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version 2

Wilroy Road Pressure Reducing Station and Offline Storage Facility Preconstruction Assessment and Damage Mitigation Report - Stage 1 and 2

Prepared for
HRSD
Virginia Beach, Virginia
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HRSD
Sustainable, Innovative Wastewater Treatment

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List of Abbreviations and Definitions

BC	Brown and Caldwell
CAP	Condition Assessment Program
CIP	Capital Improvement Program
DMT	Flat Dilatometer Test
ft	foot/feet
GER	Geotechnical Engineering Report
HRSD	Hampton Roads Sanitation District
in.	inch/inches
in./sec	inch/inches per second
LF	linear foot/feet
Manual	Transit Noise and Vibration Impact Assessment Manual
MH	Manhole
PS	Pump Station
PVC	polyvinyl chloride
RAP	Rehabilitation Action Plan
sec	second
SF	square foot
SG	Sewer Gravity (Main)
SPS	Sewer Pump Station
USEPA	United States Environmental Protection Agency
UV	Ultraviolet
VdB	vibration decibels
VDEQ	Virginia Department of Environmental Quality
VDOT	Virginia Department of Transportation
ZOI	Zone of Influence

Executive Summary

As part of Hampton Road Sanitation District's (HRSD) Regional Wet Weather Management Plan (RWWMP), the Wilroy Pressure Reducing Station (PRS) and Offline Storage Facility (OLSF) has been identified as a High Priority Project and must be constructed by the end of 2030.

HRSD developed a capital improvement plan (CIP) project NP014000 to fund the design and construction of a new PRS and a 3-million-gallon storage tank in Suffolk along the Wilroy Road corridor. Brown and Caldwell was selected as the design engineer for the project and Crowder Construction has been selected as the Construction Manager at Risk (CMAR).

This document is the combined Stage 1 and 2 Construction Impact Research and Evaluation Report that is required per Section 12 of the 2024 HRSD Design and Construction Standards. This report summarizes the construction site conditions, the likely construction means and methods, the potential construction related impacts, and recommended impact mitigation plans.

Noise generation and impacts were analyzed using the information provided in Section 7 of the *Transit Noise and Vibration Impact Assessment Manual*, published by the Federal Transit Administration in September 2018. The analysis showed that anticipated noise levels will not exceed General Assessment Construction Noise Criteria, and anticipated vibrations are high enough to cross the threshold of noticeability but are not severe enough to pose a risk of structural damage to adjacent property foundations; however, dewatering is anticipated to occur, which, depending on selected construction method, may result in ground settlement in the vicinity of the project. Potential construction disturbance is at its highest near 300 Executive Court, which is 120 feet (ft) from the anticipated construction, and at the Children's Center daycare facility which is located approximately 200 ft from the lower level excavation for the PRS.

A suggested noise, vibration, dewatering, and settlement monitoring plan has been developed for implementation by the contractor, engineer, and HRSD. It is also recommended that a community Open House be held with the contractor, engineer, and HRSD to discuss potential construction impacts and the proposed mitigation plans.

Section 1

Introduction

HRSD is undertaking a project to construct a pressure-reducing pumping station (PRS) and an offline storage facility (OLSF) located on HRSD property at 1941 Wilroy Road in Suffolk. This facility is designed to relieve pressure in the wastewater sewer force main system during wet weather events. Nearly all sanitary sewer systems have small leaks and cracks that allow water to enter the system during rain events. During larger wet weather events, the amount of water collected by the pipes increases and must be pumped to the treatment plants far down the pipeline network. This higher flow (of mostly rainwater and groundwater) increases the pipeline system pressures to a point where the pumps may no longer be able to handle the higher flows and a system backup (or sanitary sewer overflow) may occur. A pressure reducing station uses pumps to increase the capacity of the pipeline to prevent SSOs. If the PRS pump capacity is exceeded, flow can be diverted temporarily to the offline storage tank until pressures go down and the water in the tank can be returned to the system and on to the wastewater treatment plant. The stored volume is typically more than 60-70% excess groundwater or direct rainwater that entered the system through the gravity collection systems. This facility will help reduce the occurrence of SSOs, which are improvements that are mandated by the United States Environmental Protection Agency (USEPA). The facility will improve system operations, increase flow capacity, and provide reliable sanitary sewer infrastructure for areas of Suffolk and Isle of Wight County.

1.1 Project Background

As part of HRSD's Consent Decree with the USEPA and the Virginia Department of Environmental Quality (VDEQ), a Regional Wet Weather Management Plan (RWWMP) was developed in ordered to mitigate the occurrence of SSOs in the regional system under their jurisdiction. The resulting RWWMP (final revision in June 2020) was submitted to the USEPA/VDEQ and approved January 26, 2022.

After considering scoring criteria, including SSO volume reduction for each project, location of the affected SSOs, and reduction in infiltration/inflow (I/I) from each project, the RWWMP identified fifteen (15) High-Priority Projects (HPPs) separated into two rounds. Included in Round 1 of the HPPs are six (6) projects totaling more than $207 million with staggered completion dates. The completion time frame for Round 1 is 2030. The Wilroy PRS and OLSF project (NA-01) was concluded to be a HPP and is the second project identified in Round 1. See Figure 1-1 below for a general proximity map. The flow from this portion of the system eventually travels north to HRSD's Nansemond Plant at the James River.

HRSD has developed a CIP project NP014000 to fund the design and construction of the High-Priority Projects identified in Round 1. Brown and Caldwell was selected by HRSD as the design engineer for the project and Crowder Construction has been selected as the Construction Manager at Risk (CMAR).

The project is currently at 100% design. This document is the Stage 1 Construction Impact Research and Evaluation Report that is required per Section 12 of the 2024 HRSD Design and Construction Standards. This report summarizes the construction site conditions, the likely construction means and methods, the potential construction related impacts, and recommended impact mitigation plans.

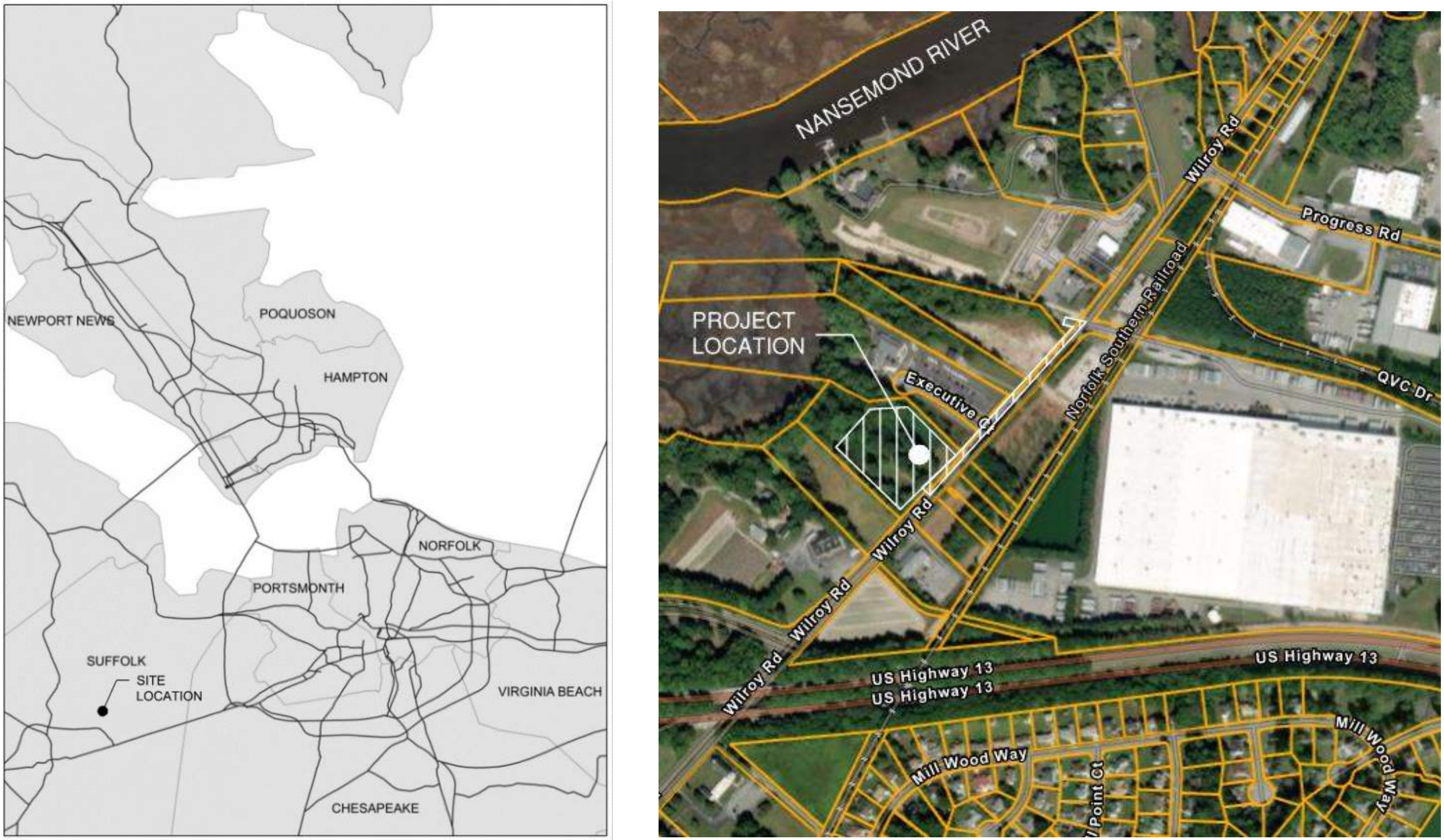


Figure 1-1. Vicinity Map

Section 2

Research and Investigations

The following sections summarize the background conditions of the project area, including the soil conditions, previous similar projects, and sensitive structures.

2.1 Soil Conditions

Several soil borings were performed during the preliminary engineering phase, as documented in the HRSD Wilroy Road Pressure Reducing Station and Offline Storage Facility Geotechnical Engineering Report (GER) submitted March 21, 2024. As shown in Figure 1 of that report, four borings were conducted throughout the site in May 2023, with one previous boring taken in 2022. Two temporary wells were installed and five Flat Dilatometer Tests (DMT) were performed to determine the strength and deformation characteristics of fine-grained soils. Depth of borings ranged from 30 feet below grade to 100 feet below grade. The native soils consisted of 2 to 4 inches of topsoil at ground surface as well as clayey sand, silty sand, sandy lean clay, fat clay, and lean clay with sand. These soils were deemed suitable for use as mat foundations and footings for the proposed tank, PRS building, and odor control facility.

Groundwater was encountered at a depth of about 6-7.5 ft below grade in most of the borings, therefore the contractor should be prepared to manage groundwater to a depth low enough to allow for the establishment of a stable working area (between 1 to 2 feet below the bottom of excavation).

2.2 Project Area Sensitive Structures

BC developed a list of potentially impacted businesses and organizations in Section 5 of the Preliminary Engineering Report, submitted June 11, 2021. Potential businesses and organizations to be impacted include:

Site Number	Address	Name of Business	Est. Distance from Nearest Work	Basements	Age of Structure
1	1901 Wilroy Road	Davis Boyz BBQ and Catering	260	No	Built 2000
2	1901 Wilroy Road	Evans Farms	375	No	Built 1945
3	300 Executive Court	The Children's Center	120	No	Built 1983
4	2017 Wilroy Road	Solomons Builders Inc.	385	No	Built 1950

5	1926 Wilroy Road	Rexel Electrical & Datacom Products BCS Inc. TRS Games	120	No	Built 1987
6	2040 Wilroy Road	Columbia Gas Transmission LLC	650	No	Sold 2018
7	1 QVC Drive	QVC	340	No	Built 1988

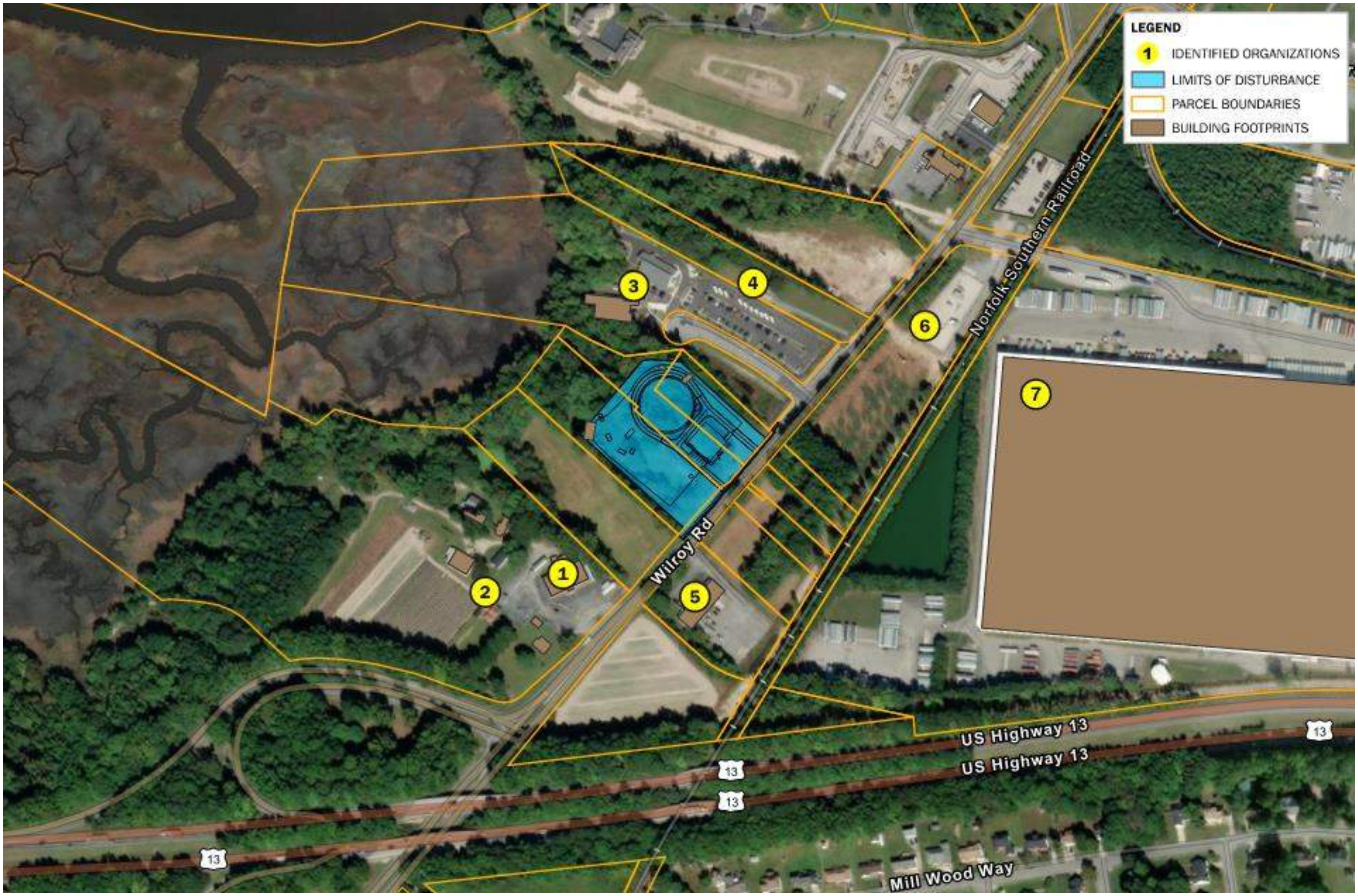


Figure 2-1. Project Area Sensitive Structures

Section 3

Anticipated Construction Means and Methods

This project consists of the construction of a pressure reducing pumping station (PRS), a 3-million-gallon offline storage facility, and an interceptor extension and water main construction along Wilroy Road. The interceptor extension and water main construction is anticipated to be open cut. Construction depths vary from 5 to 15 ft. At these depths, it is anticipated that trench boxes will be employed to provide trench excavation support and no sheeting will be employed. However, a sealed sheet pile wall method could be beneficial in limiting soil settlement from dewatering operations. As indicated by the GER, dewatering is expected in all open cut trenches and excavations.

Construction is anticipated to be executed in six main phases, as summarized in Table 3-1. Phases 2 through 4 are expected to occur concurrently, but they have been separated to identify specific impacts during construction.

Table 3-1. Summary of Construction Phases

Phase	Activity	Activity Description	Plan Sheets
1	Clearing and Grubbing	Removal of existing vegetation, trees, topsoil, and obstructions in the construction area	CD-101
2	PRS Construction	Excavation, dewatering, construction of new PRS and Odor Control	C-101 – C-302
3	Linear Work Along Wilroy	Excavation, dewatering, construction of interceptor extension and water main along Wilroy Road	C-501 – C-506
4	Tank Construction	Excavation, dewatering, construction of new Offline Storage Tank	C-101 – C-302
5	Road / Parking Construction	Roadway Construction	C-101, C-103
6	Restoration	Pavement and sidewalk restoration and final grading Backfilling of open cut trench and seeding	C-507 – C-600, L-101 – L-103

Specific equipment associated with each phase of work is provided in Appendix A.

Section 4

Potential Construction Related Impacts

Construction can generate noise, vibrations, and dewatering impacts on properties outside of the disturbed area. An overview of these potential impacts is described in the following sections. Appendix A and B provide detailed noise generation and vibration impacts analyses and include associated figures.

4.1 Noise Generation and Impacts

Noise generation and impacts were analyzed using the information provided in Section 7 of the *Transit Noise and Vibration Impact Assessment Manual* (Manual), published by the Federal Transit Administration in September 2018. As described in Section 7.1, a quantitative construction noise assessment was completed through the application of equation 7-1 to each piece of equipment anticipated to be used during each phase of construction. This equation is summarized as follows:

$$L_{eq.equip} = L_{emission} + 10\log(Adj_{Usage}) - 20\log(\frac{D}{50}) - 10G\log(\frac{D}{50}) \qquad \textbf{Eq. 7-I}$$

where:

$L_{eq,equip}$ = $L_{eq(t)}$ at a receiver from the operation of a single piece of equipment over a specified time period, dBA

$L_{emission}$ = noise emission level of the particular piece of equipment at the reference distance of 50 ft, dBA

Adj_{Usage} = usage factor to account for the fraction of time that the equipment is in use over the specified time period

D = distance from the receiver to the piece of equipment, ft

G = a constant that accounts for topography and ground effects

As an exact equipment roster and construction schedule have not been determined, a general assessment of construction noise was completed following a meeting of the engineer and contractor. For this level of assessment, a usage factor of +1 is assumed, so the term "10log(Adj_{Usage})" is equivalent to zero and is omitted from the equation. Term "G" represents ground effects, which the manual recommends to be ignored for this level of assessment, therefore the final term of the equation is also omitted. Values for the "$L_{emission}$" term were obtained from Table 7-1 of the Manual. For each phase of construction, the maximum sound level of a given construction phase (L_p) is defined as the sound generated from the two noisiest pieces of equipment. The sound generated from these two sources is summed via decibel addition, which is expressed as follows:

$$L_p = 10log_{10}(10^{\frac{L_1}{10}} + 10^{\frac{L_2}{10}} + \cdots + 10^{\frac{L_n}{10}}) \qquad \textbf{Eq. B-4}$$

where

L_1, L_2, L_n = individual source sound pressure levels to add

The highest one-hour sound generated during each phase was compared to the General Assessment Construction Noise Criteria. The noise criteria for residential areas is 90 decibels during 7:00 a.m. to 10:00 p.m. The results of the analysis are summarized in Table 4-1. Detailed results and associated figures are provided in Appendix A.

Table 4-1. Summary of Noise Impact Analysis

Phase	Noise Criteria Exceeded	Max One-Hour L_{eq} from 2 Noisiest Sources (dBA)	Max Exceedance (dBA)
1	No	81	-9
2	No	86	-4
3	No	77	-13
4	No	86	-4
5	No	80	-10
6	No	81	-9

As shown in Table 4-1, the maximum anticipated noise levels are anticipated to occur in Phases 2 and 4, however, they do not exceed general noise criteria. The highest anticipated generated noise is on site during construction of the PRS building and Offline Storage Facility, where the use of pile drivers and vibratory hammers are assumed to be used for tank foundation and sheeting/shoring. This work is anticipated to last for a few weeks early in the 2-year construction schedule.

4.2 Vibration Generation and Impacts

Vibration generation and impacts were analyzed using the information provided in Section 7 of the *Transit Noise and Vibration Impact Assessment Manual* (Manual), published by the Federal Transit Administration in September 2018. As described in Section 7.2, a quantitative construction vibration assessment was completed for each piece of equipment anticipated to be used during each phase of construction. The potential to cause both structural damage and human annoyance was assessed, as described in the following sections.

4.2.1 Potential for Structural Damage

The potential for structural damage due to vibrations produced during construction was assessed with equation 7-2 from the Manual:

$$PPV_{equip} = PPV_{ref} \times \left(\frac{25}{D}\right)^{1.5} \qquad \textbf{Eq. 7-2}$$

where:

PPV_{equip} = the peak particle velocity of the equipment adjusted for distance, in/sec

PPV_{ref} = the source reference vibration level at 25 ft, in/sec

D = distance from the equipment to the receiver, ft

Values for the "PPV_{ref}" term were obtained from Table 7-4 of the Manual. The results for each phase were compared to a peak particle velocity (PPV) value of 0.5 inch/second (in/sec), which is the construction vibration damage limit for reinforced-concrete, steel or timber buildings as provided in Table 7-5 of the manual. The results of the analysis are summarized in Table 4-2. Detailed results and associated figures are provided in Appendix B.

Table 4-2. Summary of Vibration Impact Analysis: Structural Damage

Phase	Max PPV (in/sec)	Associated Construction Activity	Recommended Vibrations Limits Exceeded (0.5 in/sec)?
1	0.007	Tree Clearing	No
2	0.055	Sheeting/Shoring	No
3	0.02	Backfill Compaction	No
4	0.115	Pile Driving	No
5	0.02	Roadway Paving	No
6	0.007	Final Site Grading	No

As shown in Table 4-2, anticipated vibration levels are well below the recommended vibration limit threshold of 0.5 in/sec for structural damage during all phases of the project. The highest anticipated vibration will be produced during pile driving for the tank foundations; however, no mitigation actions are needed. The pile driving is expected to last for a few weeks early in the 2-year construction schedule.

4.2.2 Potential for Human Annoyance

The potential for human annoyance due to vibrations produced during construction was assessed with equation 7-3 from the Manual:

$$L_{v.distance} = L_{vref} - 30log(\frac{D}{25}) \quad \textbf{Eq. 7-3}$$

where:

$L_{v.distance}$ = the rms velocity level adjusted for distance, VdB
L_{vref} = the source reference vibration level at 25 ft, VdB
D = distance from the equipment to the receiver, ft

Values for the “L_{vref}” term were obtained from Table 7-4 of the Manual. The results were then compared to a L_V value of 80 vibration decibels (VdB), which is the recommended maximum L_v for residential buildings experiencing infrequent vibration events. The results of the analysis are summarized in Table 4-3. Detailed results are provided in Appendix B.

Table 4-3. Summary of Vibration Impact Analysis: Human Annoyance		
Phase	80 Vdb Exceeded?	Max L_v (VdB)
1	No	65
2	Yes	83
3	No	74
4	Yes	90
5	No	74
6	No	65

As shown in Table 4-3, construction in Phases 2 and 4 is expected to produce vibration levels that exceed the recommended vibration limits for human annoyance. As expected, the highest produced vibration decibels are anticipated to be produced during pile driving for the tank's foundations and sheeting/shoring for construction of the PRS building. This exceedance is just outside of the site boundary and does not extend to nearby structures. Recommended mitigation actions are provided in Section 5 and detailed results are provided in Appendix B.

4.3 Dewatering Impacts on Ground and Structures Settlement

As discussed in Section 2.1, groundwater was encountered at depths of 6-7.5 ft below grade in the obtained soil borings, so dewatering is anticipated to be necessary to construct the OLSF and PRS foundations. The GER recommended that groundwater be lowered 2 ft below the bottom of the excavation to provide a stable working platform. Dewatering to these levels could result in an increase in effective soil stresses which may produce a risk of ground settlement in the vicinity of the project. Recommended mitigation actions are provided in Section 5 and detailed further in Appendix C.

4.4 Construction Impacts on Existing Topography, Hydrology, and Construction Travel Routes

All construction will take place on HRSD property at 1941 Wilroy Road except for the interceptor extension and water main construction, which should all occur within the public right-of-way or easement. For construction within existing pavement, existing drainage patterns will be maintained so no impacts to existing topography or hydrology are expected.

Construction impacts were determined to be negligible or very minimal in regard to public bus routes. The City of Suffolk Pink Route runs along Wilroy Road, heading southwest towards the project site; however, the route turns off of Wilroy Road onto Progress Road before reaching the project location and should not be impacted by this construction.

Services along Wilroy Road, including trash pickup, mail delivery, and bus services will maintain availability at all points during construction. Access to businesses along the Wilroy Road corridor of the project location will remain open at all times during construction; however, there may be delays during the Phase 3 – Interceptor Extension and Water Main Construction portion of the project due to lane closures in order to install the force main extension. Specific impacts may occur to the Children's Center for student pick up/drop off and QVC for any delivery trucks traveling on Wilroy Road. Typical traffic control of Wilroy Road during construction, as shown in the figures below, includes lane closures on a two-lane roadway using flaggers, lane closure operations through an unsignalized intersection, and shoulder closure with barrier operation.

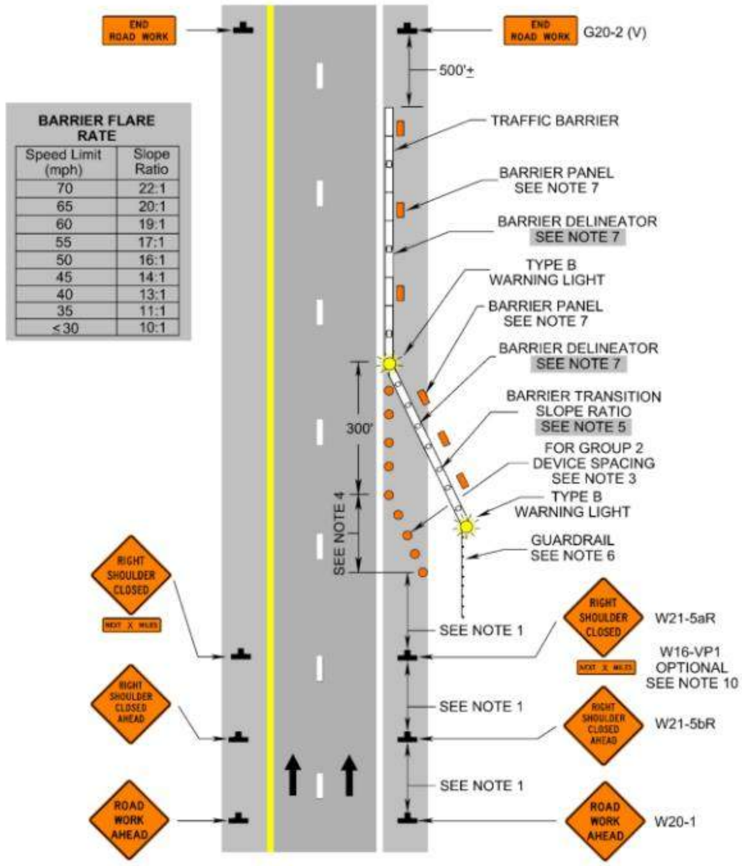


Figure 4-1. Traffic Control Detail 1

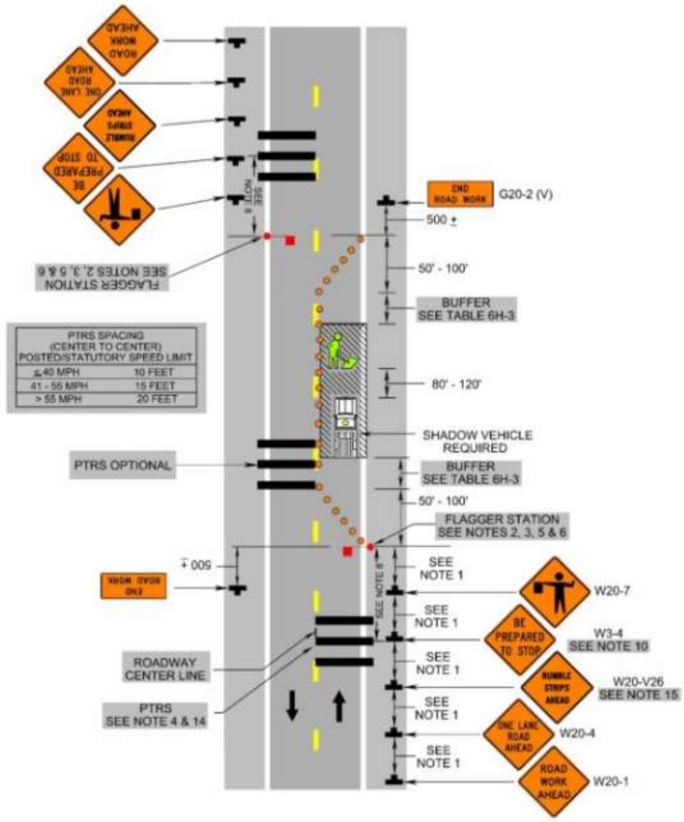


Figure 4-2. Traffic Control Detail 2

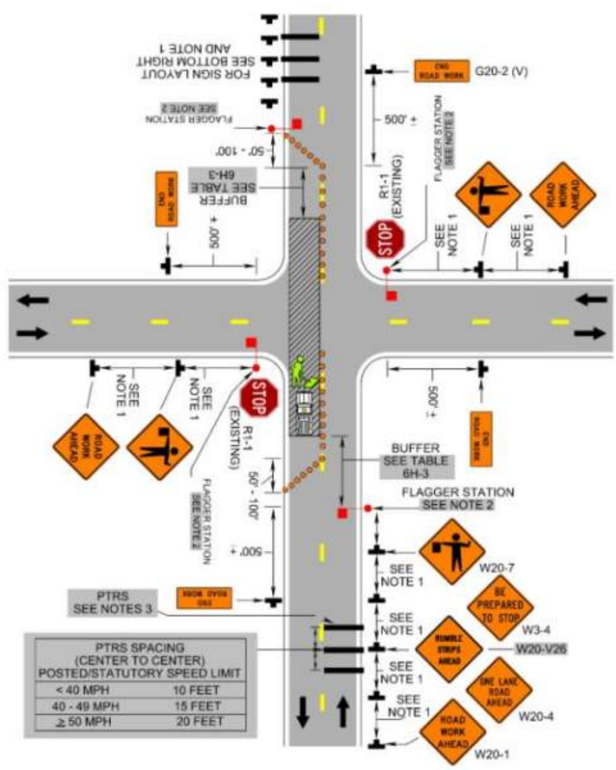


Figure 4-3. Traffic Control Detail 3

During construction of the interceptor extension and water main, stormwater runoff flow from the street may need to be directed around the excavation to maintain existing hydrology. As stated in Specification Sections 01 11 00 and 01 35 43, all areas disturbed by the Contractor, including stockpiling areas, sidewalks and access roads, shall be restored and restabilized according to the specifications.

Figure 4-4. Wilroy Road Interceptor Extension Construction Corridor, Looking Northeast

During construction, Erosion and Sediment Control (ESC) will occur in two phases, relating to site clearing and PRS and OLSF construction, respectively. Silt fence for both phases of ESC occur just within the Limits of Disturbance, covering the 3.05-acre site. On the northwestern portion of the site, temporary sediment traps will be used for the protection of the 100-foot Chesapeake Bay Protection Act Resource Protection Area (RPA) buffer which extends partially into the LOD. In Phase 1 of ESC, the temporary sediment trap will provide for 222.89 cubic yards (CY) of wet storage and 268.95 CY of dry storage. This ESC will be used during the majority of construction on the site. During Phase 2 of ESC, the focus will be building the long-term stormwater management feature (a constructed wetland). The Phase 2 temporary sediment trap will provide 41.69 CY of wet storage and 80.94 CY of dry storage.

4.5 Dust, Odor, and Other Emissions beyond Construction Zone

The most likely forms of fugitive emissions beyond the construction zone will consist of dust and odor. Dust is common to all forms of construction. As most of the construction will utilize existing pavement for delivery and removal of materials, best practices for controlling dust will be frequent road cleaning and/or application of water, as described in Specifications Section 01 35 43.3.01.G.

4.6 Locality Limitations, Requirements or Ordinances

The City of Suffolk has local requirements and ordinances that will govern the work. These requirements are summarized in the following sections.

4.6.1 City of Suffolk

The City of Suffolk code of ordinances limits operation or causing operation of equipment used in the construction, repair, alteration or demolition work on buildings, structures, alleys, or appurtenances thereto in the outdoors in any zoning district within 100 yards of a lawfully occupied dwelling between the hours of 10:00 p.m. and 6:00 a.m.; however, these limitations do not apply to construction of public projects, the repair or maintenance work performed on such project, or work performed by private or public utility companies for the repair of facilities or the restoration of services.

The work to be performed on this project will typically follow the hours as dictated by the City of Suffolk, generally assumed to be 8 a.m. to 5 p.m., Monday through Friday. In some circumstances, work outside of these timeframes may be necessary and will be communicated to the community.

Section 5

Risk Mitigation Analysis and Recommendations

As described in Section 4, this project may result in noise and dewatering impacts beyond the anticipated disturbed area of construction. The following sections provide recommended mitigation plans for consideration.

5.1 Noise Mitigation Recommendations

Since there are no specified decibel ordinances for the City of Suffolk, it is suggested in Table 7-2 of the Transit Noise and Vibration Impact Assessment Manual, published by the Federal Transit Administration in September 2018, that 90 decibels is a reasonable criterion for assessment of construction impacts in residential areas. Under this criterion, there are no phases of construction where 90 decibels is exceeded due to construction activities. Due to the proximity of construction with businesses on Wilroy Road, it is somewhat likely that business owners will notice and potentially be disturbed by construction noise. The following mitigation actions are therefore recommended:

- The contractor should be required to produce a noise mitigation plan prior to construction for review and approval by HRSD. This plan should outline how the contractor will reduce noise during construction wherever practicable. Specifically, the plan should address the following:
  - Measures to mitigate noise during construction using the 90 decibel threshold
  - Expected hours of construction (for example: 8:00 a.m. to 5:00 p.m. Monday through Friday)
- Once the plan has been approved, it is recommended that HRSD and the contractor meet with business owners on Wilroy Road to educate them on the project objectives and prepare them for the types of disturbances that they can expect during construction, including noise and vibration.
- The engineer will periodically monitor the decibel levels during the highest noise producing activities using handheld testing equipment and will immediately notify the contractor and HRSD if the anticipated levels are exceeded. The situation will be evaluated and an action plan developed to mitigate further exceedances. BC recommendation for noise monitoring locations include:
  - Edge of Limits of Disturbance (LOD)
  - The Children's Center
  - Davis Boyz BBQ and Catering
  - Businesses at 1926 Wilroy Road
- The noise impacts summarized above were modeled for the most impactful means and methods. Recognizing that there are alternative methods of construction, this report will be revised based on selected means and methods.
- The pipeline work along Wilroy Road will have a limited zone of influence for noise from construction. Trench boxes may be used for trench safety and the contractor should ensure that unnecessary noise from moving these boxes is created.

5.2 Vibration Recommendations

Since there are no specified vibration ordinances for the City of Suffolk, it is suggested in Table 7-4 of the Transit Noise and Vibration Impact Assessment Manual, published by the Federal Transit Administration that 80 VdB and 0.5 PPV are reasonable criterion for assessment of vibration impacts in residential areas. Under this criterion, the only phases where 80 VdB is exceeded is during Phases 2 and 4 of construction due to the pile driving for tank foundations and potential sheeting/shoring for the PRS construction. In other areas, maximum peak particle velocity (PPV_{ref}) and vibration decibels (VdB) produced during construction are not expected to exceed general vibration criteria. Due to the proximity of construction with businesses on Wilroy Road, it is somewhat likely that business owners will notice and potentially be disturbed by construction noise. The following mitigation actions are therefore recommended:

- The contractor should be required to produce a vibration mitigation plan prior to construction for review and approval by HRSD. This plan should outline how the contractor will reduce vibration during construction wherever practicable. Specifically, the plan should address the following:
  - Identify the highest vibration producing activities
  - Measures to mitigate vibration during pile driving activities (i.e., augering the first 5 feet prior to cast pile insertion for driving)
  - Expected hours of construction (for example: 8:00 a.m. to 5:00 p.m. Monday through Friday)
  - Options for reducing vibration producing activities if complaints are received
- Once the plan has been approved, it is recommended that HRSD and the contractor meet with business owners on Wilroy Road to educate on the project objectives and prepare them for the types of disturbances that they can expect during construction, including noise and vibration.
- The engineer will deploy vibration monitoring sensors and evaluate the data during the highest vibration producing activities and will immediately notify the contractor and HRSD if the anticipated levels are exceeded. The situation will be evaluated and action plan developed to mitigate further exceedances. The engineer's recommendation for vibration monitoring locations include:
  - Edge of Limits of Disturbance (LOD)
  - The Children's Center
  - Davis Boyz BBQ and Catering
  - Businesses at 1926 Wilroy Road
- The vibration impacts summarized above were modeled for the most impactful means and methods. Recognizing that there are alternative methods of construction, this report will be revised based on selected means and methods.
- Vibration from the construction of the pipelines along Wilroy Road should have limited impact on the surrounding area.

5.3 Dewatering Recommendations

Due to the need to dewater and the proximity of adjacent foundations to the construction area, it is possible that the lowering of the groundwater table could result in an increase in effective soil stresses which may produce a risk of ground settlement in the vicinity of the project. Groundwater modeling has been performed by the engineer that shows the cone of depression surrounding the lowest point of excavation (the lower PRS foundation) with the assumed dewatering value of 2 feet below the bottom of the excavation. With ground level at an elevation of 22 feet and the groundwater at 6 feet below grade, the cone of depression may be 17 feet to get to a dewatered elevation of -1 feet. This cone of depression radiates outward depending on soil conditions until it reaches its normal level. As the groundwater is depressed for construction, there is a chance of settlement which can be calculated using assumed parameters. The more information that is available, the accuracy of the calculations increases. The calculations performed were based on the geotechnical borings completed on the site and the published geology for the region; see Figure 5 1 for calculated limits of drawdown. Using this data, the geotechnical engineer's estimate for possible settlement is as follows:

Water Drawdown (ft)	Estimated Settlement (in)
15 - 20	¾ to 1
10 - 15	½ to ¾
5-10	¼ to ½

The closest structures to the center of the dewatering depression are the offices at 1926 Wilroy Road and the Children's Center. Both locations are estimated to fall within the 10-15 foot drawdown level and possibly experience ½ to ¾ inches of settlement. This settlement estimate is based on soil conditions obtained via soil test borings on site, identified in the Geotechnical Engineering Report, prepared by Schnabel Engineering included as reference in Appendix C – Anticipated Dewatering Impacts. This evaluation considers the same subsurface conditions encountered in our borings. While drawdown may be less at other areas around the site, settlement could differ depending on the subsurface materials underlying the surrounding areas. The dewatering impacts summarized above were modeled for the most impactful means and methods. Recognizing that there are alternative methods of construction, this report will be revised based on selected means and methods.

The amount of drawdown and settlement can be monitored during construction and the following recommendations are proposed to be implemented by the contractor and the engineer:

- The contractor should install groundwater monitoring wells on all four sides of excavation, approximately 50 feet from edge. These wells should be protected by the contractor during construction.
- The engineer will install settlement monitoring pins/plates at each of the following locations (see Figure 5-1):
  - The Children's Center
  - Davis Boyz BBQ and Catering
  - Evans Farms
  - Businesses at 1926 Wilroy Road, and
  - Wilroy Road directly adjacent to the site
  - Railroad across Wilroy Road, accessed via businesses at 1926 Wilroy Road
- During the initial dewatering effort, the engineer will monitor the groundwater levels biweekly (twice a week) and settlement pins weekly. If groundwater levels are significantly below the anticipated levels or the settlement pins move more than expected, the engineer will immediately notify the contractor and HRSD to develop an action plan. After the groundwater has been depressed for 3 weeks, monitoring will be conducted monthly.

Although the contractor is responsible for means and methods of construction, there are two anticipated construction methods for the site:

- Open pit excavation with sloped sidewalls using soil conditions to dictate the allowable slope. Some sheeting may be used to maintain the slope safely. Dewatering would be performed using wellpoints around the excavation or using a sump at the bottom of the excavation pit to collect and pump water to the sediment trap. This dewatering would be performed to the necessary elevation of -1 feet which would be 2 feet below the lowest part of the PRS foundation.
  - Pros: Less expensive, easier pit access, faster construction
  - Cons: Large cone of depression for groundwater leading to possible settlement, more material removed (at least temporarily), pit takes up more of site, high level of groundwater pumping necessary
- A sealed pit can be constructed using sheeting and shoring to isolated the excavation from the adjacent groundwater. This would require sheeting to be installed to the clay layer between 40-50 feet below grade. Once installed, a “bathtub” is created that would mitigate the impact of dewatering settlement on the nearby structures.
  - Pros: Reduce/eliminate settlement nearby, smaller pit, less groundwater pumping required
  - Cons: Very expensive (initial estimates put this at $5-10 million), makes pit access challenging for construction, slower construction, sheeting would be installed using vibration which creates more noise and vibration impacts, heavier equipment (cranes and larger excavators) required for construction

In summary, the open pit excavation is significantly lower cost but also leads to higher risk for settlement than the sealed pit option. The high cost of the sheeting and shoring would likely be much more than the cost to address any foundation settlement that may occur.

Regardless of the means selected, the contractor will be required to produce a dewatering and settlement mitigation plan prior to construction for review and approval by HRSD. This plan should outline how trenches will be dewatered and how the contractor will act to reduce settlement wherever practicable. The plan should be stamped and sealed by a geotechnical engineer.

Pipeline installation along Wilroy Road may require some dewatering however it will have a very limited zone of influence and not impact surrounding structures. Dewatering pumping should be directed by the contractor through appropriate sediment control devices and to the nearby stormwater drain and not encroach on travel lanes for Wilroy Road.

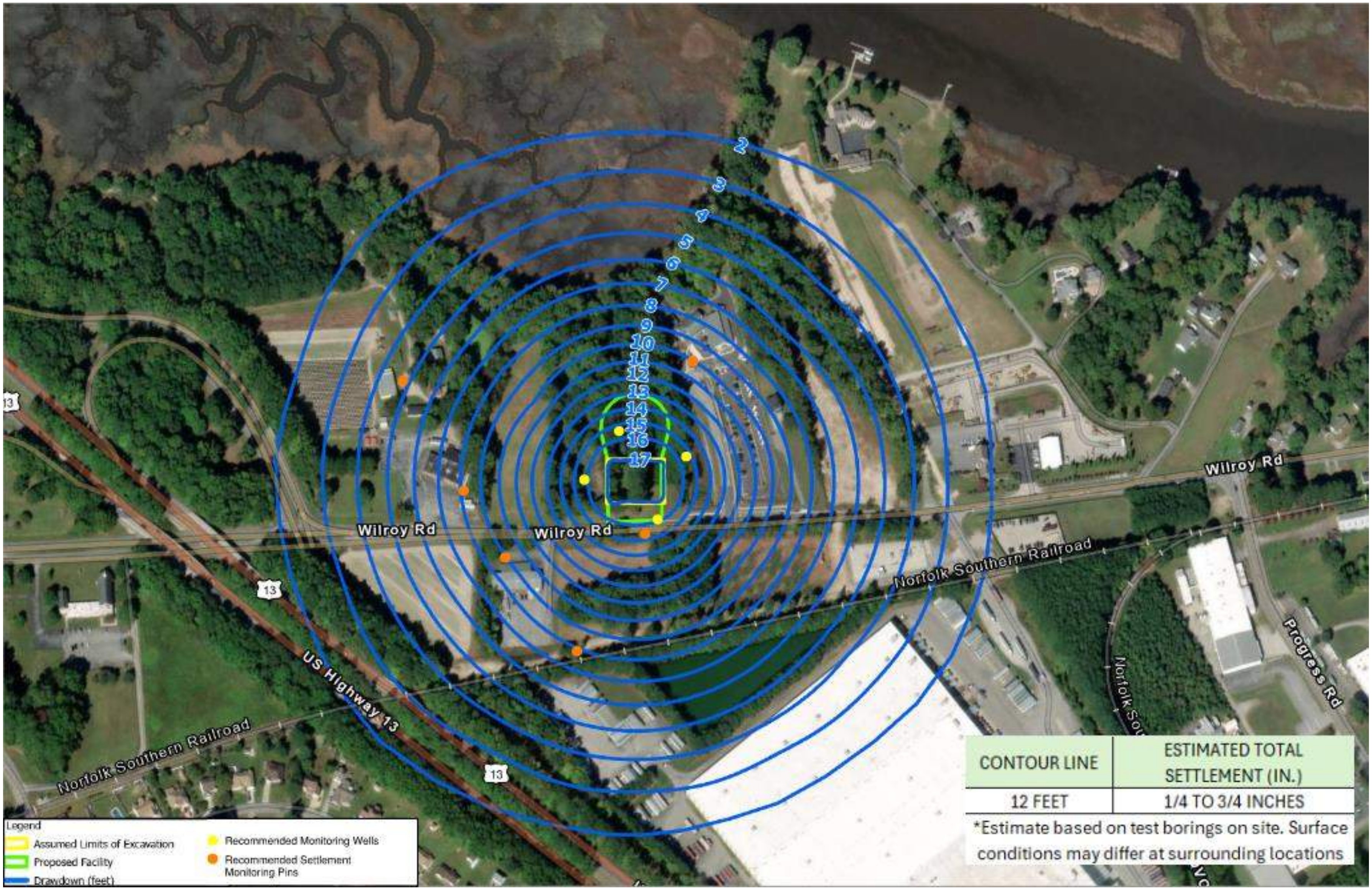


Figure 5-1. Dewatering Zone of Influence and Settlement Monitoring Recommended Locations

Section 6

References

Brown and Caldwell, *Wilroy Pressure Reducing Station and Off-Line Storage Facility Preliminary Engineering Report*, September 2022.

Federal Transit Administration, Transit Noise and Vibration Impact Assessment Manual, September 2018.

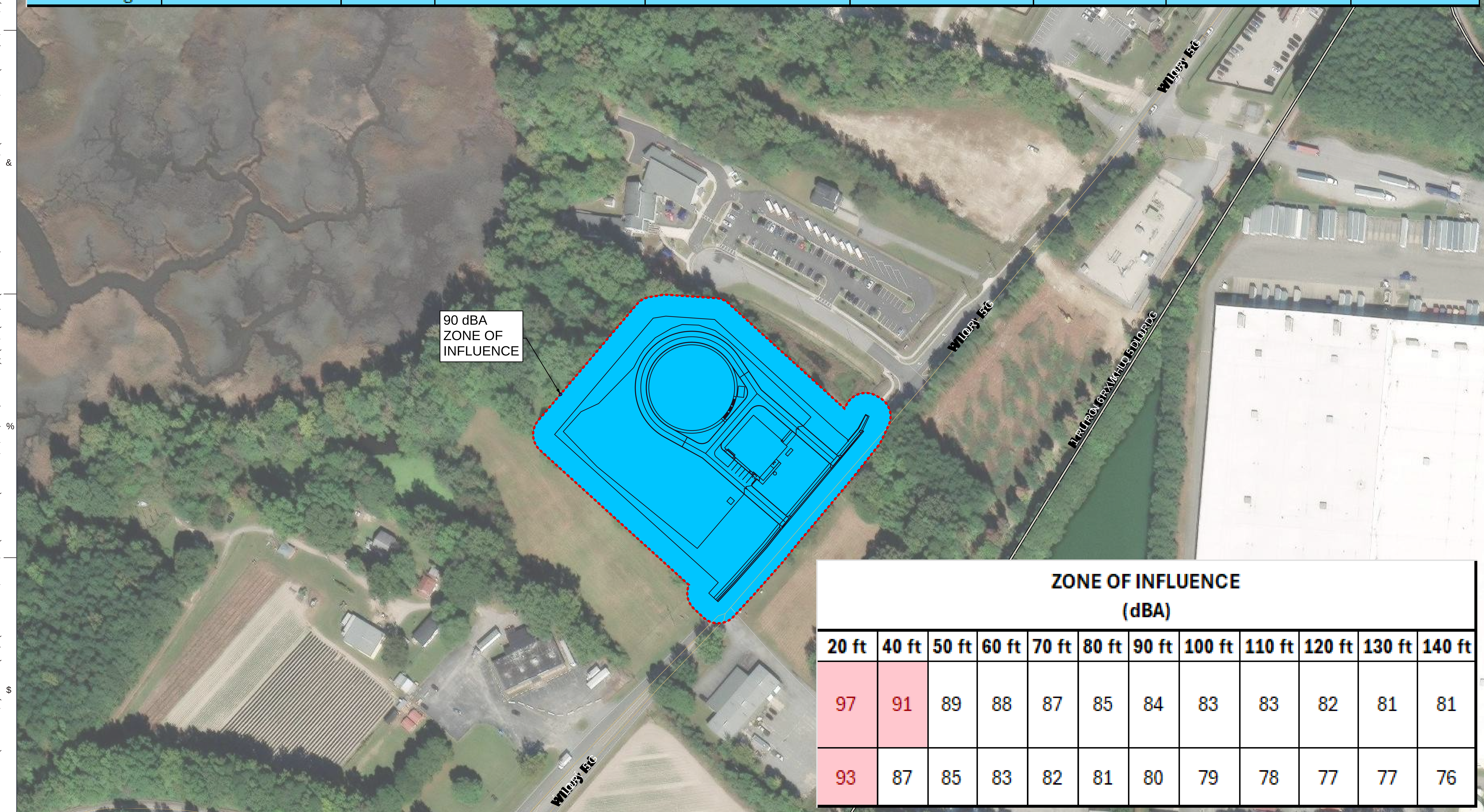
HRSD, *Design and Construction Standards, Section 12 Preconstruction Assessment and Damage Mitigation Procedures*, July 2024.

Schnabel Engineering, *HRSD Wilroy Road Pressure Reducing Station and Offline Storage Facility Geotechnical Engineering Report,* March 2024.

City of Suffolk, *Suffolk, VA Code of Ordinances Article VI. – Noise,* June, 2016.

Appendix A: Anticipated Noise Impacts

ACTIVITY	EQUIPMENT	QUANTITY	ACTIVITY DESCRIPTION	NOISE EMISSION LEVEL AT 50 FT ($L_{emission}$) (dBA)	MINIMUM DISTANCE TO RECEPTOR (ft)	ONE-HOUR L_{eq} (dBA)	MAX ONE-HOUR L_{eq} L_{eq} FROM 2 NOISIEST (dBA)	EXCEEDANCE OVER 90 dBA (dBA)
Clearing and Grubbing	Dump Truck	1	Removal of Materials	84	140	75	81	-9
	Bulldozer	1	Clearing	85	140	76		
	Chain saw	2	Tree Removal	85	140	76		
Clearing and Grubbing	Wood Chipper / Tub Grinder	1	Tree Removal	85	140	76	76	-14



ZONE OF INFLUENCE (dBA)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	110 ft	120 ft	130 ft	140 ft
97	91	89	88	87	85	84	83	83	82	81	81
93	87	85	83	82	81	80	79	78	77	77	76

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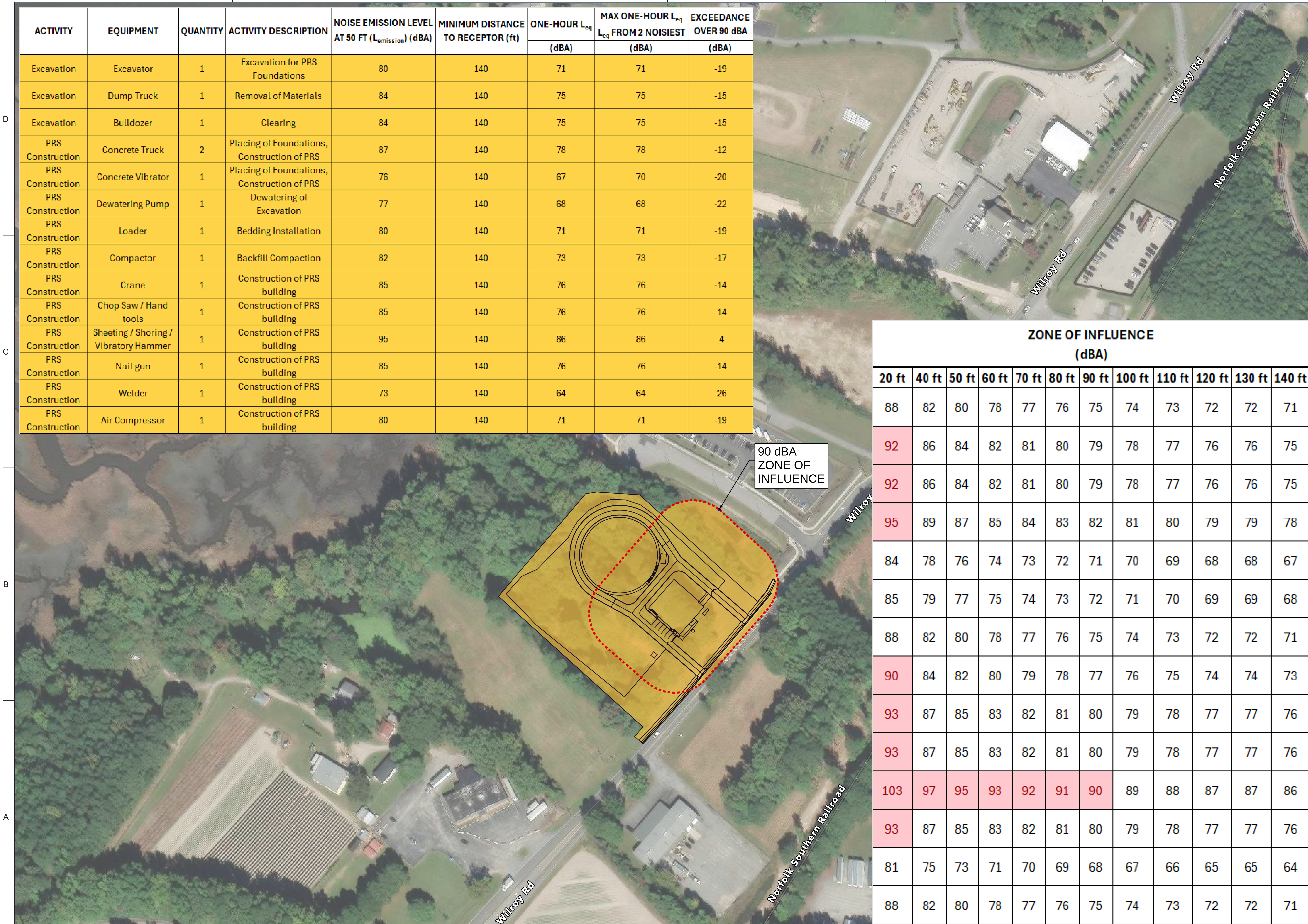
ACTIVITY	EQUIPMENT	QUANTITY	ACTIVITY DESCRIPTION	NOISE EMISSION LEVEL AT 50 FT ($L_{emission}$) (dBA)	MINIMUM DISTANCE TO RECEPTOR (ft)	ONE-HOUR L_{eq} (dBA)	MAX ONE-HOUR L_{eq} L_{eq} FROM 2 NOISIEST (dBA)	EXCEEDANCE OVER 90 dBA (dBA)
Excavation	Excavator	1	Excavation for PRS Foundations	80	140	71	71	-19
Excavation	Dump Truck	1	Removal of Materials	84	140	75	75	-15
Excavation	Bulldozer	1	Clearing	84	140	75	75	-15
PRS Construction	Concrete Truck	2	Placing of Foundations, Construction of PRS	87	140	78	78	-12
PRS Construction	Concrete Vibrator	1	Placing of Foundations, Construction of PRS	76	140	67	70	-20
PRS Construction	Dewatering Pump	1	Dewatering of Excavation	77	140	68	68	-22
PRS Construction	Loader	1	Bedding Installation	80	140	71	71	-19
PRS Construction	Compactor	1	Backfill Compaction	82	140	73	73	-17
PRS Construction	Crane	1	Construction of PRS building	85	140	76	76	-14
PRS Construction	Chop Saw / Hand tools	1	Construction of PRS building	85	140	76	76	-14
PRS Construction	Sheeting / Shoring / Vibratory Hammer	1	Construction of PRS building	95	140	86	86	-4
PRS Construction	Nail gun	1	Construction of PRS building	85	140	76	76	-14
PRS Construction	Welder	1	Construction of PRS building	73	140	64	64	-26
PRS Construction	Air Compressor	1	Construction of PRS building	80	140	71	71	-19



ZONE OF INFLUENCE (dBA)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	110 ft	120 ft	130 ft	140 ft
88	82	80	78	77	76	75	74	73	72	72	71
92	86	84	82	81	80	79	78	77	76	76	75
92	86	84	82	81	80	79	78	77	76	76	75
95	89	87	85	84	83	82	81	80	79	79	78
84	78	76	74	73	72	71	70	69	68	68	67
85	79	77	75	74	73	72	71	70	69	69	68
88	82	80	78	77	76	75	74	73	72	72	71
90	84	82	80	79	78	77	76	75	74	74	73
93	87	85	83	82	81	80	79	78	77	77	76
93	87	85	83	82	81	80	79	78	77	77	76
103	97	95	93	92	91	90	89	88	87	87	86
93	87	85	83	82	81	80	79	78	77	77	76
81	75	73	71	70	69	68	67	66	65	65	64
88	82	80	78	77	76	75	74	73	72	72	71

Brown AND Caldwell

N

0 100 200
Feet

Sources:
Aerial photo, ESRI & Affiliates, 2023

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HRSD Wilroy PRS and Tank

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APPROVED:

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157943

CLIENT PROJECT NUMBER
NP014000

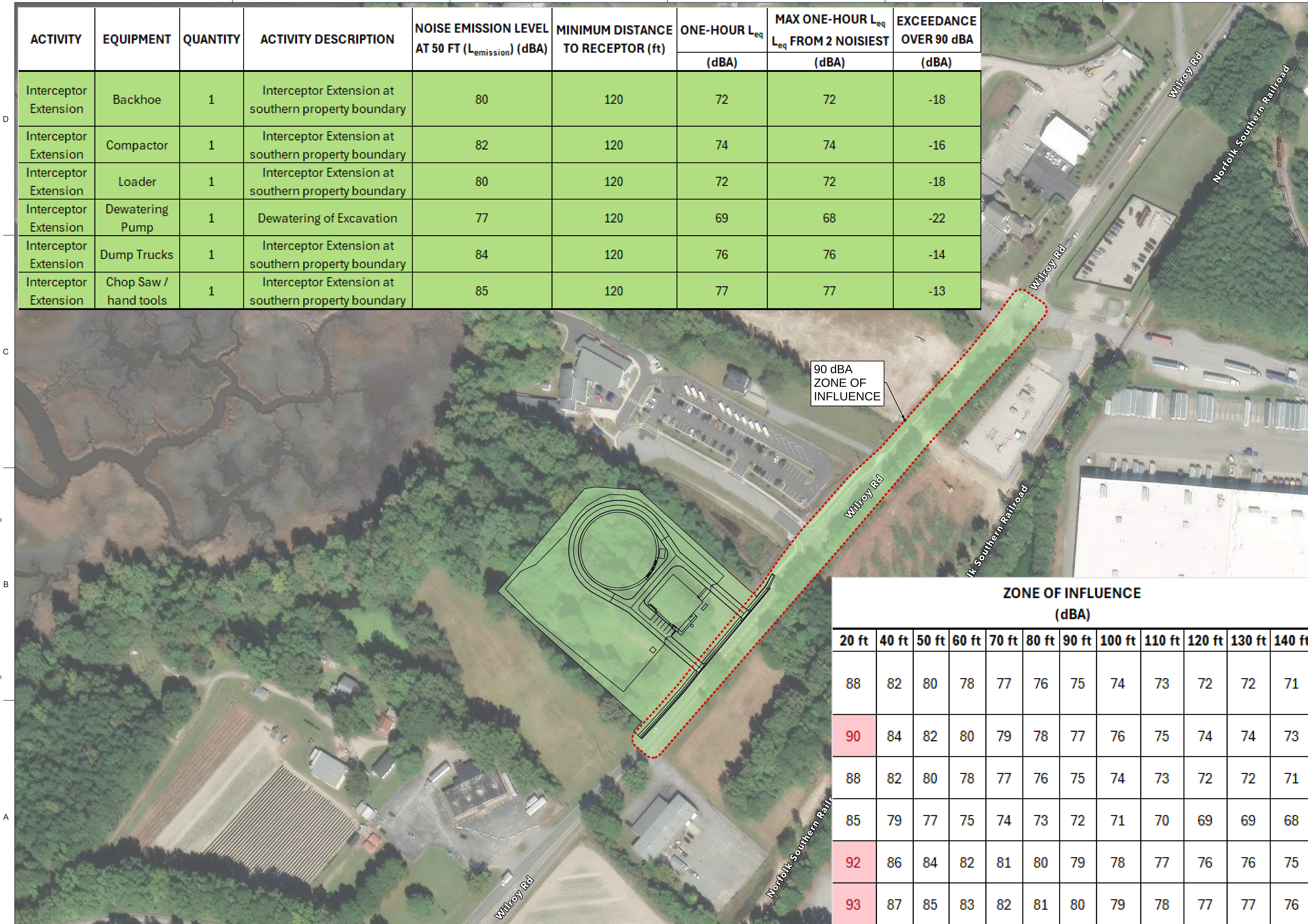
NOISE GENERATION AND IMPACTS
PHASE 2
EXCAVATION AND PRS CONSTRUCTION

DRAWING NUMBER
2

SHEET NUMBER
2 OF 6

Path: C:\USERS\TCRAWFORD\ONEDRIVE - BROWN AND CALDWELL\DOCUMENTS_DATA\GIS\WILROY FILENAME: TITLEBLOCK4GIS_2013.DWG PLOT DATE: ---- CAD USER: TINA CRAWFORD

ACTIVITY	EQUIPMENT	QUANTITY	ACTIVITY DESCRIPTION	NOISE EMISSION LEVEL AT 50 FT ($L_{emission}$) (dBA)	MINIMUM DISTANCE TO RECEPTOR (ft)	ONE-HOUR L_{eq} (dBA)	MAX ONE-HOUR L_{eq} L_{eq} FROM 2 NOISIEST (dBA)	EXCEEDANCE OVER 90 dBA (dBA)
Interceptor Extension	Backhoe	1	Interceptor Extension at southern property boundary	80	120	72	72	-18
Interceptor Extension	Compactor	1	Interceptor Extension at southern property boundary	82	120	74	74	-16
Interceptor Extension	Loader	1	Interceptor Extension at southern property boundary	80	120	72	72	-18
Interceptor Extension	Dewatering Pump	1	Dewatering of Excavation	77	120	69	68	-22
Interceptor Extension	Dump Trucks	1	Interceptor Extension at southern property boundary	84	120	76	76	-14
Interceptor Extension	Chop Saw / hand tools	1	Interceptor Extension at southern property boundary	85	120	77	77	-13



ZONE OF INFLUENCE (dBA)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	110 ft	120 ft	130 ft	140 ft
88	82	80	78	77	76	75	74	73	72	72	71
90	84	82	80	79	78	77	76	75	74	74	73
88	82	80	78	77	76	75	74	73	72	72	71
85	79	77	75	74	73	72	71	70	69	69	68
92	86	84	82	81	80	79	78	77	76	76	75
93	87	85	83	82	81	80	79	78	77	77	76

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N

0 100 200 Feet

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NOISE GENERATION AND IMPACTS
PHASE 3
INTERCEPTOR EXTENSION

DRAWING NUMBER
3

SHEET NUMBER
3 OF 6

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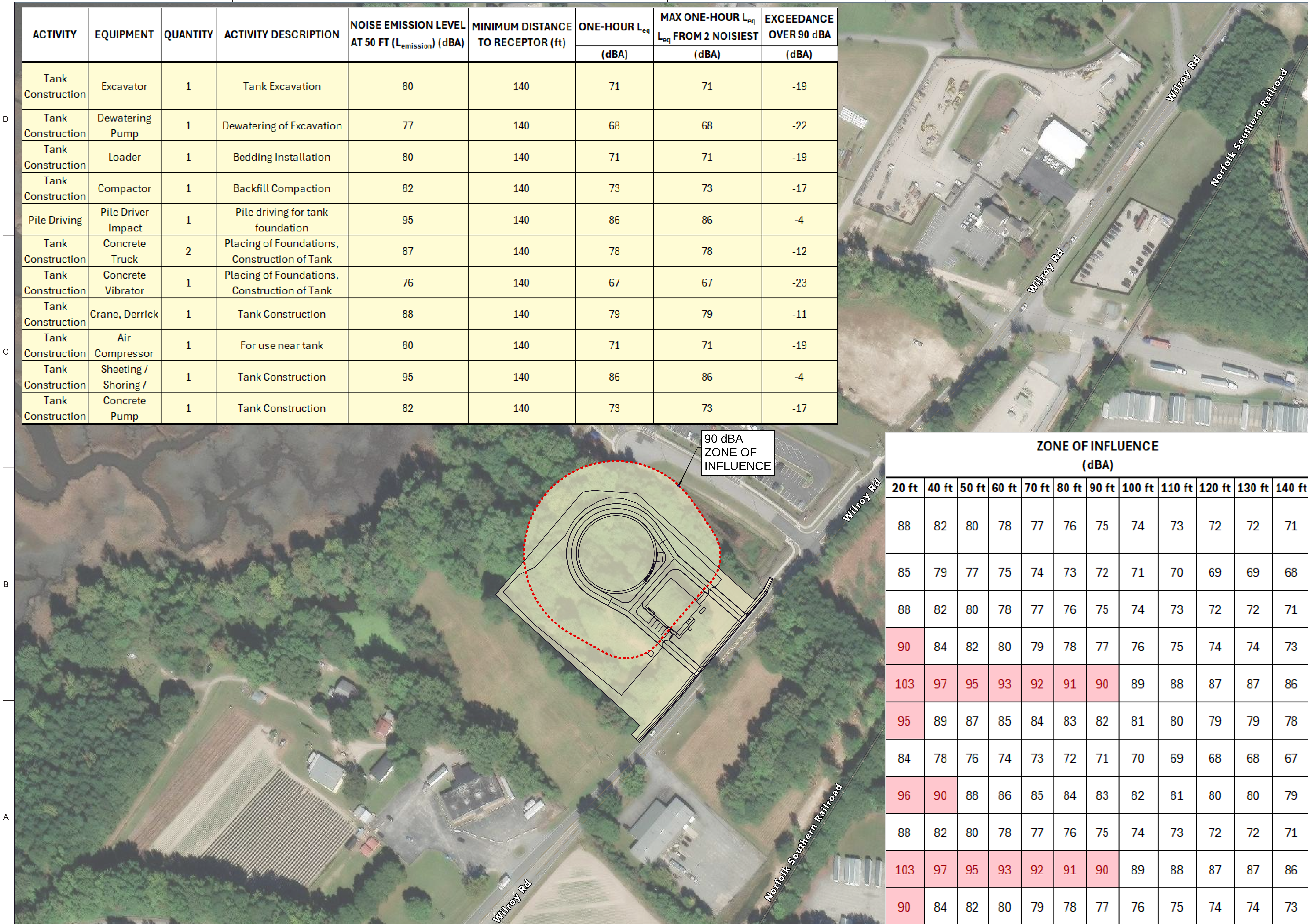
ACTIVITY	EQUIPMENT	QUANTITY	ACTIVITY DESCRIPTION	NOISE EMISSION LEVEL AT 50 FT ($L_{emission}$) (dBA)	MINIMUM DISTANCE TO RECEPTOR (ft)	ONE-HOUR L_{eq} (dBA)	MAX ONE-HOUR L_{eq} L_{eq} FROM 2 NOISIEST (dBA)	EXCEEDANCE OVER 90 dBA (dBA)
Tank Construction	Excavator	1	Tank Excavation	80	140	71	71	-19
Tank Construction	Dewatering Pump	1	Dewatering of Excavation	77	140	68	68	-22
Tank Construction	Loader	1	Bedding Installation	80	140	71	71	-19
Tank Construction	Compactor	1	Backfill Compaction	82	140	73	73	-17
Pile Driving	Pile Driver Impact	1	Pile driving for tank foundation	95	140	86	86	-4
Tank Construction	Concrete Truck	2	Placing of Foundations, Construction of Tank	87	140	78	78	-12
Tank Construction	Concrete Vibrator	1	Placing of Foundations, Construction of Tank	76	140	67	67	-23
Tank Construction	Crane, Derrick	1	Tank Construction	88	140	79	79	-11
Tank Construction	Air Compressor	1	For use near tank	80	140	71	71	-19
Tank Construction	Sheeting / Shoring /	1	Tank Construction	95	140	86	86	-4
Tank Construction	Concrete Pump	1	Tank Construction	82	140	73	73	-17



ZONE OF INFLUENCE (dBA)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	110 ft	120 ft	130 ft	140 ft
88	82	80	78	77	76	75	74	73	72	72	71
85	79	77	75	74	73	72	71	70	69	69	68
88	82	80	78	77	76	75	74	73	72	72	71
90	84	82	80	79	78	77	76	75	74	74	73
103	97	95	93	92	91	90	89	88	87	87	86
95	89	87	85	84	83	82	81	80	79	79	78
84	78	76	74	73	72	71	70	69	68	68	67
96	90	88	86	85	84	83	82	81	80	80	79
88	82	80	78	77	76	75	74	73	72	72	71
103	97	95	93	92	91	90	89	88	87	87	86
90	84	82	80	79	78	77	76	75	74	74	73

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0 100 200 Feet

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NOISE GENERATION AND IMPACTS
PHASE 4
TANK CONSTRUCTION

DRAWING NUMBER
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SHEET NUMBER
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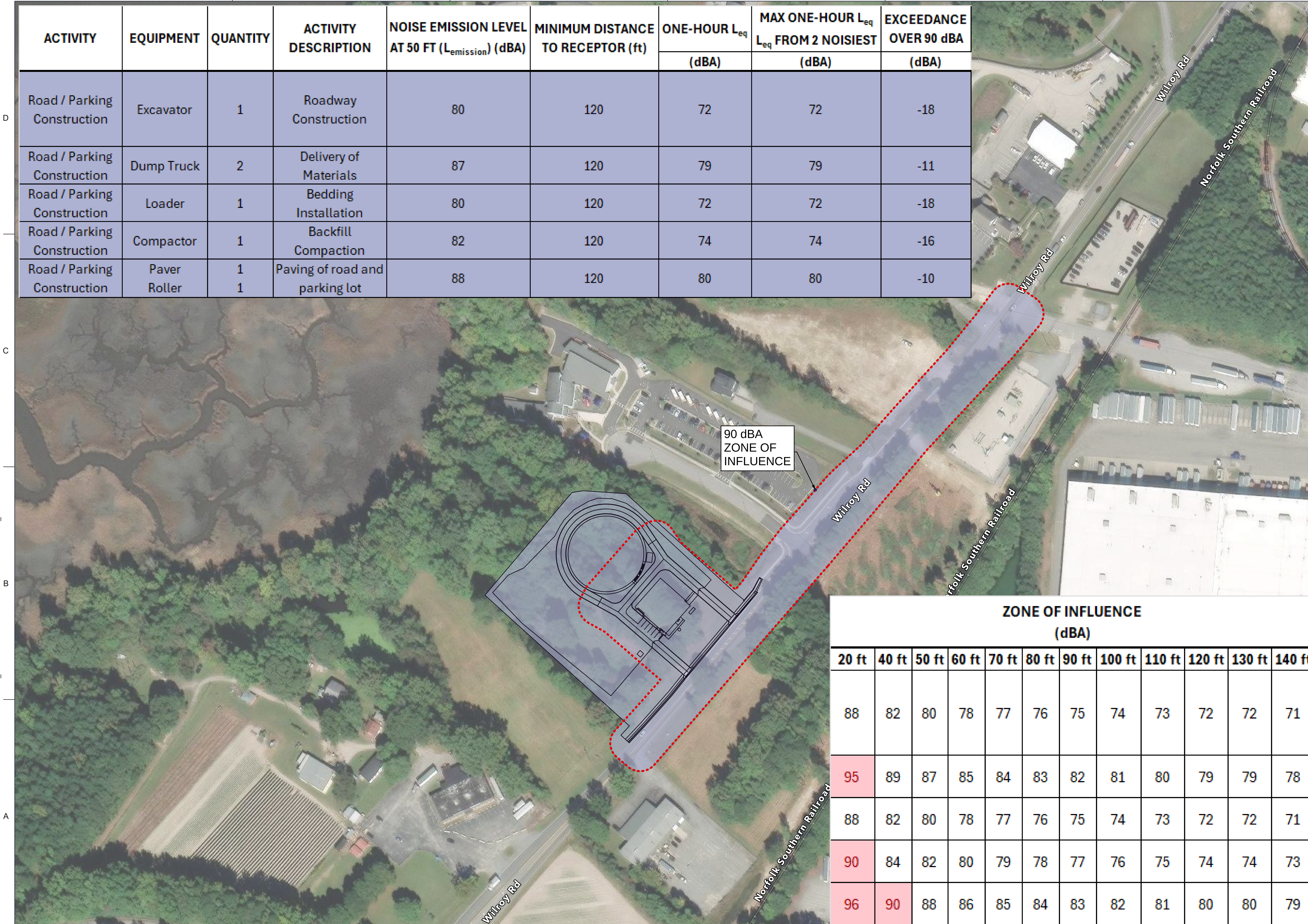
ACTIVITY	EQUIPMENT	QUANTITY	ACTIVITY DESCRIPTION	NOISE EMISSION LEVEL AT 50 FT ($L_{emission}$) (dBA)	MINIMUM DISTANCE TO RECEPTOR (ft)	ONE-HOUR L_{eq} (dBA)	MAX ONE-HOUR L_{eq} L_{eq} FROM 2 NOISIEST (dBA)	EXCEEDANCE OVER 90 dBA (dBA)
Road / Parking Construction	Excavator	1	Roadway Construction	80	120	72	72	-18
Road / Parking Construction	Dump Truck	2	Delivery of Materials	87	120	79	79	-11
Road / Parking Construction	Loader	1	Bedding Installation	80	120	72	72	-18
Road / Parking Construction	Compactor	1	Backfill Compaction	82	120	74	74	-16
Road / Parking Construction	Paver Roller	1 1	Paving of road and parking lot	88	120	80	80	-10



ZONE OF INFLUENCE (dBA)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	110 ft	120 ft	130 ft	140 ft
88	82	80	78	77	76	75	74	73	72	72	71
95	89	87	85	84	83	82	81	80	79	79	78
88	82	80	78	77	76	75	74	73	72	72	71
90	84	82	80	79	78	77	76	75	74	74	73
96	90	88	86	85	84	83	82	81	80	80	79

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N
0 100 200
Feet

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BC PROJECT NUMBER
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CLIENT PROJECT NUMBER
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NOISE GENERATION AND IMPACTS
PHASE 5
ROAD/PARKING CONSTRUCTION

DRAWING NUMBER
5

SHEET NUMBER
5 OF 6

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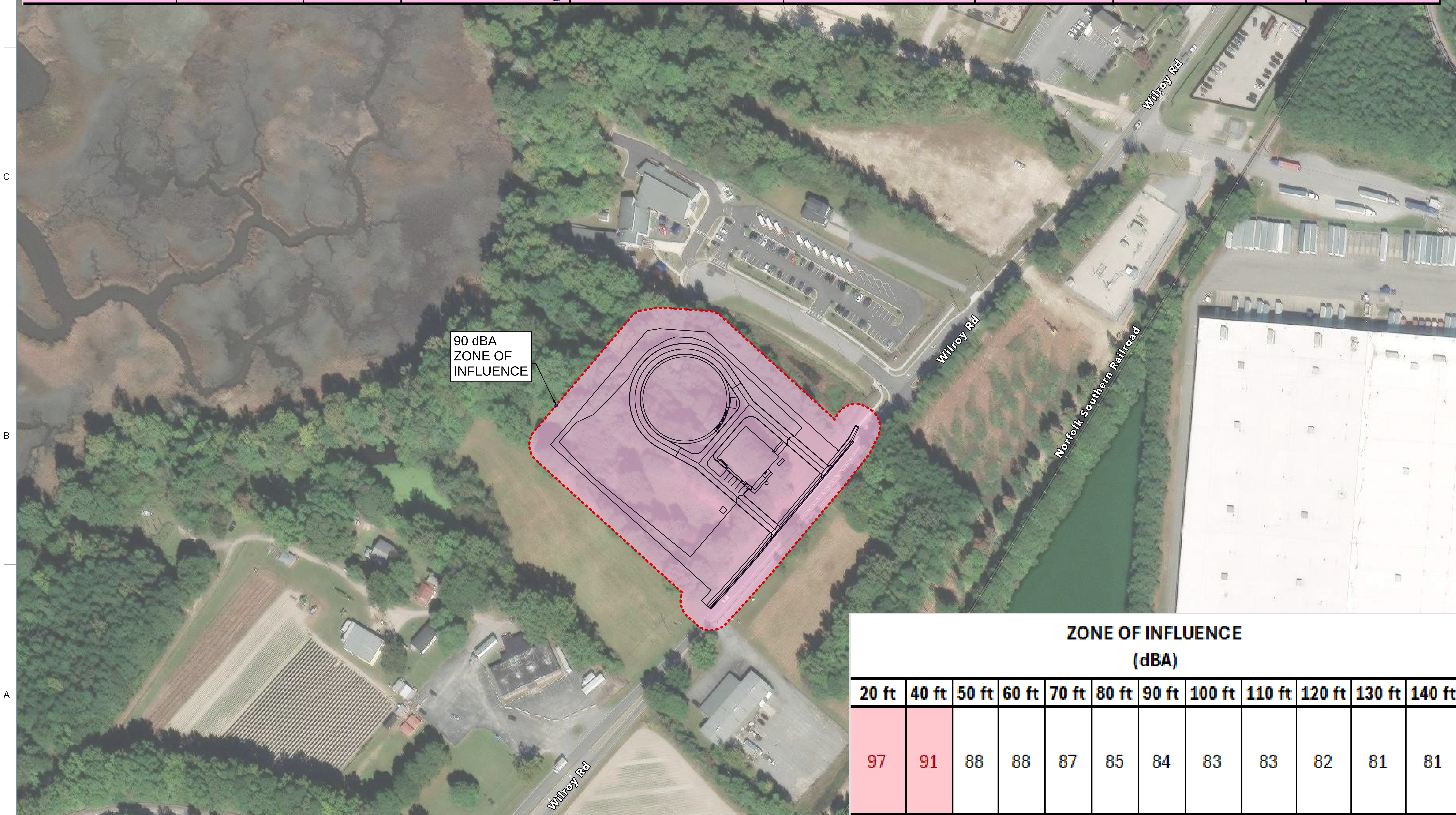
ACTIVITY	EQUIPMENT	QUANTITY	ACTIVITY DESCRIPTION	NOISE EMISSION LEVEL AT 50 FT ($L_{emission}$) (dBA)	MINIMUM DISTANCE TO RECEPTOR (ft)	ONE-HOUR L_{eq} (dBA)	MAX ONE-HOUR L_{eq} L_{eq} FROM 2 NOISIEST (dBA)	EXCEEDANCE OVER 90 dBA (dBA)
Restoration	Dump Truck	1	Installation of Materials	84	140	75	81	-9
	Bulldozer	1	Final Site Grading	85	140	76		
	Grader	1	Final Site Grading	85	140	76		



ZONE OF INFLUENCE (dBA)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	110 ft	120 ft	130 ft	140 ft
97	91	88	88	87	85	84	83	83	82	81	81

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N
0 100 200
Feet

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NOISE GENERATION AND IMPACTS
PHASE 6
RESTORATION

DRAWING NUMBER
6

SHEET NUMBER
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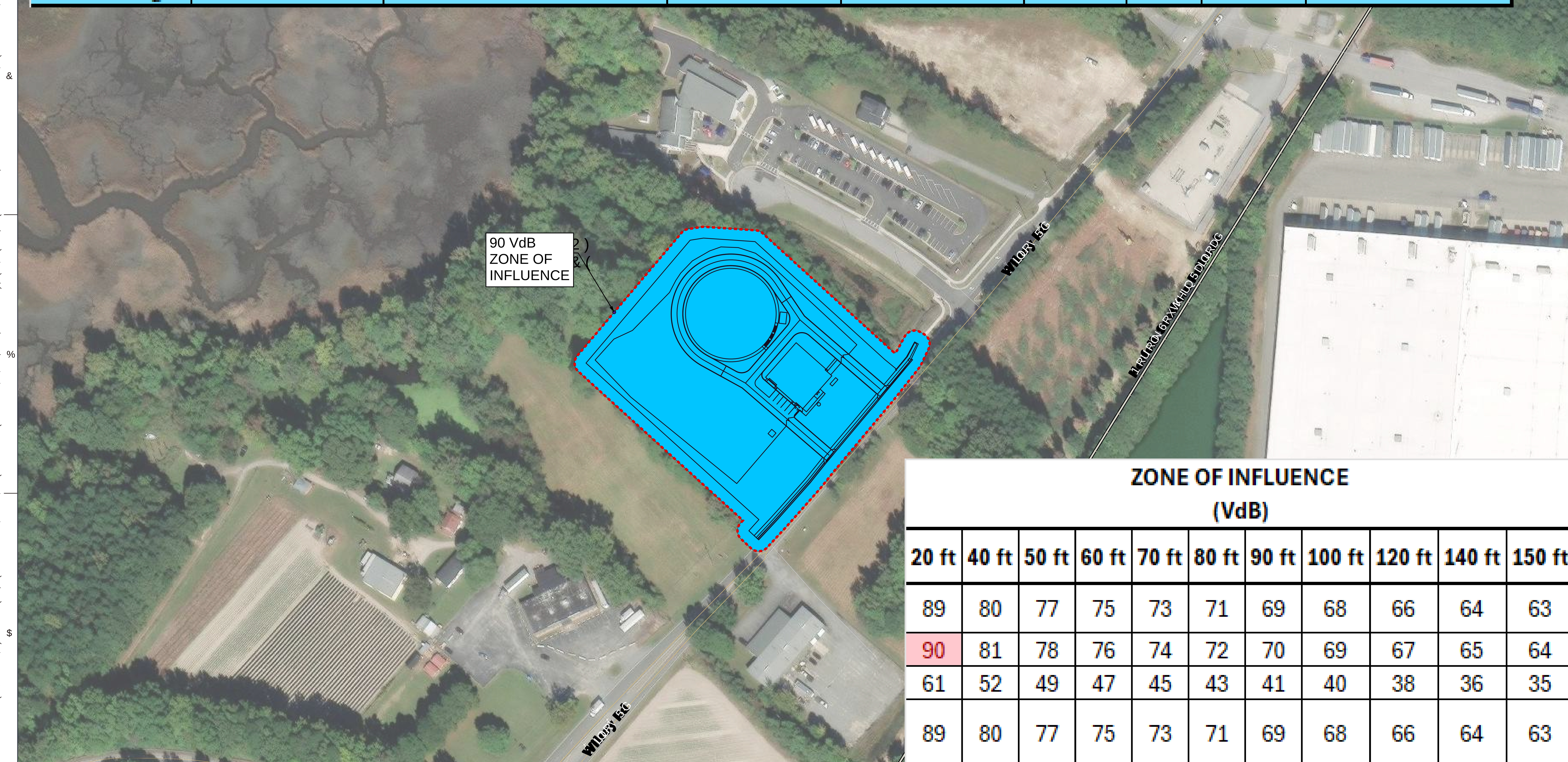
Appendix B: Anticipated Vibration Impacts

ACTIVITY	EQUIPMENT	ACTIVITY DESCRIPTION	PPV_{ref} (in/sec)	MINIMUM DISTANCE TO RECEPTOR (ft)	PPV_{equip} (in/sec)	L_{vref} (VdB)	$L_{v,distance}$ (VdB)	NOTES / ACTIONS
Clearing and Grubbing	Dump Truck	Removal of Materials	0.076	140	0.006	86	64	Barely perceptible
	Bulldozer	Clearing	0.089	140	0.007	87	65	
	Chain saw	Tree Removal	0.003	140	0.000	58	36	
Clearing and Grubbing	Wood Chipper / Tub Grinder	Tree Removal	0.076	140	0.006	86	64	Imperceptible



ZONE OF INFLUENCE (VdB)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	120 ft	140 ft	150 ft
89	80	77	75	73	71	69	68	66	64	63
90	81	78	76	74	72	70	69	67	65	64
61	52	49	47	45	43	41	40	38	36	35
89	80	77	75	73	71	69	68	66	64	63

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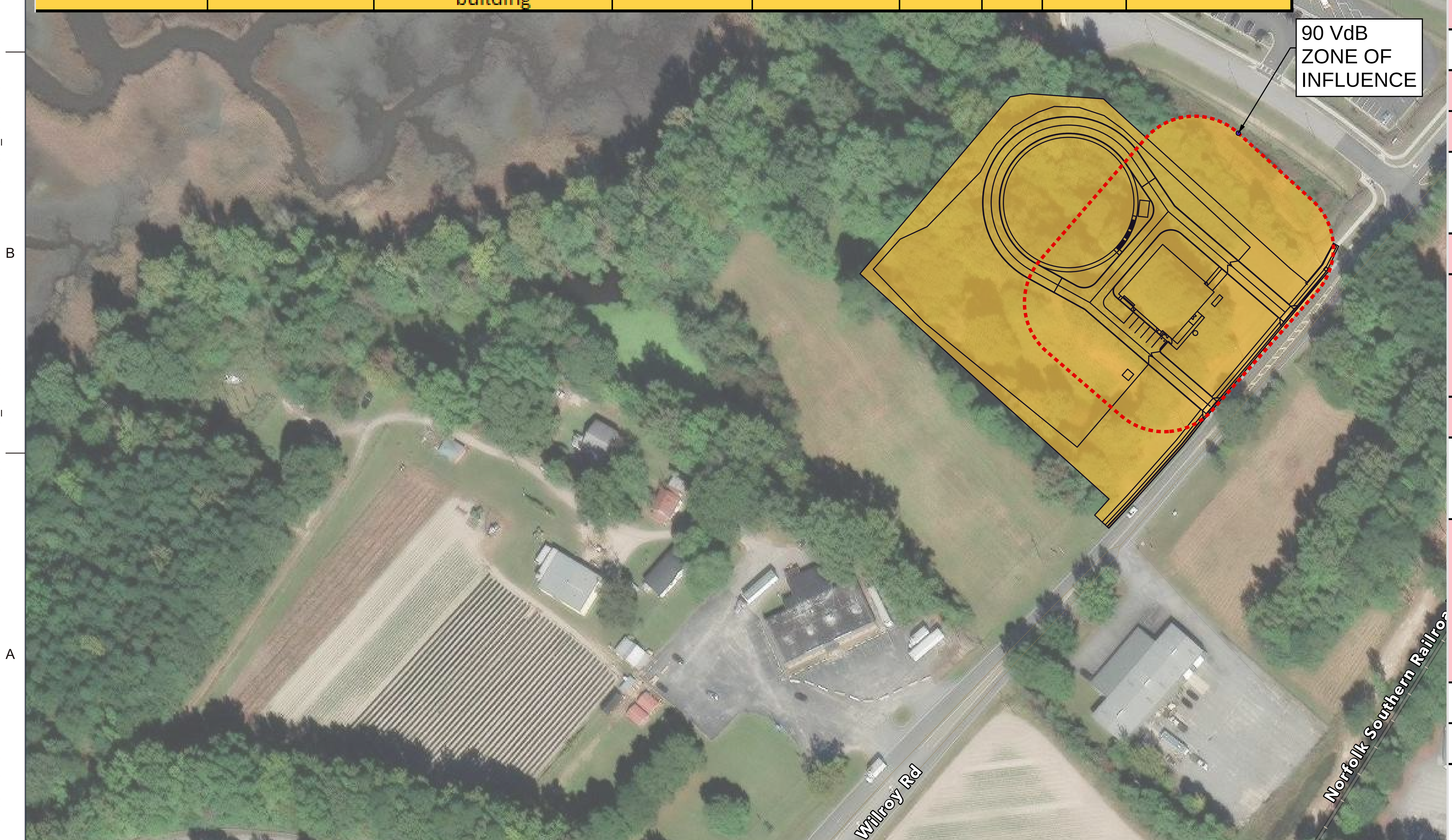
VIRGINIA BEACH, VA 23455

ACTIVITY	EQUIPMENT	ACTIVITY DESCRIPTION	PPV_{ref} (in/sec)	MINIMUM DISTANCE TO RECEPTOR (ft)	PPV_{equip} (in/sec)	L_{vref} (VdB)	$L_{v,distance}$ (VdB)	NOTES / ACTIONS
Excavation	Excavator	Excavation for PRS Foundations	0.089	140	0.007	87	65	Barely perceptible
Excavation	Dump Truck	Removal of Materials	0.076	140	0.006	86	64	Barely perceptible
Excavation	Bulldozer	Clearing	0.089	140	0.007	87	65	Barely perceptible
Dewatering	Dewatering Pump	Dewatering of Excavation	0.003	140	0.000	87	65	Imperceptible
PRS Construction	Concrete Truck	Placing of Foundations, Construction of PRS	0.076	140	0.006	86	64	Barely perceptible
PRS Construction	Loader	Bedding Installation	0.089	140	0.007	87	65	Barely perceptible
PRS Construction	Compactor	Backfill Compaction	0.21	140	0.016	94	72	Noticable, but very safe to buildings
PRS Construction	Crane	Construction of PRS	0.089	140	0.007	87	65	Barely perceptible
PRS Construction	Chop Saw / Hand tools	Construction of PRS building	0.003	140	0.000	58	36	Imperceptible
PRS Construction	Sheeting / Shoring / Vibratory Hammer	Construction of PRS building	0.734	140	0.055	105	83	Noticable, but very safe to buildings
PRS Construction	Nail gun	Construction of PRS	0.003	140	0.000	58	36	Imperceptible
PRS Construction	Welder	Construction of PRS	0.003	140	0.000	58	36	Imperceptible
PRS Construction	Air Compressor	Construction of PRS building	0.003	140	0.000	58	36	Imperceptible

ZONE OF INFLUENCE (VdB)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	120 ft	140 ft	150 ft
90	81	78	76	74	72	70	69	67	65	64
89	80	77	75	73	71	69	68	66	64	63
90	81	78	76	74	72	70	69	67	65	64
90	81	78	76	74	72	70	69	67	65	64
89	80	77	75	73	71	69	68	66	64	63
90	81	78	76	74	72	70	69	67	65	64
97	88	85	83	81	79	77	76	74	72	71
90	81	78	76	74	72	70	69	67	65	64
61	52	49	47	45	43	41	40	38	36	35
108	99	96	94	92	90	88	87	85	83	82
61	52	49	47	45	43	41	40	38	36	35
61	52	49	47	45	43	41	40	38	36	35
61	52	49	47	45	43	41	40	38	36	35



Sources:
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HRSD Wilroy PRS and Tank

REVISIONS

REV	DATE	DESCRIPTION

LINE IS 2 INCHES AT FULL SIZE

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DRAWN: TINA CRAWFORD
CHECKED:
CHECKED:
APPROVED:

FILENAME
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BC PROJECT NUMBER
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CLIENT PROJECT NUMBER
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VIBRATION IMPACTS
PHASE 2
EXCAVATION AND PRS CONSTRUCTION

DRAWING NUMBER
2

SHEET NUMBER
2 OF 6

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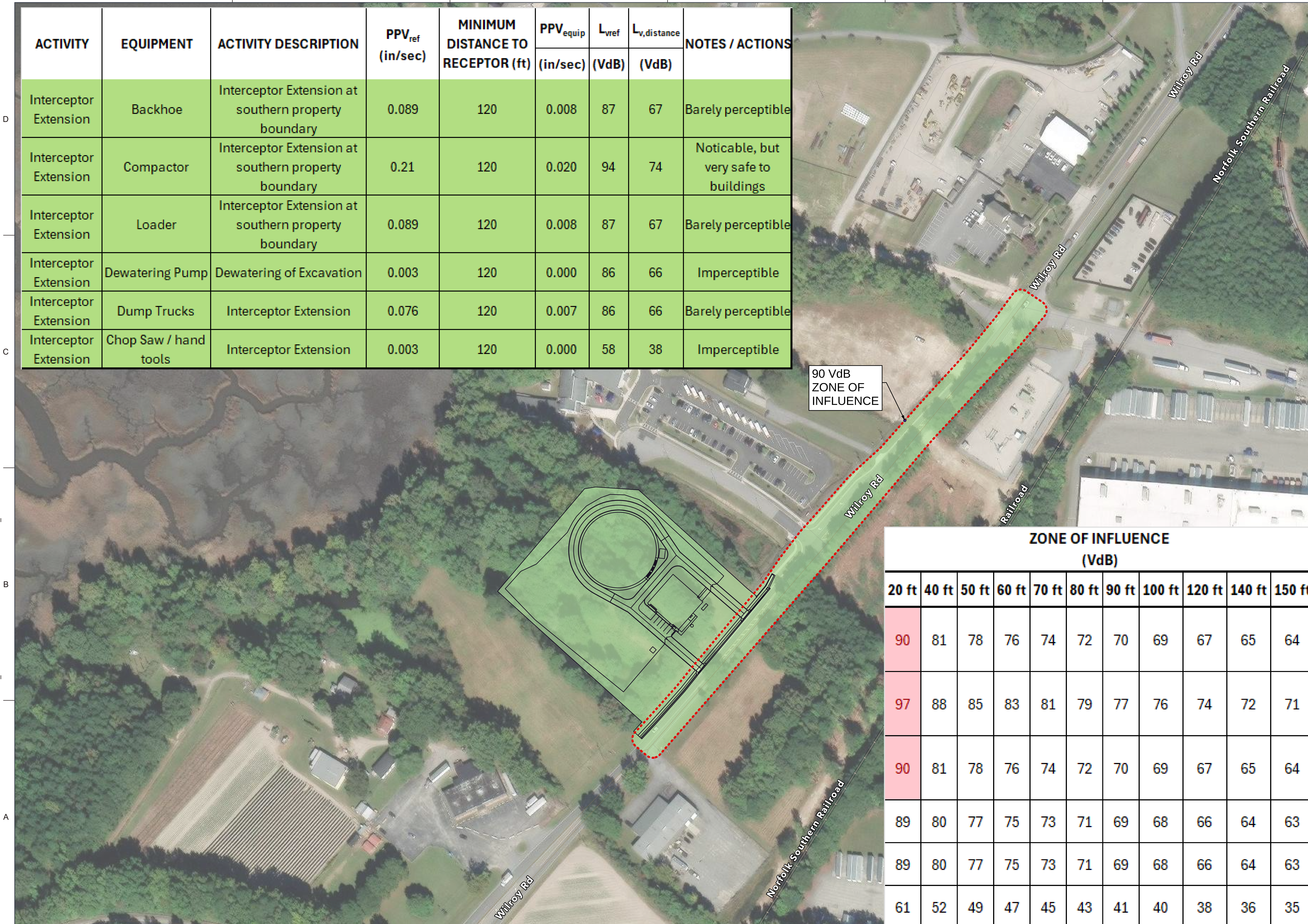
ACTIVITY	EQUIPMENT	ACTIVITY DESCRIPTION	PPV_{ref} (in/sec)	MINIMUM DISTANCE TO RECEPTOR (ft)	PPV_{equip} (in/sec)	L_{vref} (VdB)	$L_{v,distance}$ (VdB)	NOTES / ACTIONS
Interceptor Extension	Backhoe	Interceptor Extension at southern property boundary	0.089	120	0.008	87	67	Barely perceptible
Interceptor Extension	Compactor	Interceptor Extension at southern property boundary	0.21	120	0.020	94	74	Noticable, but very safe to buildings
Interceptor Extension	Loader	Interceptor Extension at southern property boundary	0.089	120	0.008	87	67	Barely perceptible
Interceptor Extension	Dewatering Pump	Dewatering of Excavation	0.003	120	0.000	86	66	Imperceptible
Interceptor Extension	Dump Trucks	Interceptor Extension	0.076	120	0.007	86	66	Barely perceptible
Interceptor Extension	Chop Saw / hand tools	Interceptor Extension	0.003	120	0.000	58	38	Imperceptible



ZONE OF INFLUENCE (VdB)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	120 ft	140 ft	150 ft
90	81	78	76	74	72	70	69	67	65	64
97	88	85	83	81	79	77	76	74	72	71
90	81	78	76	74	72	70	69	67	65	64
89	80	77	75	73	71	69	68	66	64	63
89	80	77	75	73	71	69	68	66	64	63
61	52	49	47	45	43	41	40	38	36	35

Brown AND Caldwell

N
0 100 200
Feet

Sources:
Aerial photo, ESRI & Affiliates, 2023

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HRSD

1434 AIR RAIL AVENUE
VIRGINIA BEACH, VA 23455

HRSD Wilroy PRS and Tank

REVISIONS

REV	DATE	DESCRIPTION

LINE IS 2 INCHES AT FULL SIZE

DESIGNED: RYAN FOLEY
DRAWN: TINA CRAWFORD
CHECKED:
CHECKED:
APPROVED:

FILENAME
157943-Noise Analysis.aprx
BC PROJECT NUMBER
157943
CLIENT PROJECT NUMBER
NP014000

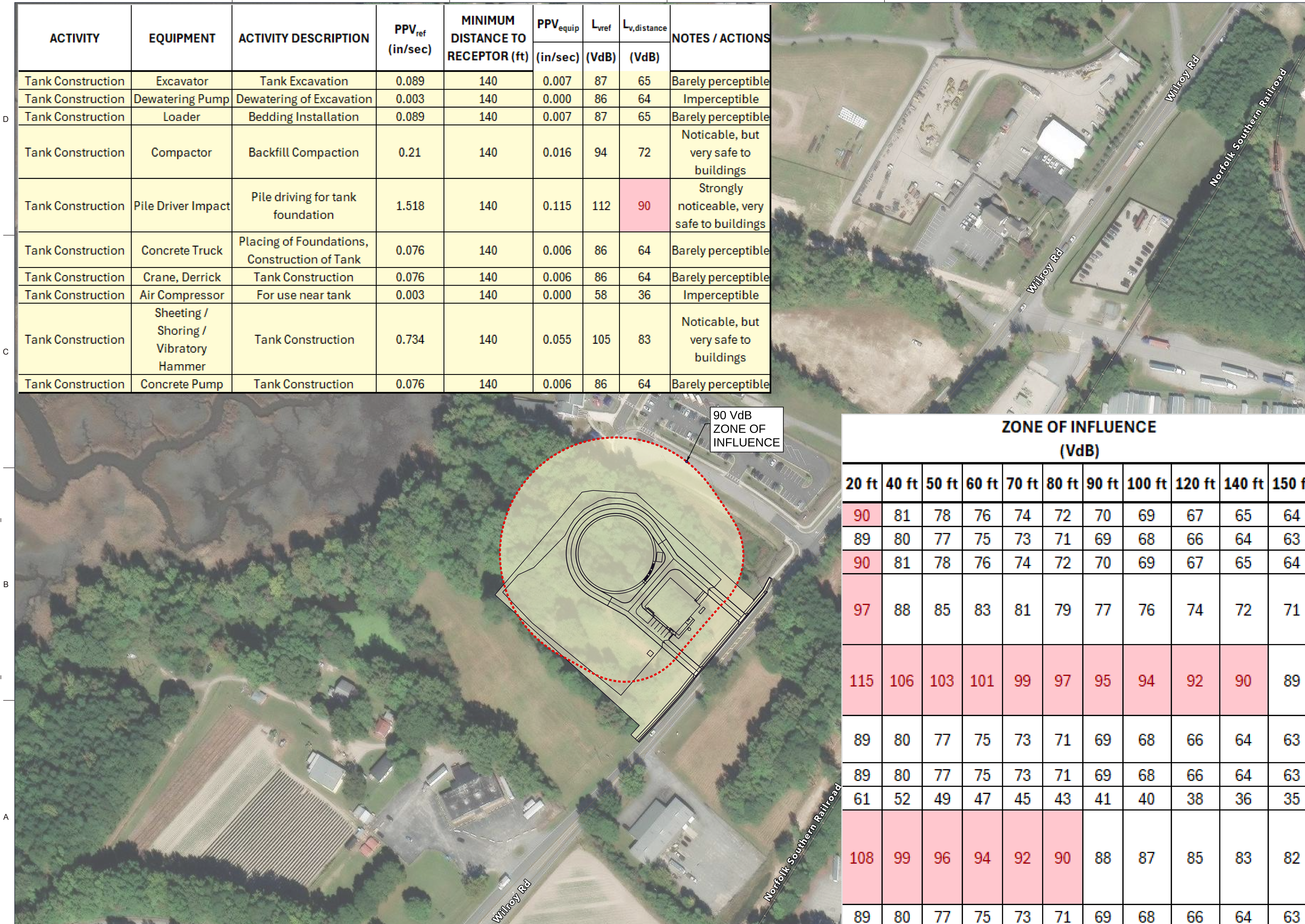
VIBRATION IMPACTS
PHASE 3
INTERCEPTOR EXTENSION

DRAWING NUMBER
3

SHEET NUMBER
3 OF 6

Path: C:\USERS\TCRAWFORD\ONEDRIVE - BROWN AND CALDWELL\DOCUMENTS_DATA\GIS\WILROY FILENAME: TITLEBLOCK4GIS_2013.DWG PLOT DATE: ---- CAD USER: TINA CRAWFORD

ACTIVITY	EQUIPMENT	ACTIVITY DESCRIPTION	PPV_{ref} (in/sec)	MINIMUM DISTANCE TO RECEPTOR (ft)	PPV_{equip} (in/sec)	L_{vref} (VdB)	$L_{v,distance}$ (VdB)	NOTES / ACTIONS
Tank Construction	Excavator	Tank Excavation	0.089	140	0.007	87	65	Barely perceptible
Tank Construction	Dewatering Pump	Dewatering of Excavation	0.003	140	0.000	86	64	Imperceptible
Tank Construction	Loader	Bedding Installation	0.089	140	0.007	87	65	Barely perceptible
Tank Construction	Compactor	Backfill Compaction	0.21	140	0.016	94	72	Noticable, but very safe to buildings
Tank Construction	Pile Driver Impact	Pile driving for tank foundation	1.518	140	0.115	112	90	Strongly noticeable, very safe to buildings
Tank Construction	Concrete Truck	Placing of Foundations, Construction of Tank	0.076	140	0.006	86	64	Barely perceptible
Tank Construction	Crane, Derrick	Tank Construction	0.076	140	0.006	86	64	Barely perceptible
Tank Construction	Air Compressor	For use near tank	0.003	140	0.000	58	36	Imperceptible
Tank Construction	Sheeting / Shoring / Vibratory Hammer	Tank Construction	0.734	140	0.055	105	83	Noticable, but very safe to buildings
Tank Construction	Concrete Pump	Tank Construction	0.076	140	0.006	86	64	Barely perceptible



Sources:
Aerial photo, ESRI & Affiliates, 2023

ZONE OF INFLUENCE (VdB)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	120 ft	140 ft	150 ft
90	81	78	76	74	72	70	69	67	65	64
89	80	77	75	73	71	69	68	66	64	63
90	81	78	76	74	72	70	69	67	65	64
97	88	85	83	81	79	77	76	74	72	71
115	106	103	101	99	97	95	94	92	90	89
89	80	77	75	73	71	69	68	66	64	63
89	80	77	75	73	71	69	68	66	64	63
61	52	49	47	45	43	41	40	38	36	35
108	99	96	94	92	90	88	87	85	83	82
89	80	77	75	73	71	69	68	66	64	63

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HRSD

1434 AIR RAIL AVENUE
VIRGINIA BEACH, VA 23455

HRSD Wilroy PRS and Tank

REVISIONS

REV	DATE	DESCRIPTION

LINE IS 2 INCHES AT FULL SIZE

DESIGNED: RYAN FOLEY
DRAWN: TINA CRAWFORD
CHECKED:
CHECKED:
APPROVED:

FILENAME
157943-Noise Analysis.aprx
BC PROJECT NUMBER
157943
CLIENT PROJECT NUMBER
NP014000

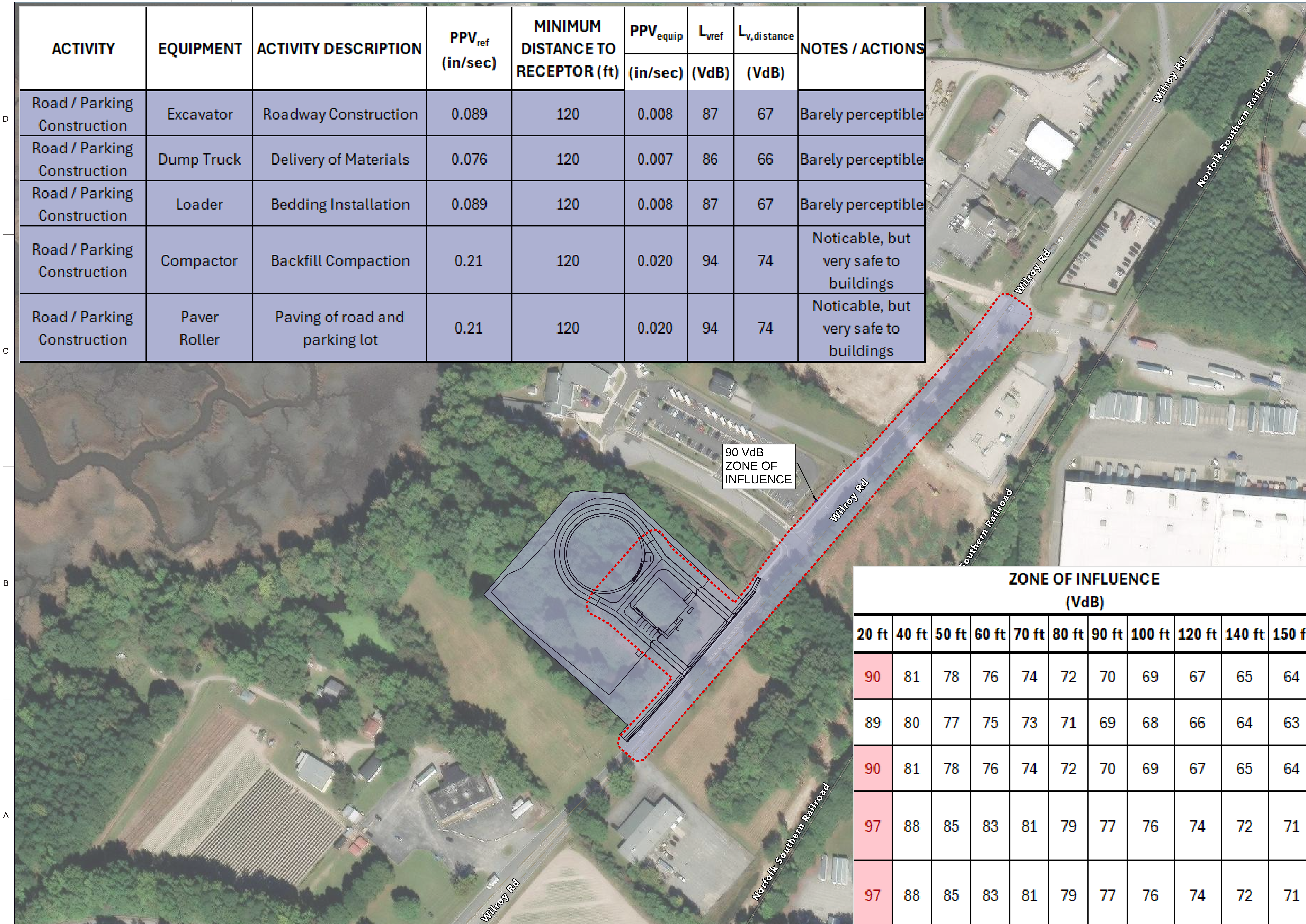
VIBRATION IMPACTS
PHASE 4
TANK CONSTRUCTION

DRAWING NUMBER
4

SHEET NUMBER
4 OF 6

Path: C:\USERS\TCRAWFORD\ONEDRIVE - BROWN AND CALDWELL\DOCUMENTS_DATA\GIS\WILROY FILENAME: TITLEBLOCK4GIS_2013.DWG PLOT DATE: ---- CAD USER: TINA CRAWFORD

ACTIVITY	EQUIPMENT	ACTIVITY DESCRIPTION	PPV_{ref} (in/sec)	MINIMUM DISTANCE TO RECEPTOR (ft)	PPV_{equip} (in/sec)	L_{vref} (VdB)	$L_{v,distance}$ (VdB)	NOTES / ACTIONS
Road / Parking Construction	Excavator	Roadway Construction	0.089	120	0.008	87	67	Barely perceptible
Road / Parking Construction	Dump Truck	Delivery of Materials	0.076	120	0.007	86	66	Barely perceptible
Road / Parking Construction	Loader	Bedding Installation	0.089	120	0.008	87	67	Barely perceptible
Road / Parking Construction	Compactor	Backfill Compaction	0.21	120	0.020	94	74	Noticable, but very safe to buildings
Road / Parking Construction	Paver Roller	Paving of road and parking lot	0.21	120	0.020	94	74	Noticable, but very safe to buildings



ZONE OF INFLUENCE (VdB)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	120 ft	140 ft	150 ft
90	81	78	76	74	72	70	69	67	65	64
89	80	77	75	73	71	69	68	66	64	63
90	81	78	76	74	72	70	69	67	65	64
97	88	85	83	81	79	77	76	74	72	71
97	88	85	83	81	79	77	76	74	72	71

Brown AND Caldwell

N

0 100 200
Feet

Sources:
Aerial photo, ESRI & Affiliates, 2023

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HRSD

1434 AIR RAIL AVENUE
VIRGINIA BEACH, VA 23455

HRSD Wilroy PRS and Tank

REVISIONS

REV	DATE	DESCRIPTION

LINE IS 2 INCHES AT FULL SIZE

DESIGNED: RYAN FOLEY
DRAWN: TINA CRAWFORD
CHECKED:
CHECKED:
APPROVED:

FILENAME
157943-Noise Analysis.aprx

BC PROJECT NUMBER
157943

CLIENT PROJECT NUMBER
NP014000

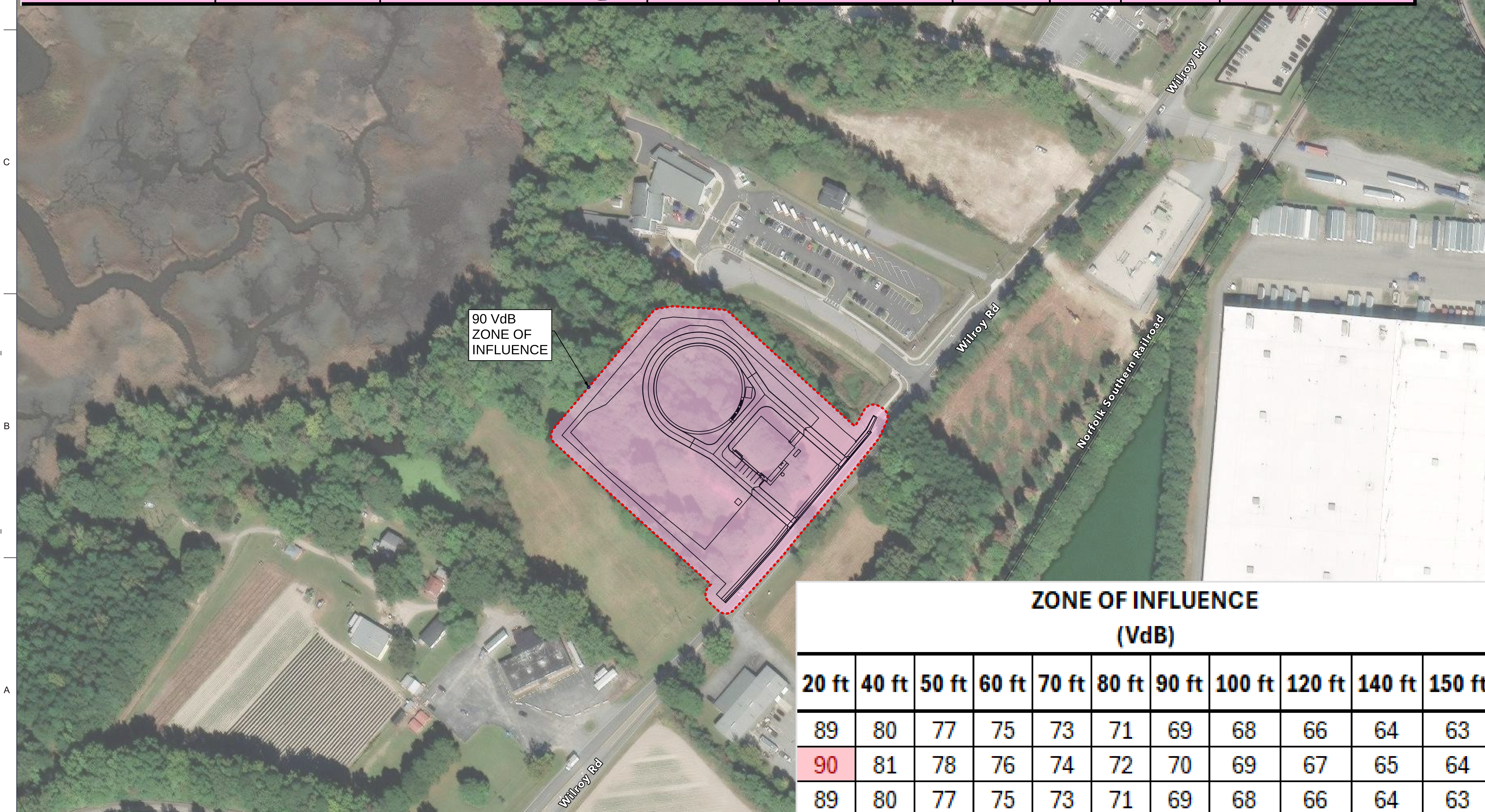
VIBRATION IMPACTS
PHASE 5
ROAD PARKING CONSTRUCTION

DRAWING NUMBER
5

SHEET NUMBER
5 OF 6

Path: C:\USERS\TCRAWFORD\ONEDRIVE - BROWN AND CALDWELL\DOCUMENTS_DATA\GIS\WILROY FILENAME: TITLEBLOCK4GIS_2013.DWG PLOT DATE: ---- CAD USER: TINA CRAWFORD

ACTIVITY	EQUIPMENT	ACTIVITY DESCRIPTION	PPV_{ref} (in/sec)	MINIMUM DISTANCE TO RECEPTOR (ft)	PPV_{equip} (in/sec)	L_{vref} (VdB)	$L_{v,distance}$ (VdB)	NOTES / ACTIONS
Restoration	Dump Truck	Installation of Materials	0.076	140	0.006	86	64	Barely perceptible
	Bulldozer	Final Site Grading	0.089	140	0.007	87	65	
	Grader	Final Site Grading	0.076	140	0.006	86	64	



ZONE OF INFLUENCE (VdB)

20 ft	40 ft	50 ft	60 ft	70 ft	80 ft	90 ft	100 ft	120 ft	140 ft	150 ft
89	80	77	75	73	71	69	68	66	64	63
90	81	78	76	74	72	70	69	67	65	64
89	80	77	75	73	71	69	68	66	64	63

Brown AND Caldwell

N

0 100 200
Feet

Sources:
Aerial photo, ESRI & Affiliates, 2023

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HRSD

1434 AIR RAIL AVENUE
VIRGINIA BEACH, VA 23455

HRSD Wilroy PRS and Tank

REVISIONS

REV	DATE	DESCRIPTION

LINE IS 2 INCHES AT FULL SIZE

DESIGNED: RYAN FOLEY
DRAWN: TINA CRAWFORD
CHECKED:
CHECKED:
APPROVED:

FILENAME
157943-Noise Analysis.aprx

BC PROJECT NUMBER
157943

CLIENT PROJECT NUMBER
NP014000

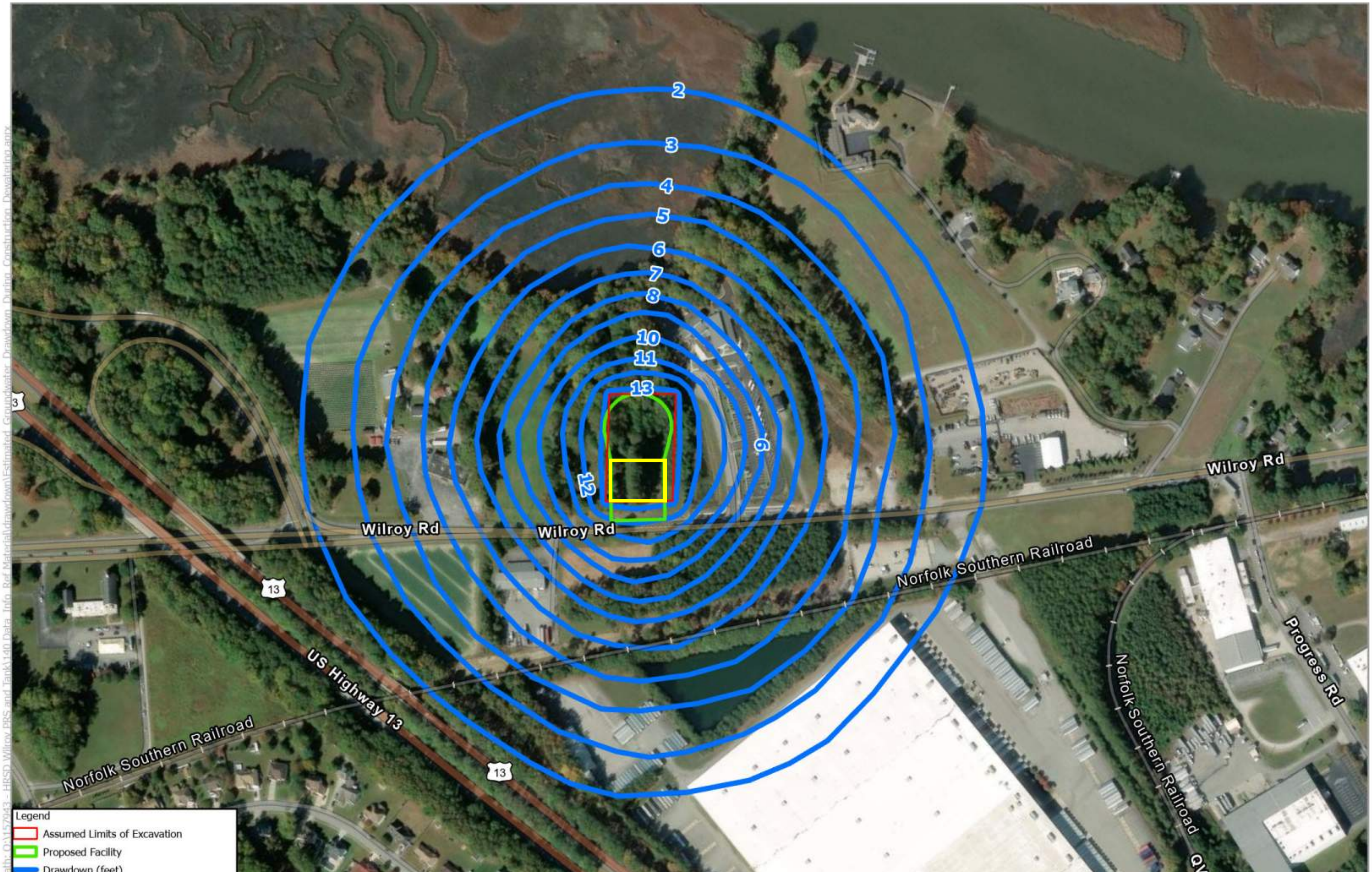
VIBRATION IMPACTS
PHASE 6
RESTORATION

DRAWING NUMBER
6

SHEET NUMBER
6 OF 6

Path: C:\USERS\TCRAWFORD\ONEDRIVE - BROWN AND CALDWELL\DOCUMENTS_DATA\GIS\WILROY FILENAME: TITLEBLOCK4GIS_2013.DWG PLOT DATE: ---- CAD USER: TINA CRAWFORD

Appendix C: Anticipated Dewatering Impacts



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Date: 8/7/2024 9:59 AM

***Contour labels are estimated drawdown (feet below static groundwater elevation)**

0 350 700 1,400 Feet

N

Estimated Groundwater Drawdown During Construction Dewatering

HRSD Wilroy Road Pressure Reducing Station and Offline Storage Facility

1941 - 1949 Wilroy Road

Suffolk, Virginia

1. PURPOSE

Excavation will be performed to construct the planned storage facility, shown on Figures 1 and 2 of the Geotechnical Engineering Report by Schnabel Engineering (Reference 1). The bottom of the excavation will be below the groundwater table; therefore, the excavation will be dewatered. The dewatering will result in a development of a groundwater table drawdown around the excavation. The purpose of this calculation is to estimate the magnitude of this drawdown. This is a preliminary estimate, as the excavation/dewatering methods and time frames are not yet specified, and hydraulic parameters of the water-bearing zone are not known. Conservative assumptions are used.

2. PROBLEM DESCRIPTION

The geologic strata at the site are described in Reference 1, section entitled Site Geology and Subsurface Conditions. The stratigraphy, from top to bottom, is:

- topsoil,
- fine-grained Tabb formation,
- coarse-grained Tabb formation,
- fine-grained Yorktown formation, and
- coarse-grained Yorktown formation.

The elevations of the top/bottom of the strata are obtained from the boring logs (Appendix A of Reference 1). The elevation of the groundwater table is specified in the section entitled Groundwater of Reference 1 as being 14 to 16 feet. The stratigraphy and the water table data are summarized on the cross section on page 6 of this calculation package.

The groundwater level that needs to be maintained in the excavation is specified in Reference 1, section entitled Excavation Dewatering to be 2 feet below the bottom of the excavation. The elevation of the excavation bottom, in turn, is defined based on the elevations of the planned construction, as indicated in Reference 1, section entitled Proposed Construction. The elevations of the bottom of the pump room and the elevation of the bearing perimeter tank foundation are both specified as 5 feet. Therefore, the groundwater level in the excavation will be maintained at the elevation of 5 feet minus 2 feet, that is 3 feet. This is also indicated on the cross section on page 5 of this package.

Based on the boring logs in Appendix A of Reference 1, a clay layer (fine-grained Yorktown formation) occurs at the depth of approximately 47 feet below ground surface. In Reference 1, section Excavation Dewatering, it is indicated that extending a sheet pile wall into this layer would hydraulically seal the excavation, in other words that the flow within the clay would be negligible. Based on that, the top of the Yorktown formation clay will be considered as the bottom of the water-bearing zone in which the excavation dewatering will occur. The water-bearing zone consists of the coarse-grained Tabb formation and coarse-grained Yorktown formation. See cross section on page 5 of this package.

Reference 1 does not contain information regarding the hydraulic properties of the water-bearing zone. However, boring logs in Appendix A of Reference 1 contain results of the grain size analyses. It is noted that samples collected from the water-bearing zone (coarse-grained Tabb and coarse-grained Yorktown) contain very high fraction of fines, between approximately 25 and 50 percent. Therefore, the hydraulic conductivity and specific yield are likely very low. Under these conditions, using individual wells to dewater an open excavation would not be practical, as the drawdown from a well in very low conductivity does not extent fat from the well. Instead, a row of well points would be used. For an excavation surrounded by a sheet pile wall, dewatering may be accomplished by extracting water from inside the enclosure, provided that boiling of the excavation bottom can be avoided.

As indicated previously, excavation could be performed either as an open or sheeted (Reference 1, section Excavation Dewatering). The former will generate a significantly greater drawdown around the excavation. Therefore, an open excavation is used in this estimate. The dewatering system is assumed to be a line of well points at some distance from the excavation perimeter, and is approximated as a trench.

3. METHOD

The static water table and the water table that will develop during dewatering are unconfined. The model used in this is calculation is that of a semi-infinite unconfined aquifer with the rapid hydraulic head change at the boundary (lowering of the water table during dewatering). The method is provided in Reference 2, as illustrated on Figure 2 of the reference (discharging aquifer). Time-dependent saturated thickness as a function of the distance from the boundary where the water table lowering occurs is calculated following the start of dewatering (eq. 33 of Reference 1).

$$h(x,t) = h_0 - (h_0 - h_1)\,[1 + C_2\, x / t^{1/2}]^{-1/\mu}$$

$$C_2 = (1/\lambda)\,(S / K)^{1/2}$$

$$\lambda^2 = [(1 - \mu)\,(1 - 2\mu) / (2\,\mu^2)]\,(h_0 + h_1)$$

$$\mu = 0.75(1 + n) - n / (2 - A) - [(2 - A)^2\,(1 + 2n) + n^2\,(2 + A)^2]^{1/2} / [4\,(2 - A)]$$

$$A = 4\,[h_0 + (1 + n)\,h_1] / [(1 + n)\,(2 + n)\,(h_0 + h_1)]$$

$$n = 1.251 + 0.099\,h_1 / (h_0 + 2\,h_1)$$

t – time elapsed from the hydraulic head change at the excavation, [d]
h_0 – initial hydraulic head at boundary of excavation, [ft]
h_1 – hydraulic head at excavation during dewatering, [ft]
S – specific yield of the water-bearing zone, [-]
K – hydraulic conductivity of the water-bearing zone, [ft/d]
x – distance from the boundary, [ft]

This is a transient model is for a water-bearing zone extending infinitely from the boundary. Under this condition, drawdown at a given point continuously increases with time. In reality, after certain time, steady state would be achieved, and the drawdown would stabilize. Therefore, this model is conservative in that it may overestimate the drawdown.

The change of hydraulic head at the boundary is determined by the level to which the excavation is dewatered.

3. PARAMETERS

Excavation

The excavation is assumed to be a rectangle encompassing the tank and the pump station, as sketched on Figure 2 of Reference 1. To account for the sloping/benching, the line of well points is assumed 50 feet away from the excavation perimeter.

Saturated thicknesses (Figures A and B)

See the cross section on page 5 of this package. Initial saturated thickness at the boundary is the difference between the static groundwater level and the top of the clay layer (bottom of the water-bearing zone):

h_0 = 16 ft – (-27) ft = 43 ft

Saturated thickness at boundary during dewatering for the open excavation case is assumed as the difference between the water level maintained in the excavation (elevation 3 feet) and the top of the clay layer:

h_1 = 3 ft – (-27) ft = 30 ft

Properties of the water-bearing zone

Hydraulic conductivity and storage coefficient of the water-bearing zone (coarse-grained Tabb and Yorktown) are not known. Values are assumed to result in the conservatively high drawdown. This corresponds to the high conductivity and low specific yield. The materials are described clayey to silty sand, with a significant fraction of fines (Reference 2, borings in Appendix A, ~ 25-50%). Based on engineering judgement, reasonable conservative estimates (high K, low S_y) of these parameters are:

$K = 1*10^{-4}$ cm/sec

$S_y = 0.01$

Dewatering period

Based on the engineering judgement, the time period requiring dewatering would be approximately 6 months. Time-frame of 200 days is assumed.

t = 200 d

4. ESTIMATE OF DRAWDOWN

The calculation is performed in a spread sheet, see page 6 of this package.

Distance from well points [ft]	Drawdown [ft]
10	12.8
25	12.5
50	12.0
100	11.1
200	9.3
300	7.8
400	6.5
500	5.4
750	3.1
1,000	1.7

Note that typical groundwater level fluctuations are in the range of 5 feet. Therefore, the drawdown would become indistinguishable from the water table fluctuations at approximately 500-750 feet from the well points.

5. SUMMARY

This calculation presents a preliminary estimate of the drawdown in the vicinity of the excavation that will be performed to construct the tank and the pump station at the HRSD Wilroy facility. The dewatering method and the parameters of the water-bearing zone are not known at this time; therefore, the estimate is preliminary and based on several assumptions. The assumptions were made to result in a conservative (high) estimate of the drawdown. The conservative assumptions are transient model, open excavation, thick saturated zone, high hydraulic conductivity and low specific yield.

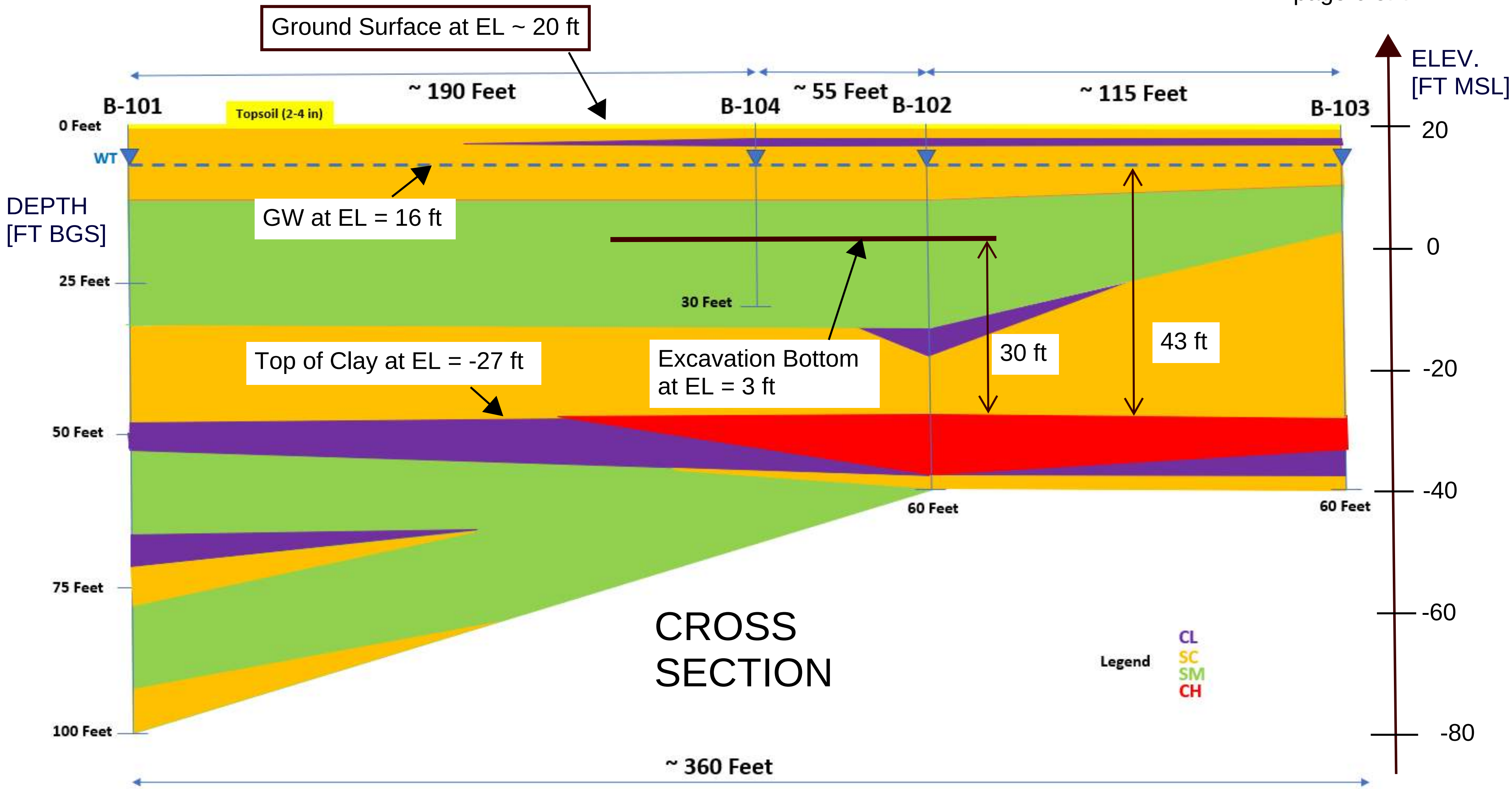
In addition, as shown on Fig 1 of Reference 1, there is a wetland/creek system adjacent to the site. This system would likely act as a source of water during dewatering and would diminish the drawdown. Employing an excavation support system, such as a sheet pile wall, would limit the flow into the excavation and also diminish the drawdown.

The results indicate that the drawdown would be approximately 13 feet at the line of dewatering well points, and would become indistinguishable from the natural groundwater level fluctuations at the distance of approximately 500-750 feet from the well points.

6. REFERENCES

1. Geotechnical Engineering Report Revision No. 1
HRSD Wilroy Road Pressure Reducing Station and Offline Storage Facility
Schnabel Engineering, March 21, 2024

2. Response of Unconfined Aquifer to Sudden Change in Boundary Head
D.A. Lockington,
Journal of Irrigation and Drainage Engineering, Vol.123, No:1, 24-27, Jan/Feb1997.



Ground Surface at EL ~ 20 ft
~ 190 Feet
~ 55 Feet
~ 115 Feet
B-101
Topsoil (2-4 in)
B-104
B-102
B-103
ELEV.
[FT MSL]
0 Feet
WT
DEPTH
[FT BGS]
GW at EL = 16 ft
25 Feet
30 Feet
30 ft
43 ft
Top of Clay at EL = -27 ft
Excavation Bottom
at EL = 3 ft
50 Feet
60 Feet
60 Feet
75 Feet
100 Feet
CROSS
SECTION
Legend
CL
SC
SM
CH
20
0
-20
-40
-60
-80
~ 360 Feet

Estimate hydraulic head at a given point, and flow rate into the constant head boundary, both as a function of time, for the case of a semi-infinite unconfined aquifer with a rapid change at the boundary.
From D.A. Lockington, Journal of Irrigation and Drainage Engineering, Jan/Feb1997.

Flow: Two dimensional flow (eq 36) $q = [C_d (h_1 - h_0) (KS)^{1/2}] / (2 t^{1/2})$

Three dimensional flow $Q = L q$

Hydraulic Head: (eq 33) $h(x,t) = h_0 - (h_0 - h_1) \{1 + (x/\lambda) [S/(Kt)]^{1/2}\}^{-1/\mu}$

Where: (eq 30) $n = 1.251 + 0.099 h_1 / (h_0 + 2h_1)$

(eq 29) $C_d^2 = (1 - 2\mu) (h_0 + h_1) / [2 (1 - \mu)]$

(eq 25) $\lambda^2 = [(1 - \mu) (1 - 2\mu) / (2 \mu^2)] (h_0 + h_1)$

(eq 27) $\mu = 0.75(1 + n) - n / (2 - A) - [(2 - A)^2 (1 + 2n) + n^2 (2 + A)^2]^{1/2} / [4 (2 - A)]$

(eq 28) $A = 4 [h_0 + (1 + n) h_1] / [(1 + n) (2 + n) (h_0 + h_1)]$

Data:

Initial head	h_0 =	43	ft	
Head at boundary after change	h_1 =	30	ft	
Specific yield	S =	0.01	-	
Hydraulic conductivity	K =	1E-04	cm/s =	0.2833 ft/d
Length	W	800	ft	

Calculate coefficients:

$n = 1.251 + 0.099 h_1 / (h_0 + 2h_1)$ = 1.27983 -

$A = 4 [h_0 + (1 + n) h_1] / [(1 + n) (2 + n) (h_0 + h_1)]$ = 0.8163 -

$\mu = 0.75(1 + n) - n / (2 - A) - [(2 - A)^2 (1 + 2n) + n^2 (2 + A)^2]^{1/2} / [4 (2 - A)]$ = -0.267 -

$\lambda^2 = [(1 - \mu) (1 - 2\mu) / (2 \mu^2)] (h_0 + h_1)$ = 995.799 ft

λ = -31.5563 $ft^{1/2}$ **<u>λ must have the same sign as μ</u>**

$C_d^2 = (1 - 2\mu) (h_0 + h_1) / [2 (1 - \mu)]$ = 44.1889 ft

C_d = 6.6475 $ft^{1/2}$

$C_1 = 0.5 C_d (h_1 - h_0) (KS)^{1/2}$ = -2.30 $ft^2/d^{1/2}$

$C_2 = (1/\lambda) (S / K)^{1/2}$ = -0.00595 $d^{1/2}$ / ft

Calculate flow rate as function of time:

time	$q = C_1/t^{1/2}$	Q = q W	
[d]	[ft^2/d]	[ft^3/d]	[gpm]
0.1	-7.273	-5818	-30.2
1.5	-1.878	-1502	-7.8
2.5	-1.455	-1164	-6.0
3	-1.328	-1062	-5.5

Calculate hydraulic heads and drawdown as functions of distance at given time:

$h(x,t) = h_0 - (h_0 - h_1) [1 + C_2 x / t^{1/2}]^{-1/\mu}$ [ft]

time "t"	distance from boundary "x" [ft]									
[d]	10	25	50	100	200	300	400	500	750	1000
200	30.2	30.5	31.0	31.9	33.7	35.2	36.5	37.6	39.9	41.3

drawdown = $h - h_0$ [ft]

time "t"	distance from boundary "x" [ft]									
[d]	10	25	50	100	200	300	400	500	750	1000
200	12.8	12.5	12.0	11.1	9.3	7.8	6.5	5.4	3.1	1.7

GEOTECHNICAL ENGINEERING REPORT
REVISION NO. 1

HRSD Wilroy Road Pressure Reducing Station and Offline Storage Facility
1941-1949 Wilroy Road
Suffolk, Virginia

Schnabel Reference 22330066.020
March 21, 2024

Schnabel ENGINEERING
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11836 Canon Boulevard, Suite 400 / Newport News, VA 23606
schnabel-eng.com

March 21, 2024

Mr. Chris Wilson, PE
Brown and Caldwell
301 Bendix Road
Virginia Beach, Virginia

Subject: Geotechnical Engineering Report, Revision No. 1, HRSD Wilroy Pressure Reducing Station and Offline Storage Facility, 1941-1949 Wilroy Road, Suffolk, Virginia, Schnabel Project 22330066.020

Dear Mr. Wilson,

SCHNABEL ENGINEERING, LLC (Schnabel) is pleased to submit our revised geotechnical engineering report for this project. This study was performed in accordance with our proposal dated December 2, 2022, as authorized by your PO # 38360 dated February 27, 2023. This revision includes recommendations for deep foundation alternatives for support the proposed storage/equalization tank.

PROJECT DESCRIPTION

Site Description

The project site is located at 1941-1949 Wilroy Road in Suffolk, Virginia. A Site Vicinity Map is included as Figure 1 at the end of this report. This site consists of two adjacent parcels, totaling approximately 5.4 acres. The site has several small structures but generally consists of woods and some grassy areas. The site is bound by Executive Court to the northeast, Wilroy Road to the southeast, 1925 Wilroy Road to the south, and wetlands to the northwest. Site slopes gently downward from about EL 22 at Wilroy Road to about EL 18 at the western end of the site.

We obtained the site information from the topographic site plan dated May 3, 2023, prepared by your office, and through our site visits.

Proposed Construction

The project will include construction of a pressure reducing station (PRS), a 3 MG offline storage/equalization tank, an odor control system, an underground fuel storage tank, and access roads. The arrangement of the site is shown on Figure 2. We understand up to about 1 ft of new fill will be placed to raise the grade around the storage tank to about EL 22.

The PRS will be about 82 by 90 ft and include four main pumps, a bypass check valve station, a generator, electrical rooms, a bathroom, and a column supported bridge crane. The floor with the generator, electrical rooms, and bathroom will be at about the existing grade, EL 21, and the pump room will be about 16 ft below the ground surface at about EL 5. The building walls will be constructed using concrete masonry units (CMU). A 1.5 MW backup generator will be located within the PRS building. There will be a 3-ton bridge crane spanning the below-grade pump room. The crane rails will be supported on independent foundations. We understand the PRS mat foundation bearing pressure will be 1,000 psf.

The storage/EQ tank will be a 160 ft diameter, concrete tank with 20 ft high walls and a concrete dome roof. The tank bottom will be about 2.8 ft thick. The tank bottom will be at about EL 16, and the bearing grade of the tank mat foundation will be at about EL 13.2, about 7 to 8 ft below the surrounding ground surface. The tank will have a vacuum flushing system with a perimeter collection trough. The perimeter foundation bearing grade below the trough will be at about EL 5. We understand the bearing pressure of the interior mat foundation and the perimeter footing will be 2,200 psf. The interior mat and the perimeter footing will be supported on deep foundations.

An activated carbon odor control system will be constructed on the south side of the storage/EQ tank. The odor control building floor slab will include a turn-down perimeter footing. We understand that the bearing pressure of the odor control building mat will be 650 psf.

We obtained the site information from the project Request for Proposal (RFP) dated August 29, 2021, Addendum No.1 dated September 21, 2021, the draft preliminary engineering report dated September 2022, the Wilroy PRS Interior Sketches and the Tank Options sketch sent to our office on May 9, 2023, and conversations with your office.

SUBSURFACE EXPLORATION AND TESTING PROGRAM

We performed a subsurface exploration and field-testing program to identify the subsurface stratigraphy underlying the site and to evaluate the geotechnical properties of the materials encountered. This program included test borings and dilatometer (DMT) soundings. Exploration methods used are discussed below. The appendices contain the results of our exploration.

Subsurface Exploration Methods

Test Borings

Our subcontractor, Fishburne Drilling, Inc., drilled four test borings under our observation between March 20 and 21, 2023. The Standard Penetration Test (SPT) was performed at selected depths in the borings. Appendix A includes specific observations, remarks, and logs for the borings; classification criteria; drilling methods; and sampling protocols. Figure 2, included at the end of this report, indicates the approximate test boring locations. We will retain soil samples up to 45 days beyond the issuance of this report, unless you request other disposition.

In addition to the borings drilled in March, we drilled several borings on this site and on the site to the south during the preliminary phases of the project. One of the preliminary borings, Boring B-01 (North), was drilled in the area of the proposed tank. The log for this boring is also included in Appendix A.

The SPT samples were obtained using a hydraulically driven automatic trip hammer (ATH). Most correlations with SPT data are based on N-values collected with a safety hammer. The energy applied to the split-spoon sampler using the ATH is about 33 percent greater than that applied using the safety hammer, resulting in lower N-values. The hammer blows shown on the boring logs are uncorrected for the higher energy. However, we correct SPT N values for the higher energy when using N values in our analyses.

Dilatometer Testing (DMT)

Our subcontractor, In-Situ Soil Testing, LC, performed five dilatometer tests (DMTs) on March 22, 2023, to depths of 20 to 100 feet below the ground surface. Details of the DMTs and test results are included in Appendix B. Figure 2 indicates the approximate DMT locations.

Soil Laboratory Testing

Our laboratory performed tests on selected samples collected during our recent and previous subsurface exploration. Our subconsultant testing lab, Enthalpy Analytical, performed ion chromatography analyses for chloride and sulfate content on selected samples.

The testing aided in the classification of materials encountered in the subsurface exploration and provided data for use in the development of our recommendations. The results of the laboratory tests are included in Appendix B and are summarized for each stratum in the Site Geology and Subsurface Conditions section of this report. Selected test results are also shown on the boring logs in Appendix A.

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Site Geology

We reviewed existing geologic data and information in our files. Based on this review, the site is generally underlain by the Sedgefield member of the Tabb Formation, and the Miocene Age soils of the Yorktown Formation. The Tabb Formation is composed of fluvial and estuarine sand, clay, organic soil, and peat. The boring logs do not indicate the presence of any peat or organic soils. The Yorktown Formation typically consists of silty sands, clayey sands and sandy clays and silts, all containing shell fragments. These soils are preconsolidated and exhibit relatively high strength. Soils of this formation are known to be sensitive to disturbance.

The above stratigraphy is typical in the area. However, in the immediate vicinity of the project site, some of the above strata have been eroded or excavated, and commonly have been replaced with recent alluvial deposits or fill.

Generalized Subsurface Stratigraphy

We characterized the following generalized subsurface stratigraphy based on the exploration and laboratory test data included in the appendices.

Ground Cover: Topsoil

The borings encountered about 2 to 4 inches of topsoil at the ground surface.

Stratum A1: Fine Grained Tabb Formation

Below the ground cover and interbedded with the coarse-grained soils of Stratum A2, four of the borings encountered a fine-grained Tabb Formation deposit consisting of sandy lean clay (CL) to a depth of 4 ft. These soils are generally moderately plastic, having liquid limits of 26 to 36 and plasticity indices of 11 to 20, indicating that these soils exhibit a moderate potential for moisture-related volume change (shrink/swell behavior). The natural moisture contents measured were about 17 to 20 percent. Based on the Standard Penetration Tests Performed, the soils of this stratum are generally medium stiff to stiff: N = 4 to 12.

Stratum A2: Coarse Grained Tabb Formation

Below the ground cover and interbedded with the fine-grained soils of Stratum A1, the borings encountered a coarse-grained Tabb Formation deposit consisting of clayey sand (SC) and silty sand (SM) to depths of 9 to 12 ft. These soils are generally moderately plastic, having liquid limits of 35 to 39 and plasticity indices of 15 to 43, indicating that these soils exhibit a low to moderate potential for moisture-related volume change (shrink/swell behavior). The natural moisture contents measured were about 17 to 20 percent. Based on the Standard Penetration Tests Performed, the soils of this stratum are generally very loose to medium dense: N = WOH to 17.

Five dilatometer tests were performed within Stratum A2. The tests resulted in dilatometer moduli of 7 to 1,119 tsf. Correlations with the dilatometer data indicate angles of internal friction of 19 to 49 degrees for this stratum.

Stratum B1: Fine Grained Yorktown Formation

Below the Tabb Formation soils of Strata A1 and A2, and interbedded with the coarse-grained soils of Stratum B2, four of the borings encountered a fine-grained Yorktown Formation deposit consisting of fat clay (CH) and lean clay (CL) with varying amounts of sand, shell fragments, and mica to depths of 37 to 72 ft. These soils are generally moderately to highly plastic, with one sample having a liquid limit of 71 and a plasticity index of 49. The natural moisture contents measured were about 34 to 48 percent. Based on the Standard Penetration Tests performed, the soils of this stratum are generally medium stiff: N = 4 to 5.

Five dilatometer tests were performed within Stratum B1. The tests resulted in dilatometer moduli of 85 to 350 tsf. Correlations also indicate undrained shear strengths of 1,440 to 4,740 tsf for the soils of this stratum.

Stratum B2: Coarse Grained Yorktown Formation

Below the Tabb Formation soils of Strata A1 and A2, and interbedded with the fine-grained soils of Stratum B1, four of the borings encountered a coarse-grained Yorktown Formation deposit consisting of clayey sand (SC) and silty sand (SM) with varying amounts of shell fragments and mica to the maximum depth of exploration, 100 ft. These soils are generally non-plastic to moderately plastic, having liquid limits up to 45 and plasticity indices up to 24. The natural moisture contents measured were about 22 to 38 percent. Based on the Standard Penetration Tests performed, the soils of this stratum are generally very loose to medium dense: N = 2 to 9.

Five dilatometer tests were performed within Stratum B2. The tests resulted in dilatometer moduli of 120 to 2,700 tsf. Correlations with the dilatometer data indicate angles of internal friction of 39 to 45 degrees for the soils of this stratum.

Groundwater

We observed groundwater in all of the borings at a depth of about 6 ft below the ground surface, about EL 14 to 16. The test boring logs in Appendix A include groundwater observations obtained during our subsurface exploration. These data include depths to groundwater encountered during drilling. Because the borings were advanced using mud rotary drilling techniques, bentonite drilling fluid was continually pumped through the drill rods to flush cuttings to the surface and maintain the sides of the borehole. Because of the presence of this drilling fluid, depths to groundwater upon drilling completion and following completion of the borings where mud rotary techniques are used are generally unreliable and are not included on the boring logs.

Our drilling subcontractor installed water observation wells adjacent to DMT sounding locations DMT-01 and DMT-03. We observed groundwater in these wells at depths of about 6.5 and 7.5 ft below the ground surface (about EL 11.5 to 15) 27 days after completion of the borings. We did not obtain long-term water level readings in the other borings since we backfilled them upon completion for safety.

The groundwater levels on the logs indicate our estimate of the hydrostatic water table at the time of our subsurface exploration. The final design should anticipate fluctuation of the hydrostatic water table depending on variations in precipitation, surface runoff, pumping, river levels, evaporation, leaking utilities, and similar factors.

Based on our groundwater observations, we expect the groundwater level on the site will be higher than the bottom of the proposed PRS and tank foundation excavations. Recommendations to address the impact of groundwater are discussed in subsequent sections.

Pipe Corrosion Potential

Laboratory analyses including pH, resistivity, oxidation reduction potential, sulfides, sulfates, and chlorides were performed to evaluate the corrosion potential of soils expected at the proposed pipe invert elevations. The results of the soil laboratory testing are summarized in Table 1 below and are presented in detail in Appendix B.

Table 1: Corrosion Potential Results

Boring	Depth (ft)	Stratum/ USCS Classification	pH	Redox Potential (mv)	Resistivity (Ohms-cm)	Sulfides	Sulfate Content (mg/kg)	Chloride Content (mg/kg)
B-102	13-20	B / SM	7.7	227	820	0	353	10.7
B-104	6-10	A / SC	6.8	337	3,500	0	22.4	16.3

We used the results of the laboratory testing, chemical analysis, groundwater levels, and the American Water Works Association (AWWA) Standard C105 method to evaluate the potential for corrosion. The AWWA method uses a point system to indicate the potential for corrosion of buried metallic structures. A score of 10 or more points indicates potentially corrosive materials. Based on these test results, the soils materials should be anticipated to achieve the recommended compaction. Moisture conditioning will be easier during the warmer, drier times of the year. Soils from below the groundwater level are expected to be saturated and unsuitable for reuse as backfill without significant drying.

Excavation Slopes

Cut slopes for excavations on the site may be constructed in accordance with OSHA regulations considering an OSHA Soil Type C. If sheeting and shoring are used to construct the tank and PRS, the earth pressure recommendations provided in the below grade walls section of our report should be used in the design of the excavation support.

Excavation Dewatering

Groundwater was recorded at depths about 2 to 10 ft higher than the bottoms of the tank and pump station excavations, so we expect the excavations will need to be dewatered to facilitate construction of these structures. The groundwater levels on the logs indicate our estimate of the hydrostatic water table at the time of our subsurface exploration. The dewatering design should anticipate fluctuation of the groundwater level, including higher than the levels recorded in the borings and wells. The proximity of the Nansemond river and surrounding swamp may also affect the dewatering design.

When sands are dewatered, the groundwater level can be lowered a significant distance away from the excavation. Dewatering can cause settlement of the surrounding soils as the dewatered materials transition from a buoyant unit weight to a total unit weight. The dewatering design should consider the lateral extent of the dewatering and the proximity of nearby structures and roadways when designing the dewatering system. Groundwater monitoring wells can be installed, and the existing structures can be monitored for settlement during dewatering operations, if needed.

The water level must be lowered enough to stabilize the bottom and sides of the excavation enough to place the bedding material, construct the tank and pump station, and backfill the excavation. For planning purposes, this level can be considered to be about 2 ft below the bottom of the excavation.

Laboratory testing performed on the sands of the Yorktown Formation indicate these soils are relatively fine sands and include about 25 to 50 percent fines. If excavation sheeting is installed (discussed below), we expect installing the sheeting into the Yorktown Formation can hydraulically seal the excavation. The depth the sheeting will need to extend into the Yorktown Formation should be evaluated by the shoring designer and the dewatering designer.

Pumped groundwater should be evaluated and disposed of according to *HRSD Design and Construction Standard, Section 35 - Standards for Capital Improvement Projects that Involve Construction Dewatering Activities*.

Temporary Excavation Support

Installing excavation sheeting can provide several benefits, including:

- Significantly reduce the volume of water to be pumped for the duration of the dewatering operation. Reducing the volume of water to be pumped will also reduce the amount of groundwater that will need to be treated and released.

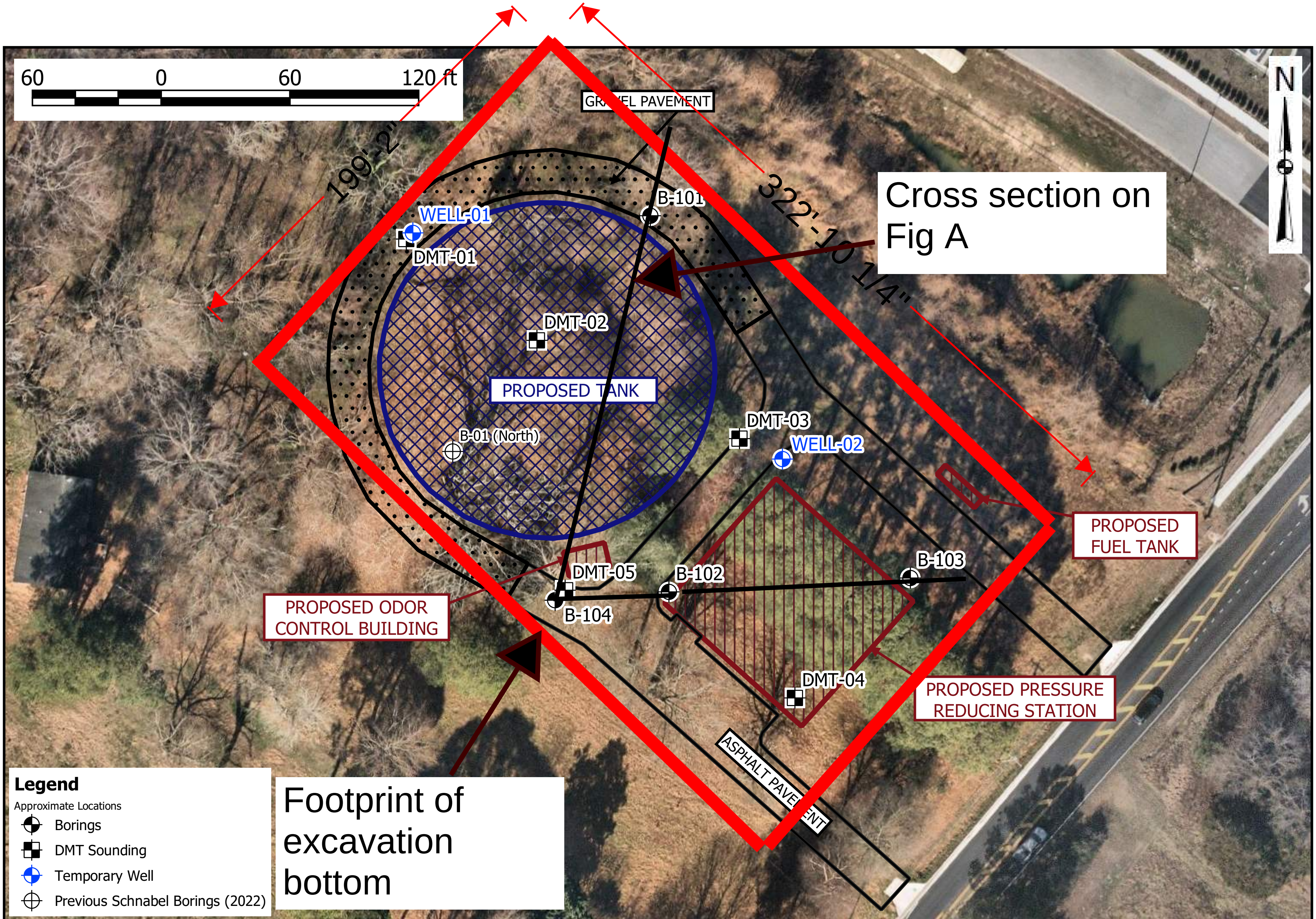
arcgis

Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community
Esri, HERE, Garmin, (c) OpenStreetMap contributors
Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community
Projection: WGS 1984 Web Mercator Auxiliary Sphere

NOT TO SCALE

Schnabel ENGINEERING	HRSD WILROY PRESSURE REDUCING STATION AND OFFLINE STORAGE FACILITY 1941-1949 WILROY ROAD SUFFOLK, VIRGINIA PROJECT NO. 22330066	SITE VICINITY MAP FIGURE 1

3/15/2023 This Map was Created In Schnabel Engineerings Site Vicinity Map Application



Schnabel ENGINEERING	HRSD WILROY ROAD PRESSURE REDUCING STATION AND OFFLINE STORAGE FACILITY 1941-1949 WILROY ROAD SUFFOLK, VIRGINIA	Figure Name: BORING LOCATION PLAN	Done: E. WALSH	Figure Number: 2
		Project Number: 22330066	Reviewed: E. MORRIS	Date: MAY 2023

APPENDIX A

SUBSURFACE EXPLORATION DATA

IDENTIFICATION OF SOIL

I. DEFINITION OF SOIL GROUP NAMES (ASTM D2487)

			SYMBOL	GROUP NAME
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels – More than 50% of coarse fraction retained on No. 4 sieve Coarse, ¾" to 3" Fine, No. 4 to ¾"	Clean Gravels Less than 5% fines	GW	WELL GRADED GRAVEL
			GP	POORLY GRADED GRAVEL
		Gravels with fines More than 12% fines	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	Sands – 50% or more of coarse Fraction passes No. 4 sieve Coarse, No. 10 to No. 4 Medium, No. 40 to No. 10 Fine, No. 200 to No. 40	Clean Sands Less than 5% fines	SW	WELL GRADED SAND
			SP	POORLY GRADED SAND
		Sands with fines More than 12% fines	SM	SILTY SAND
			SC	CLAYEY SAND
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays – Liquid Limit less than 50 Low to medium plasticity	Inorganic	CL	LEAN CLAY
			ML	SILT
		Organic	OL	ORGANIC CLAY
				ORGANIC SILT
	Silts and Clays – Liquid Limit 50 or more Medium to high plasticity	Inorganic	CH	FAT CLAY
			MH	ELASTIC SILT
		Organic	OH	ORGANIC CLAY
				ORGANIC SILT
Highly Organic Soils	Primarily organic matter, dark in color and organic odor		PT	PEAT

II. DEFINITION OF SOIL COMPONENT PROPORTIONS (ASTM D2487)

			Examples
Adjective Form	GRAVELLY SANDY	>30% to <50% coarse grained component in a fine-grained soil	GRAVELLY LEAN CLAY
	CLAYEY SILTY	>12% to <50% fine grained component in a coarse-grained soil	SILTY SAND
"With"	WITH GRAVEL WITH SAND	>15% to <30% coarse grained component in a fine-grained soil	FAT CLAY WITH GRAVEL
	WITH GRAVEL WITH SAND	>15% to <50% coarse grained component in a coarse-grained soil	POORLY GRADED GRAVEL WITH SAND
	WITH SILT WITH CLAY	>5% to <12% fine grained component in a coarse-grained soil	POORLY GRADED SAND WITH SILT

III. GLOSSARY OF MISCELLANEOUS TERMS

SYMBOLS .... Unified Soil Classification Symbols are shown above as group symbols. A dual symbol "-" indicates the soil belongs to two groups. A borderline symbol "/" indicates the soil belongs to two possible groups.

FILL.. Man-made deposit containing soil, rock and often foreign matter.

PROBABLE FILL................... Soils which contain no visually detected foreign matter but which are suspect with regard to origin.

DISINTEGRATED ROCK (DR).. Residual materials with a standard penetration resistance (SPT) between 60 blows per foot and refusal. Refusal is defined as a SPT of 100 blows for 2" or less penetration.

PARTIALLY WEATHERED ROCK (PWR)......................... Residual materials with a standard penetration resistance (SPT) between 100 blows per foot and refusal. Refusal is defined as a SPT of 100 blows for 2" or less penetration.

BOULDERS & COBBLES..... Boulders are considered rounded pieces of rock larger than 12 inches, while cobbles range from 3 to 12 inch size.

LENSES................................. 0 to ½ inch seam within a material in a test pit.

LAYERS................................. ½ to 12 inch seam within a material in a test pit.

POCKET .... Discontinuous body within a material in a test pit.

MOISTURE CONDITIONS..... Wet, moist or dry to indicate visual appearance of specimen.

COLOR .... Overall color, with modifiers such as light to dark or variation in coloration.

Schnabel Engineering	TEST BORING LOG	**Project:** Pressure Reducing & Offline Storage Facility 1941-1949 Wilroy Road Suffolk, Virginia	**Boring Number:** B-101 **Contract Number:** 22330066.020 **Sheet:** 1 of 3

Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Contractor Foreman: J. Rassio

Schnabel Representative: E. Walsh

Equipment: CME-55 (Track)

Method: 2-15/16" O.D. Tri-cone Roller Bit

Hammer Type: Auto Hammer (140 lb)

Dates Started: 3/20/23 **Finished:** 3/20/23

Location: See Location Plan

Ground Surface Elevation: 20± (ft) **Total Depth:** 100.0 ft

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered ▽	3/20	10:14 AM	6.0'	---	---

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
0.3	Topsoil; 3 inches		19.8			S-01, SPT 4+3+2+2 REC=10", 42%		TABB FORMATION
	CLAYEY SAND, fine to medium grained sand; moist, grayish brown					S-02, SPT 2+3+4+4 REC=18", 75%	MC = 21.1%	
	Change: brown and orangish brown				5	S-03, SPT 4+4+4+3 REC=14", 58%		
	Change: wet ▽	SC		A2		S-04, SPT 2+2+2+2 REC=20", 83%		
	Change: fine grained sand				10	S-05, SPT WOH/24" REC=24", 100%	LL = 36 PL = 23 MC = 41.6% % Passing #200 = 32.3	
12.0	SILTY SAND, fine to coarse grained sand; wet, greenish gray, contains shell fragments		8.0		15	S-06, SPT 2+2+2+2 REC=18", 75%	LL = 30 PL = 27 MC = 34.6% % Passing #200 = 24.9	YORKTOWN FORMATION
		SM		B2	20	S-07, SPT 2+2+3+3 REC=24", 100%		
	Change: fine grained sand				25	S-08, SPT 2+2+3+3 REC=24", 100%	MC = 31.3%	

(continued)

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
	SILTY SAND, fine to coarse grained sand; wet, greenish gray, contains shell fragments *(continued)*	SM			30	S-09, SPT 2+2+3+2 REC=24", 100%	LL = 42 PL = 26 MC = 37.3% % Passing #200 = 42.8	YORKTOWN FORMATION
32.0	CLAYEY SAND, fine to coarse grained sand; wet, greenish gray, contains shell fragments	SC	-12.0	B2	35	S-10, SPT 2+2+2+3 REC=24", 100%	LL = 45 PL = 21 MC = 34.9% % Passing #200 = 49.9 PP = 1.00 tsf	
					40	S-11, SPT 2+2+3+4 REC=20", 83%	MC = 29.2% % Passing #200 = 26.1	
	Change: fine to medium grained sand				45	S-12, SPT 1+1+2+3 REC=18", 75%	LL = 41 PL = 18 MC = 31.2% % Passing #200 = 45.3 PP = 1.75 tsf	
47.0	SANDY LEAN CLAY; wet, greenish gray, contains shell fragments	CL	-27.0	B1	50	S-13, SPT WOH+1+3+3 REC=24", 100%	PP = 1.25 tsf	
52.0	SILTY SAND, fine grained sand; wet, greenish gray, contains shell fragments, and mica	SM	-32.0	B2	55	S-14, SPT 2+1+3+3 REC=24", 100%	MC = 34.7% PP = 1.75 tsf	
					60	S-15, SPT 2+3+3+3 REC=24", 100%	MC = 34.2% % Passing #200 = 46.1	
					65	S-16, SPT 2+2+2+3 REC=24", 100%		

(continued)

Schnabel ENGINEERING **TEST BORING LOG**

Project: Pressure Reducing & Offline Storage Facility
1941-1949 Wilroy Road
Suffolk, Virginia

Boring Number: **B-101**
Contract Number: 22330066.020
Sheet: 3 of 3

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
		SM		B2				YORKTOWN FORMATION
67.0	SANDY LEAN CLAY; wet, greenish gray, contains shell fragments, and mica	CL	-47.0	B1	70	S-17, SPT 2+2+3+2 REC=24", 100%	MC = 35.3% % Passing #200 = 67.6	
72.0	CLAYEY SAND, fine to medium grained sand; wet, greenish gray, contains shell fragments	SC	-52.0		75	S-18, SPT 1+2+3+5 REC=24", 100%		
77.0	SILTY SAND, fine grained sand; wet, greenish gray, contains shell fragments, and mica	SM	-57.0	B2	80	S-19, SPT 3+2+3+3 REC=24", 100%	MC = 30.0% % Passing #200 = 42.1	
					85	S-20, SPT 3+3+3+3 REC=24", 100%		
					90	S-21, SPT 2+3+3+3 REC=24", 100%	MC = 28.9%	
92.0	CLAYEY SAND, fine to coarse grained sand; wet, greenish gray, contains shell fragments	SC	-72.0		95	S-22, SPT WOH+2+7+6 REC=24", 100%	MC = 22.4% % Passing #200 = 30.5	
						S-23, SPT 1+2+3+4 REC=24", 100%	MC = 28.9% % Passing #200 = 44.3 PP = 1.00 tsf	
100.0			-80.0		100			

Bottom of Boring at 100.0 ft.
Boring terminated at selected depth.
Boring backfilled with cuttings upon completion.

Schnabel Engineering TEST BORING LOG	Project: Pressure Reducing & Offline Storage Facility 1941-1949 Wilroy Road Suffolk, Virginia	Boring Number: B-102 Contract Number: 22330066.020 Sheet: 1 of 2

Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Contractor Foreman: J. Rassio

Schnabel Representative: E. Walsh

Equipment: CME-55 (Track)

Method: 2-15/16" O.D. Tri-cone Roller Bit

Hammer Type: Auto Hammer (140 lb)

Dates Started: 3/21/23 **Finished:** 3/21/23

Location: See Location Plan

Ground Surface Elevation: 22± (ft) **Total Depth:** 60.0 ft

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered ▽	3/21	8:55 AM	6.0'	---	---

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
0.3	Topsoil; 4 inches		21.2			S-01, SPT 3+2+2+3 REC=18", 75%	MC = 14.9%	TABB FORMATION
2.0	CLAYEY SAND, fine to medium grained sand; moist, grayish brown	SC	19.5	A2				
4.0	SANDY LEAN CLAY; moist, grayish brown	CL	17.5	A1		S-02, SPT 3+3+5+5 REC=20", 83%	LL = 36 PL = 16 MC = 19.6% % Passing #200 = 59.2 PP = 2.00 tsf	
	CLAYEY SAND, fine to coarse grained sand; moist, orangish brown and gray				5	S-03, SPT 4+6+5+2 REC=18", 75%	LL = 39 PL = 22 MC = 31.3% % Passing #200 = 41.6	
	▽ Change: wet					S-04, SPT 2+1+1+1 REC=14", 58%	MC = 42.6%	
12.0		SC	9.5	A2	10	S-05, SPT WOH/24" REC=24", 100%		
	SILTY SAND, fine to coarse grained sand; wet, greenish gray, contains shell fragments				15	S-06, SPT 2+2+2+2 REC=24", 100%	LL = 34 PL = 29 MC = 37.3% % Passing #200 = 27.6 Resistivity = 820 Ohms-cm Redox = 227 mv Sulfides = 0 pH = 7.7	YORKTOWN FORMATION
					20	S-07, SPT 2+2+2+2 REC=24", 100%		
		SM		B2	25	S-08, SPT 2+2+2+3 REC=24", 100%		

(continued)

Schnabel ENGINEERING TEST BORING LOG	Project: Pressure Reducing & Offline Storage Facility 1941-1949 Wilroy Road Suffolk, Virginia	Boring Number: B-102 Contract Number: 22330066.020 Sheet: 2 of 2

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
	SILTY SAND, fine to coarse grained sand; wet, greenish gray, contains shell fragments *(continued)*	SM		B2	30	S-09, SPT 2+3+2+3 REC=24", 100%	MC = 38.1%	YORKTOWN FORMATION
32.0	SANDY LEAN CLAY; wet, greenish gray, contains shell fragments	CL	-10.5	B1	35	S-10, SPT 1+2+3+2 REC=24", 100%	PP = 1.00 tsf	
37.0	CLAYEY SAND, fine to coarse grained sand; wet, greenish gray, contains shell fragments	SC	-15.5	B2	40	S-11, SPT 3+2+2+4 REC=24", 100%	MC = 28.2%	
					45	S-12, SPT 3+3+4+4 REC=24", 100%	MC = 23.8%	
47.0	FAT CLAY; wet, greenish gray, contains shell fragments	CH	-25.5	B1	50	S-13, SPT 1+1+3+4 REC=24", 100%	LL = 71 PL = 22 MC = 48.0% % Passing #200 = 93.3 PP = 1.50 tsf	
					55	S-14, SPT 1+2+2+3 REC=24", 100%	PP = 1.75 tsf	
57.0	CLAYEY SAND, fine to medium grained sand; wet, greenish gray, contains shell fragments	SC	-35.5	B2	60	S-15, SPT 1+2+3+2 REC=24", 100%	MC = 32.6%	
60.0			-38.5					

Bottom of Boring at 60.0 ft.
Boring terminated at selected depth.
Boring backfilled with cuttings upon completion.

Schnabel ENGINEERING | **TEST BORING LOG**

Project: Pressure Reducing & Offline Storage Facility
1941-1949 Wilroy Road
Suffolk, Virginia

Boring Number: **B-103**
Contract Number: 22330066.020
Sheet: 1 of 2

Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Contractor Foreman: J. Rassio

Schnabel Representative: E. Walsh

Equipment: CME-55 (Track)

Method: 2-15/16" O.D. Tri-cone Roller Bit

Hammer Type: Auto Hammer (140 lb)

Dates Started: 3/20/23 **Finished:** 3/20/23

Location: See Location Plan

Ground Surface Elevation: 22± (ft) **Total Depth:** 60.0 ft

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered ▽	3/20	12:50 PM	6.0'	---	---

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
0.2	Topsoil; 2 inches		21.3					TABB FORMATION
	CLAYEY SAND, fine to medium grained sand; moist, gray	SC		A2		S-01, SPT 2+2+3+3 REC=18", 75%		
2.0	SANDY LEAN CLAY; moist, gray, contains mica	CL	19.5	A1		S-02, SPT 3+4+7+6 REC=20", 83%	LL = 30 PL = 16 MC = 18.4% % Passing #200 = 50.9 PP = 2.25 tsf	
4.0	CLAYEY SAND, fine to coarse grained sand; moist, grayish brown and orangish brown ▽ Change: wet, orangish brown	SC	17.5	A2	5	S-03, SPT 2+4+4+5 REC=14", 58%		
						S-04, SPT WOH/24" REC=24", 100%	MC = 41.7%	
						S-05, SPT WOH+1+2+1 REC=24", 100%		
9.0	SILTY SAND, fine to coarse grained sand; wet, greenish gray, contains shell fragments	SM	12.5		10			YORKTOWN FORMATION
					15	S-06, SPT 2+2+2+2 REC=24", 100%	LL = NP MC = 35.0% % Passing #200 = 27.7	
17.0	CLAYEY SAND, fine to medium grained sand; wet, greenish gray, contains shell fragments	SC	4.5	B2		S-07, SPT 1+2+2+2 REC=24", 100%	MC = 36.0%	
					20			
					25	S-08, SPT 2+2+3+3 REC=24", 100%	MC = 37.5%	

(continued)

Schnabel Engineering TEST BORING LOG	Project: Pressure Reducing & Offline Storage Facility, 1941-1949 Wilroy Road, Suffolk, Virginia	Boring Number: **B-103** Contract Number: 22330066.020 Sheet: 2 of 2

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
	CLAYEY SAND, fine to medium grained sand; wet, greenish gray, contains shell fragments *(continued)*	SC		B2	30	S-09, SPT 2+2+3+2 REC=24", 100%	MC = 33.7%	YORKTOWN FORMATION
					35	S-10, SPT 2+1+2+3 REC=24", 100%		
					40	S-11, SPT 3+3+3+4 REC=24", 100%		
	Change: fine to coarse grained sand				45	S-12, SPT 4+3+4+6 REC=24", 100%	LL = 34 PL = 19 MC = 28.3% % Passing #200 = 41.3	
47.0	FAT CLAY; wet, greenish gray, contains shell fragments	CH	-25.5	B1	50	S-13, SPT WOH+2+2+5 REC=24", 100%	MC = 43.6% PP = 1.50 tsf	
52.0	SANDY LEAN CLAY; wet, greenish gray, contains shell fragments	CL	-30.5		55	S-14, SPT 2+2+2+4 REC=24", 100%	MC = 34.0% PP = 2.00 tsf	
57.0	CLAYEY SAND, fine to medium grained sand; wet, greenish gray, contains shell fragments	SC	-35.5	B2	60	S-15, SPT 3+3+3+3 REC=24", 100%	MC = 32.0%	
60.0			-38.5					

Bottom of Boring at 60.0 ft.
Boring terminated at selected depth.
Boring backfilled with cuttings upon completion.

Schnabel Engineering TEST BORING LOG	Project: Pressure Reducing & Offline Storage Facility 1941-1949 Wilroy Road Suffolk, Virginia	Boring Number: B-104 Contract Number: 22330066.020 Sheet: 1 of 2

Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Contractor Foreman: J. Rassio

Schnabel Representative: E. Walsh

Equipment: CME-55 (Track)

Method: 2-15/16" O.D. Tri-cone Roller Bit

Hammer Type: Auto Hammer (140 lb)

Dates Started: 3/21/23 **Finished:** 3/21/23

Location: See Location Plan

Ground Surface Elevation: 21± (ft) **Total Depth:** 30.0 ft

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered ▽	3/21	10:25 AM	6.0'	---	---

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
0.3	Topsoil; 3 inches		20.8			S-01, SPT 2+1+4+3 REC=18", 75%		TABB FORMATION
	CLAYEY SAND, fine to medium grained sand; moist, light brown	SC		A2				
2.0	SANDY LEAN CLAY; moist, grayish brown	CL	19.0	A1		S-02, SPT 1+4+5+6 REC=18", 75%	MC = 19.6%	
4.0	CLAYEY SAND, fine to coarse grained sand; moist, light brown and reddish brown, few gravel ▽ Change: wet	SC	17.0	A2	5	S-03, SPT 3+3+3+3 REC=12", 50%	MC = 16.4%	
						S-04, SPT 2+1+1+2 REC=14", 58%	LL = 35 PL = 19 MC = 37.6% % Passing #200 = 24.6 Resistivity = 3500 Ohms-cm Redox = 337 mv Sulfides = 0 pH = 6.8	
						S-05, SPT WOH/24" REC=24", 100%		
					10			
12.0	SILTY SAND, fine grained sand; wet, greenish gray, contains shell fragments	SM	9.0	B2				YORKTOWN FORMATION
					15	S-06, SPT 1+2+1+3 REC=24", 100%	LL = NP MC = 36.9% % Passing #200 = 26.0	
	Change: fine to medium grained sand				20	S-07, SPT 2+2+2+2 REC=24", 100%		
					25	S-08, SPT 2+2+2+4 REC=24", 100%	MC = 31.4%	

(continued)

Schnabel Engineering TEST BORING LOG	**Project:**	Pressure Reducing & Offline Storage Facility 1941-1949 Wilroy Road Suffolk, Virginia	**Boring Number:** **B-104** **Contract Number:** 22330066.020 **Sheet:** 2 of 2

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
	SILTY SAND, fine grained sand; wet, greenish gray, contains shell fragments *(continued)*	SM		B2		S-09, SPT 2+2+2+2 REC=24", 100%		YORKTOWN FORMATION
30.0			-9.0		30			

Bottom of Boring at 30.0 ft.
Boring terminated at selected depth.
Boring backfilled with cuttings upon completion.

Response of Unconfined Aquifer to Sudden Change in Boundary Head

By D. A. Lockington[1]

Abstract: Simple, analytical approximations to the solution of the one-dimensional Boussinesq equation are obtained using a weighted residual method. The approach can be applied to both the recharging and the dewatering of an unconfined, homogeneous aquifer from a fully penetrating trench. Estimates for recharge, discharge, and the elevation of the water table are given by explicit algebraic expressions. Comparison with numerical solutions illustrates the accuracy of these new formulas.

INTRODUCTION

Ground water flow in an unconfined aquifer may be approximately modeled by the nonlinear Boussinesq equation, assuming Dupuit's hypothesis of horizontal flow applies (de Marsily 1986). Solutions of the Boussinesq equation are applicable in catchment hydrology and baseflow studies (e.g., Troch et al. 1993) as well as agricultural drainage problems (e.g., Perrochet and Musy 1992) and constructed, subsurface wetlands.

Corrections to the basic nonlinear equation have been derived to account for vertical flow (e.g., Parlange et al. 1984) as well as the presence of the capillary fringe (Parlange and Brutsaert 1987). Approximate solutions to the Boussinesq equation are generally sought numerically, though important analytical solutions exist for the linearized form of the equation or its corrected form (e.g., Hall and Moench 1972; Marino 1973; Haushild and Kruse 1962; van de Giesen et al. 1994). Accurate, analytical approximations to the nonlinear equation are rare, even for simple problems. Tolikas et al. (1984) derived an approximate solution for the one-dimensional case of an aquifer being recharged when the piezometric head at its boundary is abruptly raised. Their result requires the solution of a system of nonlinear algebraic equations. The approach used is not suitable for the converse problem of aquifer dewatering (Tolikas et al. 1984).

In the present paper, a weighted residual method (Lockington 1993, 1994a,b) is used to solve the one-dimensional Boussinesq equation for both the recharging and the dewatering of a homogeneous, unconfined aquifer. The "semiinfinite" aquifer is supplied or drained by a fully penetrating trench (or canal). Estimates for recharge, discharge, and the position of the water table are given by straightforward algebraic formulas. The results are compared with numerical solutions.

RECHARGING AQUIFER

In this case, the ground-water flow problem is formulated as follows:

$$\frac{\partial h}{\partial t} = \frac{K}{S}\frac{\partial}{\partial x}\left[h\frac{\partial h}{\partial x}\right] \quad \text{with} \tag{1}$$

$$h = h_1 \quad x = 0 \quad t > 0 \tag{2}$$

$$h = h_0 \quad x > 0 \quad t = 0 \tag{3}$$

$$h = h_0 \quad x \to \infty \quad t > 0 \tag{4}$$

where $h[L]$ = piezometric head; $x[L]$ = horizontal coordinate; $t[T]$ = time; $K[L/T]$ = hydraulic conductivity; and S = specific yield. The initial, uniform saturated depth of the aquifer is h_0. At $t = 0$, the water level in the supply trench ($x = 0$) is suddenly increased to h_1 (Fig. 1).

Substitution of the variables

$$\phi = x/\sqrt{\tau} \tag{5}$$

$$H = \frac{h - h_0}{h_1 - h_0} \tag{6}$$

$$\tau = \frac{K}{S}t \tag{7}$$

transforms (1) through (4) to

$$-\frac{\phi}{2}\frac{dH}{d\phi} = \frac{d}{d\phi}\left\{[(h_1 - h_0)H + h_0]\frac{dH}{d\phi}\right\} \quad \text{and} \tag{8}$$

$$H = 1 \quad \phi = 0; \quad H = 0 \quad \phi \to \infty \tag{9}$$

Integrating (8) yields

$$-2[(h_1 - h_0)H + h_0] = \int_0^H \phi\, d\bar{H}\,\frac{d\phi}{dH} \tag{10}$$

since the flux vanishes at $H = 0$. An approximate solution, $\phi_a(H)$, will not satisfy (10) exactly, so a residual function $\varepsilon_a(H)$ is defined as

$$\varepsilon_a = 2[(h_1 - h_0)H + h_0] + \int_0^H \phi_a(\bar{H})\, d\bar{H}\,\frac{d\phi_a}{dH} \tag{11}$$

Weighted averages of the residual are forced to be zero to produce a set of integral conditions that will enable parameters in ϕ_a to be determined. The two-parameter approximation

$$\phi_a = \lambda(1 - H^{\mu}) \tag{12}$$

is proposed with the two integral conditions

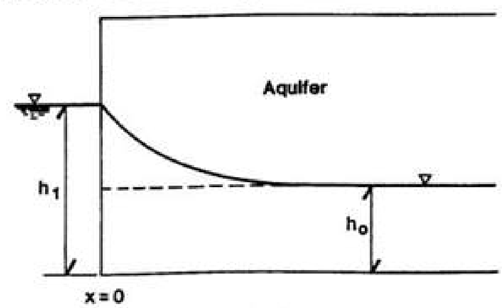


FIG. 1. Recharging Aquifer

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Note. Discussion open until July 1, 1997. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on July 24, 1995. This paper is part of the *Journal of Irrigation and Drainage Engineering*, Vol. 123, No. 1, January/February, 1997. ©ASCE, ISSN 0733-9437/97/0001-0024–0027/$4.00 + $.50 per page. Paper No. 11214.

TABLE 1. Comparison of Algebraic Solution for Recharge Coefficient, Eq. (21) with (15), (18), and (19), and Corresponding Numerical Solution Using Mathematica (*Mathematica* 1992); h_0 Is Taken as 1

h_1 (1)	C_r (numerical) (2)	C_r [Eq. (21)] (3)
1.2	1.1979	1.1982
1.5	1.2948	1.2955
2.0	1.4411	1.4425
3.0	1.6952	1.6971

$$\int_0^1 \varepsilon_a \, dH = 0 \quad \text{and} \tag{13}$$

$$\int_0^1 H^n \varepsilon_a \, dH = 0 \tag{14}$$

Note that λ and μ must have the same sign.

Following Parlange et al. (1994), the exponent, n, is taken as

$$n = 2.27932 - \frac{3h_0}{(h_1 + 2h_0)} \tag{15}$$

Substituting (11) and (12) into (13) yields λ as

$$\lambda^2 = \frac{(1 + \mu)(1 + 2\mu)}{2\mu^2}(h_0 + h_1) \tag{16}$$

while (14) becomes

$$\frac{2[(1 + n)h_1 + h_0]}{(1 + n)(2 + n)} = \frac{\lambda^2\mu^2(2 + n + 2\mu)}{(1 + \mu)(1 + n + \mu)(1 + n + 2\mu)} \tag{17}$$

Eliminating λ between (16) and (17) leads to a quadratic equation for μ that yields

$$\mu = -\frac{3}{4}(1 + n) + \frac{n}{(2 - A)} + \frac{[(2 - A)^2(1 + 2n) + n^2(2 + A)^2]^{1/2}}{4(2 - A)} \tag{18}$$

$$\text{where} \quad A = \frac{4[h_0 + (1 + n)h_1]}{(1 + n)(2 + n)(h_1 + h_0)} \tag{19}$$

The recharging flux at the boundary is defined in terms of a recharge coefficient, C_r, given by

$$C_r = \int_0^1 \phi \, dH \tag{20}$$

Substitution of (12) and (16) yields

$$C_r^2 = \frac{(1 + 2\mu)(h_1 + h_0)}{2(1 + \mu)} \tag{21}$$

Eq. (21) is compared with the numerical solution of (8) and (9) in Table 1 for a range of values of h_1, taking h_0 as unity. The numerical solution was obtained using Mathematica (*Mathematica* 1992) and was verified against the solution of Sidiropoulos and Tolikas (1984).

DISCHARGING AQUIFER

If $h_1 < h_0$, the aquifer is dewatering (Fig. 2) and the equivalent form of (10) is

$$2[(h_0 - h_1)H + h_1] = \int_H^1 \phi \, d\tilde{H} \frac{d\phi}{dH} \tag{22}$$

$$\text{where} \quad H = \frac{h - h_1}{h_0 - h_1} \tag{23}$$

The approximate solution is taken as

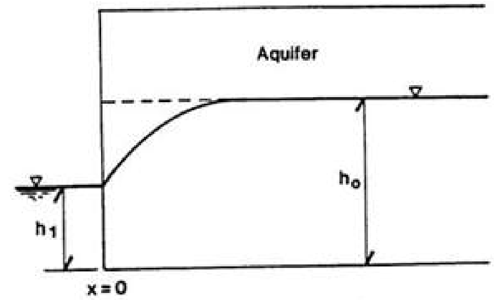

FIG. 2. Discharging Aquifer

TABLE 2. Comparison of Discharge Solution, Eq. (29) with (27), (28), and (30), with Numerical Value Calculated using Mathematica (*Mathematica* 1992); h_1 Is Taken as 1

h_0 (1)	C_d (numerical) (2)	C_d (29) (3)
1.2	1.1685	1.1687
1.5	1.2258	1.2261
2.0	1.3150	1.3154
3.0	1.4758	1.4760

$$\phi_a = \lambda[(1 - H)^{-\mu} - 1] \tag{24}$$

where λ and μ have the same sign. The associated residual is weighted with 1 and $(1 - H)^n$ then integrated over the range of H to provide two integral conditions for the determination of λ and μ. That is

$$\lambda^2 = \frac{(1 - \mu)(1 - 2\mu)}{2\mu^2}(h_0 + h_1) \quad \text{and} \tag{25}$$

$$\frac{2[h_0 + (n + 1)h_1]}{(1 + n)(2 + n)} = \frac{\lambda^2\mu^2(2 + n - 2\mu)}{(1 - \mu)(1 + n - \mu)(1 + n - 2\mu)} \tag{26}$$

As before, λ is eliminated between (25) and (26) and μ is found as the root of a quadratic equation

$$\mu = \frac{3}{4}(1 + n) - \frac{n}{(2 - A)} - \frac{[(2 - A)^2(1 + 2n) + n^2(2 + A)^2]^{1/2}}{4(2 - A)} \tag{27}$$

$$\text{with} \quad A = \frac{4[h_0 + (1 + n)h_1]}{(1 + n)(2 + n)(h_0 + h_1)} \tag{28}$$

The discharging flux is related to the discharge coefficient, C_d, which is defined as $\int_0^1 \phi \, dH$. Eq. (24) with (25) then gives

$$C_d^2 = \frac{(1 - 2\mu)(h_0 + h_1)}{2(1 - \mu)} \tag{29}$$

In this case there is no equivalent to (15). However, the exact solution is known for a constant "diffusivity" [i.e., $h_1 \sim h_0$ in (22)] (see Crank 1975) and a precise numerical solution is available for $h_1 = 0$ (Lisle et al. 1987). The most appropriate value for n can be determined in each case and an interpolation after Parlange et al. (1994) yields

$$n = 1.251 + \frac{0.099h_1}{(h_0 + 2h_1)} \tag{30}$$

In Table 2, (29) with (27), (28) and (30) is compared with a numerical solution of the governing boundary value problem obtained, as before, using *Mathematica* (1992).

EXAMPLE

As an illustration of the application of the new solutions, consider flow in a shallow sand aquifer with hydraulic con-

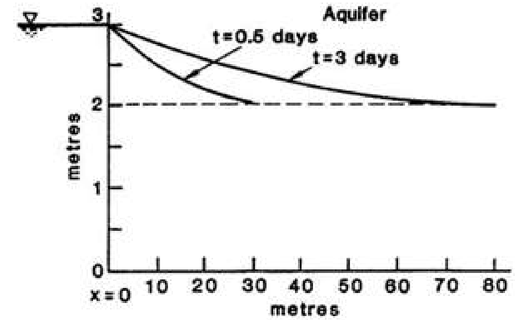

FIG. 3. Progression of Groundwater Mound in Sand Aquifer

ductivity $K = 20$ m/d and specific yield $S = 0.27$ (Fetter 1988). The aquifer is underlain by a horizontal, impermeable base taken as datum and initially has a uniform water table elevation of 2 m. The water level in a recharge trench is abruptly raised to 3 m (see Fig. 3). In this case, $n = 1.4222$, from (15); $A = 0.8943$, from (19); $\mu = 0.5215$, from (18); and (16) gives λ as 5.3453 $m^{1/2}$. Eq. (21) yields C_r as 1.8322 $m^{1/2}$.

The approximate water table profile, (12) can be written in terms of the original variables and parameters as

$$h = h_0 + (h_1 - h_0)\left(1 - \frac{x}{\lambda}\sqrt{\frac{S}{Kt}}\right)^{1/\mu} \quad \text{or} \tag{31}$$

$$h = 2 + \left(1 - 0.0217\frac{x}{\sqrt{t}}\right)^{1.9174} \tag{32}$$

Similarly, if the same aquifer is dewatering with $h_0 = 3$ m and $h_1 = 2$ m, the solution is

$$h = h_0 - (h_0 - h_1)\left(1 + \frac{x}{\lambda}\sqrt{\frac{S}{Kt}}\right)^{-1/\mu} \tag{33}$$

That is,

$$h = 3 - \left(1 - 0.0136\frac{x}{\sqrt{t}}\right)^{3.9328} \tag{34}$$

since $n = 1.2793$, from (29); $A = 0.4045$, from (28); and (27) and (25) give $\mu = -0.2543$ and $\lambda = -8.5535$ $m^{1/2}$, respectively. C_d is then 1.7340 $m^{1/2}$.

Of course, in both cases the boundary flux is given simply in terms of (21) and (29). That is, for recharge

$$-Kh_1 \left.\frac{\partial h}{\partial x}\right|_{x=0} = \frac{C_r(h_1 - h_0)\sqrt{KS}}{2\sqrt{t}} \tag{35}$$

while for a dewatering aquifer

$$-Kh_1 \left.\frac{\partial h}{\partial x}\right|_{x=0} = \frac{C_d(h_1 - h_0)\sqrt{KS}}{2\sqrt{t}} \tag{36}$$

For this example, recharge rate per meter length of trench will be 2.1288 $t^{-1/2}$ m^3/d while discharge per meter length of trench takes place at the rate of 2.0147 $t^{-1/2}$ m^3/d.

CONCLUSION

The comparison in Tables 1 and 2 demonstrates the accuracy of the new formulas for the recharge or discharge coefficient over a range of values of boundary head appropriate for the practical application of the Boussinesq equation to unconfined ground water flow (Tolikas et al. 1984). The absolute relative error compared with numerical solutions in each case is much less than 0.1%. As an example of the accuracy of the solution for the water table elevation, Fig. 4 presents (24) and a numerical solution of (8) for a draining aquifer with $h_1 = 1$ and $h_0 = 1.5$, in terms of the reduced variables, ϕ and H. The numerical solution was obtained using a shooting method Burden and Faires. The profiles are essentially indistinguishable for all practical purposes.

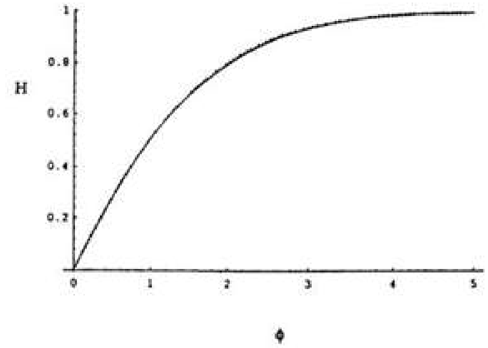

FIG. 4. Comparison of Approximate Profile, Eq. (24), with Numerical Solution for Dewatering Aquifer with $h_0 = 1.5$ m and $h_1 = 1$ m; Numerical Solution Is Dotted Line

The nonlinear Boussinesq equation has been solved by a weighted residual approach leading to a simple algebraic solution. Although derived for a semiinfinite aquifer, the nonnearity of the problem means the solution will also apply to finite aquifers until the watertable begins to change at the interior boundary.

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