80.0 feet and extended to a depth ranging from

42.0 feet to 87.0 feet below existing ground surface. The material was described as brown to red, very dense gravel, some silt, some sand, some clay. The average uncorrected field blow counts for this soil layer were 85 bpf. Laboratory testing indicates this material is non-plastic. The SPT blow counts indicate a very dense relative density.

Bedrock was encountered at depths ranging from 42.0 feet to 87.0 feet below existing ground surface, where split spoon refusal was experienced. Advanced bedrock coring tools recovered mudstone. The mudstone was described as brown, fine grained, moderately soft to moderately hard, moderately to slightly weathered, mudstone. Core recovery ranged from 49 to 100 percent, while core RQD values ranged from 22 to 80 percent.

Groundwater was observed in four installed observation wells along the alignment of the channel. The average groundwater elevation observed was 6.3 feet below the existing ground surface. Fluctuations in the level of the groundwater may occur due to seasonal variations in precipitation and other factors not evident at the time of measurement.

5.9.4. Geological Hazards

The following paragraphs provide an assessment of potential geologic hazards in the vicinity of the project.

5.9.4.1. Faults/Seismic Activity

Faults were not encountered during the subsurface investigation.

5.9.4.2. Soil Corrosion Potential

The risk of corrosion of concrete and steel was considered for soils within the proposed project site. According to a customized soil report for the site from the United States Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS), the site is rated as a high risk for corrosion of both concrete and uncoated steel. Selected soil samples were tested for acid producing potential, in accordance with NJDEP NJAC 7:13, and are summarized in Table 5-6.

Table 5-6. Summary of Acid Producing Soils

Boring ID	Depth	pH	Sulfate Ions	pH	Sulfate
		Air Dried		Oxidized	
BA-2	10-12	5.8	NO	6.8	NO
BA-4	15-17	6.4	NO	6.6	NO

Corrosivity testing is recommended prior to the commencement of construction to determine the actual potential for soil induced corrosion.

5.9.5. Geological Design Considerations

Geotechnical engineering analyses and recommendations are included herein for the ER and appurtenant structures, which includes deep foundation and retaining wall design. In addition, the results of slope stability analyses are provided below for typical sections discussed in Section 5.1. Considerations are also provided for the geolift, or vegetated retaining wall.

5.9.5.1. East Riser Design

5.9.5.1.1. Slope Stability Analyses

Selected cross sections for the ER have been evaluated for short-term, long-term, and seismic slope stability utilizing the GeoStudio SLOPE/W (2019) computer program. The SLOPE/W (2019) computer program utilizes a rotational failure surface and calculates the factor of safety based on the method selected. For this project, the Spencer Method was selected. A summary of the analyses performed, and the results are provided in the following paragraphs.

Slope stability evaluations have been completed for critical sections, designated in this section as #1, #2 and #3. Minor modifications, further discussed herein, were considered if the sections failed to satisfy design criteria. Loading conditions evaluated included long-term steady-state, rapid drawdown, seismic, and end-of-construction scenarios. The results of the analyses were evaluated against the stability factors of safety for each of the loading conditions defined on Table 4-2, Section 4.1.4.1. Two separate surcharge loads, a semi-truck and single-wide manufactured home, were assumed for the steady-state and rapid drawdown conditions based on anticipated loading conditions at various locations along the ER alignment. For the end-of-construction scenario, both a low pressure, long-reach track-hoe and semi- truck were the assumed surcharge loads Live load surcharges were not considered for the seismic conditions. A summary of the surcharge loads is provided below:

- Semi-truck (18-wheeler): 430 pounds per square (foot), derived from the HL-93 Design Truck presented in AASHTO LRFD 3.6.1.2.2
- Single-wide manufactured home: 30 psf, derived from loads provided by the U.S. Department of Housing and Urban Development (HUD) for manufactured homes
- Low pressure, long-reach track-hoe: 288 psf, derived from conventional-track hoe equipped with a low ground pressure undercarriage

The potential for a loaded dump truck to be situated near the channel hinge point exists as part of the long- term maintenance of the channel improvements. As such, the semi-truck surcharge load was considered across the entire alignment.

Long-term steady-state, seismic, and end-of-construction scenarios were evaluated for slope stability at the ordinary high and low water hydraulic loading conditions. The design water elevations utilized in the slope stability analysis for long-term steady-state, seismic, rapid drawdown, and end-of-construction scenarios are provided below (NAVD88 = MSL – 0.18 feet).

- Ordinary High Water: -0.2 feet, MSL;
- Ordinary Low Water: -2.5 feet, MSL;
- Rapid-Drawdown: 3.07 feet, MSL (Before Drawdown) to -2.69 feet, MSL (After Drawdown).

The results of the slope stability analyses were used to establish long-term and short-term surcharge offsets from the channel hinges and were evaluated for consistency with the applicable stability criteria presented in the USACE design guidance.

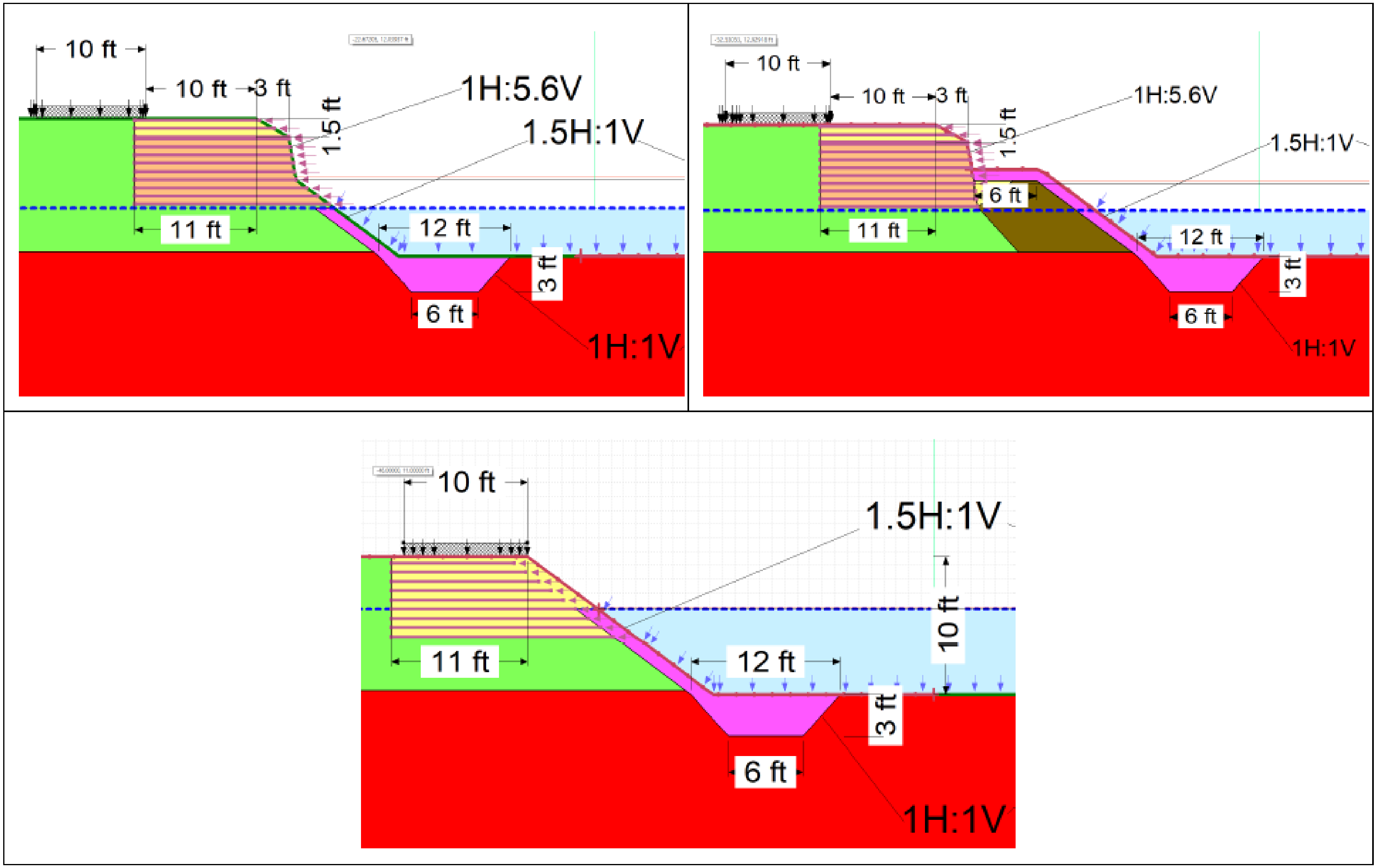


Figure 5-16. Critical Section #1 – Geolift (Top Left), Geolift with Wetland Shelf (Top Right), and Simple Reinforced Slope (Bottom)

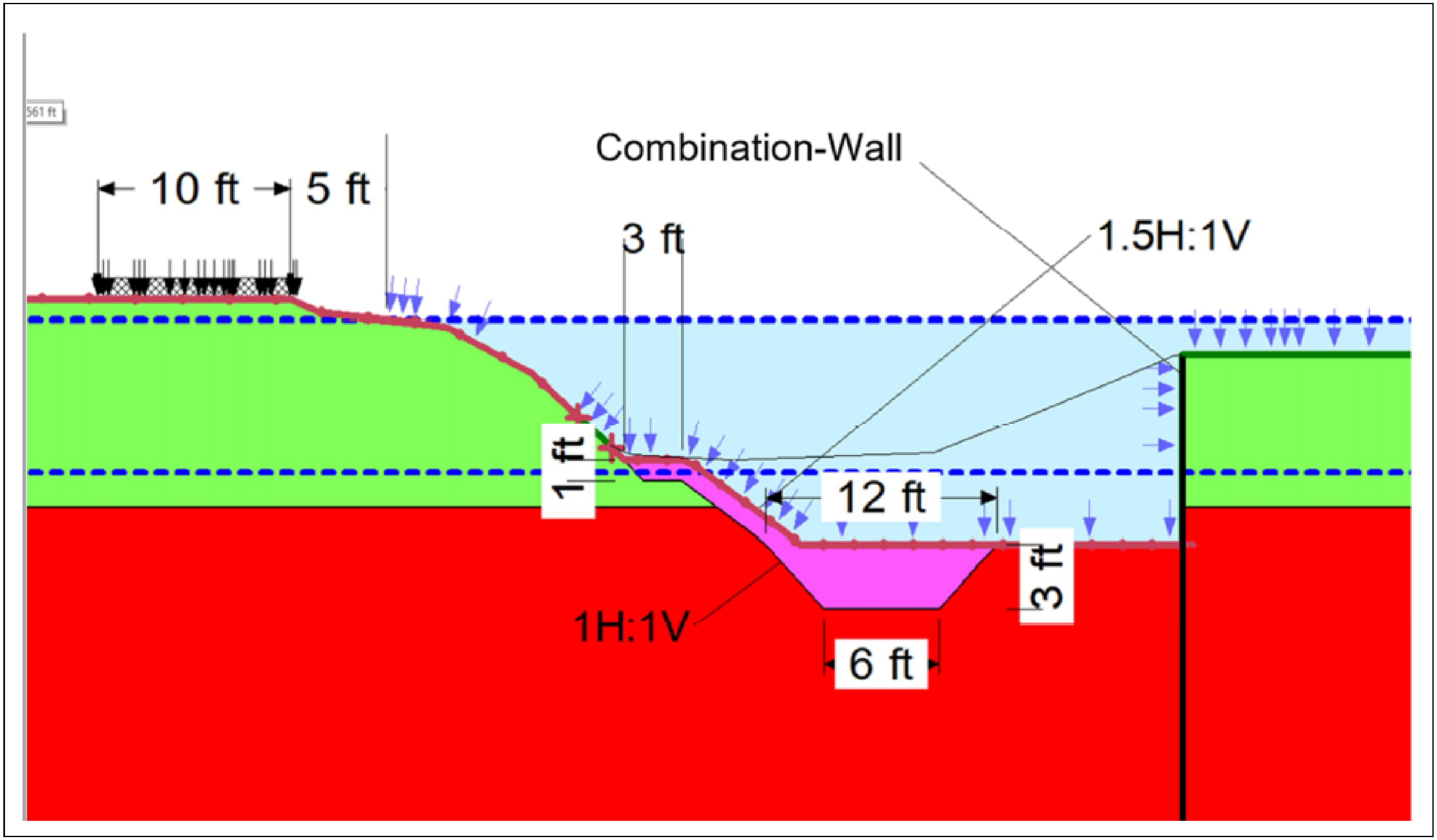


Figure 5-17. Critical Section #2 – Loomis Property

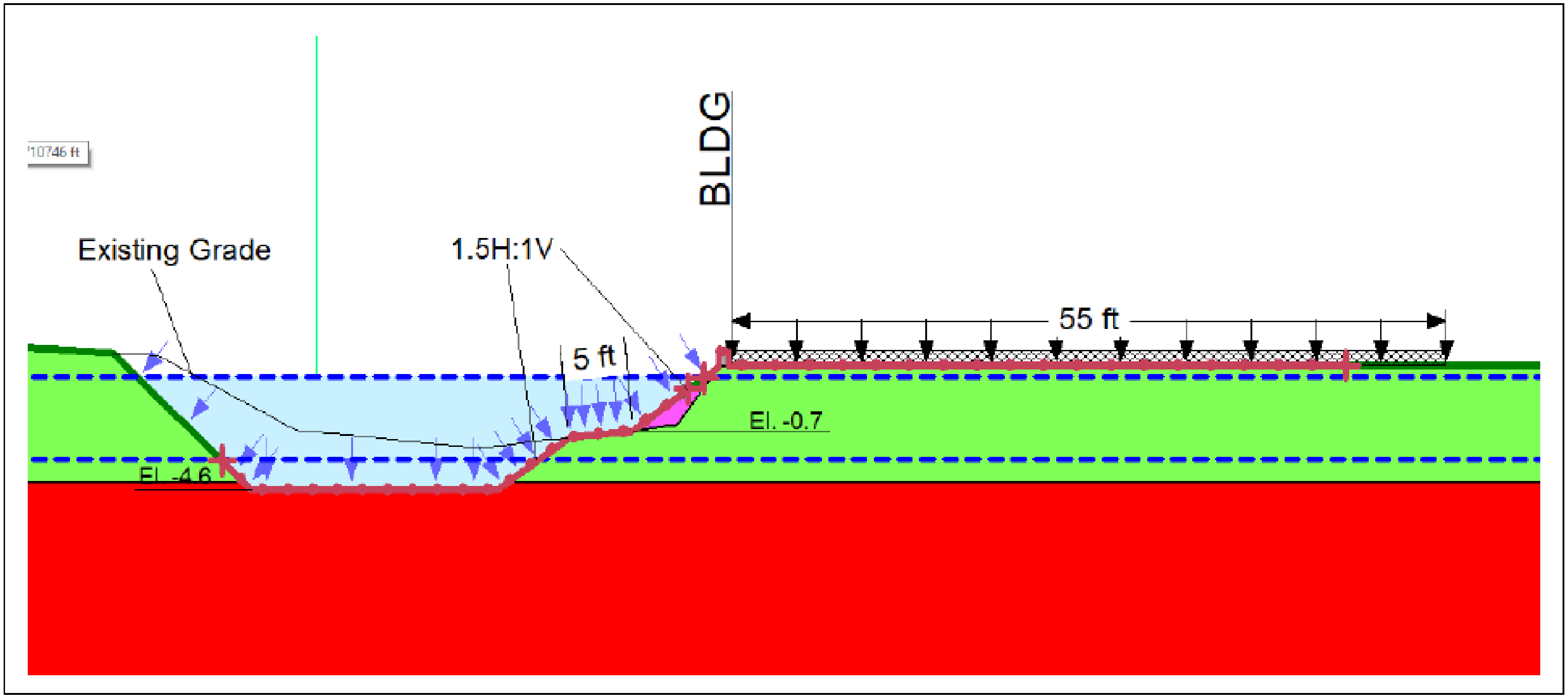


Figure 5-18. Critical Section #3 – Manufactured Home

Slope stability factors of safety were compared to USACE guidance provided for new slopes included in USACE EM 1110-2-1902. Results of the stability analysis are included below.

Table 5-7. Summary of Slope Stability Analyses

Critical Section	Description	Geometry	Loading Condition	Hydraulic Loading	Factor of Safety	
					Required[1]	Calculated
1	Simple Reinforced Slope[2]	1.5H:1.0V w/ Rip Rap Slope Protection & 3' Embedded Toe	Long Term	Ordinary High Water	1.5	1.6
				Ordinary Low Water	1.5	1.5
			Rapid Draw Down	Drawdown	1.2	1.3
			Short Term (Construction)	Ordinary High Water	1.3	1.4
				Ordinary Low Water	1.3	1.3
			Pseudostatic (Seismic)	Ordinary High Water	1.1	1.4
				Ordinary Low Water	1.1	1.3
	Geolift[2]	Geolift w/ 1.5H:1.0V Rip Rap Slope Protection & 3' Embedded Toe	Long Term	Ordinary High Water	1.5	1.5
				Ordinary Low Water	1.5	1.5
			Rapid Draw Down	Drawdown	1.2	1.2
			Short Term (Construction)[3]	Ordinary High Water	1.3	1.4
				Ordinary Low Water	1.3	1.3
			Interim Construction[4]	Ordinary High Water	1.3	1.6
				Ordinary Low Water	1.3	1.4
			Pseudostatic (Seismic)	Ordinary High Water	1.1	1.2
				Ordinary Low Water	1.1	1.1
	Geolift w/ Westland Shelf[2]	Geolift w/ 1.5H:1.0V Rip Rap Slope Protection, 6' Wetland Shelf & 3' Embedded Toe	Long Term	Ordinary High Water	1.5	1.8
				Ordinary Low Water	1.5	1.8
			Rapid Draw Down	Drawdown	1.2	1.4
			Short Term (Construction)[3]	Ordinary High Water	1.3	1.4
				Ordinary Low Water	1.3	1.3
			Interim Construction[4]	Ordinary High Water	1.1	1.3
				Ordinary Low Water	1.1	1.2
2	Loomis	3' Bench w/ 1.5H:1.0V Rip Rap Slope Protection & 3' Embedded Toe	Long Term	Ordinary High Water	1.5	1.5
				Ordinary Low Water	1.5	1.5
			Rapid Draw Down	Drawdown	1.2	1.3
			Short Term (Construction)	Ordinary High Water	1.3	1.4
				Ordinary Low Water	1.3	1.3
			Pseudostatic (Seismic)	Ordinary High Water	1.1	1.1
				Ordinary Low Water	1.1	1.2

Critical Section	Description	Geometry	Loading Condition	Hydraulic Loading	Factor of Safety	
					Required[1]	Calculated
3	Manufactured Home	5' Bench w/ 1.5H:1.0V Side Slopes - Rip Rap Protection above Bench Only	Long Term	Ordinary High Water	1.5	1.5
				Ordinary Low Water	1.5	1.6
			Rapid Draw Down	Drawdown	1.2	1.5
			Short Term (Construction)	Ordinary High Water	1.3	1.3
				Ordinary Low Water	1.3	1.3
			Pseudostatic (Seismic)	Ordinary High Water	1.1	1.2
				Ordinary Low Water	1.1	1.3

1 Factors of safety from USACE EM 1110-2-1902 for new slopes.
2 Includes biaxial geogrid spaced 8-inches vertically, extending 11-feet laterally from the channel hinge.
3 Additional guidance for equipment surcharge offsets provided in subsequent subsection.
4 Interim case considers open excavation for the embedded toe with a low pressure, long-reach track-hoe situated at the channel hinge near the ordinary high water elevation.

As indicated in the table above, the factor of safety requirements for slope stability were satisfied for each of the loading conditions and ER geometries considered.

5.9.5.1.2. Settlement of Geolifts

The ER channel improvements generally consist of increasing channel conveyance by widening and deepening the existing alignment. As a result, the new reinforced geolifts will be mostly constructed in cut sections. Often when construction occurs within the limits of a cut, settlement is negligible due to stress compensation where the increase in stress from new construction is offset by the weight of the cut material removed. However, given the presence of a soft, compressible clay strata along the alignment and a marginal increase in unit weight of the select granular fill of the geolifts relative to the foundation soils, a settlement analysis was performed to estimate the magnitude of settlement that may be expected along the channel alignment. Both consolidation settlements of fine-grained, cohesive soils and immediate settlements of granular, coarse-grained soils were estimated. However, most of the foundation settlement is expected to be long-term consolidation settlement of fine-grained soils given the majority of the overburden material is fine-grained. Immediate settlements of coarse-grained soils are expected to be negligible and occur mostly over the duration of construction.

Settlement parameters were generally derived from data collected during the subsurface investigation and laboratory testing, which included one-dimensional consolidation tests. As previously mentioned, the results of the subsurface investigation indicate that foundation soils consist primarily of a very soft lean clay strata with a maximum observed thickness of 63 feet at Boring BA-01. This stratum is generally underlain by thin granular glacial deposits and/or weathered bedrock extending to competent mudstone bedrock. The depth to bedrock and thickness of the soft clay strata decreases upstream along the channel alignment.

Settlement of the proposed geolifts was evaluated for the subsurface conditions encountered at the critical boring, BA-01, which penetrated the thickest compressible soil layer. The analysis was based on a top and bottom geolift elevation of -2.5 ft, MSL and 8 ft, MSL (NAVD88 = MSL – 0.18 feet), respectively. The settlement analysis was performed using Settle3 (Version 5.018) developed by Rocscience. The results of the settlement analysis indicate up to 1-inch of total settlement may occur along the channel alignment. Settlements are expected to decrease up-station where the thickness of the compressible soil strata is thinner. Additionally, some foundation heave may occur when excavating the existing channel to the required grades. The magnitude of heave will largely depend on the time at which the temporary excavations are left open. Some minor regrading and foundation preparation will likely be necessary to ensure the first geolift starts at the scheduled elevation of -2.5 ft, MSL. Methods to mitigate foundation heave may include reducing the time between excavation and fill placement and/or dewatering.

Long-term, consolidation settlements are expected which occur after construction and over the lifetime of the structures. Immediate settlements typically occur during construction. While minimal, some of the immediate settlements may occur during construction, however, portions of the immediate settlements may not be realized until once the structure is fully constructed.

Surcharge Offsets and Waiting Periods

The results of the slope stability analyses were utilized to establish allowable surcharge offsets, from the channel hinge, at various locations along the proposed ER alignment. In some instances, the allowable surcharge offsets can decrease after a waiting period following the completion of construction. A waiting period allows for pore pressure dissipation, and thus, strength gain in the underlying foundation soil. Waiting periods were estimated using the time to 90% consolidation determined from one-dimensional consolidation testing (See Table 5-3). The following summarizes the allowable surcharge offsets from the channel hinge point, and where applicable, the waiting periods required to decrease surcharge offsets.

Simple Reinforced Slope (Critical Section #1) – Semi-truck surcharge (≤ 430 psf) may be situated at the channel hinge (0-ft offset) during or after construction (i.e., no waiting period required).

Geolift w/ Wetland Shelf (Critical Section #1) – Semi-truck surcharge (≤ 430 psf) shall be located at least 10-feet from the channel hinge for 3 to 6 months after the end of construction, at which point the offset can be decreased to 5-feet. The 5-feet minimum offset must be maintained thereafter.

Geolift (Critical Section #1) – Surcharges up to 325 psf shall be located at least 10-feet from the channel hinge for 3 to 6 months after the end of construction, at which point the surcharge can be increased to 430 psf and the offset can be decreased to 5-feet. The 5-feet minimum offset must be maintained thereafter.

Manufactured Home (Critical Section #3) – Semi-truck surcharge (≤ 430 psf) may be situated at the channel hinge (0-ft offset) during construction. Semi-truck loading is not anticipated after construction because of the residential nature of this area.

5.9.5.1.3. Internal Reinforcement

Internal reinforcement consisting of biaxial geogrid was included in the slope stability analysis performed at critical section #1 (e.g., simple reinforced slope, geolift, and geolift with wetland shelf). The following geogrid configurations were considered and should be implemented at critical Section #1:

- Type: Biaxial Geogrid;
- Elevation of Bottom Layer: -2.5 feet, MSL, NAVD88 = MSL – 0.18 feet;
- Maximum Vertical Spacing: 8-inches;

- Minimum Lateral Extent from Channel Hinge: 11 feet;
- Minimum Tensile Strength: 270 lb/ft (at 2% strain).

The internal reinforcement at critical section #1 should meet the minimum strength requirements outlined above.

5.9.5.1.4. Combined-Wall Systems

Previous retaining wall analyses conducted through 95% Design were performed primarily for steel sheet pile walls. Following the 95% Design, the design basis was reassessed and determined Q-Case (total undrained stress) soil conditions should be evaluated. Further discussion is provided in Section 4.1.6.2 related to the design basis for the walls. It was determined that the Q-Case condition governs the design, and a more robust retaining wall consisting of a combined-wall system is required along the ER channel.

Combined-wall systems are utilized when standard steel sheet piling does not have the capacity to support the anticipated loading conditions. Combination walls are composed of two main components – a king pile (either beam or pipe piles) and intermediary steel sheet piles. The sheet piles transfer the lateral earth, surcharge, and water pressures to the king piles. The king piles are responsible for supporting most of the load. An evaluation was completed for steel combined-wall systems located along select reaches of the ER channel. The evaluation was performed in accordance with USACE EM 1110-2-2504 *Deign of Sheet Pile Walls* using the computer program PYWall v2022 created by Ensoft, Inc. PYWall considers the soil- structure interaction to estimate wall deflections, load demands, and ultimately the required embedment/lengths of the wall elements. The program outputs estimated lateral movements, shear and bending moments along the length of the wall. Evaluations of a combined-wall system were performed at the following locations. Note, the bank naming convention used below, right or left, is relative to the downstream direction (or down station).

- STA 0+08 to 3+00 (Left Bank)
- STA 19+50 to 25+68 (Right Bank)
- STA 19+50 to 19+96 (Left Bank)
- STA 39+53 to 41+19 (Right Bank)

In accordance with EM 1110-2-2504, each wall section was evaluated for three load cases (usual, unusual, extreme) at both drained (S-Case) and undrained (Q-Case) soil stress states. A S-Case stress state is present long after construction when excess pore pressures generated during construction have dissipated. In contrast, Q-Case conditions are present immediately after construction and result from the development of excess pore water pressures, like when soils are loaded quickly not allowing pore pressures to drain. The hydrostatic conditions for the three load cases evaluated were determined based on the anticipated fluctuation of loads over the service life of the wall. The usual load case considered ordinary high water (El. 0.0) hydrostatic conditions without external surcharge loads behind the wall. The unusual load case evaluated two hydrostatic loading conditions – balanced load at ordinary high water (El. 0.0) with vehicular surcharge loads and unbalanced loads resulting from low (El. -2.5) and ordinary high water conditions both with and without vehicular surcharge loading. The extreme load case modeled flooded conditions corresponding to the top of the wall (varies) with and without a surcharge load, where applicable.

Factors of safety were applied to the passive earth pressures for each soil stratum consistent with those presented in Table 5-1 of USACE EM 1110-2-2504. A factor of safety of 1.0 was used for active pressures. These factors of safety were used to determine the required depth of penetration of each wall section for rotational stability. Conversely, estimates for wall deflection, shear and bending moments along the length of the wall were estimated with a factor of safety of 1.0 for both active and passive pressures. Estimated deflections presented herein do not include effects from transient, live load surcharges like those from vehicular loading. Values determined for maximum shear and bending were used to confirm that section properties (section modulus, cross sectional area) satisfy allowable stresses for each load case considered using relationships from pages 6-1 to 6-4 of the EM.

That wall analysis presented herein is based on Nucor (Skyline) Steel's beam combined wall systems which includes steel wide flange beams and NZ shaped sheet piling. The design assumed the system will be composted of components with a steel yield strength of 50 ksi. The following sections provide the results of the analysis performed for each wall section and include the required embedment depth, estimated deflections, and maximum shear and bending moments.

STA 0+08 to 3+00 (Left Bank)

The subsurface profile utilized for the wall analysis performed between STA 0+08 and STA 3+00 was based on the critical subsurface boring, or Boring BA-02, advanced across the ER channel near the new pump station. Exploratory subsurface borings were not performed on the left bank of the channel near the wall footprint. The subsurface conditions encountered at this boring location are assumed to be generally representative of the conditions across this wall section.

The thickest very soft clay deposit was present at this boring with an observed thickness of approximately 55 feet. The undrained shear strength, S_u, of fine-grained soils often increases with depth (or effective stress) and is commonly expressed by the ratio S_u/p, where p represents the effective vertical stress. The ratio correlates approximately with a soil's plasticity index, PI, and over consolidation ratio. Figure 3-2 of USACE EM 1110-2-2504 provides the relationship between this ratio and the PI for normally consolidated clays. Based on this figure, a S_u/p ratio between 0.2 and 0.3 may be expected for the very soft clay stratum when using the average PI of 16 measured for this layer along the ER channel. A plot of the undrained shear strength (estimated from UU triaxial and pocket penetrometer field tests) versus depth was created to determine if the undrained shear strength varies with depth in accordance with the ratios shown in Figure 3-2 of the EM. The plot provided below includes graphs for ratios of 0.2, 0.25, and 0.3, along with the design S_u. A S_u/p ratio of 0.2 was utilized to determine the design undrained shear strength with depth based on the distribution of UU triaxial tests results. The lower ratio, 0.2, was also selected for determining design shear strengths because there was limited strength testing performed with depth and select UU strengths fell below the 0.2 plot. As shown in the figure below, an undrained shear strength ranging from 300 psf at the top of the layer to 575 psf near the base of the layer was utilized in the retaining wall analysis.

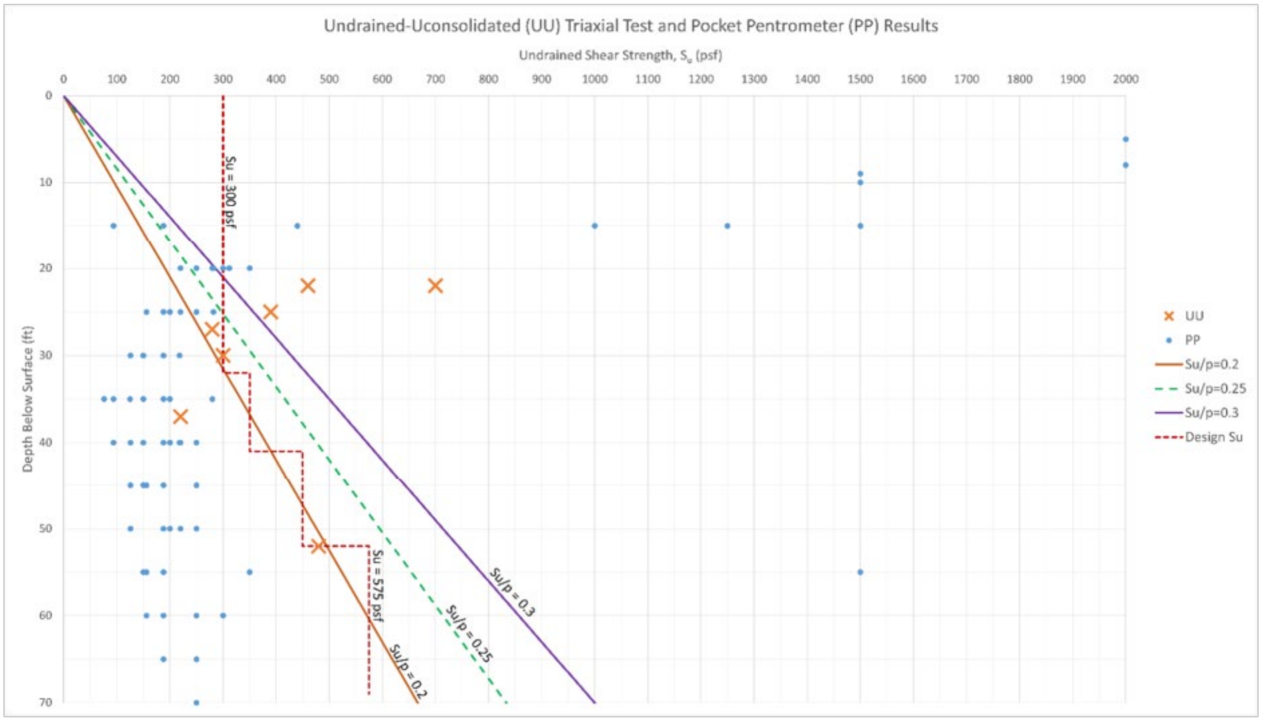


Figure 5-19. Undrained-Unconsolidated (UU) Triaxial Test and Pocket Pentrometer (PP) Results

The analysis was conducted for the maximum wall height along this wall section of 12.3 feet, occurring at STA 3+00. The results presented herein assume the retained soil height behind the wall is limited to 10.8 feet, corresponding to a minimum depth of 1.5 feet from the top of the wall. The height of retained soil behind the wall is critical in limiting wall deflections and ensuring the wall embedment satisfies rotational stability requirements. Under no circumstances shall the height of soil behind the wall exceed the conditions discussed herein and shown in the project plans without further evaluations by a Professional Engineer registered in the state of New Jersey.

External surcharge loads (e.g., vehicular loads, equipment loads) are not expected behind the wall in this area. The backslope at the critical wall height is expected to be relatively flat based on review of project cross sections, and therefore, the impacts of a sloping ground surface behind the wall were not considered. The results of the analysis performed for the retaining wall at this location is provided in Table 5-8 to Table 5-10 below, which includes the requirements for embedment and corresponding rock socket length based on estimated bedrock depths.

Table 5-8. Summary of Combined-Wall Analysis at Left Bank from STA 0+08 to 3+00

Combined-Wall System Properties[(1)]		PYWall Outputs[(2)]				Min. Embedment (ft)[(3)]	Embedment Requirements		
Combined-Wall System	Pile Spacing (in.)	Max. Moment (in- kips)	Max. Shear (kips)	Top of Wall Deflection (inch)			Estimated Depth to Bedrock (ft.)	Min. Rock Socket Length (ft.)	Estimated Total Length (ft.)
				Q- Case	S- Case				
W44x335/NZ38	78	3,042	16.8	**1.4**	**0.6**	86.0	**80.7**	**6.0**	**87.0**

Note(s):
(1) Area = 24.84 in^2/ft; Moment of Inertia = 6,112 in^4/ft; Elastic Section Modulus = 249.7 in^3/ft. Section properties satisfy allowable stresses for bending and shear in accordance with EM 1110-2-2504.
(2) FS=1.0. Deflections values shown are for the usual load case.
(3) Depth to fixity based on factors of safety recommended in the EM 1110-2-2504.

The above combined-wall system was selected based on limiting drained and undrained top of wall deflections to approximately 1-inch for the usual load case. A wall deflection marginally exceeding 1-inch was estimated for the undrained Q-Case, however, this value (1.4 inch) is within the typical range expected for normal performing retaining walls. The minimum rock socket length and total length are rounded up to the nearest foot and are based on the subsurface conditions encountered at the critical boring BA-02. It is recommended to install the intermediary sheet piles along this reach to the top of bedrock. The king piles (W-Beams) shall be socketed into competent bedrock to the minimum length shown in the table above. Bedrock was interpreted within Boring BA-02 at elevation -73.8 feet, MSL (NAVD88 = MSL – 0.18 feet). Actual subsurface conditions may differ along the proposed wall alignment as exploratory borings were not advanced on the left bank near the wall. The total length shown above is not a minimum value and represents the estimated total length. Total length values may vary where bedrock is encountered at elevations different than those assumed herein, however, the length of the rock socket shall not be less than the value shown above.

STA 19+50 to 25+68 (Right Bank) & STA 19+50 to 19+96 (Left Bank)

The subsurface profile utilized for the wall analysis performed between STA 19+50 and STA 25+68 (Right Bank) and STA 19+50 and STA 19+86 (Left Bank) was based on the critical subsurface boring, or Boring BC-02, advanced along West Commercial Avenue near the end of the wall. The subsurface conditions encountered at this boring location are assumed to be generally representative of the conditions across these wall sections. The analysis was conducted for the maximum wall height (10.7 feet) subject to a surcharge load, occurring at STA 21+10 (Right Bank). Traffic induced loads on the wall were considered due to the proximity of neighboring industry to the wall which includes a parking lot and a tractor trailer loading/unloading area. As a result, a 10-foot wide external surcharge load of 430 pounds per square feet (psf) was analyzed at a distance of 3 feet from the face of the wall. This surcharge magnitude corresponds approximately to the AASHTO HL-93 Design Truck. The ground slope behind the wall is expected to be relatively flat based on review of project cross sections, and therefore, the impacts of a sloping ground surface were not considered. The results of the analysis performed for the retaining wall at this location are provided below, which includes the requirements for embedment and corresponding rock socket length based on estimated bedrock depths.

Table 5-9. Summary of Combined-Wall Analysis at Right Bank from STA 20+58 to 25+68 and Left Bank STA 19+50 to 19+96

Combined-Wall System Properties[1]		PYWall Outputs[2]				Min. Embedment (ft)[3]	Embedment Requirements		
Combined-Wall System	Pile Spacing (in.)	Max. Moment (in- kips)	Max. Shear (kips)	Top of Wall Deflection (inch) Q- Case	Top of Wall Deflection (inch) S- Case		Estimated Depth to Bedrock (ft.)	Min. Rock Socket Length (ft.)	Estimated Total Length (ft.)
W44x335/NZ38	78	6,182	106.3	**1.3**	**0.7**	59.0	**45.0**	**14.0**	**59.0**

Note(s):
(1) Area = 24.84 in^2/ft; Moment of Inertia = 6,112 in^4/ft; Elastic Section Modulus = 249.7 in^3/ft. Section properties satisfy allowable stresses for bending and shear in accordance with EM 1110-2-2504.
(2) FS=1.0. Deflections values shown are for the usual load case.
(3) Depth to fixity based on factors of safety recommended in the EM 1110-2-2504.

The above combined-wall system was selected on the basis of limiting drained and undrained top of wall deflections to approximately 1-inch for the usual load case. A wall deflection marginally exceeding 1-inch was estimated for the undrained Q-Case, however, this value (1.3 inch) is within the typical range expected for normal performing retaining walls. The minimum rock socket length and total length shown in the table above are rounded up to the nearest foot and are based on the subsurface conditions encountered at the critical boring BC-02.

It is recommended to install the intermediary sheet piles along this reach to the top of bedrock. The king piles (W-Beams) shall be socketed into competent bedrock to the minimum length shown in the table above. Mudstone bedrock was confirmed within Boring BC-02 at elevation -39.3 feet, MSL (NAVD88 = MSL – 0.18 feet). Actual subsurface conditions may differ along the proposed wall alignment as exploratory borings were not advanced at regular station intervals along the right bank near the wall. The total length shown above is not a minimum value and represents the estimated total length. Total length values may vary where bedrock is encountered at elevations different than those assumed herein, however, the length of the rock socket shall not be less than the value shown above.

STA 39+53 to 41+19 (Right Bank)

The subsurface profile utilized for the wall analysis performed between STA 39+53 and STA 41+19 was based on the nearest critical subsurface boring, or Boring BC-02, located approximately 1,350 feet downstream of the proposed wall reach. Subsurface borings were not performed for this station range as the need for the wall was not determined until later phases of design, after the completion of the subsurface exploration. Subsurface conditions, including the depth to bedrock, were extrapolated from confirmed conditions present at borings performed along the ER channel. The subsurface conditions encountered at the nearest boring location and those assumed at this location ford design may differ from actual conditions encountered during construction.

The analysis was conducted for the maximum wall height along this wall section of 8.5 feet, occurring at STA 39+53. Traffic induced loads on the wall were considered due to the proximity of an existing neighboring parking lot. A 12-foot wide external surcharge load of 240 psf was analyzed at a distance of 3 feet from the wall face. This vehicular surcharge load represents an equivalent height of soil and was selected based on wall height and surcharge offset in accordance with AASHTO LRFD Table 3.11.6.4.2. The backslope at the critical wall height is expected to be relatively flat based on review of project cross sections, and therefore, the impacts of a sloping ground surface behind the wall were not considered. The results of the analysis performed for the retaining wall at this location are provided below, which includes the requirements for embedment and corresponding rock socket length based on estimated bedrock depths.

Table 5-10. Summary of Combined-Wall Analysis at Right Bank from STA 39+53 to 41+19

Combined-Wall System Properties(1)		PYWall Outputs(2)				Min. Embedment (ft)(3)	Embedment Requirements		
Combined-Wall System	Spacing (in.)	Max. Max. Moment (in- kips)	Max. Shear (kips)	Top of Wall Deflection (inch)			Estimated Depth to Bedrock (ft.)	Min. Rock Socket Length (ft.)	Estimated Total Length (ft.)
				Q- Case	S- Case				
W40x199/NZ14	84	2,665	19.3	**0.7**	**0.7**	55.0	**43.6**	**12.0**	**56.0**

Note(s):
(1) Area = 14.14 in^2/ft; Moment of Inertia = 2,619 in^4/ft; Elastic Section Modulus = 121.1 in^3/ft. Section properties satisfy allowable stresses for bending and shear in accordance with EM 1110-2-2504.
(2) FS=1.0. Deflections values shown are for the usual load case.

(3) Depth to fixity based on factors of safety recommended in the EM 1110-2-2504.

The above combined-wall system was selected based on limiting both drained and undrained top of wall deflections to 1-inch or less for the usual load case. The minimum rock socket length and total length are rounded up to the nearest foot and are based on assumed subsurface conditions extrapolated from nearby borings.

It is recommended to install the intermediary sheet piles along this reach to the top of bedrock. The king piles (W-Beams) shall be socketed into competent bedrock to the minimum length shown in the table above. A top of bedrock elevation of -39.7 feet, MSL (NAVD88 = MSL – 0.18 feet) was extrapolated from nearby confirmed borings advanced along the ER channel. Actual subsurface conditions and depths to bedrock may differ along the proposed wall alignment as exploratory borings were not advanced within 1,350 feet of the wall. The total length shown above is not a minimum value and represents the estimated total length. Total length values may vary where bedrock is encountered at elevations different than those assumed herein, however, the length of the rock socket shall not be less than the value shown above.

5.9.5.2. Deep Foundations for the Bridges

5.9.5.2.1. Bridges

Deep foundations are recommended as foundation supports for the Railroad Bridge 0.99 and the roadway bridges at West Commercial Avenue (STA 26+00) and Amor Ave. (STA 15+25) due to the presence of thick, very soft, and highly compressible cohesive foundation soils. Based on the geotechnical exploration and laboratory testing program, deep foundation elements consisting of steel H-piles bearing on the underlying bedrock are recommended and will rely on end bearing resistance for axial capacity. The subsurface conditions encountered at the boring locations are expected to be generally representative of the subsurface conditions at the bridge locations.

Axial resistance recommendations are provided below assuming that driven steel piles will be utilized as the primary foundation elements for the rail and roadway crossing bridges. The procedures and recommendations outlined in NJDOT, "Design Manual for Bridges and Structures" were utilized.

The results of the axial resistance, uplift resistance, and drivability analyses are based on the steel pile sizes from the 95% Design – namely HP 12x53 at West Commercial and Amor Ave. and HP 14x89 at Railroad Bridge 0.99. It was subsequently determined by the Structural Designers that more robust steel H-piles were required at each bridge location. Through 100% Design, the pile section was increased to a HP 14x73 at both Amor and West Commercial Avenues while the pile section at Railroad Bridge 0.99 was increased to a HP 14x117. Updated deep foundation analyses were not performed from 95% to 100% Design as it is presumed the larger steel H-pile sections will satisfy the design requirements previously fulfilled by the smaller piles. The values provided below for uplift and axial resistance are considered conservative estimates since they reflect values anticipated for the smaller pile sections in the 95% Design.

Axial Resistance

Axial resistance for deep foundation elements at the Railroad Bridge 0.99, West Commercial and Amor Avenue bridges is expected to be derived primarily through end bearing on bedrock. In general, the subsurface investigation indicates the structural load capacity of the H-piles will control over the nominal geotechnical axial resistance if the piles are driven to refusal on competent bedrock. A summary of axial resistances for H-piles and the corresponding top of bedrock elevations is provided below for the Railroad Bridge 0.99, West Commercial and Amor Avenue bridges

- West Commercial Avenue (STA 26+00):
  - Pile Type: HP 14x73
  - Geotechnical Axial Resistance: Structural Load Capacity
  - Top of Bedrock Elevation: -40.7 feet, MSL (NAVD88 = MSL – 0.18 feet).
  - Estimated Pile Length: 32.9 feet.
- Amor Avenue (STA 15+25):
  - Pile Type: HP 14x73
  - Geotechnical Axial Resistance: Structural Load Capacity
  - Top of Bedrock Elevation: -46.0 feet, MSL (NAVD88 = MSL – 0.18 feet)
  - Estimated Pile Length: 37.9 feet.
- Railroad Bridge 0.99:
  - Pile Type: HP 14x117
  - Geotechnical Axial Resistance: Structural Load Capacity
  - Top of Bedrock: -48.7 feet, MSL (NAVD88 = MSL – 0.18 feet)
  - Estimate Pile Length: 43.7 feet.

The top of bedrock elevations and estimated pile lengths shown above are based on subsurface profiles derived from Borings BB-01 and BB-02 at Amor Avenue, Borings BC-01 and BC-02 at West Commercial Avenue, and Boring BC-04 at Railroad Bridge 0.99. Bedrock elevations may vary across the project site and may be encountered at depths less or greater than those specified herein. As such, any deviations in subsurface conditions, not revealed in the test borings, may affect the driving conditions and the pile lengths.

A steel tensile yield strength of 50 kips per square inch (ksi) was utilized in the structural load capacity analyses of the foundation elements (results provided on axial resistance graphs) and is recommended as a minimum for pile steel strength. If modifications to the proposed deep foundation systems are required, updated axial resistance analyses will likely be required. To eliminate axial reduction factors, a minimum spacing requirement for the piles should be three pile diameters (equivalent) center-to-center.

Uplift Resistance

The factored uplift resistance was developed by applying a resistance factor of 0.25 to the ultimate skin resistance developed along the interface between the exterior of the pile and the soil. The factored uplift resistance was determined based on AASHTO LRFD (2017) Table 10.5.5.2.3-1.

Lateral Resistance

A site-specific lateral analysis utilizing the LPILE (2018) computer program was performed.

Hammer Energy

Drivability analyses were performed to estimate the ultimate driving resistance and damage potential that HP-piles will experience during installation for the Railroad Bridge 0.99, West Commercial and Amor Avenue bridges. The drivability analyses were performed utilizing the guidelines presented in the Federal Highway Administration's (FHWA) publication "Soils and Foundations Workshop Manual" and are based on a pile being driven to the to the top of bedrock elevations previously discussed.

The driving resistances were estimated under the assumed condition that no interruptions and no pile set characteristics would be experienced during the driving process. Drivability analyses were conducted utilizing the GRLWEAP (version 2010) computer program that modeled the proposed H-piles driven by a DELMAG D 5 diesel pile hammer. The pile hammer has a manufacturer's maximum energy rating of approximately 10.5 ft-kips of energy. If a pile hammer with a maximum energy rating different from that of a DELMAG D 5 is utilized, it is recommended that additional drivability analyses be conducted.

The GRLWEAP analyses indicate the DELMAG D 5 pile hammer can drive a HP-12x53 pile to the top of bedrock (elevation -40.7 and -46.0 feet, MSL, NAVD88 = MSL – 0.18 feet, at West Commercial and Amor Avenue, respectively) without developing damaging compressive or tensile stresses within the pile and without experiencing an excessive number of hammer blows per foot of driving (more than 144 bpf). Likewise, the analyses indicate the DELMAG D 5 pile hammer can drive a HP-14x89 pile to the top of bedrock (elevation -48.7 feet, MSL, NAVD88 = MSL – 0.18 feet) at Railroad Bridge 0.99 without developing damaging compressive or tensile stresses within the pile and without experiencing an excessive number of hammer blows.

5.10. Planting and Site Restoration

Planting and site restoration features are located within the riparian zone and embankment zones of the channel improvements, as shown in Figure 4-4.

5.10.1. Riparian Zone Landscape

Restoration planting in the riparian zone includes the area between the top of bank and the limits of disturbance and accommodates a wider variety of species than is possible in the embankments. Planting consists of a mixture of seed, plugs, shrubs, and trees based on available space and micro-climate. The riparian zone will be planted with species from habitats such as upland meadows and woodlands and will use suitable soil and herbivory protection as needed. Following implementation, maintenance and monitoring will be conducted as detailed in the specifications (31 97 00 – Landscaping Maintenance) and as required by permit conditions. Wetlands within the limits of work were delineated in 2019 to 2020; wetland boundaries are shown on the design plans. The Wetland Delineation and Biobenchmarking Report provides details on the wetlands delineated in 2019. Details on the additional wetland within the LPS Industries parcel at 10 Caesar Place in Moonachie, which was wetland delineated in July 2020, will be included in the USACE/NJDEP permit application. The delineation covered only the portion of that parcel within the working limits because the property owner allowed very restricted access to the property. There is an existing riparian zone mitigation area on the LPS Industries parcel. This mitigation area will be disturbed during project construction and will be restored in a new location within the LPS Industries property. Approvals for the disturbance and subsequent relocation of this mitigation area will be coordinated with NJDEP DLRP. The location of the replacement riparian zone mitigation is shown on the design plans. Because the LPS Industries would not grant additional access to the property, the presence of wetlands at this location is approximated based on observations from the July 2020 field work and review of aerial photos. All delineated wetland boundaries within the project limits are shown on the existing conditions plan included in the design set, as well as on the permit plans that will be included in the permit application. These conditions influence the plant species selection and associated planting elevation locations. The presence of buildings, existing adjacent tree canopy, and sun orientation impact the sun and shade conditions and have been considered in the planting plan layout and species selection. Proposed planting species include a variety of native plants to match existing conditions. The basis for the planting strategy within this zone is based on available planting area limits that allow for continued adjacent property uses and a proposed operations and maintenance corridor. The wetland plantings are considered where the limits of construction are adjacent to existing wetlands. Efforts have been made to minimize existing wetland disturbance to the full extent possible Vegetation will be restored to pre-construction conditions within the allowable construction limits.

5.10.2. Riparian Zone Restoration

The riparian zone of the channel is primarily intended for restoration and maintenance of vegetation and natural habitat. During preliminary design of the ER Channel Improvements, a continuous operations and maintenance corridor was investigated; however, public recreational use is not part of the project scope. The design team recommends that the corridor continue to be used for O&M purposes consistent with restoration objectives. Views into the vegetated areas of the channel improvements from existing public right-of-way areas and adjacent facility-employee areas will be maintained, while ensuring that all activities remain consistent with the restoration and maintenance goals of the project.

5.10.3. Embankment Zone Landscape

Restoration planting in the embankment zone includes the area below the top of bank to the top of base water elevation. The planting plan responds to the embankment strategy of vegetated geolift and proposed wetland shelf within the channel. Plant species that withstand both frequent and long-term inundation as well as drier conditions, and those with a range of salt tolerance are recommended. The wetland shelf is a planting area subject to periodic inundation that creates new potential wetland areas for the channel. Salinity tests were performed as part of the biobench marking study, and the use of brackish plant species was determined to be unnecessary for the locations of planting in the channel. Beginning at a point downstream of Amor Avenue, ER is a freshwater tributary (salinity ~/<0.5 parts per thousand [ppt]). Higher salinity was observed both upstream and downstream of the tide gate at Starke Road, with salinity in the mesohaline range (5 - 18 ppt). Oligohaline conditions are expected between Starke Road and upstream to Amor Avenue. Although the tide gate prevents semi-diurnal tidal fluctuation, apparently, there is some leakage past the tide gate that influences water quality. The salinity level is expected to vary based on fluvial flow/rainfall amounts and tidal stage. Plants observed along the channel banks were predominantly freshwater species, as well as plants with some salt tolerance. The plants selected for restoration plantings have taken these conditions into account, particularly at the downstream end of the ER. Native wetland species with Obligate or Facultative Wetland indicator status will be planted within the wetland shelf. Based on conveyance capacity conformance during preliminary design, perennial non-woody native plants are planned for the embankment zone planting areas. The potential to incorporate live stakes in areas of riprap was evaluated during the

100% design phase and this was deemed feasible; therefore, live stakes of woody shrubs will be planted at appropriate elevations and suitable locations within the riprap lining the channel banks.

6. PROJECT FEATURES SUMMARY

The objective of the improved conveyance system is to reduce flood risk hazards in the surrounding neighborhoods in the Boroughs of Moonachie and Carlstadt and to add ecological value to the channel riparian zone. A summary of the proposed channel improvement and modifications is provided below, and the design recommendations to satisfy these objectives are summarized on Table 6-1.

6.1. Summary of East Riser channel improvement and modifications

Results from the without project modelling show that the existing channel does not have enough capacity to convey the flow, and it induces flooding.

The proposed modifications make the channel deeper and wider. The modifications start at the downstream face of the Moonachie Avenue bridge and continues approximately 4400 feet to the ER tide gate. The channel width will be increased an average of 10ft along the profile. The figure below compares the invert of the channel in the with and without project scenarios.

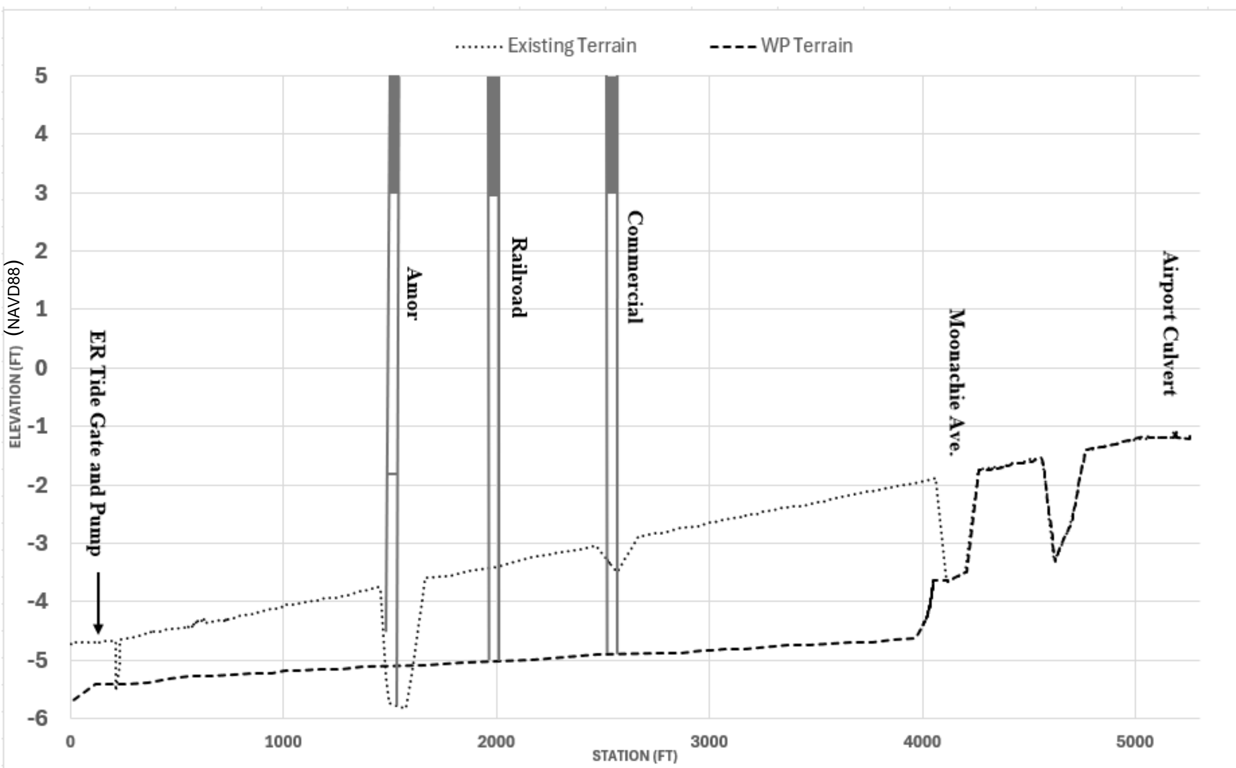


Figure 6-1. Channel invert comparison between With and Without Project Scenarios

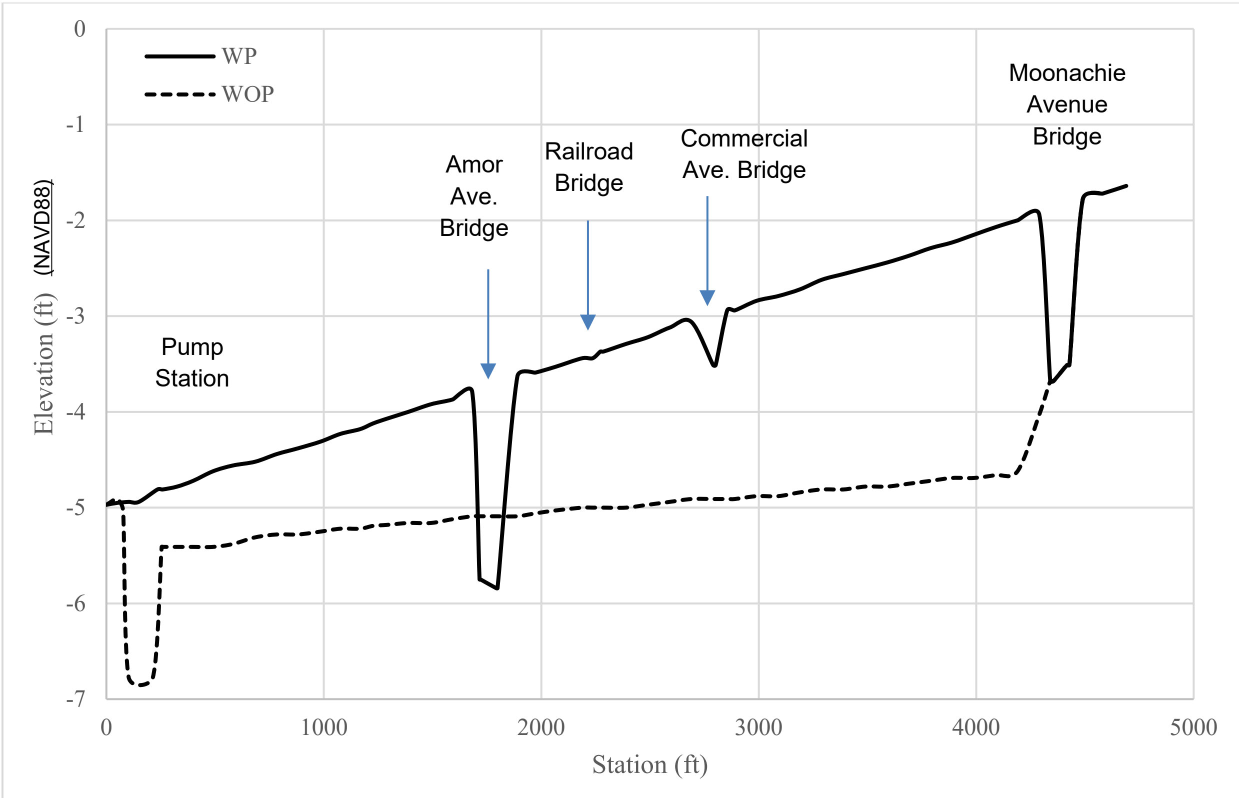


Figure 6-2. Channel invert comparison between With and Without Project Scenarios

Furthermore, the narrow bridge openings are the constraint points along the profile that are causing backwater and flooding upstream of the bridges on ER.

The proposed modification for these bridges is as follows:

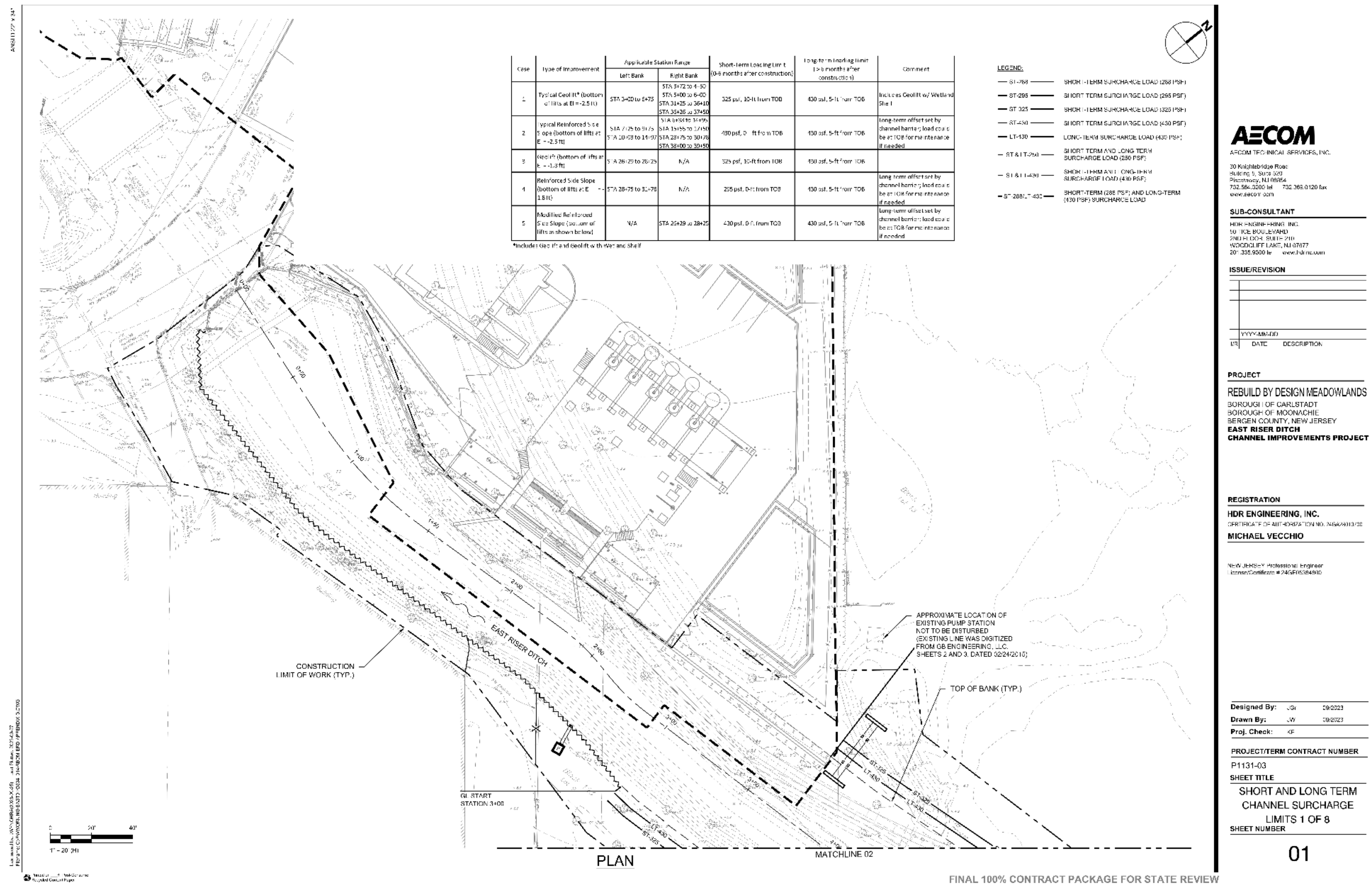
- Commercial Avenue Bridge
  - The existing opening of the Commercial Ave bridge is a 12’ span x 6’ rise elliptical culvert. The project proposes a rectangular opening with 40’ width and 8’ rise.
- Railroad Bridge
  - The existing opening of the Railroad bridge is a 21’ wide and 6.35’ high rectangular culvert. The project proposes to replace this culvert with a 44’ width and 8’ rise rectangular culvert.
- Amor Avenue Bridge
  - The existing opening of the Amor Avenue bridge is a dual ellipse culvert with 7.75’ span x 4.25’ rise. The project proposes a rectangular opening of 30’ wide and 8’ height.

Table 6-1. Design Description

Design Component	Recommendation
Channel Alignment and General Geometric Configuration	Implement alignment and geometry shown in the plans.
Operations and Maintenance Corridor	Establish 10-ft wide O&M corridor on alternating sides of the channel as shown on the plans.
Channel stabilization features	Implement features as shown on the plans
West Commercial Avenue Bridge: Bridge Replacement	Replace with prestressed concrete beam single span bridges on integral concrete abutments and driven H-piles.
Amor Avenue Bridge: Bridge Replacement	Replace with prestressed concrete beam single span bridges on integral concrete abutments and driven H-piles.
NJ Transit Railroad Bridge	Replace with Single Span, Double Track Ballasted rolled steel beam bridge supported by Precast Concrete Abutments and Driven H-Piles.
Riparian Zone Landscape	Install native vegetation to improve riparian habitat as shown in 5.10.1.1.
Riparian Zone Public Realm	Design planting layout to account for views from street crossings and private property employee areas.
Embankment Zone Landscape	Install native vegetation to improve habitat as shown in 5.10.1.3.
Temporary, Permanent, and Access Easements	Acquirement of temporary, permanent, and access easements will be evaluated using the proposed line work developed during 100% design and as shown on the plans. Refer also to Section 4.1.1.5 for a description of real estate acquisitions that are required.

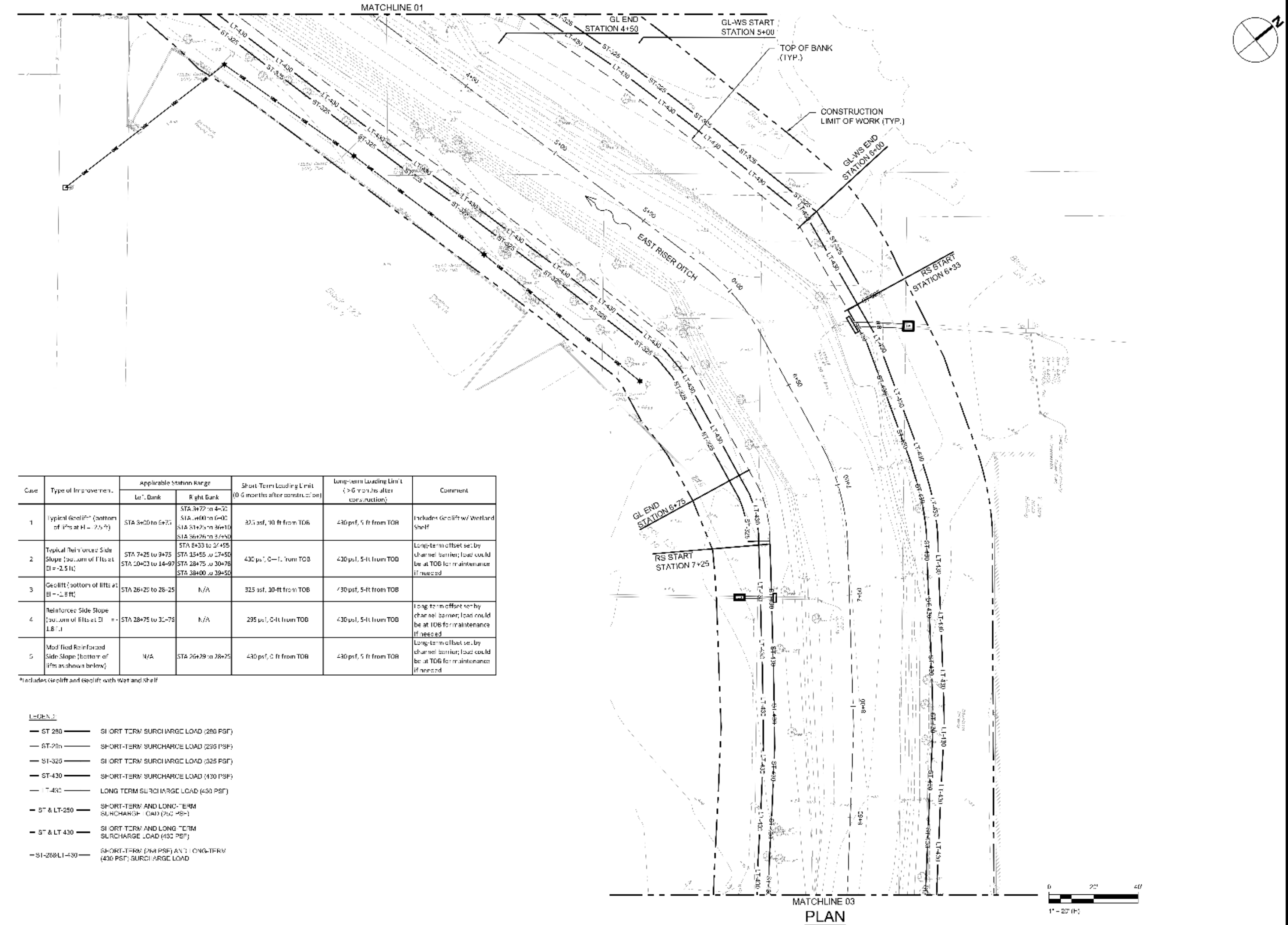
The 100% design drawings developed for the NJ Rebuild By Design Meadowlands project are provided as Attachment B-3D.

7. SHORT AND LONG-TERM CHANNEL SURCHARGE LIMITS



Case	Type of Improvement	Applicable Station Range		Short-Term Loading Limit (0-6 months after construction)	Long-term Loading Limit (>6 months after construction)	Comment
		Left Bank	Right Bank			
1	Typical Geolift* (bottom of lifts at El = -2.5 ft)	STA 3+00 to 8+75	STA 3+72 to 4+50 STA 5+00 to 6+00 STA 31+25 to 36+10 STA 35+26 to 37+50	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	Includes Geolift w/ Wetland Shelf
2	Typical Reinforced Side Slope (bottom of lifts at El = -2.5 ft)	STA 7+25 to 9+75 STA 10+03 to 14+97	STA 6+33 to 14+95 STA 15+55 to 17+50 STA 28+75 to 30+78 STA 38+00 to 39+50	430 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
3	Geolift (bottom of lifts at El = -1.8 ft)	STA 26+25 to 28+25	N/A	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	
4	Reinforced Side Slope (bottom of lifts at El = -1.8 ft)	STA 28+75 to 31+76	N/A	295 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
5	Modified Reinforced Side Slope (bottom of lifts as shown below)	N/A	STA 26+29 to 28+25	430 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed

*Includes Geolift and Geolift with Wetland Shelf



Case	Type of Improvement	Applicable Station Range		Short-Term Loading Limit (0-6 months after construction)	Long-term Loading Limit (>6 months after construction)	Comment
		Left Bank	Right Bank			
1	Typical Geolift* (bottom of lifts at El = -2.5 ft)	STA 3+00 to 6+75	STA 3+72 to 4+50 STA 5+00 to 6+00 STA 31+25 to 36+10 STA 36+26 to 37+50	325 psf, 10 ft from TOB	430 psf, 5 ft from TOB	Includes Geolift w/ Wetland Shelf
2	Typical Reinforced Side Slope (bottom of lifts at El = -2.5 ft)	STA 7+25 to 9+75 STA 10+03 to 14+97	STA 8+33 to 14+55 STA 15+55 to 17+50 STA 28+75 to 30+78 STA 38+00 to 39+50	430 psf, 0 ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
3	Geolift (bottom of lifts at El = -1.8 ft)	STA 26+29 to 28+25	N/A	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	
4	Reinforced Side Slope (bottom of lifts at El = -1.8 ft)	STA 28+75 to 31+78	N/A	295 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
5	Modified Reinforced Side Slope (bottom of lifts as shown below)	N/A	STA 26+29 to 28+25	430 psf, 0 ft from TOB	430 psf, 5 ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed

*Includes Geolift and Geolift with Wetland Shelf

LEGEND:

- ST 288 — SHORT TERM SURCHARGE LOAD (288 PSF)
- ST-295 — SHORT-TERM SURCHARGE LOAD (295 PSF)
- ST-325 — SHORT TERM SURCHARGE LOAD (325 PSF)
- ST-430 — SHORT-TERM SURCHARGE LOAD (430 PSF)
- LT-430 — LONG TERM SURCHARGE LOAD (430 PSF)
- ST & LT-250 — SHORT-TERM AND LONG-TERM SURCHARGE LOAD (250 PSF)
- ST & LT 430 — SHORT-TERM AND LONG-TERM SURCHARGE LOAD (430 PSF)
- ST-288/LT-430 — SHORT-TERM (288 PSF) AND LONG-TERM (430 PSF) SURCHARGE LOAD

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ISSUE/REVISION

I/R	DATE	DESCRIPTION
	YYYY-MM-DD	

PROJECT

REBUILD BY DESIGN MEADOWLANDS
BOROUGH OF CARLSTADT
BOROUGH OF MOONACHIE
BERGEN COUNTY, NEW JERSEY
EAST RISER DITCH
CHANNEL IMPROVEMENTS PROJECT

REGISTRATION

HDR ENGINEERING, INC.
CERTIFICATE OF AUTHORIZATION NO. 24GA28013700
MICHAEL VECCHIO

NEW JERSEY Professional Engineer
License/Certificate # 24GE05394800

Designed By:	JGr	09/2023
Drawn By:	JW	09/2023
Proj. Check:	KF	

PROJECT/TERM CONTRACT NUMBER

P1131-03

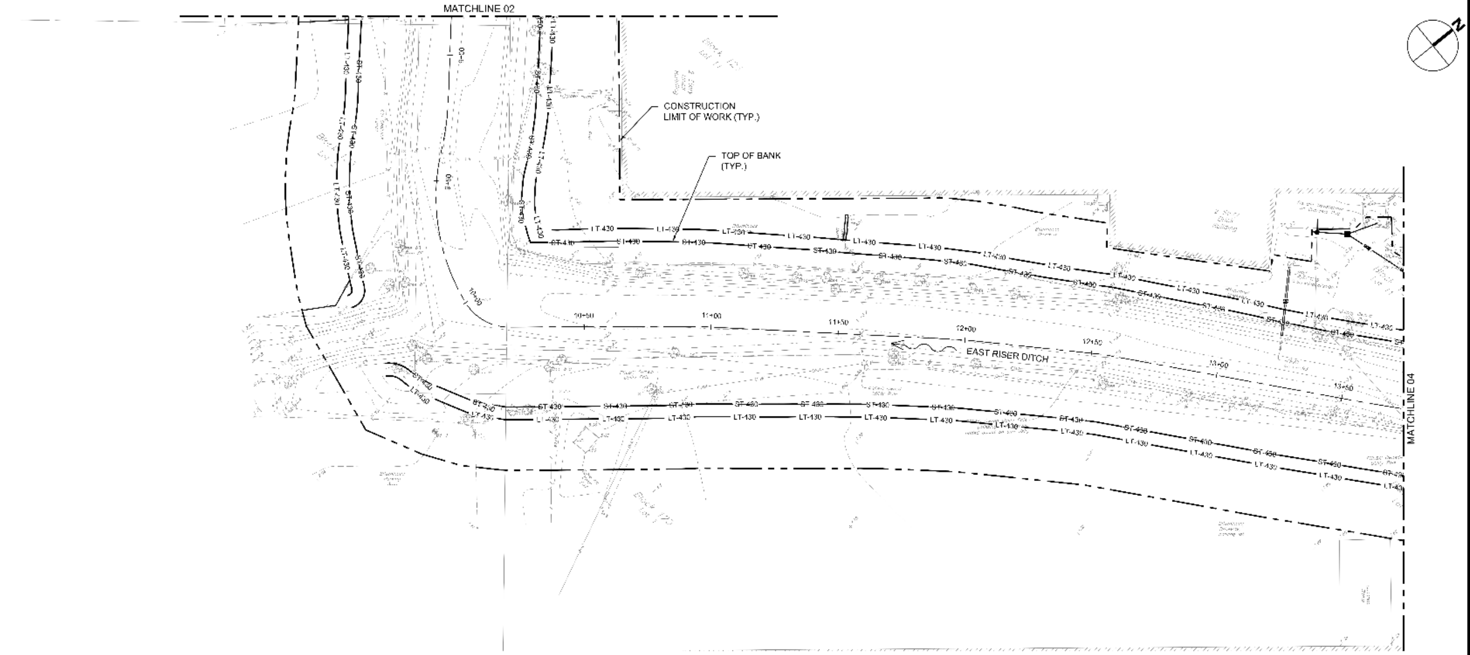
SHEET TITLE

SHORT AND LONG TERM CHANNEL SURCHARGE LIMITS 2 OF 8

SHEET NUMBER

02

FINAL 100% CONTRACT PACKAGE FOR STATE REVIEW



PLAN

LEGEND

ST 288	SHORT TERM SURCHARGE LOAD (288 PSF)
ST-295	SHORT-TERM SURCHARGE LOAD (295 PSF)
ST-325	SHORT TERM SURCHARGE LOAD (325 PSF)
ST-430	SHORT-TERM SURCHARGE LOAD (430 PSF)
LT-430	LONG TERM SURCHARGE LOAD (430 PSF)
ST & LT-250	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (250 PSF)
ST & LT-430	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (430 PSF)
ST-288/LT-430	SHORT-TERM (288 PSF) AND LONG-TERM (430 PSF) SURCHARGE LOAD

Case	Type of Improvement	Applicable Station Range		Short-Term Loading Limit (0-6 months after construction)	Long-term Loading Limit (> 6 months after construction)	Comment
		Left Bank	Right Bank			
1	Typical Geolift* (bottom of lifts at El = -2.5 ft)	STA 3+00 to 6+75	STA 3+72 to 4+50 STA 5+00 to 6+00 STA 31+25 to 36+10 STA 36+26 to 37+50	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	Includes Geolift w/ Wetland Shelf
2	Typical Reinforced Side Slope (bottom of lifts at El = -2.5 ft)	STA 7+25 to 9+75 STA 10+03 to 14+97	STA 6+33 to 14+95 STA 15+55 to 17+50 STA 28+75 to 30+78 STA 38+00 to 39+50	430 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
3	Geolift (bottom of lifts at El = -1.8 ft)	STA 26+29 to 28+25	N/A	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	
4	Reinforced Side Slope (bottom of lifts at El = -1.8 ft)	STA 28+75 to 31+76	N/A	295 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
5	Modified Reinforced Side Slope (bottom of lifts as shown below)	N/A	STA 26+29 to 28+25	430 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed

*Includes Geolift and Geolift with Wetland Shelf

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ISSUE/REVISION

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	YYYY-MM-DD	

PROJECT
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BOROUGH OF MOONACHIE
BERGEN COUNTY, NEW JERSEY
EAST RISER DITCH
CHANNEL IMPROVEMENTS PROJECT

REGISTRATION
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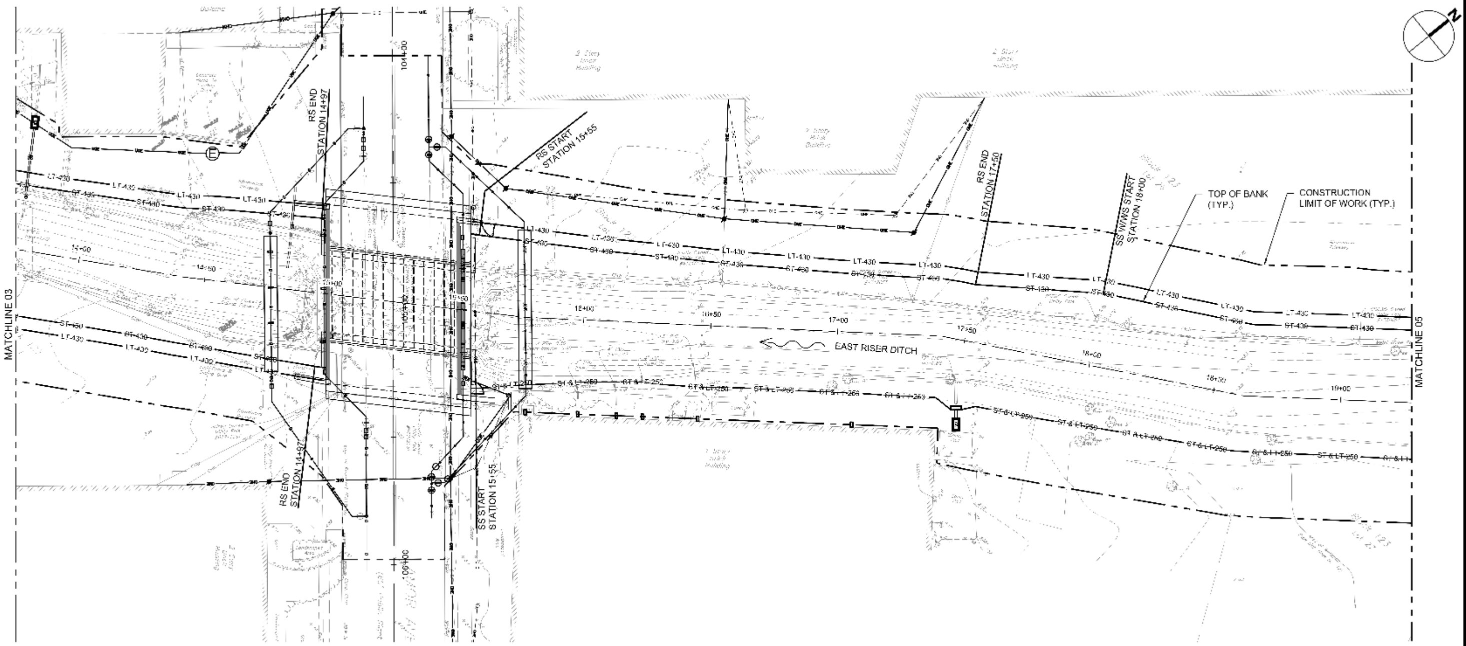
CERTIFICATE OF AUTHORIZATION NO. 24GA28013700
MICHAEL VECCHIO

NEW JERSEY Professional Engineer
License/Certificate # 24GE05394900

Designed By:	JGr	09/2023
Drawn By:	JW	09/2023
Proj. Check:	KF	

PROJECT/TERM CONTRACT NUMBER
P1131-03
SHEET TITLE
SHORT AND LONG TERM
CHANNEL SURCHARGE
LIMITS 3 OF 8
SHEET NUMBER
03

FINAL 100% CONTRACT PACKAGE FOR STATE REVIEW



PLAN

LEGEND

ST 288	SHORT TERM SURCHARGE LOAD (288 PSF)
ST-295	SHORT-TERM SURCHARGE LOAD (295 PSF)
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LT-430	LONG TERM SURCHARGE LOAD (430 PSF)
ST & LT-250	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (250 PSF)
ST & LT-430	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (430 PSF)
ST-268/LT-430	SHORT-TERM (268 PSF) AND LONG-TERM (430 PSF) SURCHARGE LOAD

Case	Type of Improvement	Applicable Station Range		Short-Term Loading Limit (0-6 months after construction)	Long-term Loading Limit (> 6 months after construction)	Comment
		Left Bank	Right Bank			
1	Typical Geolift* (bottom of lifts at El = -2.5 ft)	STA 3+00 to 6+75	STA 3+72 to 4+50 STA 5+00 to 6+00 STA 31+25 to 36+10 STA 36+26 to 37+50	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	Includes Geolift w/ Wetland Shelf
2	Typical Reinforced Side Slope (bottom of lifts at El = -2.5 ft)	STA 7+25 to 9+75 STA 10+03 to 14+97	STA 6+33 to 14+95 STA 15+55 to 17+50 STA 28+75 to 30+78 STA 38+00 to 39+50	430 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
3	Geolift (bottom of lifts at El = -1.8 ft)	STA 26+29 to 28+25	N/A	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	
4	Reinforced Side Slope (bottom of lifts at El = -1.8 ft)	STA 28+75 to 31+76	N/A	295 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
5	Modified Reinforced Side Slope (bottom of lifts as shown below)	N/A	STA 26+29 to 28+25	430 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed

*Includes Geolift and Geolift with Wetland Shelf

1" = 20'

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ISSUE/REVISION

I/R	DATE	DESCRIPTION
	YYYY-MM-DD	

PROJECT

REBUILD BY DESIGN MEADOWLANDS
BOROUGH OF CARLSTADT
BOROUGH OF MOONACHIE
BERGEN COUNTY, NEW JERSEY
EAST RISER DITCH
CHANNEL IMPROVEMENTS PROJECT

REGISTRATION

HDR ENGINEERING, INC.
CERTIFICATE OF AUTHORIZATION NO. 24GA28013700
MICHAEL VECCHIO

NEW JERSEY Professional Engineer
License/Certificate # 24GE05384900

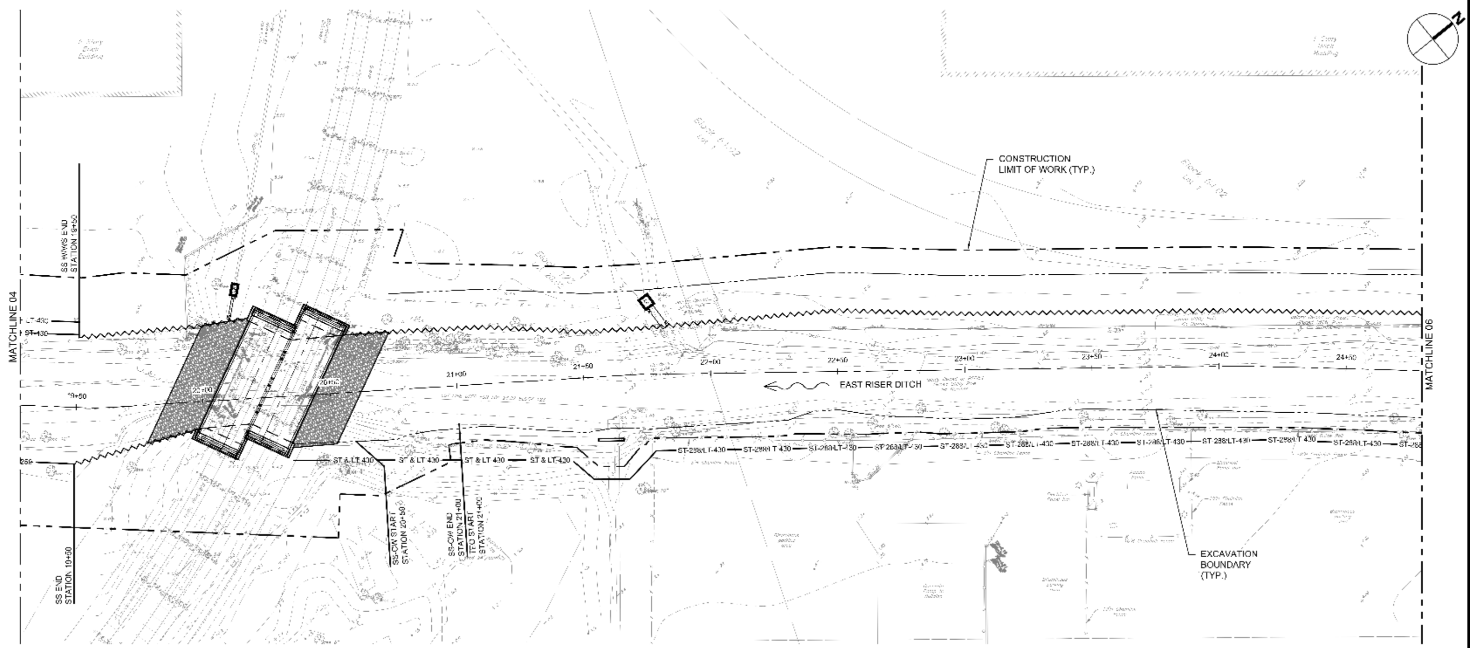
Designed By: JGr 09/2023
Drawn By: JW 09/2023
Proj. Check: KF

PROJECT/TERM CONTRACT NUMBER
P1131-03

SHEET TITLE
SHORT AND LONG TERM CHANNEL SURCHARGE LIMITS 4 OF 8

SHEET NUMBER
04

FINAL 100% CONTRACT PACKAGE FOR STATE REVIEW



PLAN

LEGEND

ST 288	SHORT TERM SURCHARGE LOAD (288 PSF)
ST-295	SHORT-TERM SURCHARGE LOAD (295 PSF)
ST-325	SHORT TERM SURCHARGE LOAD (325 PSF)
ST-430	SHORT-TERM SURCHARGE LOAD (430 PSF)
LT-430	LONG TERM SURCHARGE LOAD (430 PSF)
ST & LT-250	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (250 PSF)
ST & LT-430	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (430 PSF)
ST-288/LT-430	SHORT-TERM (288 PSF) AND LONG-TERM (430 PSF) SURCHARGE LOAD

Case	Type of Improvement	Applicable Station Range		Short-Term Loading Limit (0-6 months after construction)	Long-term Loading Limit (> 6 months after construction)	Comment
		Left Bank	Right Bank			
1	Typical Geolift* (bottom of lifts at El = -2.5 ft)	STA 3+00 to 6+75	STA 3+72 to 4+50 STA 5+00 to 6+00 STA 31+25 to 36+10 STA 36+26 to 37+50	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	Includes Geolift w/ Wetland Shelf
2	Typical Reinforced Side Slope (bottom of lifts at El = -2.5 ft)	STA 7+25 to 9+75 STA 10+03 to 14+97	STA 6+33 to 14+95 STA 15+55 to 17+50 STA 28+75 to 30+78 STA 38+00 to 39+50	430 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
3	Geolift (bottom of lifts at El = -1.8 ft)	STA 26+29 to 28+25	N/A	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	
4	Reinforced Side Slope (bottom of lifts at El = -1.8 ft)	STA 28+75 to 31+76	N/A	295 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
5	Modified Reinforced Side Slope (bottom of lifts as shown below)	N/A	STA 26+29 to 28+25	430 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed

*Includes Geolift and Geolift with Wetland Shelf

0 20' 40'
1" = 20'(H)

FINAL 100% CONTRACT PACKAGE FOR STATE REVIEW

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PROJECT

REBUILD BY DESIGN MEADOWLANDS
BOROUGH OF CARLSTADT
BOROUGH OF MOONACHIE
BERGEN COUNTY, NEW JERSEY
EAST RISER DITCH
CHANNEL IMPROVEMENTS PROJECT

REGISTRATION

HDR ENGINEERING, INC.
CERTIFICATE OF AUTHORIZATION NO. 24GA28013700
MICHAEL VECCHIO

NEW JERSEY Professional Engineer
License/Certificate # 24GE05394900

Designed By:	JGr	09/2023
Drawn By:	JW	09/2023
Proj. Check:	KF	

PROJECT/TERM CONTRACT NUMBER

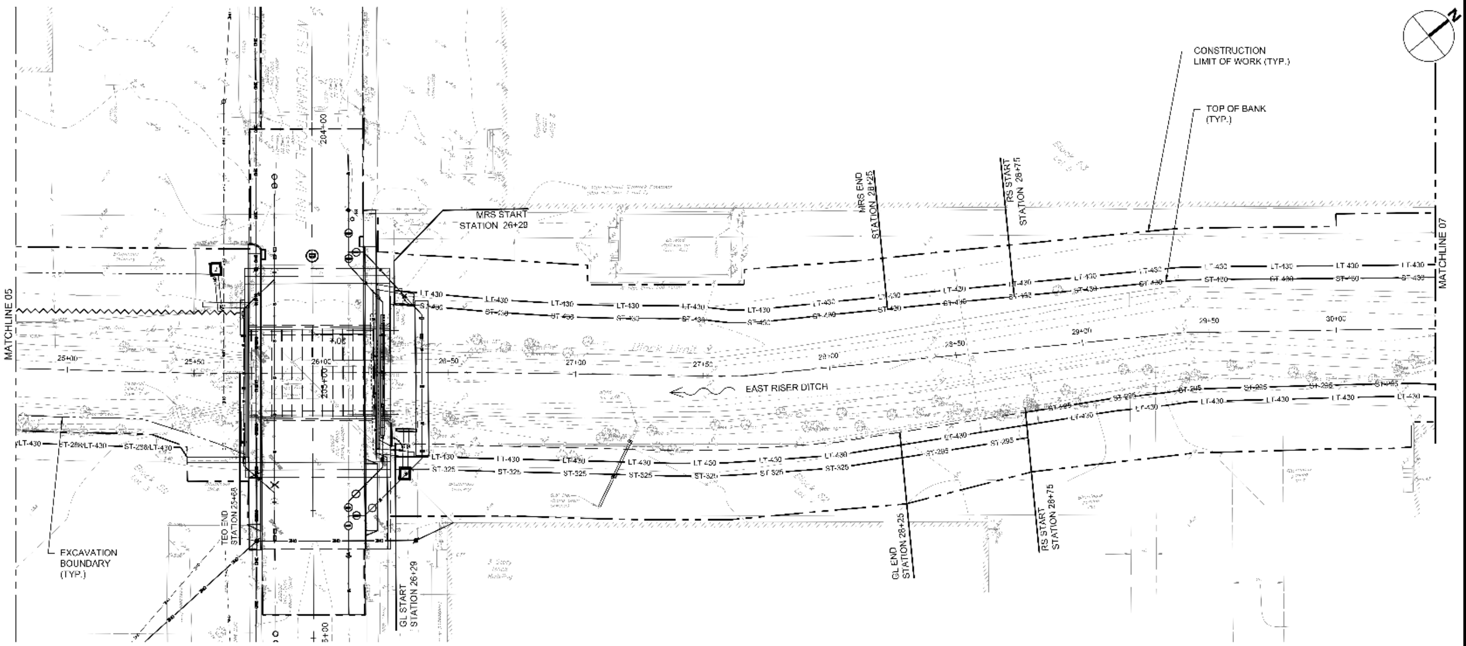
P1131-03

SHEET TITLE

SHORT AND LONG TERM CHANNEL SURCHARGE LIMITS 5 OF 8

SHEET NUMBER

05



PLAN

LEGEND

ST 288	SHORT TERM SURCHARGE LOAD (288 PSF)
ST-295	SHORT-TERM SURCHARGE LOAD (295 PSF)
ST-325	SHORT TERM SURCHARGE LOAD (325 PSF)
ST-430	SHORT-TERM SURCHARGE LOAD (430 PSF)
LT-430	LONG TERM SURCHARGE LOAD (430 PSF)
ST & LT-250	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (250 PSF)
ST & LT-430	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (430 PSF)
ST-288/LT-430	SHORT-TERM (288 PSF) AND LONG-TERM (430 PSF) SURCHARGE LOAD

Case	Type of Improvement	Applicable Station Range		Short-Term Loading Limit (0-6 months after construction)	Long-term Loading Limit (> 6 months after construction)	Comment
		Left Bank	Right Bank			
1	Typical Geolift* (bottom of lifts at El = -2.5 ft)	STA 3+00 to 6+75	STA 3+72 to 4+50 STA 5+00 to 6+00 STA 31+25 to 36+10 STA 36+26 to 37+50	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	Includes Geolift w/ Wetland Shelf
2	Typical Reinforced Side Slope (bottom of lifts at El = -2.5 ft)	STA 7+25 to 9+75 STA 10+03 to 14+97	STA 6+33 to 14+95 STA 15+55 to 17+50 STA 28+75 to 30+78 STA 38+00 to 39+50	430 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
3	Geolift (bottom of lifts at El = -1.8 ft)	STA 26+29 to 28+25	N/A	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	
4	Reinforced Side Slope (bottom of lifts at El = -1.8 ft)	STA 28+75 to 31+76	N/A	295 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
5	Modified Reinforced Side Slope (bottom of lifts as shown below)	N/A	STA 26+29 to 28+25	430 psf, 0 ft from TOB	430 psf, 5 ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed

*Includes Geolift and Geolift with Wetland Shelf

1" = 20' (H)

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SUB-CONSULTANT
HDR ENGINEERING, INC.

ISSUE/REVISION

PROJECT
REBUILD BY DESIGN MEADOWLANDS
BOROUGH OF CARLSTADT
BOROUGH OF MOONACHIE
BERGEN COUNTY, NEW JERSEY
EAST RISER DITCH CHANNEL IMPROVEMENTS PROJECT

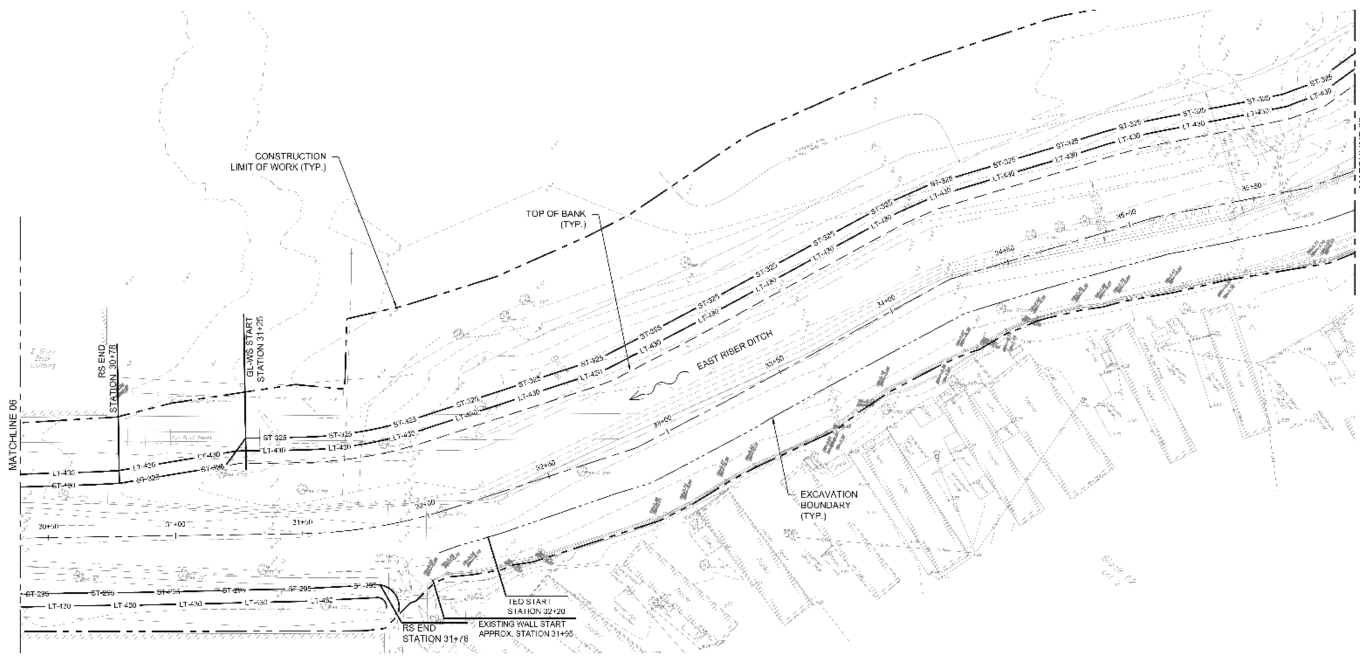
REGISTRATION
HDR ENGINEERING, INC.
MICHAEL VECCHIO

PROJECT/TERM CONTRACT NUMBER
P1131-03

SHEET TITLE
SHORT AND LONG TERM CHANNEL SURCHARGE LIMITS 6 OF 8

SHEET NUMBER
06

FINAL 100% CONTRACT PACKAGE FOR STATE REVIEW



PLAN

LEGEND

ST 288	SHORT TERM SURCHARGE LOAD (288 PSF)
ST-295	SHORT-TERM SURCHARGE LOAD (295 PSF)
ST-325	SHORT TERM SURCHARGE LOAD (325 PSF)
ST-430	SHORT-TERM SURCHARGE LOAD (430 PSF)
LT-430	LONG TERM SURCHARGE LOAD (430 PSF)
ST & LT-250	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (250 PSF)
ST & LT-430	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (430 PSF)
ST-288/LT-430	SHORT-TERM (288 PSF) AND LONG-TERM (430 PSF) SURCHARGE LOAD

Case	Type of Improvement	Applicable Station Range		Short-Term Loading Limit (0-6 months after construction)	Long-term Loading Limit (> 6 months after construction)	Comment
		Left Bank	Right Bank			
1	Typical Geolift* (bottom of lifts at El = -2.5 ft)	STA 3+00 to 6+75	STA 3+72 to 4+50 STA 5+00 to 6+00 STA 31+25 to 36+10 STA 36+26 to 37+50	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	Includes Geolift w/ Wetland Shelf
2	Typical Reinforced Side Slope (bottom of lifts at El = -2.5 ft)	STA 7+25 to 9+75 STA 10+03 to 14+97	STA 6+33 to 14+95 STA 15+55 to 17+50 STA 28+75 to 30+78 STA 38+00 to 39+50	430 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
3	Geolift (bottom of lifts at El = -1.8 ft)	STA 26+29 to 28+25	N/A	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	
4	Reinforced Side Slope (bottom of lifts at El = -1.8 ft)	STA 28+75 to 31+76	N/A	295 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
5	Modified Reinforced Side Slope (bottom of lifts as shown below)	N/A	STA 26+29 to 28+25	430 psf, 0 ft from TOB	430 psf, 5 ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed

*Includes Geolift and Geolift with Wetland Shelf

0 20' 40'
1" = 20' (ft)

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ISSUE/REVISION
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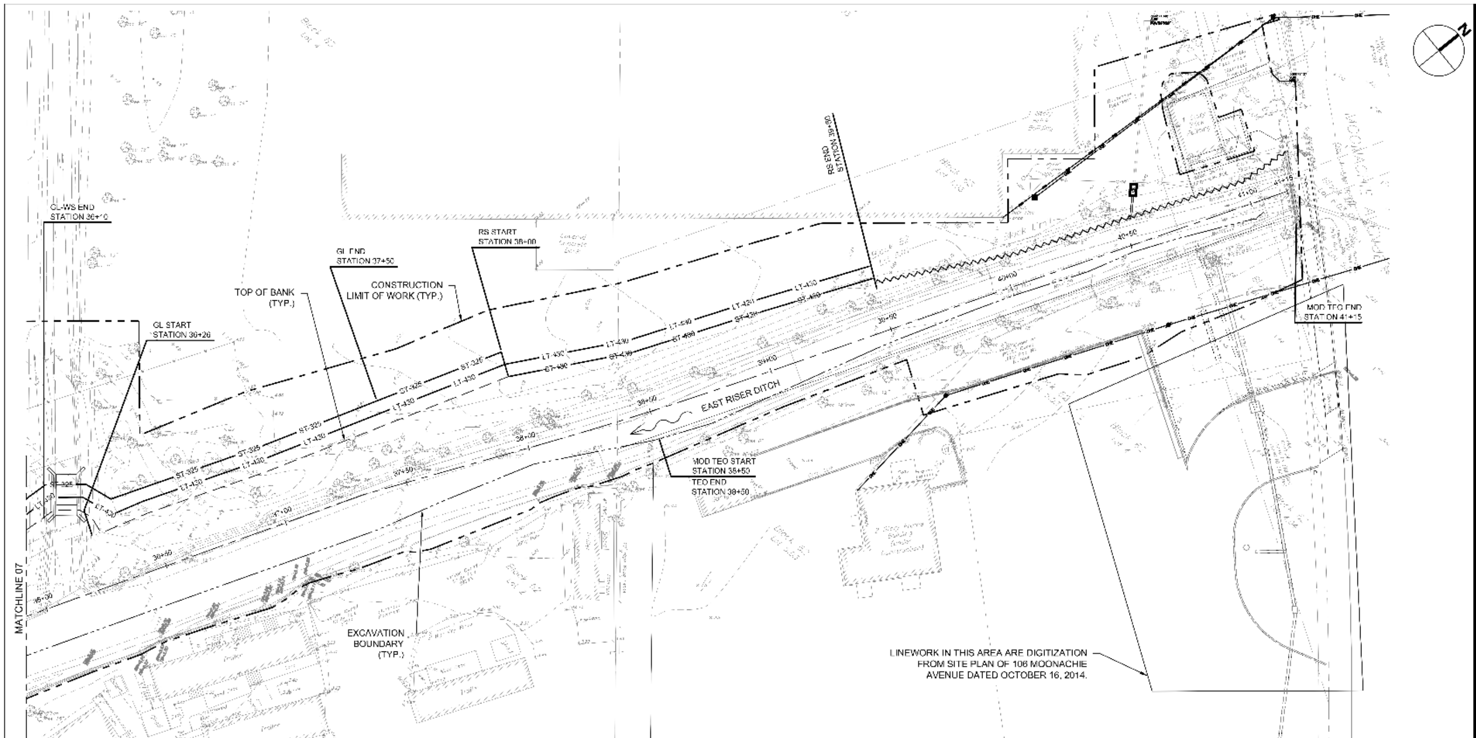
PROJECT
REBUILD BY DESIGN MEADOWLANDS
BOROUGH OF CARLSTADT
BOROUGH OF MOONACHIE
BERGEN COUNTY, NEW JERSEY
EAST RISER DITCH
CHANNEL IMPROVEMENTS PROJECT

REGISTRATION
HDR ENGINEERING, INC.
CERTIFICATE OF AUTHORIZATION NO. 24GA28013700
MICHAEL VECCHIO
NEW JERSEY Professional Engineer
License/Certificate # 24GE05394900

Designed By: JGI 09/2023
Drawn By: JRY 09/2023
Proj. Check: KF

PROJECT/TERM CONTRACT NUMBER
P1131-03
SHEET TITLE
SHORT AND LONG TERM CHANNEL SURCHARGE LIMITS 7 OF 8
SHEET NUMBER
07

FINAL 100% CONTRACT PACKAGE FOR STATE REVIEW



PLAN

LEGEND:

ST 288	SHORT TERM SURCHARGE LOAD (288 PSF)
ST-295	SHORT-TERM SURCHARGE LOAD (295 PSF)
ST-325	SHORT TERM SURCHARGE LOAD (325 PSF)
ST-430	SHORT-TERM SURCHARGE LOAD (430 PSF)
LT-430	LONG TERM SURCHARGE LOAD (430 PSF)
ST & LT-250	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (250 PSF)
ST & LT-430	SHORT-TERM AND LONG-TERM SURCHARGE LOAD (430 PSF)
ST-288/LT-430	SHORT-TERM (288 PSF) AND LONG-TERM (430 PSF) SURCHARGE LOAD

Case	Type of Improvement	Applicable Station Range		Short-Term Loading Limit (0-6 months after construction)	Long-term Loading Limit (>6 months after construction)	Comment
		Left Bank	Right Bank			
1	Typical Geolift* (bottom of lifts at El = -2.5 ft)	STA 3+00 to 6+75	STA 3+72 to 4+50 STA 5+00 to 6+00 STA 31+25 to 36+10 STA 36+20 to 37+50	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	Includes Geolift w/ Wetland Shelf
2	Typical Reinforced Side Slope (bottom of lifts at El = -2.5 ft)	STA 7+25 to 9+75 STA 10+03 to 14+97	STA 6+33 to 14+95 STA 15+55 to 17+50 STA 28+75 to 30+78 STA 38+00 to 39+50	430 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
3	Geolift (bottom of lifts at El = -1.8 ft)	STA 26+29 to 28+25	N/A	325 psf, 10-ft from TOB	430 psf, 5-ft from TOB	
4	Reinforced Side Slope (bottom of lifts at El = -1.8 ft)	STA 28+75 to 31+76	N/A	295 psf, 0-ft from TOB	430 psf, 5-ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed
5	Modified Reinforced Side Slope (bottom of lifts as shown below)	N/A	STA 26+29 to 28+25	430 psf, 0 ft from TOB	430 psf, 5 ft from TOB	Long-term offset set by channel barrier; load could be at TOB for maintenance if needed

*Includes Geolift and Geolift with Wetland Shelf

1" = 20' FT

FINAL 100% CONTRACT PACKAGE FOR STATE REVIEW

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AECOM TECHNICAL SERVICES, INC.

SUB-CONSULTANT
HDR ENGINEERING, INC.

ISSUE/REVISION
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PROJECT
REBUILD BY DESIGN MEADOWLANDS
BOROUGH OF CARLSTADT
BOROUGH OF MOONACHIE
BERGEN COUNTY, NEW JERSEY
EAST RISER DITCH
CHANNEL IMPROVEMENTS PROJECT

REGISTRATION
HDR ENGINEERING, INC.
MICHAEL VECCHIO
NEW JERSEY Professional Engineer

Designed By: JG 09/2023
Drawn By: JW 09/2023
Proj. Check: KF

PROJECT/TERM CONTRACT NUMBER
P1131-03

SHEET TITLE
SHORT AND LONG TERM CHANNEL SURCHARGE LIMITS 8 OF 8

SHEET NUMBER
08