EXHIBIT 5

EXHIBIT 5

ATTACHMENT J

Thacker Pass Project
Engineering Design Report
Clay Tailings Filter Stack, Waste Rock Storage Facilities, Coarse Gangue Stockpile, Mine Facilities & Process Plant Stormwater Management

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No. 21993

NewFields Project No. 475.0385.000
April 2, 2020

TABLE OF CONTENTS

LIST OF TABLES

LIST OF CLAY TAILINGS FILTER STACK & PROCESS PLANT STORMWATER DRAWINGS

TABLE OF CONTENTS
(Continued)

LIST OF CLAY TAILINGS FILTER STACK & PROCESS PLANT STORMWATER DRAWINGS (Cont'd)

TABLE OF CONTENTS
(Continued)

LIST OF CLAY TAILINGS FILTER STACK & PROCESS PLANT STORMWATER DRAWINGS (Cont'd)

LIST OF WASTE ROCK STORAGE FACILITIES AND COARSE GANGUE STOCKPILE DRAWINGS

LIST OF MINE SURFACE WATER CONTROL FEATURES DRAWINGS

TABLE OF CONTENTS
(Continued)

LIST OF MINE SURFACE WATER CONTROL FEATURES DRAWINGS (Cont'd)

LIST OF APPENDICES

TABLE OF CONTENTS
(Continued)

LIST OF APPENDICES (Cont'd)

1 INTRODUCTION

NewFields Mining Design & Technical Services (NewFields) was commissioned by Lithium Nevada Corporation (LNC) to design the Clay Tailings Filter Stack (CTFS), East and West Waste Rock Storage Facilities (WRSF), Coarse Gangue Stockpile (CGS), Process Plant Ponds, and associated infrastructure in support of the application for a Water Pollution Control Permit (WPCP). Information presented in this report was prepared in accordance with the requirements outlined in the State of Nevada Regulations Governing the Design, Construction, Operation and Closure of Mining Operations, Nevada Administrative Code (NAC) 445A.

The Thacker Pass Project is located approximately 20 miles west-northwest of Orovada, 62 miles north-northwest of Winnemucca, between the Kings River Valley to the west, the Quinn River Valley to the east, the Montana Mountains to the north, and the Double H Mountains to the south in an area known as Thacker Pass. The elevation in the Project area ranges from approximately 4,200 to 5,650 feet above mean sea level (amsl). A location map is provided in on the cover of the **Drawings**.

1.1 Scope of Work

The general objectives of the project included the following elements:

- Establish design criteria (Appendix A) for use as a basis to complete the Project design;
- Complete geotechnical field investigations and laboratory testing to assess subsurface conditions and to develop input parameters for geotechnical evaluations of mine waste materials (Appendices B & C);
- Provide geotechnical evaluations for the Plant Site, CTFS, CGS and West and East WRSF (Appendix D);
- Complete a site wide hydrologic analysis;
- Complete hydraulic stormwater analysis and civil designs for the CTFS, Reclaim Pond and West and North Diversion Channels; CGS Sediment Pond; West and East WRSF sediment ponds, Process Plant entrance road culvert design; and review and stamp NA Coal's hydraulic and civil designs for the Mine Facilities sediment ponds, diversion channels and culverts. (Appendix E);
- Prepare Technical Specifications (**Appendix F**); and
- Generate drawings to support permitting and construction (**Drawings**).

1.2 Project Background

Subsurface exploration of the McDermitt Caldera and Thacker Pass Project area started in 1975. At this time, Chevron began a uranium exploration program in the volcanic rocks located throughout the McDermitt Caldera. The United States Geological Survey (USGS) notified Chevron on the presence of anomalous concentrations of lithium associated with the caldera. Chevron initiated a clay analysis program, which confirmed the presence of high lithium concentrations using airborne gamma ray spectrometry, although their exploration program continued to focus on uranium (Advisian, 2018).

Chevron drilled 234 holes in the 1970s and 1980s that broadly outlined the lithium deposit. Between 1980 and 1987, Chevron conducted a drilling program that focused on lithium targets and conducted extensive metallurgical testing to determine the viability of extracting lithium from the clays.

In 2007, Western Lithium USA Corporation (WLC) began an exploration drilling program focused on the southern portion of the caldera. WLC drilled 232 exploration holes over the course of four years in the Project area, which identified an anomalously high-grade lithium deposit. As part of a merger, WLC officially changed its name to Lithium Americas Corporation (LAC) in March of 2016 and ownership of the Project was placed in LAC's Nevada-based subsidiary, LNC.

LNC continued exploration drilling in 2017 and 2018, drilling an additional 142 holes. The WLC/LNC drilling exploration program drilled a total of 374 HQ (2.5") core holes for a total of 113,951 feet with a range of depths from 20 feet to 760 feet. The average depth of drilling is 302 feet. The HQ core was drilled with either a truck or track mounted core rig capable of 1,500 feet of depth. The drilling proved a viable resource that is available for mining and extraction.

1.3 Project Overview

LNC proposes to construct, operate, reclaim, and close an open pit lithium mining and processing operation, the Thacker Pass Project, located on public lands in northern Humboldt County, NV. Pending the required authorizations and permits, construction for Phase 1 will commence in 2021 and mine production will begin in late 2022. Construction and operation will consist of the open pit mine, West and East WRSF, CGS, CTFS, stockpile areas, roads, ponds, diversion channels, diversion berms, processing facilities, and mine support facilities. **Appendix A** lists the design criteria used to design the facilities.

It is expected that approximately 50 million dry tons of ore will be mined from the open pit. Once the pit is opened and established, as the pit is mined, it will be concurrently backfilled with waste rock. Initially, excavation will start on the western side of the overall pit extents. The West WRSF will be located southwest of the pit and will store 26.4 M CY of excavated mine waste rock material and the East WRSF will be located to the east of the pit and store 5.8 M CY. The CGS, located southeast of the East WRSF, will have a storage capacity of approximately 26.1 M CY. Three growth media stockpiles will store material salvaged from proposed disturbance. Two of these stockpiles will be located southeast of the pit, near the ROM ore stockpile. The third growth media stockpile will be located northeast of the East WRSF.

The Mine Facilities Area located south of the pit and north of SR293, will consist of a truck shop, warehouse, fuel bay, wash station and other ancillary buildings. Stormwater for the area will gravity flow to diversion channels and berms, which will direct flows into sediment ponds. Culverts will be used to convey flows under roads.

Located south of the pit and east of the Mine Facilities Area will be a run-of-mine (ROM) ore stockpile with a storage capacity of 0.5 M CY.

The ROM ore will be dozed into a conveyor trap and fed to an attrition scrubber which will separate the lithium-rich fine clay from the coarse low-grade material. The solids are mixed with water into slurry form and pumped to the Process Plant where the coarse-grained low-grade material (coarse gangue) is separated from the high grade fine-grained material that continues through the process plant. The coarse low-grade material will be stockpiled in the CGS. The Process Plant produces lithium carbonate and lithium hydroxide, which is sold on the market. The clay tailings, neutralization solids and various salts generated as a result of the processing will be sent to the CTFS.

The Sulfuric Acid Plant is located on the south end of the Process Plant and will generate sulfuric acid for use in the leaching process and will also generate steam for energy that will provide power to the project. Maintenance, laboratory, office, and other processing support facilities will be located in this general area as well.

The CTFS will be located east of the Process Plant and is designed to store 70 M CY of structural and non-structural tailings with the capability to expand. The design storage capacity is based on an initial tailings production rate of approximately 2.75 M dry tons per year for up to 25 years.

The base of the CTFS will have an 80-mil high-density polyethylene (HDPE) double-sided textured liner placed above a layer of liner bedding and overlain with collection pipes and a 2-foot thick layer of overliner. A perimeter road will be built around the facility. No solution will be applied to the CTFS; however, seepage that is squeezed from the clay tailings during the consolidation process and any precipitation that falls on the facility will be collected by an underdrain collection system. The underdrain system is designed to provide positive drainage toward a reclaim pond located south of the CTFS. Solution from the reclaim pond will be pumped to the Process Plant as make-up water or left to evaporate.

Upgradient stormwater will be directed around project facilities through diversion channels and into natural drainage ways. Runoff generated from disturbed areas such as the CGS, and East and West WRSF, Mine Facilities Area or Process Plant Area will be routed into sediment basins before release to existing drainage ways. The stormwater plan details and calculations are included in **Appendix E**.

1.4 Use of this Report

This report has been prepared exclusively for LNC. No third party, other than NewFields, shall be entitled to rely on any information, conclusions, opinions or other information contained herein without the express written consent of LNC.

Supporting data upon which our recommendations are based are presented in the following sections of this report. The recommendations presented herein are governed by NewFields' interpretation of the physical properties of the soils encountered in the field investigation, projected groundwater conditions, and the layout and design data generated and discussed in this report. If subsurface conditions other than those described in this report are encountered, or if project details are changed, NewFields should be informed so that our recommendations can be reviewed and amended, as necessary.

2 SITE CHARACTERIZATION

2.1 Climate

Precipitation data from various frequency storm events were obtained using the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 Point Precipitation Frequency Estimates at the mine location from the NOAA website. Average monthly evaporation values for the Rye Patch Dam weather station (approximately 85 miles south-southeast of the project Thacker Pass), were obtained from the Western Regional Climate Center (WRCC) website. Average monthly precipitation data was collected from LNC's site climate monitoring station and compared with the other nearby stations. Reported precipitation, and evaporation values, and storm events are summarized in the Design Criteria in **Appendix A**. **Table 2-1** presents values for the various storm events used in the design.

Table 2-1: Design Storm Events (24-hour duration)

Recurrence Interval (years)	Precipitation Depth (inches)
2	1.13
25	1.96
100	2.48
500	3.12
From NOAA Atlas 14: Latitude: 41.696° N and Longitude: 118.02° W; Elevation = 4,622.8 ft (2018)	

2.2 Local and Regional Geology

The Thacker Pass Project is located in north-central Nevada at the northern end of the Basin and Range tectonic province. Regional geology stretches from southern Oregon to Mexico and is characterized by a series of extension-related normal faults trending roughly north-south resulting in a repetitive series of mountain ranges separated by valleys. The project site is bounded to the north by the Montana Mountains; to the south by the Double H Mountains; to the west by the Kings River Valley; and to the east by the Quinn River Valley.

Local geology of the Thacker Pass Project is controlled by the McDermitt Volcanic Field, a volcanic complex containing four large calderas (or "super volcanoes") that formed in the middle Miocene. The McDermitt Volcanic Field is located within the southeastern-propagating swarm of volcanism from the Steens Mountain into north-central Nevada. The largest and southeastern most caldera of the McDermitt Volcanic Field, the McDermitt Caldera, hosts the ore body of the Thacker Pass Project. Prior to collapse of the McDermitt Caldera at 16.33 million years ago (Ma), volcanism in the northern portion of the McDermitt Volcanic Field and locally small volumes of lavas erupted near the present-day Oregon-Nevada border. These lavas and the flood basalts are exposed along walls of the McDermitt Caldera and are approximately 16.5 to 16.3 million years old (Advisian, 2018).

A large lake formed in the caldera basin following the eruptions in the McDermitt Volcanic Field. Associated caldera lake sediments that host the Thacker Pass deposit were deposited on top of the horsts and grabens formed during the faulting associated with the Tuff of Thacker Creek. The lake captured sediments that were eroded from the surrounding drainages.

Lacustrine claystone sediments which host lithium ore are found intimately interbedded with thin, repetitive water lain ash sequences. Ash layers are well sorted, medium to coarse sized lapilli grains deposited across wide extents, particularly in the Southwest Basin where thick sequences of basal ash beds were encountered across multiple exploration boreholes. Diagenesis at depth has silicified claystone beds in finely laminated, mudstone sequences. The ratio of ash to claystone in these lacustrine units is a continuum, with thick sequences of ash beds found more abundantly in basal lacustrine deposits in the Southwest Basin Area, and greater components of claystone found in the open pit footprint. The rhyolitic Tuff of Long Ridge is found underlying lacustrine sediments and is present in latite textures of felsic phenocrysts to a fine-grained groundmass. In some instances, the Tuff of Long Ridge was deposited as viscous lava, forming flows and pseudo bedding planes. These deposits are referred to as Rheomorphic Tuffs.

2.3 Subsurface Conditions

A preliminary site investigation was completed by AMEC in 2011. NewFields conducted a site investigation in February 2019 that included 31 boreholes and 29 test pits. Four of the boreholes were extended to depths between 100 and 150 feet below ground surface (bgs). The other twenty-seven were extended to depths of 30 to 50 feet bgs. Another site investigation was completed in December 2019 that included five additional boreholes to 100 feet depth and 21 additional test pits.

Subsurface conditions can generally be classified as a thin veneer of growth media, approximately 12 inches to 24 inches in thickness, overlying alluvium overburden consisting of loose to very dense fine to coarse sands and gravels with varying amounts of clay, silt, sand and gravel overlying residuum composed of slightly weathered to highly weathered basalt. In the open pit area, the alluvium directly overlies claystone with varying amounts of interbedded ash (AMEC, 2011). Throughout the site, thin seams and lenses of low plastic clay and silt were observed in select borings at relatively shallow depths. The thickness of alluvium overburden varies significantly across the site, with recorded thicknesses between 10 feet to over 65 feet.

There is no general trend of overburden thickness or bedrock elevation across the site, primarily due to the degree of weathering and the basalt depositional process.

The site generally slopes to the South-southeast at approximately 4 to 6 percent gradient with isolated slopes up to 15 to 20 percent gradient. Based upon the topography there is significant relief across the entire project; approximately 400 feet of elevation change across the pit area, approximately 160 feet of change across the CTFS, 400 feet of change across the East WRSF and CGS, and approximately 70 feet of elevation change across the Process Plant site. **Appendix D** contains the full geotechnical investigation results.

2.3.1 Groundwater

Prior to 2018, LNC had six active monitoring wells; these wells continue to be monitored and recorded on a quarterly basis. Additional wells were installed at strategic locations in August 2018 by Piteau Associates for groundwater monitoring. The results show the groundwater levels between 70 and 180 feet below ground surface (bgs) in the pit area.

- Groundwater generally flows from north to south-southwest, following the general topographic trend. Groundwater elevations have remained steady over the monitoring period, indicating that the groundwater conditions are at steady state.
- NewFields encountered groundwater in five deep borings during the two site investigations completed in 2019.
  - In BH19-02 groundwater was encountered at a depth of 93 feet bgs,
  - BH19-04 at 93 feet,
  - BH19-05 at 83 feet,
  - BH19-33 at 60 feet,
  - BH-34 at 97 feet,
  - BH19-35 at 90 feet.

The remaining boreholes did not encounter groundwater. Based on the geotechnical investigation, groundwater is not anticipated in the upper 50 feet bgs. In general, groundwater is not expected to influence construction and operation of the process plant, CTFS, East and West WRSF and CGS. Refer to the **Table 2-2** for borehole depth to groundwater findings by NewFields in 2019. All boreholes not included in the table did not encounter groundwater. **Appendix D** includes compete geotechnical investigation results.

Table 2-2: Depth to Water Encountered in the NewFields 2019 Geotechnical Investigations

Piezometer or Borehole	Depth to Groundwater (feet)	Ground Water Elevation (feet amsl)
BH19-02	93.2	4835.1
BH19-04	92.5	4632.9
BH19-05	83.5	4715.4
BH19-33	60.2	4623.8
BH19-34	97.5	4728.5
BH19-35	90.3	4626.7

2.3.2 Surface Water

The topographical arrangement and site terrain straddles the southern end of the Humboldt Range. Surface water drains into two hydrographic basins: Quinn River Basin to the east and Kings River to the west. The CTFS, CGS, East WRSF, Process Plant Facilities and the east half of the Mine Facilities area are located entirely within the Quinn River Basin, as shown on Figure 000 **in Appendix E.2**. The West WRSF and the western edge of the Mine Facilities Area is located within the Kings River Basin. The pit straddles the hydrographic basin boundary. The surface water at the Project Area is associated with outflow from ephemeral creeks and runoff from precipitation as a result of storm events and snowmelt. Most surface drainages at the site are ephemeral and flow seasonally or during storm events or sustained periods of heavy precipitation. A small section of Pole Creek crosses the Plan of Operation (POO) boundary at its northeast corner; it is not located near any planned mine facilities. On the western edge of the project site, a small section of Thacker Creek crosses the POO boundary. No area within 1,300 feet of the creek will be disturbed in the current mine plan. Several existing natural drainages enter the property boundary from the north and traverse the project site. These drainages and runoff from the site discharge to one of the two existing drainage ways located just north of southern plan of operations boundary, roughly parallel to SR 293. The drainage way east of the hydrographic boundary flows east into Crowley Creek. The drainage within the Kings River Basin flows west into Thacker Creek. Surface water delineation and subsequent consultation with the United States Army Corps of Engineers (USACE) has determined there are no Waters of the

United States (WOTUS) within or immediately adjacent to the Project Area (USACE, 2019). The current Jurisdictional Determination was approved on February 8, 2019 and is included in **Attachment I** of the WPCP. A summary of the stormwater design plan and its supporting calculations are located in **Appendix E**.

2.3.3 Stormwater Controls

Stormwater controls have been designed to route upgradient runoff around the proposed project infrastructure and to accommodate and contain on-site runoff from design storm events. The intent of the stormwater controls is to:

- Divert non-contact water (i.e. water that has not come in contact with disturbed ground or process solutions) around the project facilities and discharge to downstream watercourses.
- Convey sediment-laden runoff (i.e. water that comes off stripped surfaces and roadways) as necessary to sediment collection basins prior to discharging to downstream watercourses.
- Contain precipitation from a design storm event that has come in contact with process solution.

For all surface water controls, the hydrological modeling was performed using HEC-HMS, a precipitation-runoff simulation computer program developed by the USACE to calculate the magnitude and timing of the peak flows and volumes resulting from specific storm events. HEC-15 (FHWA, 2005) was then used to estimate channel flow depths and riprap sizing based on the cross-sectional geometry, minimum channel profile slope, and peak flows. The required channel depths and riprap sizing were determined for each channel segment longitudinal slope. In steep sections of the channels, a rock chute calculation was completed based on Natural Resources Conservation Service (NRCS, 1998) design procedures to determine the appropriate channel dimensions and riprap sizing.

2.4 Seismic Hazard

NewFields completed a seismic hazard assessment on July 18, 2019. The results are presented in **Appendix D**. Probabilistic ground motions associated with various risk levels were assessed using the USGS unified hazard tool. The hazard tool application is based on the 2014 USGS national seismic hazard maps and adjusts for the site soil class. The reported peak ground acceleration (PGA) for a 2 percent and 10 percent chance of exceedance in 50 years, which corresponds to a return period of 475 years and 2,475 years, are presented in **Table 2-3**. Deaggregation of the seismic hazard indicates that the mean event is a 6.6 moment magnitude at 14 miles from the site.

Table 2-3: Probabilistic Design Accelerations

Return Period	Reported PGA (g)
475-Year	.09
2,475-Year	.26

2.4.1 Site Classification

The results of the geotechnical subsurface investigation near the process facilities and CTFS (NewFields, 2019) determined that the upper 100 feet consist of 20 to 60 feet of very dense silty sand and gravel fan deposits overlying weathered basalt. The deepest boring near the proposed pit, which is west of the CTFS, was 50 feet and consisted of dense to very densely bedded ash and clay with average Standard Penetration Test (SPT) resistance (N value) of greater than 50 blows per foot. In accordance with the 2015 IBC and ASCE 7-16, the site classifies as very dense soil and soft rock, Site Class C.

The maximum considered earthquake response accelerations at short and long periods, S_S and S_1, respectively, were determined using an online calculator provided from the Structural Engineers Association of California (SEAC) (SEAC, 2019). All relevant seismic design values for structures are listed in **Table 2-4**.

Table 2-4: Code Based Seismic Parameters

Site Soil Class	C
Mapped MCE_R, five (5) percent damped, spectral response acceleration parameter at short periods (Site Class C), S_S	0.50g
Mapped MCE_R, five (5) percent damped, spectral response acceleration parameter at a period of one (1) second (Site Class B), S_1	0.18g
Design, five (5) percent damped, spectral response acceleration parameter at short periods, S_{DS}	0.43g
Design, five (5) percent damped, spectral response acceleration parameter at period of one (1) second, S_{D1}	0.18g

2.4.2 Recommended Design Ground Motions

Ground motions associated with design-level earthquakes were developed for the project site using both site specific code-based procedures and publicly available information from the United States Geological Survey (USGS). Based on all the available information, NewFields recommends the following:

- Earthen structures (such as the CTFS) should be designed considering a MCE PGA equal to 0.44g based on the most conservative results of the Deterministic Seismic Hazard Analysis (DSHA) in

Appendix D.4 and a OBE of 0.09g based on the 475-year return probabilistic event. This is in compliance with the NAC guidelines; and

- Design of structures should be completed using the code based spectral response parameters listed in **Table 2-4**.

2.4.3 Other Seismic Hazards

Potential seismic hazards for any site include ground rupture, slope instability, seismic induced settlement, and liquefaction or strain softening of subsurface deposits. Ground rupture is not expected to be a hazard for the project site or associated facilities since near-surface faulting and active faults are not documented within the project site. Liquefaction, which can occur within loose, saturated granular deposits, is not expected to be a hazard for the project site due to the depth to groundwater and the dense conditions in the near surface overburden. Similarly, potential seismic settlement from liquefaction of saturated, deep deposits is not expected based on our understanding of the subsurface conditions.

3 GEOTECHNICAL INVESTIGATION

As previously discussed, a Prefeasibility Study (PFS) level site investigation was completed for the development of the Thacker Pass Project (AMEC, 2011). The study included a site investigation and subsequent geotechnical recommendations including preliminary geotechnical recommendations for the open pit and foundation and earthwork recommendations for the various facilities associated with the project.

The general location of the process plant and individual structures orientated on the various process plant pads have been altered since the PFS was completed. In February and December 2019, NewFields completed geotechnical investigations to assess the geotechnical conditions in the subsurface near the West and East WRSF, Mine Facilities, CGS, Process Plant and CTFS.

In general, sufficient data is available to suitably characterize the subsurface beneath the majority of the site, but additional data may be necessary to confirm conditions beneath the sulfuric acid plant.

The locations of the NewFields 2019 borings and test pits associated with the recent investigation, as well the previous investigation, are shown on **Drawing A050**.

3.1 2019 Field Investigations

A site investigation was completed by AMEC in 2011 based on an initial project site layout (AMEC, 2011). The project elements subsequently changed and as a result, NewFields completed an additional site investigation between February and April 2019, which included 31 boreholes and 29 test pits. A supplementary site investigation was completed within the footprint of the CTFS in December 2019. This program consisted of five boreholes extended to depths of 100 feet and

21 test pits. The boreholes were advanced using a CME-850 track-mounted drill rig, and each borehole was drilled with 4.25-inch diameter hollow stem auger in soil and diamond bit rock coring methods when in bedrock. Eight boreholes were extended to depths of 100 to 150 feet below ground surface (bgs), with the remaining twenty-seven boreholes extended to depths of 30 to 50 feet bgs. Test pits were excavated with a CAT 320E excavator to depths of 7 to 19 feet bgs. NewFields logged the lithologies and characteristics of subsurface materials based on recovery from the driven samples, soil cuttings brought to the surface on the auger flights and excavator buckets and recovered rock core. All boreholes were abandoned in accordance with the Nevada Administrative Code (NAC) 534 for Underground Water and Wells. The geotechnical borings which did not encounter groundwater were abandoned by backfilling the holes with bentonite from the terminal depth to within 20 feet of the ground surface. Boreholes were then backfilled with neat cement grout to the ground surface. Water was encountered in five boreholes, which were subsequently backfilled from terminal depth to the ground surface with a neat cement grout, in accordance with the NAC 534 regulations.

The borehole and test pit logs summarize the results of material classifications and observations made at each borehole or test pit location. These records include drilling or excavation depth, description of each strata encountered, strata delineation, estimates of strata density, and location of samples retained for laboratory analysis. The logs represent our interpretation of the contents of the field logs and the results of the laboratory tests on select field samples. Borehole and test pit as-installed locations and logs are presented in **Appendices B.1** and **B.2**.

Drawing A050 shows the location of the geotechnical investigation completed at the site. The results of NewFields Geotechnical Investigation were presented in a factual report (October 2019) and presented in **Appendix B.3.**

3.2 Laboratory Testing Program

Soil and rock samples obtained during the field investigation were labeled, packaged and transported to the NewFields laboratory in Elko, Nevada where the majority of the soil testing was completed. Bulk samples tested for corrosivity potential were sent to Sunland Analytical Laboratory. Samples obtained from the field investigation were tested for index properties, natural moisture and unit weight, specific gravity of soil solids, moisture content/unit weight relationships, and corrosivity potential. Individual laboratory data sheets are presented in **Appendix C** and summarized in **Table C-1**. The results of the NewFields Laboratory Testing were presented in a factual report (October, 2019) and presented in **Appendix B.3**

Soil classification involved particle size analyses and Atterberg limits which were used to divide soils into groups such that the engineering properties of the soils within each group are similar. Each sample was categorized according to the Unified Soil Classification System (USCS), which is based on the material gradation and plasticity.

3.2.1 Index Properties

The index properties of soils were evaluated by particle size analyses and Atterberg limits tests. Results indicate that the materials encountered were predominantly composed of fine to coarse grained silty sand with varying amounts of gravel particles.

Atterberg limits results indicate the plasticity index (PI) ranges from non-plastic to high plasticity with the majority of fine-grained materials exhibiting non-plastic behavior. Based on the measured gravimetric water content, the majority of the plastic materials are at or below the plastic limit. The samples yielded an average moisture content of 13.5 percent as measured on a dry weight-basis (i.e. geotechnical definition). The apparent specific gravity of soil solids was measured as 2.54.

3.2.2 Moisture Content – Unit Weight Relationship

The relationships between unit weight (density) and moisture content was established for a bulk sample using Proctor compaction test procedure. The modified Proctor test (ASTM D1557) was performed on a bulk test pit sample to determine the maximum dry unit weight and the corresponding optimum moisture content. The sample yielded a maximum dry unit weight of 78.3 pcf and an optimum moisture content of 34.0 percent.

3.2.3 Corrosivity Potential

Laboratory soil resistivity, pH, and water-soluble sulfates and chlorides tests were conducted on soils obtained from select areas to assess their corrosivity potential, and results are presented in **Table 3-1**.

Table 3-1: Results of Corrosivity Testing Potential

Sample	Depth (ft)	Material Type	pH	Resistivity (ohm–cm)	Sulfates (ppm)	Chlorides (ppm)
BH19-12	2.5-6.5	Silty SAND (SM)	7.65	150	691.9	1246.9
BH19-13	7.5-10.5	Silty SAND (SM)	7.88	780	45.5	103.2
BH19-26	10-11.5	SAND (SW-SM) with gravel and silt	7.85	750	295.2	97.2

The average pH of the native soil was approximately 7.8, which is considered mildly alkaline. The measured resistivity ranged from 150 to 750 ohm-cm, which indicates the soil has a high corrosion potential for steel (American Petroleum Institute, 1991). The average measured chlorides ranged from 97 to 1247 parts per million (ppm), which indicates the soil is mildly corrosive to corrosive to steel. The measured water-soluble sulfates in the soil ranged from 46 to 690ppm, which indicates negligible sulfate exposure for concrete (American Concrete Institute, 1994).

3.3 Clay Tailings Assessment

Samples of leached solids (LFilterCake), neutralization solids (NFilterCake), and sulfate salts (Salt) were provided by LNC and transported to the NewFields AMRL/AASHTO accredited laboratory in Elko, Nevada where the material testing was conducted. Select laboratory tests were performed on individual components (LFilterCake, NFilterCake, and Salt) along with testing performed on composite filtercake samples both with and without salt. The composite filtercake samples are identified as the "tailings" that will be stored in a geomembrane lined facility at the project site. The results of NewFields Tailings Assessment were presented in a Technical Memorandum, TM-07 (December 2019) and are presented in **Appendix C.6.**

The tailings with salt samples were reconstituted at a ratio of 64.1 percent LFilterCake, 17.3 percent NFilterCake, and 18.6 percent Salt, as measured by dry weight. The salts were hydrated with 11.1 percent tap water prior to reconstitution with the tailings. The tailings without salt samples were reconstituted at a ratio of 78.7 percent LFilterCake and 21.3 percent NFilterCake, as measured by dry weight.

It should be noted that all moisture contents presented in this memorandum were completed as per ASTM D2216 and are reported on a dry basis (Weight of water/Weight of dry solids) as this is the common reporting practice for geotechnical reporting.

Index testing included moisture content and Atterberg limits testing, which were used to assess the relationship between as-received moisture and the materials plasticity. Moisture content – unit weight relationships were developed from bulk samples of tailings, both with and without salt. Strength properties of tailings are estimated based upon Unconsolidated Undrained (UU) and Consolidated Undrained (CU) triaxial testing. This laboratory testing program included:

- Atterberg Limits (ASTM D4318)
- Natural Moisture Content (ASTM D2216)
- Modified Proctor Moisture – Unit Weight Relationship (ASTM D1557)
- Unconsolidated Undrained Triaxial Compression (ASTM D2850)
- Consolidated Undrained Triaxial Compression (ASTM D4767)

Individual laboratory testing results for the clay tailings are summarized in **Tables 3-2**, **3-3** and **3-4.** Individual laboratory data sheets are presented in **Appendix C.7.**

3.3.1 Clay Tailings Index Property Testing

The index properties of the materials were evaluated by particle size analysis, moisture content and Atterberg limits testing. The Atterberg limits test was used to measure the moisture content of the upper and lower limits of the range in which the soil is in the plastic state and are only performed on the soil fraction passing the No. 40 sieve (0.42 mm). The moisture content at the upper limit is known as the liquid limit (LL) and the moisture content at the lower limit is designated as the plastic limit (PL). The numerical difference between the LL and the PL, termed the plasticity index (PI), is a measure of the soil plasticity. Generally, soils that exhibit a PI between 5 and 10 are low plasticity, between 10 and 20 correlate to medium plasticity and between 20 and 40 correlate to high plasticity. Particle size analysis and Atterberg limits results indicate that the materials classify as an elastic silt (MH) with varying amounts of fine sand and medium plasticity.

Samples of the individual components were preserved at their as-received moisture content by double sealing bulk samples in airtight plastic bags and storing in sealed buckets. Gravimetric moisture contents for all samples tested ranged between 55 and 75-percent. Most materials had a moisture content above their LL, with the exception of the tailings material without salt.

Table 3-2: RESULTS OF LABORATORY INDEX TESTING

Material	Liquid Limit	Plastic Limit	Plasticity Index	As-Received Moisture Content
LFilterCake	53	40	13	55.7
NFilterCake	64	47	18	68.5
Salt	-	-	-	74.1
Tailings w/Salt	51	40	11	60.9
Tailings w/out Salt	71	59	12	59.3

3.3.2 Clay Tailings Laboratory Compaction Testing

Two moisture-unit weight relationship tests using the modified Proctor method (ASTM D1557) were completed on bulk samples of tailings, one without salt and one with salt. The samples yielded maximum dry unit weights ranging from 70 to 72 pounds per cubic foot (pcf) and optimum moisture contents (OMCs) ranging from 45 to 46 percent. In general, the sample with salt yielded a higher dry unit weight and lower moisture content.

Table 3-3: Results of Laboratory Compaction Testing

Material	Laboratory Compaction	
	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)
Tailings w/out Salt	70.1	46.0

Tailings with Salt	72.4	45.3

3.3.3 Clay Tailings Shear Strength

The shear strength of remolded tailings samples were measured by triaxial compression testing under isotropic Unconsolidated Undrained (UU) and Consolidated Undrained (CU) conditions. A bulk sample of tailings without salt was air dried to the OMC and six individual specimens were selected. Three of the tailings specimens were mixed with the salt and three were kept without salt. A second bulk sample of tailings was air dried to three percent over OMC and two tailings specimens were reconstituted with salt. All eight of these tailings specimens were then remolded at 95 percent of the maximum dry unit weight into 2.8-inch diameter by 5.6-inch tall test specimens.

The UU samples were confined at 25 pounds per square inch (psi) during testing while the CU samples were backpressure saturated and consolidated at 25 and 50 psi, respectively. Mohr-Coulomb strength parameters were developed from the test measurements as shown in **Table 3 - 4**. Consolidated, drained parameters (effective stress) were calculated by subtracting the measured internal pore pressure from the chamber and axial applied stresses.

Table 3-4: Shear Strength Properties

Material	Dry Unit Weight (pcf)	Moisture Content (%)	CU Triaxial Testing				UU Triaxial Testing
			Effective Stress		Total Stress		Undrained Shear Strength (psf)
			Friction Angle (degrees)	Cohesion (psf)	Friction Angle (degrees)	Cohesion (psf)	
Tailings	66.6	45	40	65	19	400	6300
Tailings + Salt	68.8	45	40	180	20	390	700
Tailings+ Salt	66.6	54	42	0	22	0	-

3.4 Coarse Gangue Stockpile Assessment

A sample of Coarse Gangue Stockpile (CGS) material was provided by LNC and transported to the NewFields AMRL/AASHTO accredited laboratory in Elko, Nevada for testing. Select laboratory testing was performed on this sample to obtain engineering parameters for the CGS stability analysis presented in **Appendix D.3**. The laboratory testing program for this sample included:

- Grain Size Distribution (ASTM D422)
- Atterberg Limits (ASTM D4318)
- Direct Shear (ASTM D3080)

Laboratory testing results and individual data sheets for the CGS material are located in **Appendix C.8**.

4 GROWTH MEDIA STRIPPING AND STOCKPILES

Growth media consisting of soils and alluvium will be salvaged from the footprint of proposed disturbances during initial construction and throughput the Project area as mining progresses. Where present, growth media will be stripped and stockpiled for use in future reclamation and closure activities. Growth media will be stockpiled in three major stockpiles.

Growth media stockpiles will be constructed with slopes no steeper than 3H:1V. LNC will implement measures to mitigate erosion, weathering, and leaching of salts and nutrients during storage of growth media. LNC will seed the growth media stockpiles if necessary to create a temporary vegetative cover, which binds the soil and limits loss due to wind and water erosion of the stockpile to avoid increased sediment concentration in surface runoff. All stockpile areas will be constructed with erosion control measures to reduce sediment from leaving the stockpile site.

5 CLAY TAILINGS FILTER STACK (CTFS)

Lithium processing will produce tailings comprised of acid leach filter cake (clay material), neutralization filter cake, magnesium sulfate salt and sodium/potassium sulfate salts, collectively referred to as clay tailings. LNC proposes to place the clay tailings in the CTFS, which will be a permanent lined storage facility, located east of the process plant.

LNC will convey tailings material from the tailings filter press located in the filtration and neutralization building at the process plant to the CTFS. Clay tailings and the collective salt wastes will be transported via two separate conveyors from the process plant and will form two distinct stockpiles within the CTFS footprint. Material from these temporary stockpiles will be placed on the CTFS in conformance with the stacking plan. The conveyor system will be able to transport an average of 500 dry tons per hour during initial production with a potential to increase if required.

Approximately 70 M CY of clay tailings will be placed on the CTFS. The CTFS will be constructed with a structural zone in the exterior of the facility and a non-structural zone in the interior. An interior non-structural zone will be constructed with layers of tailings and salts compacted at 85% of Modified Maximum Dry Density (MMDD) as determined by ASTM D1557. Surrounding this core, a structural zone will be constructed at 95% compaction of MMDD and with 4H:1V sideslopes. The structural zone will be stacked against the nonstructural zone at a slope of 1H:1V with a 3 feet thick chimney drain between them that extends from the overliner layer to the surface. The chimney drain will consist primarily of sands and gravels to provide a hydraulic break between the two zones to dissipate potential pore pressure. The CTFS will be fully lined with an

HDPE geomembrane and underlain with a six-inch liner bedding material. The facility will include an underdrain seepage collection system between the geomembrane and the clay tailings, which will allow seepage water and stormwater to drain to the reclaim pond.

5.1 Mass Grading

The CTFS mass grading will generally follow the native ground that slopes from northwest to southeast at slopes ranging between three and six percent with an average slope of around 3.5 percent. Most of the fill will be placed in the natural drainages that traverse the site and for the perimeter roads around the facility. Most of the cut is in the channels, the Reclaim Pond and along ridgelines across the facility. Drawing A070 shows isopach shading indicating the cut and fill thicknesses of the CTFS and the north and west diversion channels.

The perimeter access road around the CTFS is 32 feet wide with 2.5H:1V slopes. The perimeter haul road is 128 feet wide with 2.5H:1V slopes. The throughway is 80 feet wide with six-foot-high safety berms along fill areas. The road is a minimum five feet high as measured from the inside toe and crest and is sloped inwards towards the pad at two percent as shown on Drawing A107.

Cell divider berms located in the interior of the CTFS are there to separate the facility into stages and maintain containment of stormwater runoff solution in the event of a significant storm event. The cell divider berms are made of common fill approximately 4.5 feet high and rounded for an approximate total width of 34 feet wide. They extend from the north perimeter berm to the south channel with the exception of the divider berm between Cell 1 and Cell 2 which at a natural ridgeline that can be used as a divider berm. Drawings A215 and A216 show the cell divider berm sections.

After the major cut and fills within the facility are completed, the upper 6-inches of the exposed fill or cut surface shall be scarified, moisture conditioned and compacted to provide a liner bedding for the geomembrane. If the existing ground is rocky and does not consist of fine-grained materials smaller than 3/4-inch then material will have to borrowed and placed over the rocky surface.

5.2 Liner System

The CTFS has been designed with a single liner system consisting of 80-mil HDPE double-sided textured geomembrane layer over top of the liner bedding layer.

5.2.1 Liner Bedding

The liner bedding will consist primarily of insitu or borrowed fined grained materials moisture conditioned and compacted to form a smooth firm surface for which to place the geomembrane. Laboratory testing will be performed on liner bedding samples prior to (Control tests) and during construction (Record tests). In-situ moisture/density tests will be completed to assure conformance to the specifications as outlined in **Appendix F**.

The HDPE geomembrane will be textured on both sides to increase the frictional resistance between the liner bedding and overliner materials that will be in contact with the geomembrane. Geomembrane materials will be subjected to testing at the plant site by the manufacturer as well as conformance testing performed by a third-party laboratory to ensure quality. During installation, the liner will be subjected to a QA/QC testing and inspection program as outlined in the Technical Specifications included in **Appendix F**.

A QA/QC program will be implemented during installation to ensure that the geomembrane is installed according to the manufacturer's recommendations, to monitor the integrity of the seams; and to ensure that the minimum thickness of the overlying cover materials (overliner) is maintained.

5.3 Underdrain System

Precipitation that falls upon the CTFS will seep through the tailings and be collected by the underdrain system and directed to the Reclaim Pond. A seepage calculation was completed which showed a maximum seepage rate of up to 74 gpm could flow to the Reclaim Pond at ultimate facility buildout as a result of tailings consolidation. The seepage calculation is presented in **Appendix E**. The underdrain system consists of a network of collection pipes placed on top of the geomembrane and a layer of overliner material placed over the pipes. No solution will be applied to the CTFS other than for periodic surface dust suppression; therefore, the only fluid collected by the underdrain system will be stormwater, natural infiltration or pore water squeezed out of tailings due to long-term consolidation of the tailings material. The facility is divided into six cells of similar size for permanent placement of tailings plus one cell in the southwest corner used for temporary clay tailings and salt stockpiles but the underdrain systems for all the cells are connected.

5.3.1 Solution Collection Piping System

The solution collection piping system will consist of four-inch diameter dual wall smooth interior perforated corrugated polyethylene (CPe) (ADS N-12 equivalent) secondary collection pipes located on the geomembrane and spaced 200 feet apart in a herringbone pattern. The secondary collection pipes drain to a 12-inch diameter dual wall smooth interior perforated (CPe) (ADS N-12 equivalent) collection header pipe situated in the topographic low points of each cell in the CTFS as shown on Drawing A210. The collection header pipes connect into a 12-inch diameter dual wall smooth interior perforated CPe pipe that runs along the channel on the south side of the CTFS. At the CTFS solution outlet channel the 12-inch dual walled perforated CPe pipe connects to a solid 12-inch diameter HDPE DR 17 underdrain outlet pipe which will convey flow into a Parshall Flume for measuring the seepage flow rate and then into the CTFS Reclaim Pond as shown on Drawing A222.

5.3.2 CTFS Stormwater Overflow Pipes

Seven 36-inch diameter HDPE DR 17 stormwater overflow pipes are located at the CTFS Solution Outlet Channel approximately 24 inches above the invert of the 12-inch diameter underdrain outlet pipe. These pipes are designed to convey the CTFS stormwater runoff during a 100-year/24-hour storm event under the haul road and into the CTFS Solution Outlet Channel, which then drains into the Reclaim Pond.

5.3.3 Overliner

The Overliner layer consists of a 24-inch thick drainage medium consisting of minus 1.5-inch sand and gravel mixture that covers the surface of the HLP as shown on Drawings A215 and A216. This single layer will provide protection to the geomembrane during tailings placement and will have a high transmissivity to promote lateral drainage of seepage and stormwater runoff from the CTFS. Overliner will initially be processed from native soils on site and later the coarse gangue stockpile can be used as an overliner source as the material will consist of washed sands.

5.3.4 Chimney Drain

A chimney drain was designed to separate the tailings in the structural zone from the non-structural zone. The non-structural zone will consist primarily of salts and also contain tailings with elevated moisture contents due to weather or upset conditions in the process plant. The chimney drain will consist primarily of sand and serve as a hydraulic break to relieve potential pore pressure built up between the two zones. Piezometers will be installed on either side of the chimney drain to monitor pore pressures in each zone as shown on Drawings A400 and A410.

5.4 Reclaim Pond

Stormwater collected by the underdrain system will be directed to the Reclaim Pond. The pond is a double geomembrane lined pond with an operating capacity of 9.2 million gallons, can store a 100-year, 24-hour storm event runoff volume of 17.8 million gallons and has 3.6 million gallons of storage available in the top 3 feet of freeboard. The total pond volume to the crest is 30.6 million gallons.

The reclaim pond will be double lined with a 60-mil HDPE double-sided textured geomembrane liner on bottom overlain by a 200-mil thick layer of geonet and an 80-mil HDPE double-sided textured geomembrane liner above the geonet. Water collected in the pond will not be discharged as part of the stormwater management. The water will be pumped to the processing plant to be used as make-up water for processing operations or will evaporate. The pond will be equipped with a leak detection and recovery system consisting of a collection sump between the two liners and a riser pipe laid along one of the slopes, providing access for monitoring and recovering any leakage through the primary liner.

5.4.1 Leak Collection and Recovery System

Stormwater collected by the underdrain system will be directed to the Reclaim Pond. The Reclaim Pond has a sump located in the southeast corner. The sump is a total of five feet deep with the lower 2 feet of the sump being the leak collection and recovery sump (LCRS) and the upper 3 feet serving as the pond surface water sump. The LCRS is between the primary and secondary geomembranes and the pond surface water sump is located above the primary geomembrane. The LCRS has bottom dimensions of ten feet by ten feet with 2.5H:1V slopes and has select gravel wrapped in geotextile on top of the secondary geomembrane. A 12-inch diameter HDPE DR 21 pipe with slots cut into the lower ten feet is positioned into the LCRS, which serves as a pump sleeve. A submersible pump will be positioned inside of the pump sleeve and connected to a discharge pipe that will pump leakage from between the layers to the crest of the pond and back onto the primary geomembrane. Drawing A230 shows a section of the LCRS sump.

5.4.2 Pond Pump Back System

The upper three feet of the five-foot-deep CTFS Reclaim Pond sump is a recessed sump above the primary geomembrane that allows the sloping pumpback system to evacuate water out of the pond. The sloping pumpback system is a submersible pump attached to an 8-inch diameter HDPE pipe sleeved inside of an 18-inch diameter HDPE pipe, which serves as a pump sleeve. The pump will pump water out of the pond through the 8-inch pipe back to the Leaching Tanks in the Process Plant if required. The pumpback pipe was designed to pump out 500 gallons per minute. The pump will be designed by the Process Plant designer based on operational preferences. The 18-inch diameter HDPE pipe sleeve will be held down by sheet of 80-mil HDPE liner ballasted with two concrete filled six-inch diameter HDPE pipes. At the crest of the pond the 18-inch diameter HDPE pipe transitions to a flanged stainless-steel pipe that is braced and welded to a steel plate embedded into a six feet wide by six feet long by three feet deep reinforced concrete anchor block. Drawings A235 shows a section of the pond sump and sloping pumpback system.

5.5 Perimeter Haul Road

A perimeter road will be constructed around the CTFS, providing access to and containment of the facility. Along the northern and eastern perimeters of the CTFS, the road will have a crest elevation at least five feet above the top of the geomembrane and a crest width of 32 feet. On the western side of the CTFS, including the area adjacent to the temporary tailings stockpile area, the perimeter road will have a typical crest width of 131 feet (80-ft driving width) and a crest elevation at least eight feet above the top of geomembrane. Along the southern CTFS perimeter, a solution collection channel of varying depth will run along the inside perimeter of the haul road. The CTFS perimeter road in this section will have a crest elevation nine feet above the bottom-of-channel (top of geomembrane) and crest width of 131 feet.

All sections of the CTFS perimeter haul road will be constructed with 2.5H:1V side slopes and safety berms on both sides of the roadway crest. The HDPE double-sided textured geomembrane will be extended from the CTFS base grading, up the inside embankment of the road, and will be anchored into the roadway crest on the embankment-side of the CTFS-side safety berm. Wearing course will consist of an 18-inch thick layer on the haul road surface and a six-inch thick layer on the access road surface.

5.6 CTFS Diversion Channels

Two diversion channels were designed to the north and west of the CTFS to divert stormwater runoff of undisturbed areas around the facility. The stormwater diversions are designed with a maximum 2.5H:1V cut and fill slopes and can convey stormwater runoff from a 500-year/24-hour storm event. The diversion channel varies from 30 feet to 60 feet in width and 2.5H:1V slopes as defined on Drawing A311. There are three culvert crossings designed for the CTFS West Diversion Channel. One is for haul truck traffic entering into the CTFS and the other two are for conveyor crossings from the Process Plant for the clay tailings stockpile and the salt stockpile. A layout of the conveyor crossings is shown on Drawing A305. The channel armoring requirements are shown on Drawing A310.

The CTFS North Diversion Channel is approximately 80 feet wide with 2.5H:1V slopes and diverts water around to the east side of the CTFS. The riprap requirements for the channel are shown on Drawing A313.

The hydrology calculations for each stormwater diversion are provided in **Appendix E**.

6 COARSE GANGUE STOCKPILE (CGS)

Coarse gangue is produced in the classification stage of the mineral processing and is conveyed into the CGS after going through a dewatering process. LNC will convey the coarse gangue material to the CGS located east of the open pit. The gangue material will include lithium content whose economic value cannot be extracted at this time with a rate of return meeting LNC's criteria, using the proposed technique.

The CGS will be placed above existing ground that has been stripped of one foot of growth media. The stripped growth media will be placed in the growth media stockpile. The coarse gangue material will be placed directly onto prepared subgrade. The stockpile is expected to accommodate approximately 26M CY of material during the first 10 years of operation, and with the ability to expand. The design basis, calculations, design drawings and specifications for the coarse gangue stockpile are included in **Appendix A.**

The CGS is currently conservatively designed per a stacking plan provided by LNC with 50 ft lift heights and 75.5 ft benches graded between each lift to provide an overall stacking slope of 5.5H:1V and intermediate lift slopes of 4H:1V as shown on Drawing C135. Additional stability analysis completed by NewFields show that the coarse gangue sand stockpile can be stacked to 3H:1V slopes and still meet the minimum stability factor of safety if the sands are adequately dewatered during the classification process. Additional strength testing of the coarse gangue material will be conducted during operations and side slope requirements may change in the future. Geochemistry and transport analysis completed by SRK and Piteau shows that the CGS does not require a liner at the foundation.

6.1 CGS Sediment Pond

Stormwater runoff from the CGS will drain to the low point on the south side of the facility and through two 54-inch diameter corrugated metal pipes (CM) into the CGS Sediment Pond as shown on drawing C135.

The CGS Sediment Pond is designed to contain runoff from a 2-year/24-hour storm event and slowly drain over a period of three days through the perforations in the 42-inch diameter HDPE DR 17 riser pipe. Runoff from storm events up to 25-year/24-hour can flow out the top opening of the riser pipe, which has steel mesh over it to keep potential debris from getting lodged inside. The sediment pond is designed to store two feet of sediment and have three feet of freeboard above the spillway invert. Storm events greater than 25-year/24-hour and up to 100-year storm events will drain out of the overflow spillway into the CTFS West Diversion Channel. The peak flow from a 100-year/24-hour storm event can pass through the spillway with one foot of freeboard to the crest of the pond. Drawing C140 shows the layout of the CGS Sediment Pond. Sediment will be removed from the facility once the sediment design capacity has been reached.

Riprap is installed at the inlet and outlets of the sediment pond. The riprap size and thickness is shown on Drawing C140. A layer of non-woven geotextile will be installed beneath all riprap.

7 WASTE ROCK STORAGE FACILITIES (WRSF)

Waste rock material generated from the open pit mining operation will be placed in two proposed WRSFs, located west and east of the pit as shown on Drawing C010. LNC plans to haul waste rock to either WRSF based on operational requirements such as capacity and haul cycle efficiency. The Thacker Pass waste rock and ore have a low potential for acid generation, according to the results of the static testing (NPR greater than 3 for all material types) and confirmed with the kinetic testing program. For this reason, the WRSFs were designed as unlined facilities. The design criteria for the WRSFs is included in **Appendix A** and the stability analysis evaluation is included in Appendix D. Calculations for the sediment ponds are included in **Appendix E**.

7.1 West WRSF

The West WRSF area is 160 acres and is designed with a storage capacity of 26.4M CY at 3.5H:1V slopes. The maximum thickness is 275 feet and the existing topography at the base slopes from north to southwest.

Waste rock will be placed in the West WRSF during of the initial stages of mine operation and will shift later to the East WRSF once it has reached maximum capacity. The WRSF facility will be placed above existing ground that has been stripped of one foot of growth media. The stripped growth media will be placed in the growth media stockpile.

7.1.1 West WRSF Sediment Pond

Stormwater runoff from the West WRSF will drain to the low point in the southwest corner of the facility into the West WRSF Sediment Pond as shown on drawing C105.

The north half of the West WRSF Sediment Pond is in cut and the south half is in fill. The south embankment is 15ft wide with upstream and downstream slopes at 2.5H:1V. The cut slopes on the north side of the pond are at 2.5H:1V. The pond is designed to contain runoff from a 100-year/24-hour storm event to reduce the potential for stormwater runoff in disturbed areas from flowing into Thacker Creek, which is located approximately 2,000 feet to the southwest. The sediment pond is designed to store two feet of sediment and have three feet of freeboard above the spillway invert. Storm events greater than the 100-year/24-hour storm events will drain out of the overflow spillway onto natural ground. The peak flow from a 500-year/24-hour storm event can pass through the spillway with one foot of freeboard to the crest of the pond. Drawing C110 shows the layout of the West WRSF Sediment Pond.

Water draining into the pond will be left to evaporate or infiltrate into the ground. A submersible pump can also be used to pump out water within the pond as well. Sediment will be removed from the facility once the sediment design capacity has been reached.

Riprap is installed at the inlet and outlet of the sediment pond. The riprap size and thickness are shown on Drawing C110. A layer of non-woven geotextile will be installed beneath all riprap.

7.2 East WRSF

The East WRSF area is 137 acres and is designed with a storage capacity of 5.8 M CY at 3.5H:1V slopes with potential to expand. The maximum design thickness is 75 feet and the existing topography at the base slopes from northwest to southeast.

Based on the current mining plan the waste rock from the pit will be placed in the East WRSF during the later stages of mine operation after the West WRSF has reached maximum capacity. The WRSF facility will be placed above existing ground that has been stripped of one foot of topsoil. The stripped topsoil will be placed in the growth media stockpile.

7.2.1 East WRSF Sediment Pond

Stormwater runoff from the East WRSF will drain to the low point in the south corner of the facility into the East WRSF Sediment Pond as shown on drawing C120.

The northern portion of the East WRSF Sediment Pond is in cut and the southern portion is in fill, which ties into the crest of the haul road. The south embankment is a minimum of 15ft wide with upstream and downstream slopes at 2.5H:1V. The cut slopes on the north side of the pond are at 2.5H:1V.

The East WRSF Sediment Pond is designed to contain runoff from a 2-year/24-hour storm event and slowly drain over a period of three days through the perforations in the 36-inch diameter HDPE DR 11 riser pipe. Runoff from storm events up to 25-year/24-hour can flow out the top opening of the riser pipe, which has steel mesh over it to keep potential debris from getting lodged inside. The sediment pond is designed to store two feet of sediment and have three feet of freeboard above the spillway invert. Flows greater than those generated by 25-year/24-hr storm event will drain out of the overflow spillway, across the haul road and into a natural drainage. The peak flow from a 100-year/24-hour storm event can pass through the spillway with one foot of freeboard to the crest of the pond. Drawing C125 shows the layout of the East WRSF Sediment Pond. Sediment will be removed from the facility once the sediment design capacity has been reached.

Riprap is installed at the inlet and outlets of the sediment pond. The riprap size and thickness are shown on Drawing C125. A layer of non-woven geotextile will be installed beneath all riprap.

8 MINE FACILITIES

The mine facilities that are being designed by others will be located southeast of the mine pit as shown in the **Drawings** and will be accessed via the mine facilities access road from SR 293. The main mine facilities area consists of a parking lot, shop/office building, fuel island, wash bay, tire pad and storage area, substation, and ready line.

The ROM stockpile and two of the three growth media stockpiles will be located within the mine facilities area as well, east of the main mine facilities. LNC will haul ore recovered from open pit operation to the ROM stockpile located south of the pit. LNC proposes to construct and operate mineral processing facilities in the attrition scrubbing and classification areas to separate the lithium-rich, fine clay material from the coarse gangue.

8.1 Mining Roads

Mine access and haul roads have been designed to provide access to from the pit to the Mine Facilities Area. The access roads are designed for small equipment traffic and haul roads are designed for large haul truck traffic and other support equipment. Numerous culverts have been designed around the mine facilities area as shown on Drawing 002. The culvert summary shown on Drawing CULV01 provides the drainage area, flow rate, culvert length, elevations, slope, diameter, and number of culverts in the Mine Facilities area. The hydrology calculations for each culvert are provided in Appendix E.

8.2 Mine Stormwater Diversions

Stormwater diversion channels and berms have been designed to direct stormwater runoff from undisturbed areas around the Mine Facilities and to direct stormwater runoff from disturbed areas into the sediment ponds. The stormwater diversions are designed with 3H:1V cut and fill slopes and can convey stormwater runoff from a 100-year/24-hour storm event. A layout of the diversion channels and berms in the Mine Facilities area is shown on Drawing 002. The diversion channel characteristics such as peak flow rate, dimensions, slope, velocity, riprap size and thickness are included in their respective drawings in the Mine Surface Water Control Features drawing set. The hydrology calculations for each stormwater diversion are provided in **Appendix E**.

8.3 Mine Facilities Sediment Ponds

Three sediment ponds were designed for the mine facilities area: Facility Sediment Pond #1, Facility Sediment Pond #2 and Mine Sediment Pond #1. The cut and fill slopes for each facility are 3H:1V for the pond area and the spillway side slopes are at 4H:1V. The southern portion of the facilities are in fill and the northern portions are in cut. The embankment crest widths are 15 feet wide. All of the sediment ponds are designed to contain runoff from a 2-year/24-hour storm event and slowly drain over a period of three days through the perforations in the 36-inch diameter HDPE riser pipe. Runoff from storm events up to 25-year/24-hour can flow out the top opening of the riser pipe, which has steel mesh over it to keep potential debris from getting lodged inside. Flows greater than those generated by a 25-year/24-hr storm event will drain out of the overflow spillway into the natural drainages. The peak flow from a 100-year/24-hour storm event can pass through the spillway with at least one foot of freeboard to the crest of the pond. Drawing 002 shows the location of each of the sediment ponds. Sediment will be removed from the pond basins once the sediment design capacity has been reached.

8.3.1 Facility Sediment Pond #1

Stormwater runoff from the west end of the mine shop/office facility area and the area directly south of the pit will drain into Facility Sediment Pond #1. A diversion channel to the north of the pond and a diversion berm to the east of the pond direct stormwater runoff into the pond. The sediment pond was designed to store three feet of sediment and has three feet of freeboard above the spillway invert. The peak flow from a 100-year/24-hour storm event can pass through the spillway with a minimum of one foot of freeboard to the crest of the pond. Riprap is installed at the inlets and riser pipe outlet of the sediment pond. The pond water elevations, volumes, riprap size and thickness are shown on Drawing FP1-2. A layer of non-woven geotextile will be installed beneath all riprap.

8.3.2 Facility Sediment Pond #2

Stormwater runoff from the eastern portion of the mine shop/ office facility area, ROM Stockpile and Scrubber Pad will drain into Facility Sediment Pond #2. A diversion channel to the west of the pond and a diversion berm to the north of the pond will direct stormwater runoff into the pond. The sediment pond was designed to store three feet of sediment and has 2.5 feet of freeboard above the spillway invert. The peak flow from a 100-year/24-hour storm event can pass through the spillway with a minimum of one foot of freeboard to the crest of the pond. The pond water elevations, volumes, riprap size and thickness are shown on Drawing FP2-2. A layer of non-woven geotextile will be installed beneath all riprap.

8.3.3 Mine Sediment Pond #1

Stormwater runoff from the northeastern portion of the Pit and newly constructed haul roads will drain into the Mine Sediment Pond #1. The sediment pond is located in a natural drainage and was designed to store four feet of sediment and has 3.5 feet of freeboard above the spillway invert. The peak flow from a 100-year/24-hour storm event can pass through the spillway with a minimum of one foot of freeboard to the crest of the pond. Riprap is installed at the inlets and outlets of the sediment pond. The pond water elevations, volumes, riprap size and thickness are shown on Drawing MP1-3. A layer of non-woven geotextile will be installed beneath all riprap.

9 PROCESS PLANT

The Process Plant is designed by others and will be located south of the CGS and west of the CTFS as shown on Drawing A010 and will be accessed via two separate roads from SR 293. One entrance will be for reagent delivery trucks and the other entrance will be for all others. The Process Plant will process lithium rich fine clay and produce clay tailings and sulfate salts, which will be conveyed to the temporary stockpile area at the CTFS. The lithium carbonate and lithium hydroxide will be sold as concentrate.

9.1 Process Plant Entrance Roads

The Process Plant entrance roads are separated to keep reagent truck traffic separated from all other traffic at the process plant. While the roads have separate entrances at SR 293, they come together when crossing the main natural drainage south of the Process Plant as shown on Drawing A010. Culverts were designed along the entrance road to pass the runoff from a 25-year/24-hour storm event. For larger storm events, water will flow over the road.

There are three areas where culverts were designed along the entrance road. The largest is just north of SR293 where the road crosses the main drainage south of the site. Seven 60-inch diameter culverts are required to convey water under the road. The downstream slope of the road has riprap for erosion protection for larger storm events where water will flow over the road. A layout of the drainage crossing is presented on Drawing A324.

Hydrology and Hydraulic calculations for the culverts and the watershed map is provided in **Appendix E.**

9.2 Process Plant Sediment Pond

The Process Plant Sediment Pond is located in a natural drainage southeast of the Process Plant. The sediment pond is designed to contain runoff from a 2-year/24-hour storm event and slowly drain over a period of three days through the perforations in the 24-inch diameter HDPE riser pipe. Runoff from storm events up to 25-year/24-hour can flow out the top opening of the riser pipe, which has steel mesh over it to keep potential debris from getting lodged inside. Storm events greater than 25-year/24-hour frequency will drain out of the overflow spillway into the natural drainages. The peak flow from a 100-year/24-hour storm event can pass through the spillway with a minimum of one foot of freeboard to the crest of the pond. Drawing A300 shows the layout of the sediment pond. Sediment will be removed from the pond basin once the design capacity has been reached.

The sediment pond was designed to store two feet of sediment and has two feet of freeboard above the spillway invert. Riprap is installed at the inlets and riser pipe outlet of the sediment pond. The pond water elevations, volumes, riprap size and thickness are shown on Drawing A300. A layer of non-woven geotextile will be installed beneath all riprap.

10 GEOTECHNICAL EVALUATION

This section summarizes our geotechnical recommendations based on the proposed construction and subsurface conditions encountered beneath the CTFS, CGS, WRSF, and Process Plant. Design parameters and a discussion of geotechnical considerations related to construction of the various components of these facilities are included herein.

At this time, information regarding the Process Plant building types, foundation types and structural loads are not available. All recommendations provided herein are preliminary and will be revised when further information becomes available.

10.1 Plant Foundation Recommendations

The results of NewFields Process Plant Site Soil and Foundation Report were presented in a summary report (November 2019) that is included in **Appendix D.1.**

10.2 CTFS, CGS, and WRSF Stability Assessments

The results of the stability analyses for the CTFS, CGS, and WRSF are presented in the following subsections along with descriptions of the material properties and seismic parameters used in the stability models. The results of NewFields Stability Assessments were presented in a Technical Memorandum, TM-08 (January 2020) and TM-09 (February 2020) and are included in **Appendices D.2 and D.3.**

Stability analyses were performed using the computer program SLIDE v8 by RocScience. SLIDE is a two-dimensional slope stability program for evaluating circular or noncircular failure surfaces in soil or rock slopes using limit equilibrium methods. The Spencer's method, which is appropriate for all slope geometries and soil profiles, was utilized within the stability model and assumes all interslice forces are parallel and have the same inclination. The factor of safety can be defined generally as the resisting forces along a potential failure plane divided by the gravitational and dynamic driving forces. Both static and seismic conditions were analyzed.

In July 2019, NewFields completed a deterministic seismic hazard analysis of the Thacker Pass site, which is presented in **Appendix D.4**. The analysis involved review of regional geology and using the unified hazard tool software program from USGS to determine site classification and peak ground acceleration. The corresponding PGA for the 475-year (OBE) and 2,475-year (MDE) events are 0.09g and 0.26g, respectively. Based on these seismic hazard parameters, and the Hynes-Griffin and Franklin analytical method, a reduced pseudostatic seismic coefficient of 0.13g (one-half of the PGA) is valid and was used to evaluate for post closure pseudostatic conditions.

To assess the stability of slopes during seismic loadings, a pseudostatic approach was used where the potential slide mass is subjected to an additional, destabilizing horizontal force which represents the effect of earthquake motions and is directly related to the PGA. Very simply, the seismic force is the weight of the slide mass multiplied by a horizontal pseudostatic earthquake coefficient (k_H). Since the earthquake motion is not a constant, horizontal destabilizing force, using the full PGA for k_H has been shown to be overly conservative. Hynes-Griffin and Franklin (1984) discussed the concept that using one-half of the PGA for the horizontal pseudostatic earthquake coefficient more closely simulates actual earthquake loading, and with the resulting minimum factor of safety being equal to at least 1.0, slope deformations will be within tolerable limits. Thus, a seismic coefficient equal to one-half the PGA, or 0.13g, was adopted for the pseudostatic stability analyses.

The CTFS, CGS and WRSFs have each been evaluated as an engineered structure and designed as a waste rock storage facility. Minimum acceptable factors of safety for static and pseudostatic conditions were established as 1.3 and 1.05, respectively. The results of the stability analyses are presented in **Appendices D.2** and **D.3.**

10.2.1 Material Properties

Design parameters utilized in the stability evaluations for the CTFS and CGS were conservatively selected based upon laboratory index and strength test data in conjunction with observations from the field investigation and historical experience with similar materials. Design parameters utilized for the stability evaluations for the WRSFs were conservatively selected based upon previous reporting and experience with similar materials. The claystone material is reported by AMEC to have an International Society for Rock Mechanics (ISRM) hardness of S6/R0 and a Rock Quality Designation (RQD) ranging from 0 to 91. This implies that once excavated the material may exhibit engineering behavior similar to a stiff soil rather than a competent or intact rock. The AMEC report also states that the claystone appears to weather and breakdown into a high plastic soil upon exposure to the elements. The engineering parameters for the facility foundations were developed from laboratory index and strength test data in conjunction with observations from the field investigation, previous reporting by others, and historical experience with similar materials.

Material properties used in the stability analysis were based on available laboratory test data and experience with similar materials. Based upon triaxial laboratory testing results, the cohesion within the tailings materials is very sensitive to relatively small changes in moisture contents. For this reason, any effects that cohesion may have on strength have been assumed to be negligible for this stability analysis. It is recommended that long term monitoring and testing be performed to ensure that these assumptions are correct. The material properties used in the stability analyses are summarized in the following paragraphs and in **Table 9-1.**

Table 10-1: Properties Used in the Stability Analyses

Material	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Alluvium – Foundation	110	32	0
Drainage Layer	110	35	0
Liner Interface	110	16	0
Non-Structural Tailings	90	16	0
Structural Tailings	100	20	0
Coarse Gangue Material	110	31	0
Waste Rock (Claystone – Clay Soil)	100	18	200

10.2.2 Model Development

Both static and pseudostatic loadings were evaluated for a critical cross section through the ultimate CTFS, CGS and WRSF configurations. This critical location was selected based upon existing topography, proposed grading of the facility foundations (if required) and proposed grading of the facility slopes. The locations of the critical cross sections are presented in **Appendices D.2** and **D.3**. During pseudostatic analysis the tailings material parameters are reduced to account for strain softening during potential deformation.

10.2.3 Stability Analysis Results

Results of the slope stability analyses for the cross sections under consideration are summarized in **Table 10-2** and figures presented in **Appendices D.2** and **D.3**. These figures detail the critical cross section and the failure planes with the lowest factors of safety. Based on this evaluation, the CTFS, CGS and WRSF's will remain stable for static loading conditions. The initial slopes and benches for the East and West WRSF and CGS were provided by North American Mining who is completing the mining plan for LNC. Stability analysis was completed on these facility configurations and it was determined that the slopes used in the design of the East WRSF and CGS may be overly conservative. Since the initial analysis was completed, NewFields completed additional analysis, which shows the East WRSF could be constructed at a steeper slope of 3.5H:1V. Preliminary results also show that 3H:1V slopes are achievable for the coarse gangue stockpile while meeting the minimum factor of safety. Additional samples should be collected during initial operations to determine if steepening the slopes is possible so the facility footprint can be reduced.

Table 10-2: CGS and WRSF Summary of Stability Analysis

Location	Static FoS	Pseudostatic OBE FoS	Pseudostatic MDE FoS
CTFS – Overall Stability	1.3	-	0.7
CGS – Overall Stability (5.5H:1V)	3.6	2.3	2.0
CGS – Inter-Bench Stability (4H:1V)	2.6	1.9	1.7
CGS – Overall Stability (3H:1V)	1.9	1.5	1.3
East WRSF – Overall Stability (5.5H:1V)	2.9	1.7	1.4
East WRSF – Inter-Bench Stability (4H:1V)	2.2	1.5	1.3
West WRSF – Overall Stability (3.5H:1V)	1.3	0.9	0.8

*The current design thickness of the East WRSF is only 75 feet but to allow stable slopes for potential future expansion, the facility will be constructed to the same slope as the West WRSF given the materials will be the same.

Pseudostatic loading conditions indicated that the factor of safety could be less than 1.05 for the CTFS and West WRSF under both the OBE and MDE events and thus a deformation analysis was completed to estimate potential slope movements as presented in **Section 10.2.4**.

10.2.4 Seismic Slope Deformation Analysis

Since the pseudostatic stability evaluation for the CTFS and West WRSF resulted in calculated minimum factors of safety less than 1.05 for the OBE and MDE event, potential seismic deformations of the facility slopes were evaluated using a simplified method. Bray and Travasarou developed a semi-empirical relationship for estimating the magnitude and probability of permanent slope displacements that utilizes a non-linear, fully coupled stick-slip sliding block model to estimate dynamic performance of soil slopes. The response spectrum and moment magnitude of the design earthquake were based on data obtained from the USGS.

Results of the CTFS deformation analysis indicate that for the MDE event, potential slope displacements between 17 to 32 inches could be expected. This estimate is for movement along the entire slope length for the maximum height of 400 feet. It is our professional opinion that these slope movements are acceptable and any potential slope deformation from the MDE seismic event would not result in an excursion of the tailings outside containment.

Results of the WRSF deformation analysis indicate that for the OBE and MDE events potential slope movements between five and 50 inches could be expected. This estimate is for movement along the entire slope length for the maximum thickness of 275 feet. This amount of displacement may cause minor surficial sloughing but will not impact the overall integrity of the facility. Since the West WRSF potential slope movements were acceptable, the same slope was designed for the East WRSF.

11 HAUL AND ACCESS ROADS

LNC will primarily use haul trucks in the Project area for the following activities:

- Movement of ore to the ROM stockpile
- Movement of waste rock material to the WRSFs (during the first years of operation)
- Movement of tailings and salt from the temporary stockpiles to the CTFS.

The haul road maximum gradient will be less than ten percent with an 80-foot road throughway width. Roads will be sloped away from the centerline with the exception of the CTFS perimeter haul roads and have a wearing course thickness of 18 inches. Haul roads in the mine area will be constructed according to MSHA standards. Secondary access roads will be approximately 32 feet in width with a minimum 1.5 percent grade from the centerline and have a wearing course thickness of six inches.

Facility entrance roads off SR293 will be classified as private roads. All site roads will allow for emergency vehicle access minimum requirements. The Process Plant road layout is designed to support the anticipated site traffic for construction, operations and maintenance requirements of the facility. The design considers anticipated vehicle traffic, equipment turning requirements and clearances and ensures access requirements are met.

LNC will construct ditches on the side of roads to capture road runoff as needed. Runoff from haul and secondary roads will be collected and routed to stormwater sediment ponds as needed. Dust control measures used for road grading will include watering before and after grading activities and reduction of equipment speeds during operations, if necessary. Chemical treatment may be used for additional dust suppression.

12 CLOSURE

The Tentative Plan for Permanent Closure (TPPC) is included as **Attachment Q** in the WPCP submittal. The temporary closure plan is included as **Attachment P** of the WPCP submittal; however, no temporary or seasonal closures of the mine are planned during its operation. The following provides a summary of the closure activities described in the TPPC.

Closure and major reclamation activities will occur in the first two years following cessation of mining. Monitoring and maintenance will continue for five years post closure until the final bond release. Post-production reclamation activities will include recontouring, cover placement, placement of growth media, seeding activities, and removal of infrastructure and fluid management.

Throughout the Project's operational phase, concurrent reclamation will occur in areas where final configurations are complete. LNC will begin reclamation activities at the earliest practicable time within areas of the Project that are considered inactive, without potential, or completed. Early initiation of reclamation will stabilize soil, reduce dust, and naturalize runoff.

Earthwork reclamation will ensure that potential visual impacts resulting from development of the proposed Project are minimized. Regraded stockpile slopes will be covered by a layer of growth media. Cover over the clay tailings will include a compacted clay layer overlain with cover soil. This cover soil will consist of coarse gangue or benign pit waste rock with growth media. Growth media will be salvaged from the growth media stockpiles. The proposed reclamation seed mix and application rates are included in **Attachment Q** of the WPCP. The seed mix is designed to provide species that can exist in the environment of northwestern Nevada. The Noxious and Invasive Weed Management Plan, provided in the Plan of Operations, outlines the strategies for proactively preventing noxious and invasive weeds.

In accordance with NAC 445A, the permanent stormwater diversions that will remain during the post-closure period will be designed to handle the 500-year/24-hour design storm event at closure. Regraded slope angles, revegetation (e.g., growth media placement), and BMPs will limit erosion and reduce sediment in runoff. Silt fences, waddles, sediment traps, and other BMPs will help prevent migration of eroded material until reclaimed slopes and exposed surfaces have demonstrated erosional stability.

In general, facility reclamation practices will include decommissioning, demolition, waste removal, backfilling, regrading, placing growth media, and revegetating Project facility areas.

Reclamation efforts will occur on both an interim and, whenever possible, concurrent basis throughout the Project's operational phase. Specific reclamation activities for the main Project infrastructure are further described in **Attachment Q** of the WPCP.

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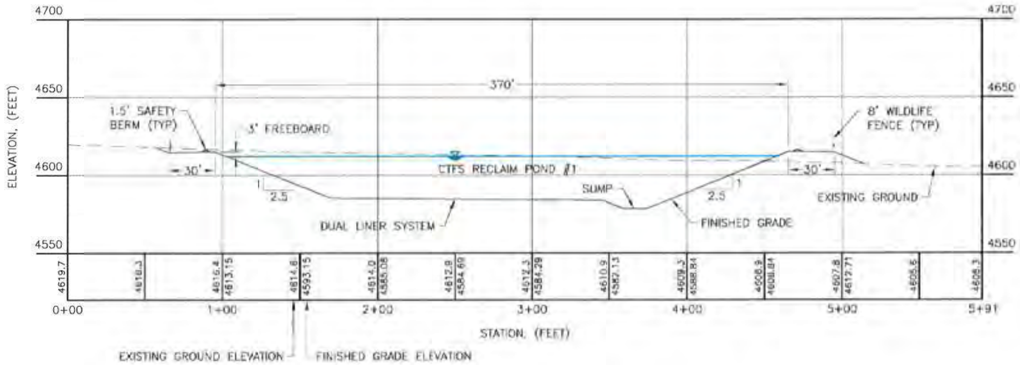

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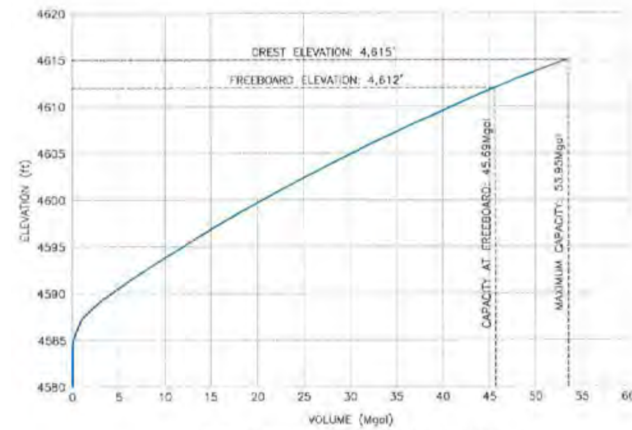
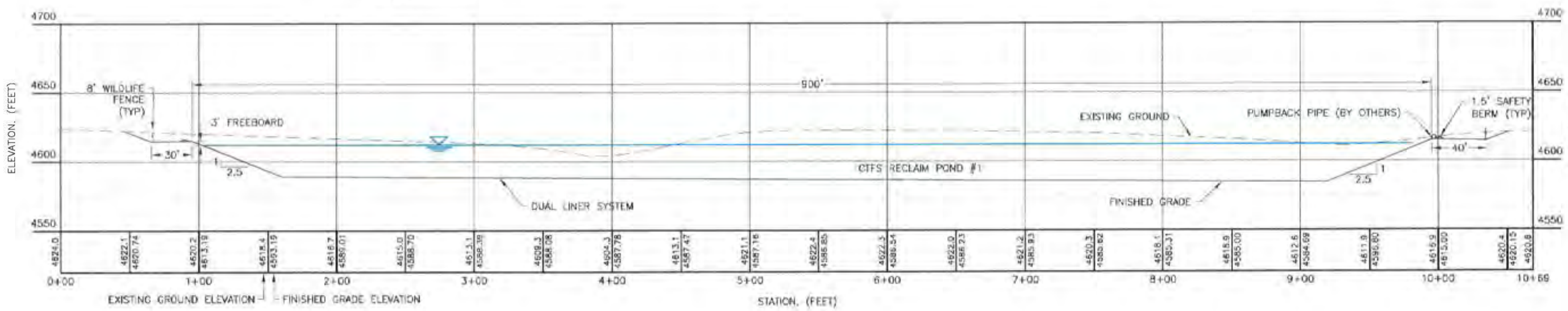
A220
CTFS RECLAIM POND CROSS SECTION (N-S)

0 40 80 FEET HORIZONTAL
0 40 80 FEET VERTICAL
NO EXAGGERATION



CTFS RECLAIM POND FILLING CURVE



U
A220
CTFS RECLAIM POND CROSS SECTION (W-E)

0 40 80 FEET HORIZONTAL
0 40 80 FEET VERTICAL
NO EXAGGERATION

PROFESSIONAL ENGINEER - STATE OF NEVADA
MATTHEW T. HALEY
Exp.06/30/21
CIVIL
No. 21993

REV	DATE	DESCRIPTION	TECH	ENG
0	4/[illegible]	ISSUED FOR CONSTRUCTION	RL	MTH

APPROVED BY: MTH
CHECKED BY: RTB
DESIGNED BY: MTH
DRAWN BY: RL

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CLIENT: LITHIUM NEVADA CORP.
PROJECT: THACKER PASS PROJECT
TITLE: CTFS RECLAIM POND SECTIONS AND DETAILS
FILENAME: 0385.000.077D
DRAWING NO.: A221
REVISION: 0

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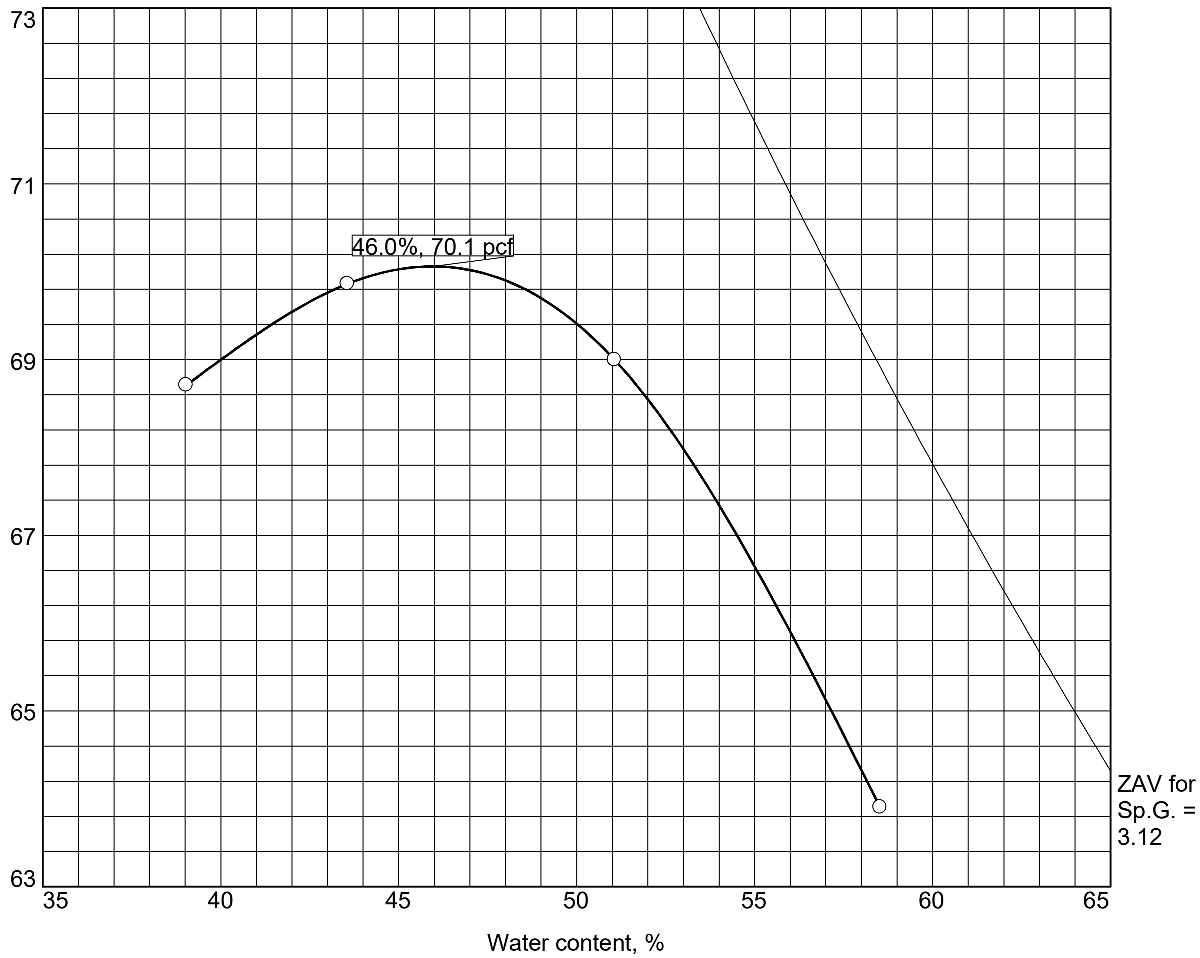
COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method B Modified

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
				3.119			0	

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 70.1 pcf Optimum moisture = 46.0 %	Blend without salt

Project No. 475.0385.000 **Client:** Lithium Nevada

Project: Thacker Pass

Date: 11/12/

○ **Location:** Tailings Blend w/o Salt **Sample Number:** 19-393-01

NewFields

Remarks:

*Measured Specific Gravity (19-057-01)

Figure 19-393-01

Tested By: JH **Checked By:** JH