Subsurface Investigation and Foundation Design Report

Newberry County Capital Sales Tax Project No. 6 Ten Water Point Locations for the Consolidated Fire District

Newberry County, South Carolina

March 14, 2018

Executive Summary

HDR Engineering Inc., of the Carolinas, (HDR) understands that the County of Newberry (the County) is planning to install one (1) 40,000-gallon fiberglass reinforced plastic (FRP) underground fire suppression water storage tank, on ten different sites across the eastern, central, and northeastern portions of the county. HDR was contracted to design the foundations for these tanks.

It is our understanding that County wishes to install DARCO, Inc., brand tanks, or a commercially available equivalent FRP tank product. For the purposes of this design, we utilized design criteria and recommendations for DARCO, Inc. brand FRP tanks.

Our subsurface investigation consisted of a site reconnaissance, advancing one (1) 50-foot boring, collecting disturbed and undisturbed samples, and performing geotechnical laboratory determination of soil index and strength properties. Our foundation design consisted of bearing capacity and settlement analyses for each site and general construction related recommendations.

Sufficient bearing capacity to support tank installation was exhibited at each site. Anticipated bearing pressure for the foundation pads supporting the water storage tanks were calculated to be approximately 1.6-kips/ft^2. The existing in-situ vertical stress at the foundation level is approximately 1.8-kips/ft^2. The foundations will be embedded approximately 15-feet below the existing ground surface elevation and will consist of a graded aggregate pad. The foundations will be supported on medium dense to dense residual soils, either silty sand or silt underlain by weathered rock and rock. Given the nature of the foundation soils, and the fact that the total bearing stresses beneath the tank foundations will not exceed the existing vertical stresses, post-construction settlements are anticipated to be small, less than ¼ inch, at each of the proposed sites. The majority of the settlement is expected to occur during tank installation and within two weeks of the completion of construction at each site. Since the predominant foundation soil type is silt, negligible, if any, secondary settlement would be expected to occur.

We have included key portions of a manufacturer's installation recommendations relative to specific sections of this report for ease of reference. The complete manufacturer installation recommendations, as well as their installation manual may be downloaded from their website.

The results of our field investigation, laboratory testing and analyses, as well as foundation design and construction recommendations are presented herein.

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1 Introduction

1.1 Purpose

The Newberry County Consolidated Fire District (the County) contracted HDR to perform engineering services to support installation of fire suppression water supply tanks on ten sites in eastern, central, and northeastern Newberry County, South Carolina.

1.2 Scope

This report is intended as a report of the findings of the subsurface investigation and foundation design recommendations for installation of an approximately 10-foot by 70-foot, 40,000-gallon fiberglass reinforced plastic (FRP) underground tank at each of the ten sites selected by the County. It is our understanding that the tanks will be used to supply firefighting water in rural areas where fire hydrants are unavailable. Our design recommendations are made in accordance with National Fire Protection Association (NFPA-22) requirements, manufacturer installation recommendations, generally accepted engineering practices, and our experience on similar projects. This report is not intended to be, nor should it be construed as, a construction specification.

2 Sites

The ten site locations were provided to us by the County. Table 2-1 lists the ten sites, in non-specific order assigned by HDR, as well as GPS coordinates. A map of the sites is provided in Appendix A of this report.

Table 2-1. Site Locations and Coordinates

Site Number and Description	Northing	Easting
1. Fire Station	34.401767	-81.573638
2. Fire Station	34.456252	-81.473912
3. Fire Station	34.312155	-81.396802
4. Fire Station	34.369818	-81.447944
5. Active Recycling Center	34.375666	-81.410662
6. Church	34.449300	-81.496040
7. Cleared Field (Near Residence)	34.391241	-81.474446
8. Turkey Farm	34.464223	-81.451891
9. Cleared Field (Near Residence)	34.342335	-81.450075
10. Wooded Area, adjacent to cleared field (Near Residence)	34.366501	-81.526476

Table 2-1. Site Locations and Coordinates

Site Number and Description	Northing	Easting

Source: Newberry County

2.1 General Site Conditions

The ten sites were physically situated in eastern-central and northeastern Newberry County, South Carolina. The sites were generally flat to gently sloping and all but Site #10 are either cleared, grassed, or cultivated agricultural lands. Site #10 is located in a moderately wooded area adjacent to a private residence that will require clearing prior to construction. Table 2-2 below provides site specific information.

Table 2-2. General Site Conditions

Site	Rock Observed	Clearing Required	Topography	Utilities	Soil Boring	Comments
1	None.	None.	Gently Sloping	None	B8	Fire station on-site
2	None.	None.	Gently Sloping	None	B5	Fire station on-site
3	None.	None.	Gently Rolling to Flat	None	B1	Fire station on-site
4	Some visible near ground surface.	None.	Gently Sloping to Flat	Some Type of Warning Siren	B3	Fire station on-site
5	None.	None.	Gently Rolling	Overhead Power Lines, Probably Underground	B2	Active Recycling Center
6	None.	None.	Flat	Overhead Power Lines, Storm Drainage	B7	Church on-site
7	None.	None.	Flat	None	B10	In a cleared field
8	None.	None.	Flat	Well on Tree Line, Overhead Lines	B6	Active turkey farm on-site
9	None.	Tall grass.	Gently Sloping	Underground and Overhead Power Lines, Storm Drainage	B4	Boring in field across the street from a private residence with several outbulidings
10	None.	Moderately wooded.	Gently Sloping	Overhead Power Lines Near Road, None at Site	B9	Boring in wooded lot adjacent to private residence

2.2 General Site Geology

The ten Newberry County sites lie within the Piedmont physiographic province of South Carolina. The Piedmont consists of moderate to steep topography with soils predominantly derived from weathered rock and alluvial deposits.

Each of the proposed sites lie within the Charlotte Terrane, a complex of metasedimentary and metavolcanic rocks that are predominantly schistose with some gneissic zones, intruded by plutonic sequences including predominantly granite and minor gabbroic intrusions (Hibbard et al, 2013). The rock types underlying the sites vary within the aforementioned units. Sites 1 and 3 are within schistose/gneissic metamorphic rocks. Sites 2, 6, and 8 lie within a unit mapped as mylonitic gneiss, and Sites 4, 5, 7, 9, and 10 lie within the Newberry Granite (SCGS, 2017). Figure 2-1 presents the location of the sites relative to the geologic units.

The soils underlying each of the sites are residual soils derived in place from rock that underlies the sites. The soils underlying the sites vary somewhat based on USDA NRCS mapping. At Site 1, the soils are mapped as the Cecil sandy clay loam with 2 to 7 percent slopes and moderately eroded, and Hard Labor sandy clay loam with 2 to 6 percent slopes. Both are identified as residuum weathered from gneiss. At Site 2, the soils are mapped as Cecil sandy clay loam with 2 to 6 percent slopes weathered from gneiss and the Wynott-Winnsboro complex, 2 to 6 percent slopes described as residuum weathered from grano-diorite. Site 3 soil consists of the Cecil sandy clay loam derived from weathered schist or gneiss with 2 to 6 percent slopes. At Site 4 the Helena sand loam with 2 to 10 percent slopes derived from weathered granite and the Santuc loamy course sand, 2 to 6 percent slopes also residual soil derived from granite make up the soils. Site 5 soils consist of the Hard Labor sandy loam, 2 to 6 percent slopes, derived from weathered granite, and Wynott-Winnsboro complex, 2 to 6 percent and 6 to 10 percent slopes consisting of sandy loam, clay, and sandy clay loam derived from weathered diorite and gabbro. Site 6 soils consist of the Cecil sandy clay loam with slopes from 2 to 6 and 6 to 10 percent that are residual soils derived from weathered gneiss, the Rion Sandy loam, 15 to 25 percent slopes and Pacolet sandy clay loam 15-25 percent slopes both residual soils of weathered gneiss. Site 7 soils consist of the Cecil sandy clay loam having 2 to 6 and 6-10 percent slopes. These soils are residual soils derived from weathered granite. Site 8 soils consist of the Cecil sandy clay loam having 2 to 6, 6-10, and 7-15 percent slopes, and the Pacolet sandy clay loam having 15-25 percent slopes. All soils are residual soils derived from weathered gneiss. Site 9 soils consist of the Wynott Winnsboro complex having 2-6 percent slopes that is residual soil derived from weathered diorite or gabbro, the Cecil sandy clay loam with 2 to 6 and 6 to 10 percent slopes that is residual soil derived from weathered granite, and the Pacolet sandy clay loam having 15-25 percent slopes. Site 10 soils consist of the Cecil Sandy clay loam having 2 to 6 percent slopes also a residual soil derived from weathered granite. The predominant soil type across all ten sites is sandy clay loam. Sandy clay loam consists of 45 or greater percent sand, less than 28 percent silt, and 20-35 percent clay. Coarse grained residual soils in the Piedmont are generally medium dense while fine grained residual soils typically have a consistency of medium stiff to very stiff.

While faults and shear zones are mapped in Newberry County (SCGS, 2017) there are no faults classified as active with the Quaternary (1.6 million years) in the vicinity of the ten sites (USGS, 2017).

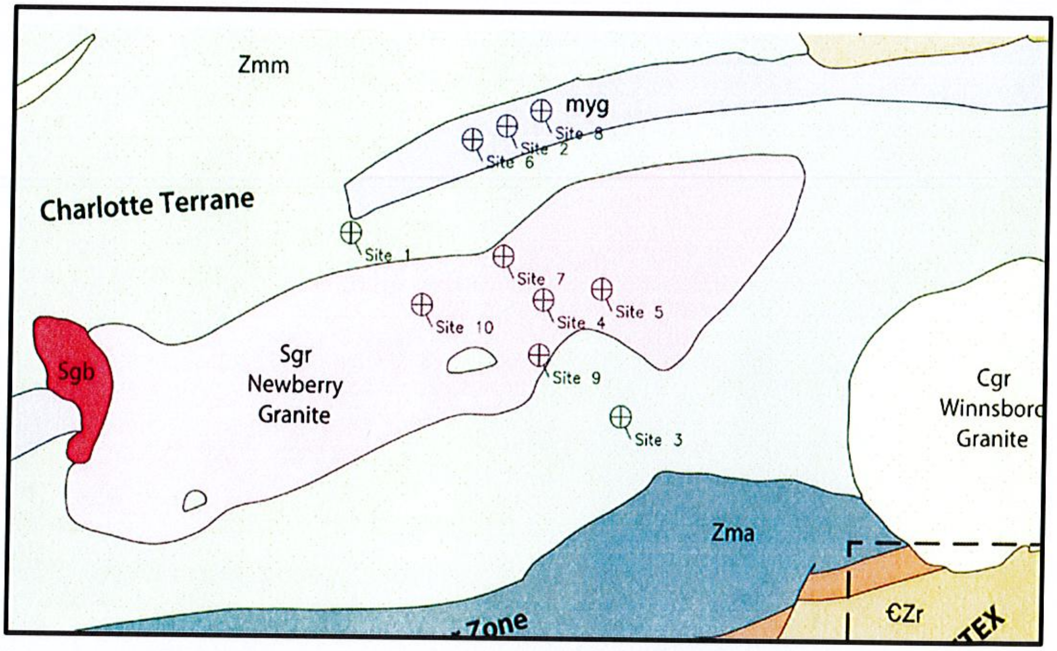

Figure 2-1. Geologic Units.

(Zmm-Charlotte Terrane (Schistose and Gneissic meta-sedimentary and metavolcanic rocks); Sgr-Newberry Granite (granitic to diorite intrusive igneous rocks).; myg-mylonitic gneiss; Zma-migmatitic gneiss and amphibolite.)

Source: United State Department of Agriculture Natural Resource Conservation Service, Web Soil Survey, https://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx

2.3 General Groundwater Conditions

Groundwater was encountered in boring B-10 at an approximate depth of 45-feet BEGSE, but it was not encountered in any of the other borings at the time of drilling. Variations in the groundwater elevation should be anticipated, since groundwater conditions can change based on rainfall, seasonal changes, construction activity, and other factors.

2.4 Screening for Seismic Hazards

HDR conducted a screening for seismic hazards at the proposed site locations to estimate potential for seismically induced shear strength loss (SSL) and ground motions. For estimation of seismic demands we have based our analyses on Reference 8.

The first step in our screening process involved the estimation of Seismic Site Class based on the IBC criteria, as detailed in Reference 8. The majority of the ten sites were classified as Site Class D, which is defined as a site with "Stiff Soil" having an average shear wave velocity of 600 to 1,200 feet/sec in the upper 100 feet of the soil profile.

Site 3 was classified as Site Class C, "Very Dense Soil or Soft Rock" with an average shear wave velocity of 1,200 to 2,500 feet/sec. A classification of Site Class D was considered to be representative for all the proposed sites. Our estimation of Site Class was based on the soil borings with Standard Penetration Testing (SPT) blow counts conducted at each of the sites.

The USGS Seismic Hazards Mapping Tool, Reference 9, was then used to construct the horizontal acceleration response spectrum at the ground surface for both the Maximum Considered Earthquake (MCE) and the Design Earthquake using IBC criteria. Our analyses indicate that the design Peak Ground Acceleration (PGA) at the project sites will be 0.14g, or lower, where "g" is the acceleration due to gravity at sea level. The analyses for the response spectra are included in Appendix B of this report.

The soils encountered at the project site are primarily medium to dense mixes of residual sand and silt transitioning to rock or weathered rock at depths ranging from about 26 to 50 feet. Based on the observed soil conditions, the magnitude of the PGA at the project site, and the depth of embedment of the water tank footings (about 15 feet), we estimate that the foundation soils supporting the proposed water tanks have a low to very low potential for seismically induced shear strength loss caused by liquefaction. Ground improvements or other measures related to mitigation of seismic hazards are not needed or recommended, based on our screening.

3 Water Storage Tanks

3.1 Tank Dimensions and Loads

It is our understanding that the County will install 40,000-gallon FRP underground tanks made by DARCO, Inc., or a commercially available equivalent FRP tank product, at each of the proposed sites. For the purposes of this design, we utilized design criteria and recommendations for DARCO, Inc. brand FRP tanks. It is recommended that the Contractor confirm compatibility of the brand to be utilized with the DARCO installation recommendations presented herein. According to DARCO, the 40,000-gallon fiberglass reinforced plastic tanks measure approximately 10-feet in diameter, are 69-feet long, and weigh approximately 12,500-lbs empty.

3.2 Manufacturer Design Recommendations

The manufacturer provides installation recommendations for use with installation of their water tanks, and these recommendations were reviewed by HDR. Key portions of those recommendations regarding excavation, backfill, compaction, bedding material, anti-flotation anchors ("dead men"), construction Quality Assurance/Quality

Control, and traffic slabs, relative to the following sections of this report are reprinted for ease of reference in Appendices E through G.

The complete manufacturer installation brief may be downloaded at http://darcoinc.com/wordpress/wp-content/uploads/2016/08/FINISHED-IB-031416.pdf. The complete manufacturer installation manual may be downloaded at http://darcoinc.com/wordpress/wp-content/uploads/2017/03/Online-copy-FRP-Manual-2017.pdf.

4 Subsurface Investigation

4.1 Soil Borings

HDR drill crews mobilized to the sites on July 27, 2017, drilling one (1) boring to a target depth of 50-feet below existing finish grade elevation (BEGSE), or to auger refusal, whichever was shallower, at each of the proposed sites.

The borings were advanced utilizing 3.25-inch inside diameter continuous flight hollow-stemmed augers. In the upper 10-feet of each boring, disturbed samples were continually collected and standard penetration testing (SPT) was conducted. From 10-feet to boring termination depth, sampling and SPT testing were conducted at intervals of every 5-feet.

The boring locations were intended to correlate to the approximate center of the proposed tank location based the site location information provided by the County. Soil types were visually classified during drilling based on observation by HDR personnel in general accordance with the Unified Soil Classification System (USCS) as outlined in Reference 10. Boring logs were prepared for each boring. The logs contained the soil descriptions classified using USCS classifications with general correlations to the AASHTO soil classification system, SPT "N" counts, and lab tests results where applicable. The boring logs are provided in Appendix B of this report.

4.1.1 Boring Results

Table 4-1 provides a summary of the borings conducted at each site.

Table 4-1. Boring Information

Boring Number	Site Number	Boring Termination Depth (ft-BEGSE)	Rock Encountered
1	3	45.1	Yes, at 26.2 feet
2	5	47.8	Partially Weathered Rock
3	4	40.4	No
4	9	50.5	No
5	2	50.5	No

Table 4-1. Boring Information

Boring Number	Site Number	Boring Termination Depth (ft-BEGSE)	Rock Encountered
6	8	50.2	No
7	6	50.5	No
8	1	45.5	No
9	10	40.1	No
10	7	50.2	No

In general, site soils were classified as silty Sand (SM). There were some site soils that were classified as elastic Silt (MH), and three sites yielded soils classified as Silt (ML). Sandy soils from existing ground surface elevation to a depth of 20-feet BEGSE ranged in relative consistency from loose to very dense. Silts exhibited relative consistencies ranging from stiff to hard. Rock or partially weathered rock (PWR) was found shallower than 50-ft BEGSE on only two of the sites. However, splitspoon refusal, defined as an SPT blow count of 50 blows with a spoon advance of 2 inches or less (i.e. SPT N Count of 50/2"), typically occurred on most sites between 40 and 50-feet BEGSE. In the Piedmont soils that characterize these sites, splitspoon refusal typically occurs in PWR. Although soils on most sites could not be definitively characterized as PWR, it should be expected that PWR may be encountered at all ten sites.

The Boring Logs contained in Appendix B of this report provide site specific soils information including soil classifications, relative density/consistency, sample and testing information, and boring termination depth.

4.2 Sampling and Laboratory Testing

At each interval where SPT testing was conducted, disturbed soil samples were also collected. From the samples collected in each boring, selected samples were identified for laboratory testing to classify soil types and determine index properties.

Thin-walled metal tubes (Shelby Tubes) were intended to be collected from each boring from 18-feet to 20-feet BEGSE, to provide samples for laboratory shear strength testing. Recovery from Shelby tubes was only possible for three of the sites. Additionally, to develop site specific compaction standards to be used in construction, a composite sample of the top ten (10) feet of soil was collected. The results of the geotechnical laboratory testing are provided in Appendix C of this report. Table 4-2 on the following page summarizes sampling and testing plan.

Table 4-2. Sampling and Geotechnical Laboratory Testing Summary

Boring No.	Site No.	No. Samples	Shelby Tube	Tested Disturbed Sample No.'s	Laboratory Testing				
					Grain Size Hydrometer	Atterberg Limits	CU	Proctor	Unconfined Compression
1	3	13 and (3) Rock Cores	Yes	SS-7, SS-9	✓			✓	✓
2	5	15	Yes	SS-2, ST-1	✓	✓	✓	✓	
3	4	10 and (3) Rock Cores	Yes	SS-6, ST-1	✓			✓	
4	9	14	Yes	SS-5, ST-1	✓	✓		✓	
5	2	14	Yes	SS-2, SS-9	✓	✓		✓	
6	8	15	No	SS-7, ST-1	✓	✓	✓	✓	
7	6	14	Yes	SS-4, SS-9	✓	✓		✓	
8	1	13	Yes	SS-7, ST-1	✓	✓		✓	
9	10	12	Yes	SS-7, ST-1	✓	✓	✓	✓	
10	7	11	Yes	SS-7, SS-5	✓	✓		✓	

5 Foundation Design

5.1 Applied Loads

Applied loads for the water tank foundations were estimated as discussed in this section. Some loads were gathered from published literature. Other loads were calculated based on interpretation of these published values. The sources for our assumptions are detailed in the References Section of this report. Other load values were calculated from the results obtained from lab testing.

According to the County, no traffic slabs will be installed directly over the tanks at any of the sites. However, if the County does utilize a traffic slab over a tank, manufacturer recommendations may be provided by the tank manufacturer. Excerpts of the DARCO, Inc. recommendations pertaining to traffic slab installation over a tank are provided for reference in Appendix G of this report.

Table 5-1. Applied Loads

Load	Unit Wt.	Load (kip)	Derivation	Comments
Tank Water	62.4-lbs/ft^3	333.7	40,000 gal x 62.4-lbs/ft^3	The unit weight of water of 62.4-lbs/ft^3 is a generally accepted, published value.
Concrete Slab*	150-lbs/ft^3	255	85-ft x 20-ft x 1-ft	(1) 150- lbs/ft^3 is a generally accepted value for reinforced concrete. (2) The manufacturer recommends that the pad extend at least 1/2-diameter beyond each end and sidewall of the tank.
Concrete Dead Men	150-lbs/ft^3	33	(1-ft x 2-ft x11-ft)x 150 lbs/ft^{3}x10 dead men	Dimensions and number of dead men recommended by the manufacturer.
Stone Backfill	110-lbs/ft^3	1,856	(75-ft x 15-ft x15-ft) x 110-lbs/ft^3	(1) The unit weight of #57 stone is a value published by the SCDOT for this aggregate material (see the following section). (2) The design value was reduced by the volume occupied by the tank.

Concrete traffic slab over the tank is a construction option and the County may choose not to use them. To conservatively estimate foundation loads, each site was modeled as if a slab would be constructed on it.

5.2 Theory

Foundation design for the proposed water tanks consisted of verifying the bearing capacity of the bearing soil stratum, and estimating settlements beneath the proposed foundations at each site, as detailed in this section of the report. The water tanks will be supported on mat foundations of gravel 75-feet long by 15-feet wide by 1-foot thick. The tank and anchors are installed directly on this bedding layer.

Site specific bearing capacity was estimated using the Brinch-Hansen Equation, outlined in Reference 11, reprinted on the following page, as Equation 5-1.

$$(c'*N_C*F_{SC}*F_{DC})+(Q*N_Q*F_{SQ}*F_{DQ})+(0.4*\gamma*B*N_\gamma*F_{S\gamma}*F_{D\gamma}) \quad \textbf{Equation 5-1}$$

Settlement was analyzed using the Schmertmann Method as adapted by Bowles, as it appears in Reference 12. The equation is reprinted below as Equation 5-2.

$$\Delta_H=q_0*B'*(1-\mu_2/E_s)*I_S*I_F \quad \textbf{Equation 5-2}$$

NFPA-22 requires that Bearing Capacity be determined using a Factor of Safety of 3.0. The manufacturer guidelines recommend ideal soil strength parameters and minimum bearing capacity recommendations as follows:

Minimum Bearing Capacity: 2,500-lbs/ft^2 (vertical support)

Minimum Cohesive Strength: 500-lbs/ft^2 (sidewall stability)

We used these parameters as the acceptance criteria for our design.

Due to the silty nature of the foundation soils at each of the sites, the soils exhibited negligible cohesive strength. The lack of cohesion prompted additional excavation in order to satisfy the US Occupational Safety and Health Administration (OSHA) excavation safety requirements. Excavation dimension requirement determination is detailed in Section 6.2.2 of this report. The foundation soils at each of the sites, under the loading conditions described in Section 5.1 of this report, exceeded the minimum bearing capacity criteria.

Neither NFPA-22 nor the manufacturer recommendations provided a maximum settlement amount. NFPA-22 requires that settlements not "impair the structural integrity of the tank." Based on our analyses, the total bearing pressure at the base of the tank foundations following the end construction will be less than the existing vertical pressure prior to excavation. Therefore, we anticipate total and differential vertical settlements resulting from the proposed construction to be minimal, and less than about ¼ inch. The majority of the settlement is expected to occur during and within two weeks of the completion of construction at each of the sites. Since the predominant foundation soil type is silt and sand, negligible, if any, secondary settlement is expected. Differential settlement can be a concern with structures of this size, especially in soils of the nature encountered on these sites. Compaction as recommended in Section 6 of this report will greatly reduce the chances of the occurrence of differential settlements.

A full listing of the sources we used in our analyses and design is contained in Section 7.0 of this report.

5.3 Gravel Bedding and Backfill Material

The manufacturer recommendations require the tank to be bedded on a gravel pad that meets the minimum guidelines described on the following page.

- Rounded "pea gravel" ¼ to ½-inch diameter or crushed rock chips retained on the ¼ to ½-inch screen sizes
- Must have few stones (5% or less) that are greater than ½-inch in size
- Must have a unit weight of at least 100-lbs/ft^3
- Washed and free of fines and organics so that no more than 5% passes the #8 sieve

Source: DARCO, 2016 FRP Tank Installation Brief

The gravel used in our analyses is #57 Stone, with an assumed unit weight of 110-lbs/ft^3, and a gradation from the SCDOT Standard Specifications for Highway Construction as shown in Figure 5-1 on the following page. We recommend that for bedding and backfill, gravel meeting the #57 stone gradation be used on this project. A copy of this gradation is provided for reference in Appendix E of this report.

Gradation of Coarse Aggregates										
Percentage by Weight Passing Sieves Having Square Openings										
Sieve Designation	Aggregate No.									
	CR-14	5	56	57	67	6M	8M	78	789	89M
2-inch	100	--	--	--	--	--	--	--	--	--
1½-inch	95 - 100	100	100	100	--	--	--	--	--	--
1-inch	70 - 100	90 - 100	90 - 100	95 - 100	100	100	--	--	--	--
¾-inch	--	20 - 55	40 - 85	--	90 - 100	90 - 100	100	100	100	--
½-inch	35 - 65	0 - 10	10 - 40	25 - 60	--	--	95 - 100	90 - 100	95 - 100	100
⅜-inch	--	0 - 5	0 - 15	--	20 - 55	0 - 20	75 - 100	40 - 75	80 - 100	98 - 100
No. 4	10 - 40	--	0 - 5	0 - 10	0 - 10	0 - 5	10 - 35	5 - 25	20 - 50	20 - 70
No. 8	--	--	--	0 - 5	0 - 5	--	--	--	--	2 - 20
No. 16	--	--	--	--	--	--	0 - 5	0 - 5	0 - 6	--
No. 100	--	--	--	--	--	--	0 - 2	--	0 - 2	0 - 3

Figure 5-1. SCDOT Coarse Aggregate Gradation.
Source: South Carolina Department of Transportation, 2007 Standard Specifications for Highway Construction, Appendix

6 Construction Recommendations

6.1 General

We recommend that the Contractor follow the tank installation, handling, and storage procedures outlined in References 2 and 3 of this report, as well as the Project Specifications prepared by HDR for construction of this project. We also recommend that one site be completely excavated, have the tank installed, and be completely backfilled to finished grade elevation *before* beginning work at another location. Under no circumstances should an excavation be left open for a total of more than 3 days.

Groundwater was only encountered in boring B-10, but not in any of the other soil borings at the time of drilling. Groundwater elevation can vary based on rainfall, seasonal factors and construction activity. The Contractor should anticipate the potential presence of groundwater in the excavations during construction. A pump should be maintained at the site to expeditiously remove accumulating rainfall, stormwater runoff, or groundwater intrusion. It is also strongly recommended that excavations do remain open longer than 24 hours without placing bedding and backfill material. Under no circumstances should an excavation remain open over a weekend or other period of extended time away from any site.

6.1.1 Generalized Installation Process

We anticipate that tank installation will entail the generalized steps in the sequence outlined below. For specific steps and recommendations for each step, the reader is directed to the corresponding sections of this report, the Project Plans and specifications, and the manufacturer recommendations.

General Installation Sequence

- Clearing and grubbing.
- Excavation to the lines, grades, and a minimum depth of 15-feet BEGSE, as indicated in the construction plans.
- Preparing the subgrade.
- Constructing the deadmen anchors as recommended in Section 6.4 of this report.
- Installing 12-inches of bedding material (#57 stone) that meets the gradation recommendations outlined in the previous section.
- Placing the tank and deadmen.
- Backfilling around the tank with #57 stone to a minimum of 1-foot above the top of the tank as recommended in Section 6.3 of this report.
- Using a vibratory plate compactor, place and compact the stone and backfill soil.
- Completing construction and installing appurtenances as indicated in the Project Plans and Specifications.

6.2 Excavation

The dimensions outlined in Table 6-1 were assumed in our analysis. Excavation dimensions include the manufacturer recommended additional excavation of a minimum of 2-feet beyond each end of the tank and a minimum of 2-feet beyond each tank sidewall. To facilitate the installation of deadmen anchors and to allow for CQA/QC testing, we had designed the excavation for 3-feet of additional excavation beyond the sidewalls and ends of the tanks.

Additionally, to allow for 1-foot of bedding material under the tank, it will be necessary to excavate 1-foot below tank base elevation. Therefore the excavation dimensions shown in Table 6-1 are recommended and a schematic of the excavation is shown in Figure 6-1:

Table 6-1. Excavation Dimensions

Tank Area Excavation (Irregularly Shaped)	
Dimension	**(feet)**
Length	75
Width	16
Depth	15*
End/Side Slope Excavation	
Depth	11
Length	16.5
Slopes	1.5H:1V

****Depth includes 1-foot of additional excavation to allow for installation of bedding material.***

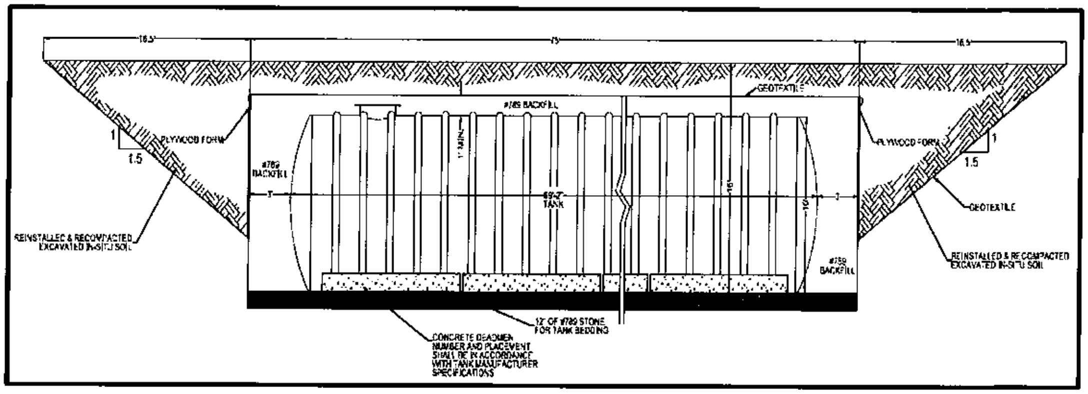

Figure 6-1: Excavation Layout Schematic (NTS)

6.2.1 Excavation Stability

Both NFPA and the manufacturer recommendations require vertical sidewall stability within the excavation. Both sources refer the Engineer to OSHA excavation requirements to ensure the safety of workers installing and backfilling the tank. A cursory examination of the soils from each site was made and compared to OSHA excavation requirements by soil type. HDR has not performed any type of engineering analysis for slope stability. The Contractor should perform an independent analysis to confirm our results prior to performing any construction activities.

OSHA Requirements

HDR, using 29 CFR § 1926 Subpart (P) OSHA Excavation and Trenching Regulations, References 14 and 15, developed these preliminary excavation recommendations for each site. The regulations required first determining the soil type and generalized soil strength parameters, including cohesive strength. These factors then dictated the shape and maximum slope of sidewall and endwall excavations.

Based on the boring logs and lab results, HDR assessed the soils as Class B, soils as defined by the excerpt from the OSHA Regulations:

> *Type B* – Includes cohesive soil with an unconfined compressive strength greater than 0.5 tsf (48 kPa) but less than 1.5 tsf (144 kPa) and granular cohesionless soils (such as angular gravel, similar to crushed rock, silt, silt loam, sandy loam, and, in some cases, silty clay loam and sandy clay loam). See *Appendix A to Subpart P of Part 1926, paragraph (b) — Definitions (Type B)*, for a detailed definition of Type B soil.
>
> *Source: OSHA 2226-10R, 2015 Trenching and Excavation Safety*

Some sites exhibited varying soil strata within 20-feet BEGSE, which could trigger different OSHA slope configuration. However, even in a layered condition, the differing soil layers were still met OSHA Type B soils criteria.

Therefore, HDR recommend the tank excavations, for each site, with the slope configuration shown in Figure 6-2.

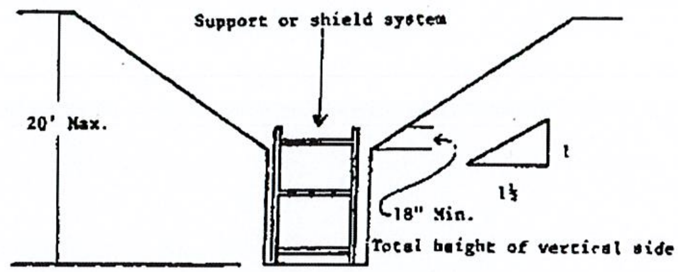

Figure 6-2. OSHA Type B Soils Allowable Excavation Configuration.
Source: OSHA Website

6.3 Preparation for Bedding

The subgrade should be prepared by removing any loose soils, organic materials, and/or other deleterious materials from the base of the excavation. Groundwater entering the excavation and collecting in the bottom should be pumped out continuously. It should be verified that it is uniformly excavated to a depth of 15-feet BEGSE. Using either a vibratory plate compactor or a remote controlled, walk-behind vibrating sheeps foot roller (a "Rammax"), the subgrade should be compacted to an in-situ relative compaction of 95%, as determined by the Standard Test Methods for Density of Soil and Rock in Place by the Sand Replacement Method in a Test Pit, Method A (ASTM D-4914). "Jumping Jack" type compaction equipment is not recommended for use on this project.

6.4 Tank Anti-Flotation Anchoring

To counteract buoyancy forces of a partially-filled tank, periods of sustained rainfall, temporary rise in groundwater surface elevation beyond those encountered during drilling, or other temporary or permanent situations whereby water could enter the tank excavation, we recommended that foundation anchors ("deadmen") be installed.

We have designed for and require that the tanks be anchored using deadmen anchors, as outlined in the manufacturer recommendations. Those guidelines recommend using six (6) 24-inch wide by 12-inch thick by 11-feet long, half-round, anchors on each side of the tanks, for a total of twelve anchors. Alternate configurations for the deadmen are available through the manufacturer.

Excerpts, including a schematic, downloaded from the manufacturer's website and other pertinent information from the DARCO, Inc. FRP Tank Installation Brief and the DARCO Installation Instruction Manual, are reprinted in Appendix D of this report.

6.5 Backfill and Cover

Manufacturer recommendations require the tank to be fully encased in the gravel backfill, detailed in Section 5.2 of this report, to a minimum of 1-foot higher than the top of tank elevation, as depicted in Figure 6-1. It is recommended that gravel backfill be placed using a tremie or chute, rather than directly dumping (i.e. "tailgating"), to direct the gravel into the excavation without damaging the tank.

Gravel should be placed in 12-inch loose lifts and compacted with a vibratory plate compactor, as directed in the manufacturer installation recommendations.

Compacted soil backfill may be used to backfill the excavation above the gravel backfill to the ground surface, possibly with reinforced concrete slabs installed around key appurtenances. As shown on Figure 6-1, a portion of the side and end slopes may be backfilled with compacted soil; however the Contractor may choose to use gravel backfill in these areas at his discretion.

If soil backfill is to be used, once gravel backfill has reached a height of 1-foot above the top of the tank and prior to placement of any soil, a non-woven geotextile separation fabric, such as US Fabrics US-180NW or equivalent should be installed to prevent migration of soil into the gravel backfill. A copy of the product reference sheet is provided in Appendix E of this report for reference.

According to OSHA excavations standards a wide excavation will be necessary. To help reduce construction cost and to facilitate construction, the manufacturer recommendations allow tank installation using a treated plywood separation wall that completely surrounds the tank. The wall is then backfilled with gravel. If such a wall is used, it should be situated no less than 3-feet beyond each side and each end of the tank, as shown in Figure 6-1. The wall should be constructed and installed as outlined in the excerpts from the manufacturer recommendations, reprinted in Appendix E of this report.

Soil used as backfill should meet the following criteria:

- Free of organics, trash, roots, and other deleterious materials
- Have less than 5% of soil retained on the #4 sieve and no more than 25% passing the #200 sieve
- Have a USCS classification of SC, SM, SW-SC, or SW-SM
- Have Liquid Limit ≤ 40, a Plasticity Index ≤ 15

Backfill soils should be compacted to 95% of the Maximum Dry Density and within $\pm$ 2% of the Optimum Moisture Content as determined in an AMRL certified geotechnical testing laboratory, through the Standard Proctor Determination (ASTM D698).

6.6 Quality Control Recommendations

We recommend that a Construction Quality Assurance/Quality Control (CQA/CQC) Plan be developed prior to commencing construction.

The plan should include CQA/CQC testing recommended in the manufacturer installation recommendations, excerpts of which have been provided in Appendix F of this report.

The CQA/CQC plan should contain procedures, references, and methods for addressing areas that do not comply with required standards for the following:

- Hydrostatic leak testing
- Methods for determining that the subgrade meets the minimum relative density recommendations outlined in Section 6.3
- Procedures for probing from the "4:00 position to the 8:00 position" for every lift, after it has been properly compacted
- Methods for determining the compaction of soil backfill

7 References

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