BUREAU OF INDIAN AFFAIRS BANNOCK PUMPING PLANT MODERNIZATION PROJECT

GEOTECHNICAL ENGINEERING REPORT

Fort Hall Reservation
Power County, Idaho

for

HDR, Inc.
April 21, 2025

CIANI & HATCH ENGINEERING

Geotechnical Engineering Report, Revision 1

Bureau of Indian Affairs
Bannock Pumping Plant Modernization Project
Fort Hall Reservation, Power County, Idaho

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Bannock Pumping Plant Modernization Project

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ABBREVIATIONS AND ACRONYMS

ACI	American Concrete Institute
AL	Atterberg Limit
ASCE	American Society of Civil Engineers
ASTM	ASTM International
bgs	below ground surface
CHE	Ciani & Hatch Engineering, PLLC
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
HVAC	Heating, Ventilation, & Air-Conditioning
ISPWC	Idaho Standards for Public Works Construction
ksf	kips per square foot
MCE_R	Mapped Risk-Targeted Maximum Considered Earthquake
MDD	Maximum Dry Density
NFPA	National Fire Protection Association
PCCP	Prestressed Concrete Cylinder Pipe
pcf	pounds per cubic foot
pci	pounds per cubic inch
USGS	United States Geological Survey

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1.0 INTRODUCTION

This report presents geotechnical engineering conclusions and recommendations based on the results of Ciani & Hatch Engineering, PLLC's (CHE) field investigation and laboratory testing program for the Bannock Pumping Plant Modernization project (project) located on the Fort Hall Reservation in Power County, ID. The general project location is shown on the Project Vicinity Map (Figure 1); the site is shown relative to existing features and infrastructure on the Geotechnical Exploration Site Plans (Figures 2-1 and 2-2).

This report was previously issued on January 16, 2025. Following confirmation of foundation loading, additional foundation support options were incorporated into this report. This report supersedes the report issued on January 16, 2025.

1.1 PROJECT UNDERSTANDING

This report was prepared based on our discussions with, and information provided by, HDR, Inc. (HDR). At the time of this report, we have been provided the 30% design drawings dated September 5, 2024. CHE understands the proposed improvements to existing facilities at the pumping plant includes expansion of the below-grade, cast-in-place valve vault and construction of a new electrical building and associated transformer, generator, and propane tank in the adjacent switchyard. At the location of the siphon, improvements include the expansion of the siphon foundation to accommodate the installation of a new rotating screen, construction of a new concrete debris management slab, and installation of a new steel catwalk structure. We understand that the new debris management slab will include a 4-foot-tall pre-cast block wall. CHE understands slope cuts will be used for the proposed site excavations and shoring recommendations are not needed for the proposed improvements.

1.2 SCOPE OF SERVICES

The purpose of our design services was to explore the subsurface soil and groundwater conditions at the site as a basis for providing geotechnical design recommendations for the project. Our scope of services has been provided in accordance with our proposal for geotechnical engineering services dated May 7, 2024. The services in that proposal were incorporated into the subcontract between HDR and CHE executed July 1, 2024, and amended August 13, 2024.

2.0 EXISTING CONDITIONS

The following sections provide a description of the existing surface conditions, the geologic setting, and a summary of the subsurface soil and groundwater conditions observed during CHE's field investigation.

2.1 SURFACE CONDITIONS

The pumping plant site is relatively flat and vegetated with native grasses and shrubs and sits at the toe of an approximately 75-ft-tall slope immediately southwest of the pumping plant and switchyard. Topography in the immediate vicinity of the siphon is relatively flat with cultivated farmland to the northeast, southeast, and southwest. The land northwest of the siphon is predominantly undeveloped.

The pumping plant structure, located approximately 200 feet south of the intersection of the Bannock Drain and the Taghee Canal, was constructed in 1965. The age of any additions or renovations after the initial construction is unknown. This structure is constructed from conventional masonry units and houses heavy equipment including pumps and an overhead crane. Stair-step cracking is visible in the exterior of this structure, following the block path. The pumping plant structure connects under the gravel access road to the south to three 36-inch Prestressed Concrete Cylinder Pipe (PCCP) pipelines extending to the southwest

and one 42-inch PCCP extending to the south. The switchyard lies to the northwest of the pumping plant and connects to the pumping plant via underground electric powerlines. Overhead powerlines extend from the switchyard to the northwest.

The siphon structure, located approximately 1,600 feet to the northeast of the pumping plant, was constructed prior to 1985, based on our review of publicly available aerial imagery. A small power transformer connected to the overhead power lines and several below-grade tanks/structures lie to the south of the western side of the structure. A steel catwalk crossing over Bannock Creek connects the east and west sides of the siphon. The slopes to Bannock Creek were approximately 2H:1V (horizontal to vertical) and heavily vegetated. At the time of our exploration program and during our November 2024 site visit, the water surface in Bannock creek was approximately 9 feet below the top of the creek bank. Buried power extended from the substation through the approximate centerline of the access road to the north, across the catwalk, then south to the trash rack of the siphon. Overhead powerlines are present along the southern side of the site and continue to the east.

2.2 GEOLOGIC SETTING

Geologic information for the project area was obtained from the “Geology of the Michaud and Pocatello quadrangles, Bannock and Power Counties, Idaho” geologic map (Trimble, 1976). Surficial deposits mapped within the project areas consist predominantly of young alluvium and loess from the Holocene and Upper Pleistocene epochs.

Alluvium in this area is attributed to Bannock Creek and is predominantly comprised of silt and sand. Loess refers to windblown deposits, predominantly comprised of silt and sand mixtures with variable degrees of cementation. Loess is typically deposited loosely, resulting in the formation of pinhole structures, which can collapse when inundated/saturated with water and subjected to loading, leading to excessive settlement. Loess has been documented to be greater than 25 feet thick to the east of Bannock Creek and more than 200 feet thick to the west. Based on the proximity to Bannock Creek and our review of Idaho Department of Water Resources Well-Driller logs, we anticipate the loess soils extend to depths of 20 to 40 feet in the project area.

The soils encountered during CHE’s field investigation program were generally consistent with the mapped geology. In addition to loess and alluvium, we encountered variable thicknesses of fill throughout the project site.

2.3 SUBSURFACE CONDITIONS

Subsurface conditions were explored between August 7 and 8, 2024, by advancing eight test pit explorations (CHE-TP1-24 through CHE-TP8-24) to depths between 3 and 15 ft below ground surface (bgs). The approximate locations of the explorations are shown on Figures 2-1 and 2-2.

CHE personnel monitored the field explorations, collected representative soil samples, and maintained detailed logs of the subsurface soil and groundwater conditions. Summary test pit logs and a description of the geotechnical field investigation are provided in Appendix A.

An additional site visit was completed on November 18, 2024, after the irrigation had been shut off for the season. The purpose of our visit was to evaluate subgrade soil conditions for the proposed rotating screen and to obtain subgrade soil samples. Samples of subgrade material were obtained at the new debris management slab and in the base of the Taghee Canal.

Soil samples were transported to STRATA’s geotechnical laboratory in Boise, Idaho, for further examination and testing. Test results and a description of the geotechnical laboratory testing program are provided in Appendix B.

2.3.1 Soil Conditions

The soils observed underlying existing surface conditions (i.e., natural vegetation, gravel surfacing) were characterized into three general units:

Fill: Fill was encountered in every test pit except CHE-TP3-24 to depths up to 2 feet at the pumping plant and deeper at the siphon. Fill generally consisted of reworked native silt and contained gravel and cobbles as well as construction debris (such as timber or slag). Fill was typically light brown in color and ranged in consistency/density across the project site.

Loess: Loessal soils were encountered in all test pits except CHE-TP8-24. These soils consisted of soft to very stiff light brown silt. Pinhole structure was observed on the sidewalls of the test pit excavations and in the excavated soil samples. Cementation was observed in some layers, ranging from weak to moderate.

Alluvium: The alluvial deposits were encountered below approximately 5 feet bgs in Test Pits CHE-TP4-24, CHE-TP5-24, and CHE-TP7-24, all of which were adjacent to the eastern slope at the location of the siphon structure. These deposits were a brown to dark brown mixture of silt and clay with variable sand content. Organic debris, including rootlets and twigs, were encountered within this stratum. These materials were generally soft to stiff and moist.

3.0 CONCLUSIONS AND RECOMMENDATIONS

A summary of the primary geotechnical considerations is provided below. This summary is provided for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- Based on laboratory test results, the loess deposits encountered at the project site are prone to collapse-consolidation settlement when loaded and saturated. For an allowable bearing capacity of 2 ksf, we anticipated up to 0.5 inch of collapse settlement. To further limit collapse consolidation settlement, an allowable bearing pressure of 1 ksf should be used.
- New structures and equipment may be supported on shallow or mat foundations, depending on the design bearing pressure. Spread footings are not recommended for structures with a bearing pressure greater than 1 ksf due to the potential differential settlement resulting from collapse consolidation settlement of loess soils.
- Due to the presence of collapsible loess soils, extra care should be taken to divert surface water away from structures and equipment pads. Infiltration facilities, if planned, should be located away from structures and in coordination with the project geotechnical engineer.
- Groundwater was not encountered in our explorations. Based on nearby, historic well logs, we anticipate groundwater is over 20 feet below existing site grades.
- The Debris Management Slab was designed as a Portland Cement Concrete Pavement (PCCP) subject to highway loading, as requested by HDR, to determine the appropriate slab thickness and base material support. The results of our analysis show that 7 inches of reinforced PCCP supported on 6 inches of crushed surfacing base course is appropriate for the debris management slab.
- Chemistry Suite testing (pH, Resistivity, Sulfates) of samples from across the project site indicate a low concern for soil corrosivity.

Specific geotechnical engineering recommendations are presented in the following sections of this report.

3.1 EARTHQUAKE ENGINEERING

3.1.1 Seismicity

Southeast Idaho is a seismically active region with low to moderate seismic intensity and hazard. Earthquakes in this area are generally low in magnitude but can occasionally meet moderate levels. The seismic activity in the area is associated with the faulting in volcanic rock that occurred during the subsidence and formation of the Eastern Snake River Plain. Potential hazards from earthquakes include ground shaking, liquefaction, lateral spreading, and surface fault rupture. Our opinions regarding the likelihood of these seismic hazards occurring at the site are presented below.

3.1.2 Ground Shaking

There is a risk of earthquake-induced ground shaking at the site. The severity of ground shaking will mostly be a function of the earthquake's magnitude and proximity to the site. In our opinion, strong ground shaking should be considered in the design of the planned structures at the site. We recommend that the seismic ground shaking at the site be evaluated in accordance with the ASCE 7-22 design code.

3.1.3 ASCE 7-22 Seismic Design Information

We understand the project is being designed in accordance with the National Fire Protection Association (NFPA) 5000-2024 Building Construction and Safety Code, which references ASCE 7-22 for earthquake loading requirements. ASCE 7-22 requires site-specific data to a depth of 100 ft to determine seismic site class. Where soil properties are not known in sufficient detail, default site conditions should be used in accordance with Section 11.4.2.1. Spectral response accelerations shall be based on the most critical spectral response acceleration at each period of Site Class C, Site Class CD, and Site Class D. The ASCE 7-22 risk-targeted maximum considered earthquake (MCE_R) parameters for Site Class C, CD, and D are presented in Table 1.

Table 1. ASCE 7-22 Seismic Design Parameters

ASCE 7-22 Parameter	Recommended Value		
	Site Class C	Site Class CD	Site Class D
S_s (percent g) ***Short Period Spectral Response Acceleration***	0.42	0.42	0.42
S_1 (percent g) ***1-Second Period Spectral Response Acceleration***	0.13	0.13	0.13
PGA_M (percent g) ***Peak Ground Acceleration Adjusted for Site Effects***	0.19	0.22	0.23
T_L (second) ***Long-Period Transition Period***	6	6	6
S_{DS} (percent g) ***Short Design Earthquake Spectral Response Acceleration***	0.31	0.36	0.40
S_{D1} (percent g) ***1-Second Design Earthquake Spectral Response Acceleration***	0.12	0.17	0.23
S_{MS} (percent g) ***MCE_R, 5%-damped, spectral response acceleration parameter at short periods adjusted for site effects***	0.46	0.54	0.60
S_{M1} (percent g) ***MCE_R, 5%-damped, spectral response acceleration parameter at a period of 1-second adjusted for site effects***	0.18	0.26	0.35

3.1.4 Liquefaction Potential

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in development of excess pore pressures in saturated soils and subsequent loss of shear strength. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to structures. Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our field exploration and laboratory testing, we consider the risk of liquefaction at the project site to be low.

3.1.5 Surface Fault Rupture

The nearest mapped fault systems to the project site include the Wasatch fault zone, located 33 miles southeast, and the Idaho Rift System fault, located 32 miles northwest (USGS, 2024). Based on the fault distances from the site, the risk of fault displacement resulting in ground rupture at the site is considered low.

3.2 FOUNDATION SUPPORT

The project site is underlain by loess soils, which are susceptible to collapse consolidation settlement under foundation loading when wetted. Loess soils were encountered in the depths explored in the test pit explorations completed for this study. Loessal soils form when clay and silt-sized particles are loosely deposited by wind, resulting in the formation of small voids between particles called pinholes. These soils may be relatively stable under loading, but collapse upon saturation, resulting in potentially excessive

settlement. Based on the design bearing elevation of the structures at the site and the anticipated thickness of the loess to depths greater than 20 feet, removal and replacement of the unsuitable soil is not feasible.

Two collapse consolidation tests were completed on site soils. The tests indicate that foundation soils have a collapse potential of 0.05 to 0.5 percent under 1 ksf and 2 ksf foundation loads, respectively.

The wetting horizon is the downward movement of water through a soil zone. The wetted depth of the soil will depend on the duration of time the soil is exposed to water head. Zones of silt within the soil become wetted at different times and exhibit different collapse potential based on their deposition and mineralogy. CHE evaluated the hypothetical wetting horizon for our site and selected a depth of 9 ft for evaluation of collapse consolidation settlement (Li & Vanapalli, 2015). This wetted depth corresponds to an estimated collapse settlement of 0.1 to 0.5 inch for soil under 1 ksf and 2 ksf foundation loads, respectively.

The primary source of water at the project site is precipitation from rain and snow events. The site should be graded such that water flows away from structures and concrete mats to reduce potential for surface water infiltration and subsequent wetting of subgrade soils.

3.2.1 Shallow Foundations

We recommend new building and equipment pads with a design bearing pressure greater than 1 ksf be supported on mat foundations to evenly load foundation soils and reduce effects of differential settlement. Support of the new pedestrian bridge/catwalk structure and other structures with a design bearing pressure equal to 1 ksf or less can be provided with conventional shallow spread footings. We recommend the leading edge of the pedestrian bridge footings be set back a minimum of 4 feet from the top of the slope of the canal. We recommend minimum widths of 16 and 48 inches for continuous wall and isolated spread footings, respectively. The design frost depth for Power County is 32 inches; therefore, we recommend that all foundation elements be founded at least 32 inches below the lowest adjacent finished grade.

For new structures with shallow spread footings, conventional floor slabs-on-grade are appropriate when the structures are supported on proof-rolled native soils or on structural fill placed over stable native soils, provided the subgrade soils are prepared as described in Section 3.8. Based on the test pits completed at the site, we anticipate that stiff to very stiff loessal silt will be present at the floor slab subgrade elevation.

3.2.1.1 ALLOWABLE BEARING PRESSURE

For mat foundations supported on native soils or on structural fill extending down to these soils, an allowable soil bearing pressure of 2 ksf may be used where 0.5 inch of settlement is acceptable. For shallow spread footings supported on native soils or on structural fill extending down to these soils, or where settlement must be limited to less than 0.5 inch, an allowable soil bearing pressure of 1 ksf should be used. The zone of structural fill below the foundation should extend beyond the edge of the foundation a distance at least equal to the thickness of the structural fill. The zone of structural fill should be compacted to at least 95 percent of the maximum dry density (MDD) in general accordance with ASTM International (ASTM) D1557. The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

3.2.1.2 SUBGRADE PREPARATION

We recommend the subgrade soils be prepared and evaluated as described in Section 3.8 of this report. Structural foundations and slabs-on-grade should be constructed on a 6-inch-thick layer of clean crushed rock to provide uniform support and to act as a capillary break. The 6-inch crushed rock layer should be underlain with a woven geosynthetic that provides enhanced lateral drainage, such as Mirafi H_2R*i* or approved equivalent. The woven geosynthetic should be placed on approved subgrade or structural fill

extending to competent site soils. Where existing fill is exposed, the subgrade soils should be recompacted to a firm condition.

If water vapor migration through the mat foundation or slab-on-grade is objectionable and/or moisture cannot be tolerated by the proposed structures, the capillary break material should be covered with a heavy plastic sheet to act as a vapor retarder. The vapor barrier should have sufficient thickness to withstand foot traffic during placement of reinforcement and concrete and be constructed in accordance with the American Concrete Institute (ACI 302.1R). The contractor should be made responsible for maintaining the integrity of the vapor barrier during construction.

3.2.1.3 MODULUS OF SUBGRADE REACTION

For mat foundations or slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 55 pci may be used.

3.2.1.4 FOUNDATION SETTLEMENT

We estimate that footings founded as recommended above will experience between 0.5 and 1 inch of post-construction settlement, which includes elastic settlement and the potential for 0.1 to 0.5 inch of collapse-consolidation settlement of loessal soils. Differential settlement between comparably loaded column footings or along a 25-foot section of mat foundation or continuous wall footing should be less than 0.5 inch. We expect most of the post-construction foundation settlements will occur during construction as loads are applied. Loose or disturbed structural fill or native soils not removed from the foundation excavation prior to placing capillary break and concrete will result in additional settlement.

3.2.1.5 LATERAL RESISTANCE

Lateral foundation loads may be resisted by passive resistance on the sides of the foundation and by friction on the base of the shallow foundations. For shallow foundations supported on native soils, the allowable frictional resistance may be computed using a coefficient of friction of 0.35 applied to vertical dead-load forces.

Passive resistance may be estimated using an equivalent fluid density of 300 pounds per cubic foot (pcf) above the groundwater table. These values assume that the footings and below-grade elements are poured directly against structural fill placed and compacted as recommended.

The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

3.2.1.6 FOUNDATION DRAINAGE

We recommend that foundation drains be installed around the perimeter of the foundations for the proposed electrical building and for concrete pads supporting settlement-sensitive equipment. The perimeter footing drains should consist of at least 4-inch-diameter perforated pipe placed on a 2-inch bed of, and surrounded by, 6 inches of drainage material enclosed in a drainage geotextile that meets the requirements of Section 718.05 of the Idaho Transportation Department (ITD) Standard Specifications to prevent fine soil from migrating into the drainage material. Appropriate cleanouts for drainpipe maintenance should be installed.

We recommend that the drainpipe consist of either heavy-wall perforated pipe (JM Eagle Solvent Weld PVC, or equal) or rigid corrugated smooth interior polyethylene pipe (ADS N-12, or equal). Perforations for the pipe should consist of two rows of holes, 0.5 inch in diameter at 5 inches on center and 120-degree angle apart. We do not recommend using flexible tubing for foundation drainpipes. The drainage material should conform to Drain Rock in accordance with Table 1 of Section 802.2.2 of the 2020 Idaho Standards for Public Works Construction (ISPWC), but with the nominal maximum aggregate size limited to 1 inch.

The perimeter drains should be sloped to drain by gravity to a suitable discharge point away from structures (preferably a storm drain). We recommend that the cleanouts be covered and placed in flush-mounted utility boxes. Water collected in roof downspout lines for the new electrical building must not be routed to the footing drain lines.

3.2.1.7 CONSTRUCTION CONSIDERATIONS

Immediately prior to placing concrete, all debris and loose soils that have accumulated in the foundation excavations during forming and steel placement must be removed. Debris or loose soils not removed from the foundation excavations will result in increased settlement.

Due to the presence of moisture-sensitive loess soils, we recommend that work be completed during the dry season to limit subgrade soil exposure to moisture. If wet-weather construction cannot be avoided, we recommend that all footing subgrades be protected using concrete blankets or a lean concrete mud mat. Concrete blankets should have sufficient overlap to prevent water migration onto the subgrade soils. The mud mats or blankets should be placed the same day that the foundation subgrade is excavated and approved for foundation support.

We recommend that all completed foundation excavations be observed by a representative of our firm prior to placing mud mat/concrete blankets, reinforcing steel, and structural concrete. Our representative will confirm that the bearing surface has been prepared in a manner consistent with our recommendations and that the subsurface conditions are as expected.

3.3 DEBRIS MANAGEMENT SLAB

At the siphon structure, a debris management slab will be constructed to allow for trash and debris to be pulled from the Taghee Canal, dried, and then burned. This slab will be partially supported by a 4-foot-tall precast block wall in the northwest corner of the siphon improvement area, adjacent to Bannock Creek. Based on our discussions with the project team, we understand that the debris management slab is to be designed as a PCCP assuming highway loading. For design of pavement structures, traffic loading is converted into equivalent single axle load (ESAL). Design of the debris management slab was completed using 4.5 million rigid ESALs and a 40-year design life following methodology for rigid pavement design provided in the 1993 AASHTO Guide for Design of Pavement Structures.

Based on the test pits we completed throughout the debris management slab area (CHE-TP4-24 through CHE-TP7-24), we anticipate approximately 2 to 3.5 feet of undocumented fill comprised predominantly of reworked native soils with occasional cobbles and slag. Subgrade soil conditions were evaluated using the Idaho R-Value test to evaluate the strength and load-bearing capacity of subgrade materials. We evaluated the required thickness of the slab in accordance with the 1993 AASHTO Guide for Design of Pavement Structures rigid pavement design methodology and our experience. Based on our analysis, the debris management slab should be 7 inches thick. The PCCP should include reinforcement, which will be designed by the structural engineer. To prevent cracking or structural damage caused by temperature fluctuations, joints should be installed at intervals not exceeding 15 feet.

3.3.1 Subgrade Preparation

We recommend that concrete slabs-on-grade be constructed on a 6-inch-thick layer of clean crushed rock to provide uniform support and to act as a capillary break. The capillary break material should consist of crushed surfacing base course (CSBC) with a maximum particle size of 1 inch and negligible sand and silt. The 6-inch CSBC layer should be placed on a subgrade separation geotextile that meets the requirements of Section 718.07 of the ITD Standard Specifications. Prior to placing the geotextile, the exposed subgrade should be prepared as described in Section 3.8 of this report. Subgrade conditions should be observed and

proof-rolled during construction and prior to placing capillary break to evaluate the presence of unsuitable subgrade soils. If the subgrade soils are excessively loose or soft, it may be necessary to excavate localized areas and replace them with compacted structural fill. Where existing fill is exposed, the subgrade soils should be recompacted to a firm condition.

3.4 ROTATING SCREEN SLAB-ON-GRADE

At the time of our initial exploration program in August of 2024, the Taghee Canal was carrying water and inspection of subgrade soils was not feasible. We returned to the project site on November 18, 2024, to evaluate the condition of the canal subgrade. Based on our site visit, we anticipate that the channel bed subgrade is predominantly comprised of native fine-grained soils (silt and clay) which were saturated at the time of our visit. Subgrade conditions in the base of the canal were evaluated by probing with a 0.5-inch-diameter steel probe rod. Subgrade conditions were observed to be very soft. At the time of our site visit we were able to penetrate over 30 inches into the canal subgrade using little effort.

We understand that the new rotating screen will require extension of the existing slab-on-grade, adding approximately 22 feet to the slab. We recommend that concrete slabs-on-grade for the rotating screen be constructed on an 18-inch-thick layer of clean crushed rock to provide uniform support and added capacity to site soils. The 18-inch crushed rock layer should be placed on a subgrade separation geotextile that meets the requirements of Section 718.07 of the ITD Standard Specifications. Prior to placing the geotextile, the exposed subgrade should be prepared as described in Section 3.8 of this report. Subgrade conditions should be observed prior to placing the geotextile and capillary break to evaluate the presence of unsuitable subgrade soils. If the subgrade soils are excessively loose or soft, it may be necessary to excavate localized areas and replace them with compacted structural fill.

Provided the recommendations described above are followed, the slab may be designed with an allowable bearing pressure of 950 psf.

3.5 CANAL LINER

We understand that portions of the irrigation canal channel will be lined with concrete as part of the proposed improvements. Canal embankment slopes should be shaped to at least 2 inches above the top edge of the lining, graded to a maximum slope of 2H:1V, compacted to a firm condition, and then lightly scarified. Concrete can be placed directly on native soils, provided they have been prepared as recommended.

3.6 EARTH RETAINING STRUCTURES

We understand that the transformer for the pumping plant will have a below-grade vault extending up to 4 feet below the structural mat foundation and that the debris management slab at the siphon will have a 4-foot-tall precast gravity block wall at the northwest corner, adjacent to the slope leading to Bannock Creek. The following recommendations assume a maximum wall height of 4 feet and that vehicle traffic on or near these walls is limited to occasional passenger vehicles.

3.6.1 Below-Grade Walls

Permanent below-grade walls may be designed using the earth pressure diagrams presented on Figure 3. Earth pressures are provided for two conditions: Not Restrained Against Rotation and Restrained Against Rotation. Walls that are free to rotate/move laterally at the top a distance of at least one thousandth the height of the wall (H/1000) should be designed using active earth pressures provided for the Not Restrained Against Rotation case. Walls that will be restrained from rotation/lateral movement should be designed using at-rest earth pressures provided for the Restrained Against Rotation case.

Foundation surcharge loads from adjacent buildings should be incorporated into the design of the below-grade walls using the surcharge pressures presented on Figure 4. If vehicles can approach the tops of exterior walls to within half the height of the wall, a traffic surcharge of 250 psf should be added to the wall pressure. Other surcharge loads, such as from construction equipment or construction staging areas, should be considered on a case-by-case basis and can be provided, if necessary, as design progresses. In the event that walls exceeding 6 feet are planned for any structure, walls must be designed incorporating the additional seismic lateral earth pressure.

The lateral earth pressures provided on Figure 3 assume that the ground surface behind the wall is horizontal/flat. If the ground surface behind the wall is sloped steeper than 4H:1V, CHE should be contacted to provide revised earth pressures.

These recommendations assume that all below-grade retaining walls will be provided with adequate drainage. The values provided for soil bearing, frictional resistance, and passive resistance presented for foundation design are applicable to retaining wall design. Walls located in level ground areas should be founded at a depth of 32 inches below the adjacent grade.

3.6.2 Gravity Block Retaining Walls

Gravity block retaining walls may be designed using the soil properties provided in Table 2. The wall design should include a 350 psf surcharge load to account for debris and occasional traffic loading. Additionally, if the wall face is within 4 feet of the crest of the Bannock Creek slope, global stability of the wall considering the embankment slope geometry should be checked.

Table 2. Gravity Block Retaining Wall Soil Properties

Zone	Friction Angle, f (deg)	Cohesion, c (psf)	Unit Weight, g (pcf)
Reinforced Soil	34	0	130
Retained Soil	30	0	120
Foundation Soil	30	0	105
Leveling Pad	40	0	135

Notes:

a. Soil properties for reinforced soil, retained soil, and leveling pad material assume conformance with the structural fill material and placement/compaction requirements described in Section 3.8 of this report.

3.6.3 Wall Drainage

In areas where temporary cut slopes and conventional cast-in-place techniques are used to construct below-grade walls, a conventional footing drain should be located on the outside of the wall. Positive drainage should be provided behind cast-in-place and gravity block walls by placing a 2-foot-wide zone of free-draining material against the wall. Free-draining material should consist of Drain Rock in accordance with Table 1 of Section 802.2.2 of the 2020 ISPWC, but with the nominal maximum aggregate size limited to 1 inch. The zone of wall drainage material should extend from the base of the wall to within 2 feet of the ground surface. The wall drainage material should be covered with 2 feet of less-permeable material, such as the on-site silt soils that is properly moisture conditioned and compacted. A nonwoven geotextile separator should be placed between the wall drainage material and overlying cover soil. A 4-inch-diameter perforated drainpipe should be installed within the free-draining material at the base of each wall. The foundation drain recommended in Section 3.2.1.6 can be incorporated into the bottom of the drainage zone and used for this purpose.

The pipes should be laid with minimum slopes of 0.25 percent and discharged into the stormwater collection system to convey the water off site. The pipe installations should include a cleanout riser with cover located at the upper end of each pipe run. The cleanouts could be placed in flush-mounted access boxes. Collected downspout water should be routed to appropriate discharge points in separate pipe systems.

3.7 GRAVEL ACCESS ROAD

Site improvements include construction of an all-weather gravel surfacing roadway to support vehicular traffic around the pumping plant and switchyard. We understand that this surface will be used for maintenance and operation activities. We anticipate that the vehicles that will be utilizing this length of road will be primarily pickup trucks (up to 1 ton) and occasional tractors and dump trucks (approximately two tractors and two dump trucks per week averaged over a 1-year period).

3.7.1 Subgrade Preparation

Subgrade areas should be prepared as recommended in Section 3.8 of this report. We recommend that a proof-roll of the subgrade, compacted subbase course, and base course be observed by a representative from our firm prior to constructing the gravel access road. Soft or yielding areas observed during proof-rolling may require over-excavation and replacement with compacted structural fill.

3.7.2 Gravel Roadway Section

The gravel section recommended herein is based on AASHTO 1993, *AASHTO Guide for Design of Pavement Structures*, following the design methodology for Aggregate-Surfaced Roads, our experience, and observations during the geotechnical exploration program. We designed the aggregate-surfaced road using a rutting criteria of 1 inch and a 0.5% growth rate over a 20-year design life. Based on our discussions with the design team, we approximated 8,000 ESALs for design of the gravel roadways section. This ESAL value was developed using the following estimated daily traffic loading:

- 10 pick-up trucks (FHWA Class 3 vehicle)
- 0.28 tractors (FHWA Class 6 vehicle) representing approximately two tractors per week averaged over a 1-year period.
- 0.28 dump trucks (FHWA Class 8 vehicle) representing approximately two dump trucks per week averaged over a 1-year period.

ESALs are estimated using the assumed traffic classification volume and load equivalency factors (LEFs). We assigned a LEF of 0.004, 1.5, and 2.0 for FHWA class 3, 6, and 8 vehicles, respectively.

We recommend a roadway section consisting of a 6-inch thickness of crushed rock base course overlying 15 inches of granular subbase. The subbase and base course should be compacted to at least 95 percent of the MDD (ASTM D1557).

We understand that an asphalt surface course is not being considered. As such, roads and surfaces that do not have an asphalt surface course will require regular maintenance including re-grading of the aggregate surface.

3.8 EARTHWORK

CHE understands that earthwork will include excavating and backfilling for foundations, slabs-on-grade, below-grade walls, and utility trenches.

3.8.1 Clearing and Site Preparation

Construction of the proposed structures will require demolition of existing utilities, gravel access roads, and other pertinent structures. Existing debris should be excavated and removed from the site. Demolished concrete structures may be ground up and reused on-site as structural fill; otherwise, it should be hauled off site. Existing utilities and associated trench backfill should be removed from below the proposed building footprints. All excavations that extend below slabs-on-grade or foundation subgrades should be backfilled with structural fill.

Areas to be developed or graded should be cleared of surface and subsurface deleterious matter including any debris, brush, and associated stumps and roots. Graded areas should be stripped of organic soils. Based on our site observations, we estimate that stripping depths will typically be on the order of 8 to 12 inches to remove the rooted layer. Deeper stripping and grubbing depths will be required to remove localized deeper deposits of topsoil.

The organic soils can be stockpiled and processed for landscaping purposes or may be spread over disturbed areas following completion of grading. If spread out, the organic strippings should be placed in a layer less than 1-foot thick, should not be placed on slopes greater than 3H:1V, and should be track-walked to a uniformly compacted condition. Materials that cannot be used for landscaping or protection of disturbed areas should be removed from the project site.

3.8.2 Excavation Support

At this time, we expect that temporary open cut slopes will be suitable for excavations required for the project and understand temporary shoring is not anticipated for the site. The following sections summarize the general excavation recommendations for temporary cut slopes and permanent cut-and-fill slopes.

3.8.2.1 TEMPORARY CUT SLOPES

The stability of open cut slopes is a function of soil type, groundwater seepage, slope inclination, slope height, and nearby surface loads. The use of inadequately designed open cuts could impact the stability of adjacent work areas, existing utilities, and endanger personnel. In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to variable soil and groundwater conditions. Therefore, the contractor should have the primary responsibility for deciding whether or not to use open cut slopes rather than some form of temporary excavation support, and for establishing the safe inclination of the cut slope.

Acceptable slope inclinations should be determined during construction. All open cut slopes and temporary excavation support should be constructed or installed and maintained in accordance with the requirements of the appropriate governmental agency.

For planning purposes, temporary unsupported cut slopes more than 4 feet high may be inclined no steeper than 1.5H:1V in the fill and native loess. It may be necessary to flatten the cut slopes if seepage is present on the cut face or if localized sloughing occurs. We recommend that a representative from our firm observe excavation of temporary cuts over 6 feet in height to assess stability and confirm recommended cut slope inclination.

The above guidelines assume that surface loads such as equipment loads and storage loads will be kept a sufficient distance away from the top of the cut so that the stability of the excavation is not affected. We recommend that this distance be not less than half the height of the cut or 5 feet, whichever is greater.

Water entering excavations must be collected and routed away from prepared subgrade areas. We expect that this may be accomplished by installing a system of drainage ditches and sumps along the toe of the cut slopes. Some sloughing and raveling of the cut slopes should be expected. Temporary covering, such

as heavy plastic sheeting with appropriate ballast, should be used to protect these slopes during periods of wet weather. Surface water runoff from above cut slopes should be prevented from flowing over the slope face by using berms, drainage ditches, swales, or other appropriate methods.

If temporary cut slopes experience excessive sloughing or raveling during construction, it may become necessary to modify the cut slopes to maintain safe working conditions. Slopes experiencing problems can be flattened, regraded to add intermediate slope benches, or additional dewatering can be provided if the poor slope performance is related to groundwater seepage.

3.8.3 Subgrade Preparation & Protection

All topsoil, existing fill, and organic soils should be removed from structure and hardscape subgrade areas. The debris management slab and roadway section may utilize undocumented fill as subgrade. Prior to placing concrete, structural fill, and/or pavement base course materials, all subgrade areas should be evaluated by means of a 0.5-inch-diameter steel probe rod or by proof-rolling to confirm bearing capacity recommendations are met and to locate any soft or pumping soils. Proof-rolling can be completed using heavy, tire-mounted equipment such as a loaded dump truck or a 5-ton smooth-drum vibratory roller. If soft or pumping soils are observed, they should be removed and replaced with properly compacted structural fill.

Site soils contain significant fines content (silt/clay) and will be highly sensitive to increases in moisture content. The exposed subgrade soils can deteriorate rapidly in wet weather and under equipment loads. The contractor should take necessary measures to prevent site subgrade soils from becoming disturbed or unstable. CHE recommends construction occur only during dry weather. However, if constructed during periods of wet weather, CHE recommends subgrade soils be protected with concrete blankets, a lean concrete mud mat, or other appropriate measures. The blankets, mud mat, or other appropriate measures should be placed immediately following subgrade approval. Construction traffic during the wet season should be restricted to specific areas of the site, preferably areas that are surfaced with crushed rock not susceptible to wet weather disturbance.

3.8.4 Utility Trenches

Trench excavation, pipe bedding, and trench backfilling should be completed using the general procedures described in the 2020 ISPWC, or other suitable procedures specified by the project civil engineer.

Utility trench backfill should consist of structural fill and should be placed in lifts of 8 inches or less (loose thickness) such that adequate compaction can be achieved throughout the lift. Sand backfill, containing less than 5 percent fines, may be compacted in loose lifts not exceeding 12 inches. Each lift must be compacted prior to placing the subsequent lift. Prior to compaction, the backfill should be moisture conditioned to within 3 percent of the optimum moisture content, if necessary.

3.8.4.1 SOIL CORROSIVITY

We understand that a series of underground pipes are planned to connect the valve vault of the pumping plant to the thrust block to the southwest and that these pipes are planned to be approximately 3 to 5 feet below ground surface. Corrosivity of site soils was evaluated through chemistry suite laboratory testing, which includes pH, resistivity, and sulfates concentration. The results of these tests are presented below in Table 3, and in the remarks of the logs in Appendix A.

Table 3. Corrosivity Testing Results

Test Pit No.	Depth (ft)	pH	Resistivity (ohm-cm)	Sulfates (ppm)
CHE-TP2-24	6.0-7.0	8.45	3,403	37
CHE-TP3-24	3.0-4.0	8.73	5,790	ND[1]
CHE-TP7-24	1.5-2.0	8.5	2,195	45

1. ND = non-detectable sulfate content

Soils with higher than 1,500 ohm-cm of resistivity and a pH higher than 5.5 are generally considered non-corrosive. Additionally, for structural elements, soils are considered corrosive if the sulfate concentration is higher than 1,500 ppm and/or the pH is 5.5 or less (Caltrans 2022). Based on the results of these tests, we anticipate that the subsurface soils encountered in the project area exhibit a low corrosion potential to construction materials.

3.8.5 Structural Fill

All fill that will support foundations, floor slabs, or pavements and hardscape areas, or to be placed against retaining walls or in utility trenches, should meet the criteria for structural fill presented below.

3.8.5.1 MATERIALS

Materials used to construct the building pad, placed under foundations and hardscape, and used to backfill utility trenches are classified as structural fill for the purpose of this report. Quality of structural fill material varies depending upon its use as described below:

- Structural fill placed below new buildings should consist of crushed aggregate granular structural fill, with a maximum particle size of 2 inches, as described in Section 802.2.2 of the 2020 ISPWC.
- To provide uniform support, utility line bedding material should meet the requirements of Pipe Bedding, as described in Section 305 of the 2020 ISPWC.
- Utility trench backfill should consist of crushed aggregate granular structural fill, with a maximum particle size of 2 inches, as described in Section 802.2.2 of the 2020 ISPWC.
- Structural fill placed around perimeter footing drains, under slab drains, and cast-in-place wall drains should consist of Drain Rock in accordance with Table 1 of Section 801.2.2 of the 2020 ISPWC, but with the nominal maximum aggregate size limited to 1 inch.
- Structural fill placed as capillary break below slabs should consist of Type 1 Crushed Aggregate ¾-inch minus in accordance with Section 802 of the 2020 ISPWC.
- Base course placed as the running surface of the aggregate-surfaced road should conform to Type 1 Crushed Aggregate ¾-inch minus crushed aggregate in accordance with Section 802.2 of the 2020 ISPWC.
- Subbase course placed for the aggregate-surface road should consist of 1-inch Uncrushed Aggregate in accordance with Table 1 of Section 801.2.2 of the 2020 ISPWC, with the percent passing the No. 200 limited to 10 percent.
- Geosynthetics placed for subgrade stabilization should consist of Type II Subgrade Separation Geotextile that conforms to the requirements of Section 2050, Part 2.3 of the 2020 ISPWC.
- Geosynthetics placed to prevent the migration of fines (i.e. around footing drains) should consist of drainage geotextile that conforms to the requirements of Section 2050, Part 2.1 of the 2020 ISPWC.

3.8.5.2 REUSE OF ON-SITE NATIVE SOILS

The native soil contains a high percentage of fines and will be sensitive to changes in moisture content. As a result, the on-site soils will likely require significant moisture conditioning in order to meet the required compaction criteria during dry weather conditions and will not be suitable for reuse during wet weather. Furthermore, most of the fill soils required for the project have specific gradation requirements, and the on-site soils do not meet these gradation requirements. Therefore, imported structural fill meeting the requirements described above should be used where structural fill is necessary. Site soils that cannot be reused as structural fill should be disposed of at a suitable off-site location, or in landscaped areas where several inches of post-construction settlement is acceptable.

3.8.5.3 FILL PLACEMENT AND COMPACTION CRITERIA

Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in loose lifts not exceeding 12 inches in thickness if using heavy compactors, or 4 inches if using hand-operated equipment. The actual thickness will be dependent on the structural fill material used and the type and size of compaction equipment. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Compaction of all structural fill should be in accordance with the ASTM D1557 (modified proctor) test method. Structural fill should be compacted to the following criteria:

1. Structural fill placed below foundations and concrete slabs-on-grade should be compacted to at least 95 percent of the MDD.
2. Structural fill placed behind below-grade walls should be compacted to between 90 to 92 percent of the MDD. Care should be taken when compacting fill near the face of below-grade walls to avoid over-compaction (and, hence, overstressing the walls). Hand operated compactors should be used within 5 feet behind the wall. The contractor should keep all heavy construction equipment away from the top of retaining walls a distance equal to half the height of the wall, or at least 5 feet, whichever is greater.
3. Structural fill in new pavement and hardscape areas, including utility trench backfill, should be compacted to at least 90 percent of the MDD, except that the upper 2 feet of fill below final subgrade should be compacted to at least 95 percent of the MDD.
4. Non-structural fill, such as fill placed in landscape areas, should be compacted to at least 90 percent of the MDD, unless otherwise specified by the landscape architect.

3.8.5.4 WEATHER CONSIDERATIONS

Surficial site soils are predominantly classified as silt and clay, have a high percentage of fines, and are considered moisture sensitive. Imported fill could also be moisture sensitive if the fines content is above 10 percent. When the moisture content of these soils is more than a few percentage points above or below optimum, the soil becomes unstable and soil compaction and operation of heavy equipment on these soils is difficult. Disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather or in wet conditions. During wet weather, CHE recommends:

- The ground surface be sloped so surface water is directed away from the work area to an approved collection/dispersion point.
- Earthwork activities do not take place during periods of heavy precipitation or in wet conditions.
- Measures are taken to prevent site soils and soil stockpiles from becoming wet or unstable.

- The amount of fines in imported fill materials is limited to 5 percent or less, by dry weight, based on the fraction passing the ¾-inch sieve.
- A smooth-drum roller is used to seal the ground surface prior to periods of precipitation to reduce the extent to which the soil becomes wet or unstable.
- Foundation subgrade soils be protected with concrete blankets, lean concrete mud mats, or other appropriate coverings.
- Construction traffic is restricted to specific areas of the site, preferably areas surfaced with materials not susceptible to wet weather disturbance.
- Contingencies are included in the project schedule and budget to allow for the above measures.

3.9 SEDIMENTATION AND EROSION CONTROL

In our opinion, the erosion potential of the on-site soils is low to moderate, if properly managed. Construction activities including stripping and grading will expose soils to the erosional effects of wind and water. The proximity to the slope leading to Bannock Creek at the siphon and trash rack structures pose risk of additional erosion if construction vehicles or work performed is too close to these slopes. The amount and potential impacts of erosion are partly related to the time of year that construction occurs. Wet-weather construction will increase the amount and extent of erosion and potential sedimentation.

Erosion and sedimentation control measures may be implemented by using a combination of interceptor swales, straw bale barriers, silt fences, and straw mulch for temporary erosion protection of exposed soils. All disturbed areas should be finish graded and seeded as soon as practical to reduce the risk of erosion. Erosion- and sedimentation-control measures should be installed and maintained in accordance with the requirements of the county or other applicable procedures specified by the project civil engineer. Measures to control excessive dust should be considered as part of the overall health and safety management plan.

4.0 FINAL DESIGN AND CONSTRUCTION SUPPORT

A geotechnical engineer familiar with the project should conduct a design review of the plans and specifications for compliance with the recommendations presented in this report. CHE also recommends that monitoring, testing, and consultation be provided during construction to confirm that the conditions encountered are consistent with those observed in the field explorations, to provide expedient recommendations should conditions be revealed during construction that differ from those anticipated, and to evaluate whether geotechnical-related construction activities comply with projects plans, specifications, and the recommendations contained in this report.

Geotechnical-related activities include evaluation of foundation and pavement subgrade, placement and compaction of backfill material, observation of temporary slopes, and other earthwork activities. CHE would be pleased to provide these services for you.

5.0 LIMITATIONS

Ciani & Hatch Engineering, PLLC (CHE) has prepared this report for the exclusive use of HDR, the Bureau of Indian Affairs, and their authorized agents for explicit application to the Bannock Pumping Plant Modernization project located on the Fort Hall Reservation in Power County, Idaho. No other party is entitled to rely on the information, conclusions, and recommendations included in this document without the express written consent of CHE. Reuse of the information, conclusions, and recommendations provided herein for extensions of the project or for any other project, without review and authorization by CHE, shall be at the user's sole risk.

This report, along with its conclusions and interpretations, should not be considered a guarantee of the subsurface conditions. The analysis and recommendations provided in this report rely on the data gathered from each specific test pit location and other information detailed herein. This report does not account for potential variances that might exist between different test pit or testing locations, across the site, or due to the influence of weather conditions or events. The true subsurface conditions can only be confirmed during the actual earthwork construction phase. In the event of any observed variations, we recommend you promptly notify CHE so that we can confirm that the soil conditions encountered are consistent with those observed in our explorations and provide supplementary recommendations as needed.

An evaluation of environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials, or conditions was not completed and is not part of CHE's scope. If the potential for such contamination or pollution is a concern, an environmental consultant should be retained to complete such studies.

CHE recommends the project design and specifications are reviewed by CHE to verify recommendations have been interpreted and implemented as intended. Should site or project conditions change, CHE should be retained to review the recommendations and conclusions in this report to determine if the recommendations are still applicable or to provide supplemental information, if needed.

Within the limitations of scope, schedule, and budget, CHE's services have been provided in a manner in accordance with the level of skill and care ordinarily exercised in the field of geotechnical engineering currently practicing in the same area and under similar conditions. No other warranty, either expressed or implied, should be understood.

CHE cannot guarantee the accuracy and content of electronic files (email, text, table, figures, etc.). The master file of this document is stored by CHE and will serve as the official document of record.

6.0 REFERENCES

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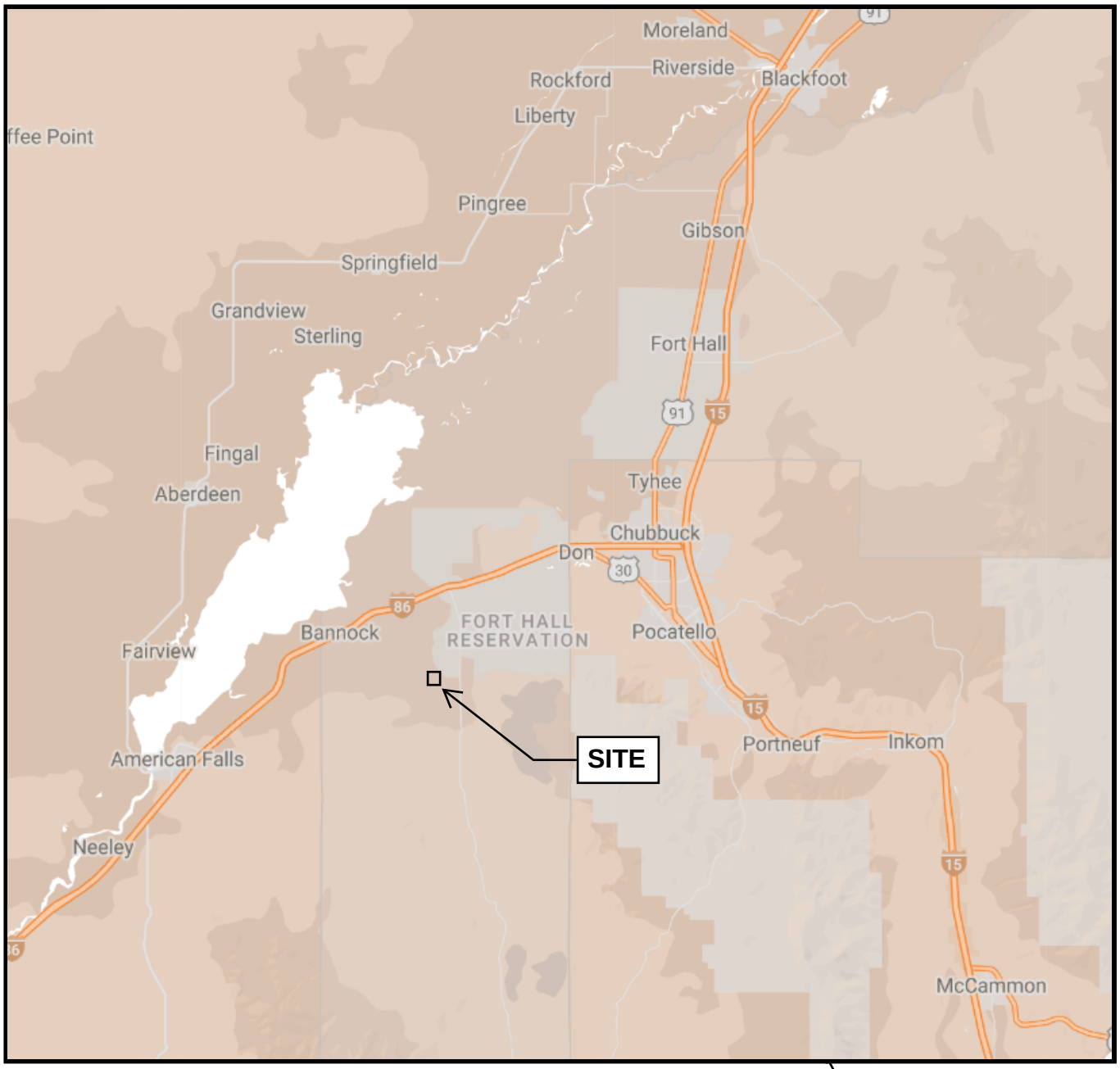
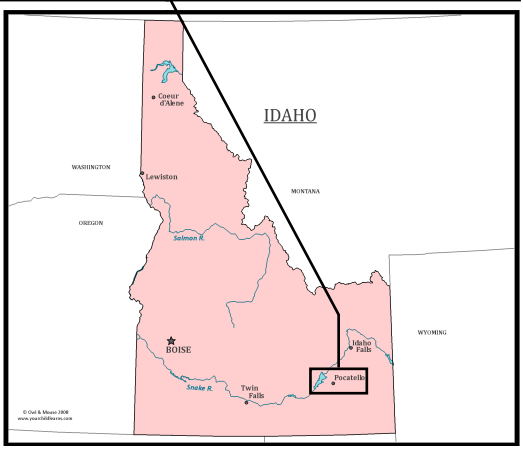
Figures





NOTES:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes only. It is intended to assist in showing features discussed in an attached document. Ciani & Hatch Egineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.

Map Source: Map data 2024 Google

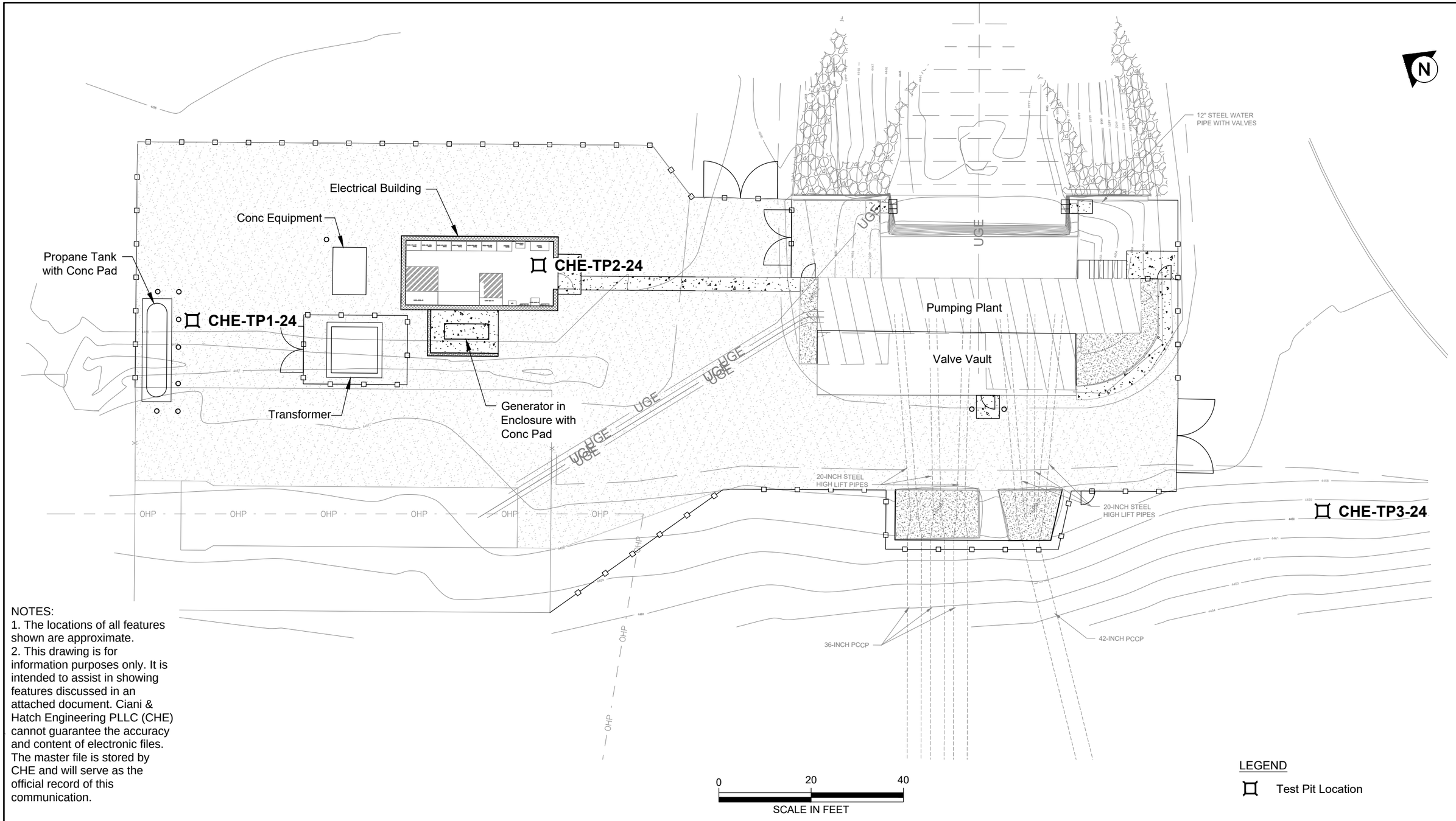
CIANI & HATCH ENGINEERING

Project Vicinity Map

Bureau of Indian Affairs
Bannock Pumping Plant Modernization Project
Fort Hall Reservation, Power County, Idaho

FIGURE

1



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Survey Reference:
Base drawings provided by HDR, dated 9/5/2024.

LEGEND

Test Pit Location

CIANI & HATCH ENGINEERING

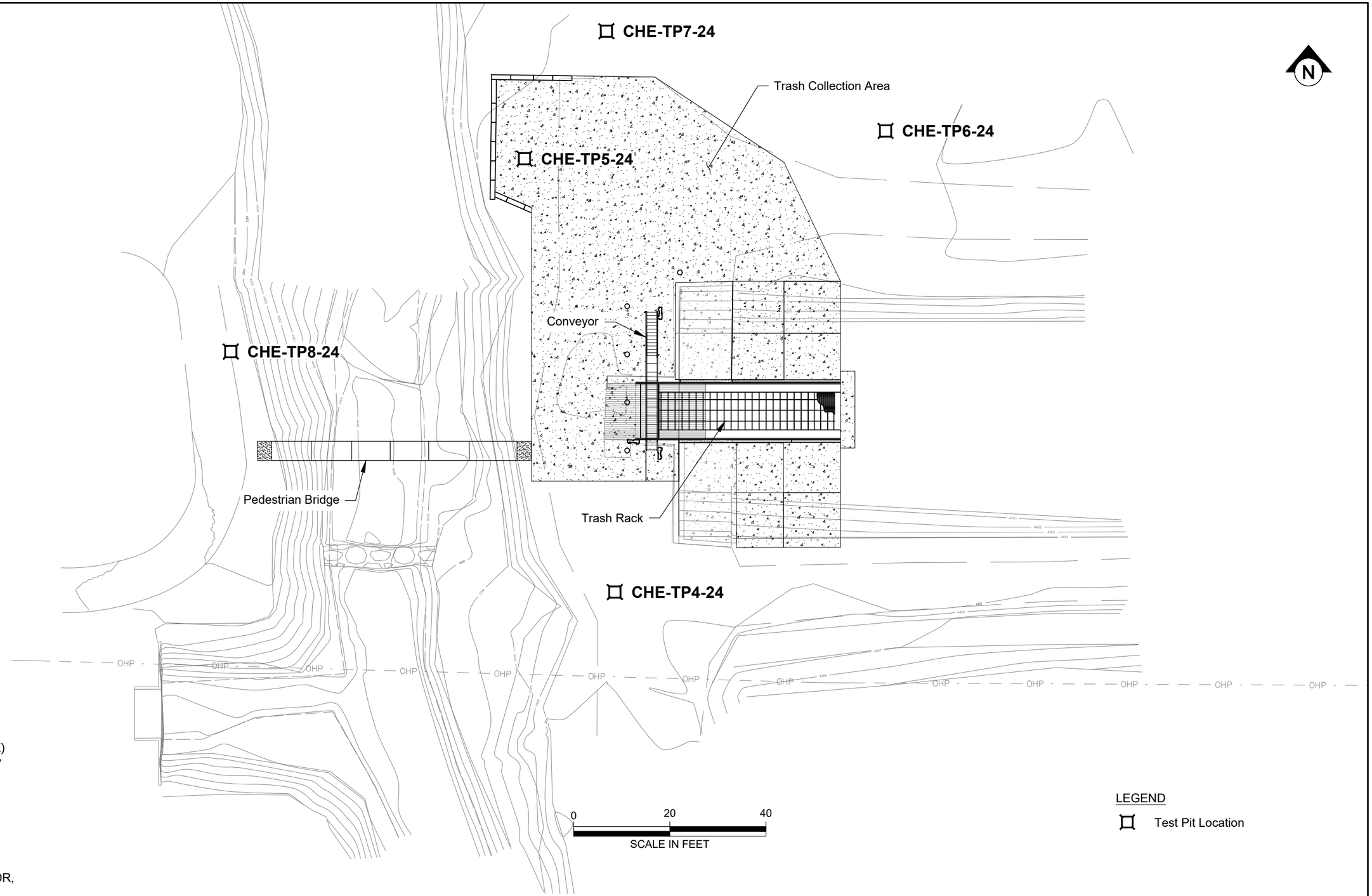
PROJECT MANAGER	M. Hatch	PROJECT NUMBER	4000-001-00
DRAWN BY	C. Taylor	SCALE	As Shown
CHECKED BY	M. Hatch	FILE NAME	4000-001-00 Site Plan.dwg
APPROVED BY	W. Ciani	DATE	September 2024

Geotechnical Exploration Site Plan - Pumping Plant & Electrical Building

Bureau of Indian Affairs
Bannock Pumping Plant Modernization Project
Fort Hall Reservation, Power County, Idaho

FIGURE

2-1



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LEGEND

Test Pit Location

CIANI & HATCH ENGINEERING

PROJECT MANAGER	M. Hatch	PROJECT NUMBER	4000-001-00
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CHECKED BY	M. Hatch	FILE NAME	4000-001-00 Site Plan.dwg
APPROVED BY	W. Ciani	DATE	September 2024

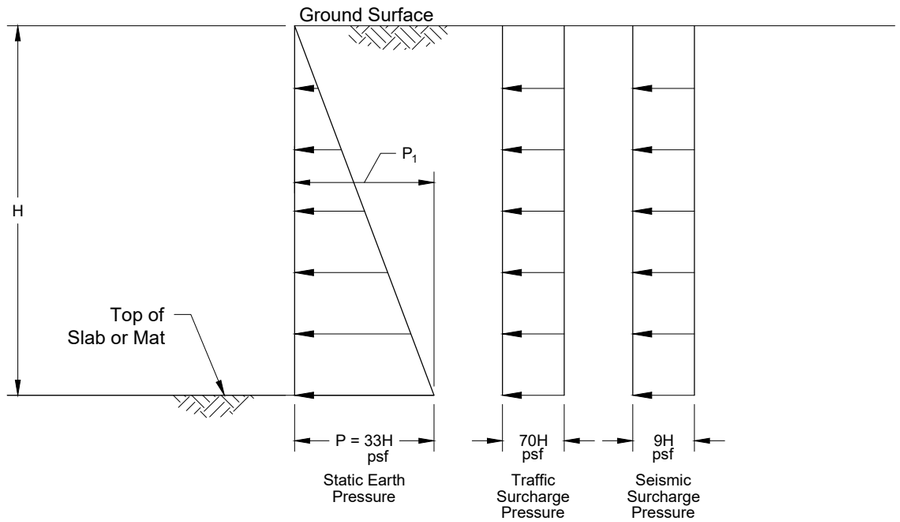
Geotechnical Exploration Site Plan -Trash Rack

Bureau of Indian Affairs
Bannock Pumping Plant Modernization Project
Fort Hall Reservation, Power County, Idaho

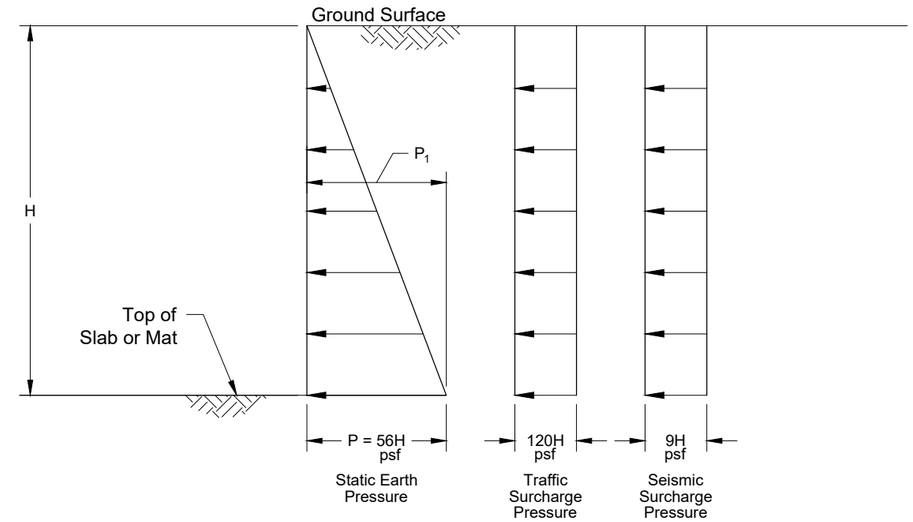
FIGURE

2-2

Below Grade Walls
Not Restrained Against Rotation



Below Grade Walls
Restrained Against Rotation



Not to Scale

NOTES

1. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 4.
2. This pressure diagram is appropriate for permanent below grade walls and retaining structures which are not adjacent to temporary shoring walls and are not restrained against movement.
3. Walls are assumed to be restrained if top movement during backfilling is less than H/1000, where H is the wall height.
4. These lateral soil pressures assume that the ground surface behind the wall is horizontal.
5. The static earth pressure does not include a factor of safety and represents the actual anticipated static earth pressure.
6. Ciani & Hatch Engineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.

LEGEND

H = Height of Wall, Feet

P_1 = Maximum Static Earth Pressure
Pounds per Square Foot

CIANI & HATCH ENGINEERING

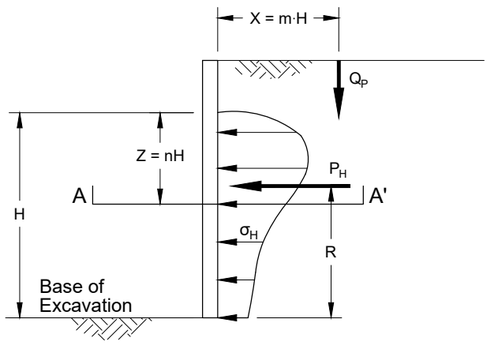
PROJECT MANAGER	M. Hatch	PROJECT NUMBER	4000-001-00
DRAWN BY	C. Taylor	SCALE	As Shown
CHECKED BY	M. Hatch	FILE NAME	NA
APPROVED BY	W. Ciani	DATE	10/01/2024

Permanent Below-Grade Walls

Bureau of Indian Affairs
Bannock Pumping Plant Modernization Project
Fort Hall Reservation, Power County, Idaho

FIGURE
3

Lateral Earth Pressure From Point Load, Q_P (Spread Footing)



FOR $m \le 0.4$

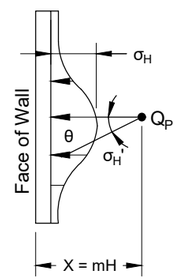
$$\sigma_H = \frac{0.28 Q_P n^2}{H^2(0.16+n^2)^3}$$

FOR $m > 0.4$

$$\sigma_H = \frac{1.77 Q_P m^2 n^2}{H^2(m^2+n^2)^3}$$

$$\sigma_H' = \sigma_H \cos^2(1.1\theta)$$

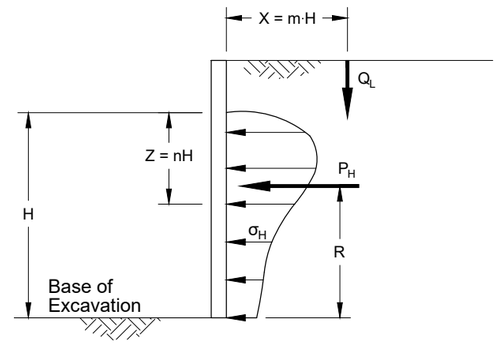
M	$P_H\left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



SECTION A-A'

Pressures from Point Load Q_P

Lateral Earth Pressure From Line Load, Q_L (Continuous Wall Footing)



FOR $m \le 0.4$

$$\sigma_H = \frac{0.2 n Q_L}{H(0.16+n^2)^2}$$

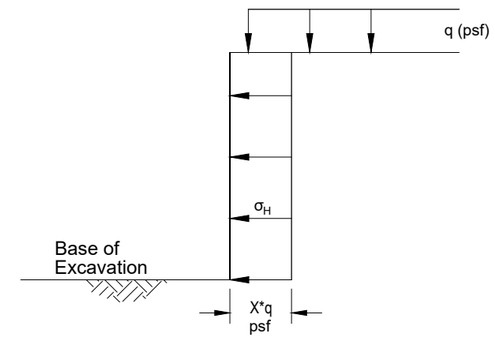
FOR $m > 0.4$

$$\sigma_H = \frac{1.28 m^2 n Q_L}{H(m^2+n^2)^2}$$

$$\text{Resultant } P_H = \frac{0.64 Q_L}{(m^2+l)}$$

M	R
0.1	0.60H
0.3	0.60H
0.5	0.56H
0.7	0.48H

Uniform Surcharges, q (Floor Loads, Large Foundation Elements, or Traffic Loads)



σ_H = Lateral Surcharge Pressure From Uniform Surcharge

X = Lateral Pressure Coefficient as Shown in Table Below

	X
Wall Not Restrained Against Movement	0.24
Wall Restrained Against Movement	0.41

Not to Scale

NOTES

1. Procedures for estimating surcharge pressures shown above are based on Manual 7.02 Naval Facilities Engineering Command, September 1986 (NAVFAC DM 7.02).
2. Lateral earth pressures from surcharge should be added to earth pressures presented on Figures 3.
3. See report text for where surcharge pressures are appropriate.
4. Ciani & Hatch Engineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.

LEGEND

Q_P = Point load in pounds
Q_L = Line load in pounds/foot
H = Excavation height below footing, feet
σ_H = Lateral earth pressure from surcharge, psf
q = Surcharge pressure in psf
θ = Radians
σ_H' = Distribution of σ_H in plan view
P_H = Resultant lateral force acting on wall, pounds
R = Distance from base of excavation to resultant lateral force, feet

CIANI & HATCH ENGINEERING

PROJECT MANAGER	M. Hatch	PROJECT NUMBER	4000-001-00
DRAWN BY	C. Taylor	SCALE	As Shown
CHECKED BY	M. Hatch	FILE NAME	NA
APPROVED BY	W. Ciani	DATE	10/01/2024

Recommended Surcharge Pressure

Bureau of Indian Affairs
Bannock Pumping Plant Modernization Project
Fort Hall Reservation, Power County, Idaho

FIGURE

4

Appendix A.

CHE FIELD EXPLORATIONS

APPENDIX A

FIELD EXPLORATIONS

Subsurface conditions at the site were explored by advancing eight test pits (CHE-TP1-24 through CHE-TP8-24). The test pits were completed to depths between 3 and 15 feet below existing site grades. The excavation was performed by Precision Underground subcontracted to Ciani & Hatch Engineering, PLLC (CHE) on August 7 and 8, 2024.

CHE personnel coordinated and monitored the explorations, obtained representative soil samples, maintained detailed logs of the subsurface soil observed, and described the soil encountered by visual and textural examination. Each representative soil type was described using the Unified Soil Classification System (USCS), in general accordance with ASTM International Standard D2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedures)*. Summary logs of the explorations are presented on Figures A-2 through A-9. The stratigraphic contacts indicated on the logs represent the approximate boundaries between soil types; actual transitions may vary and be more gradual. The relative densities noted on the boring logs are based on the DCP data obtained in the borings and our field observations. The soil and groundwater conditions illustrated are specific to the dates and locations mentioned in the report and may not necessarily reflect conditions in other areas or at different times. A key to the exploration logs is presented on Figure A-1.

Test pit soil samples were taken from the excavation sidewalls, base of the excavation, and bucket of the excavator. Relatively undisturbed samples were obtained using a 2.8-inch-outside-diameter Shelby tube sampler (ASTM D1587).

No groundwater was encountered in our explorations at the time of our site visit. At the location of the Siphon, surface water in Bannock creek was approximately 9 feet below existing grade.

Upon completion of fieldwork, all test pits were backfilled with excavated soils and tamped into place with the excavator bucket to roughly match the existing surface.

CHE returned to the project site on November 18, 2024, to obtain a sample of subgrade material in the area of the proposed Debris Management Slab and to evaluate the subgrade soil conditions within the Taghee Canal at the location of the rotating screen slab extension. We evaluated the condition of the subgrade by probing with a 0.5-inch-diameter steel probe rod. The native channel material was comprised of saturated fine-grained soil, with probe depths of 30 inches.

CHE obtained two bulk samples during our November 2024 site visit. A sample of the channel material was collected approximately 12 feet east of the existing structure at the centerline of the channel and a sample of the subgrade material was collected from within the Debris Management Slab, approximately 37 feet north of the existing siphon structure.

Soil samples were transported to the STRATA geotechnical laboratory in Boise, Idaho, for further examination and testing. Test results are presented in Appendix B.

Soil Classification Chart
(Unified Soil Classification System)

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	LETTER DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	Well-graded gravels, gravel-sand mixtures, little or no fines
				GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
	MORE THAN 60% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	Silty gravels, gravel-sand-silt mixtures
				GC	Clayey gravels, gravel-sand-clay mixtures
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	Well-graded sands, gravelly sands, little or no fines
				SP	Poorly graded sands, gravelly sands, little or no fines
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	Silty sands, sand-silt mixtures
				SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL	Organic silts and organic silty clays of low plasticity
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	Inorganic silts, micaceous or diatomaceous fine sand or silty soils
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS				PT	Peat, humus, swamp soils with high organic contents

OTHER MATERIALS	LETTER SYMBOL	GRAPH SYMBOL
Asphalt Concrete	AC	
Cement Concrete	CC	
Base Course	BC	
Wood	W	

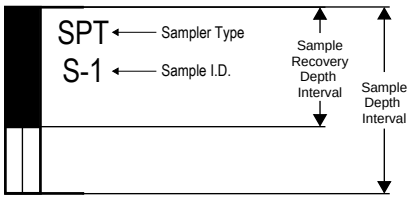
Field and Lab Testing

%F	Percent passing the No. 200 Sieve
SA	Grain size analysis
CA	Combined grain size analysis and hydrometer
CHEM	Chemistry Suite
CONSOL	Consolidation Test
AL	Atterberg Limits
LL	Liquid Limit
PL	Plastic Limit
PI	Plasticity Index
MC	Moisture Content
WOH	Weight of Hammer
WOR	Weight of Rod

Sampler Types

SHELBY	Shelby Tube Sampler
SPT	Standard Penetration Test
CALI	California Sampler
GRAB	Grab sample

Sampler Information



Geology Boundaries

Soil strata contact between soil geologic units

Soil strata contact within the same geologic unit

Soil Density/Consistency

Fine-Grained Soils	
N-value blows per foot	Qualitative Consistency Description
<2	Very soft
2 to 4	Soft
4 to 8	Medium stiff
8 to 15	Stiff
15 to 30	Very stiff
> 30	Hard

Granular Soils	
N-value blows per foot	Qualitative Density Description
<4	Very Loose
4 to 10	Loose
10 to 30	Medium dense
30 to 50	Dense
> 50	Very dense

Percent Minor Constituents

With:

5-15% (silt/clay) in sand or gravel
15-30% (snd/gravel) in silt or clay

Notes:

1. Each representative soil type was described using the Unified Soil Classification System (USCS), in general accordance with ASTM International standard Standard D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedures). Dual letter symbols (e.g., SP-SM for sand or gravel) indicate soil with an estimated 5-15% fines. Multiple letter symbols (e.g., ML/CL) indicate borderline or multiple soil classifications.
2. Soil density or consistency descriptions are based on judgment using a combination of sampler penetration blow counts, drilling or excavating conditions, field tests, and laboratory tests, as appropriate.
3. These logs are for information purposes only. They are intended to assist in showing features discussed in an attached document. Ciani & Hatch Engineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.
4. Descriptions on the logs are constrained to the specific limited exploration locations at the time the explorations were made. These logs are not warranted to be representative of subsurface conditions at other locations or times. These logs are to be interpreted with the discussion in the report text for a proper understanding of subsurface conditions.

CIANI & HATCH ENGINEERING

Key to Exploration Logs

Bannock Pumping Plant Modernization
Fort Hall Reservation
Power County, Idaho

FIGURE

A-1

CIANI&HATCH ENGINEERING

Testpit No: CHE-TP1-24

UTM	: 12T	**Excavator**	: CAT 420	**Job Number**	: 4000-001-00
Latitude	: 43.836914	**Excavator Supplier**	: Precision Underground, LLC	**Client**	: HDR, Inc.
Longitude	: -112.645048	**Logged By**	: JAH	**Project**	: Bannock Pumping Plant Modernization
Elevation	: 4462(ft)	**Reviewed By**	: MSH	**Location**	: Fort Hall Reservation, ID, USA
Total Depth	: 14 ft	**Date**	: 08/07/2024	**Loc Comment**	: BPP NW

Elevation (ft)	Depth (ft)	Sample Number	Sample Type	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content	Fines Content	Remarks
4462		S-1	GRAB			ML	Light brown SILT with sand and gravel (stiff to very stiff, dry) (Fill)			Rootlets to 8" bgs
4461							Light brown SILT, with pinhole structure and no to weak cementation (medium stiff to stiff) (Loess)			
4460		S-2	GRAB							
4459		S-3	SHELBY	CONSOL				15.1		Dry Unit Weight: 101.8 pcf
4458		S-4	GRAB	MC				14.6		
4457										
4456										
4455		S-5	GRAB			ML				
4454										
4453										
4452		S-6	GRAB							
4451										
4450										
4449		S-7	GRAB							
4448							**CHE-TP1-24 Terminated at 14ft**			

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

Figure A-2

CIANI&HATCH ENGINEERING

Testpit No: CHE-TP2-24

UTM	**: 12T**	**Excavator**	**: CAT 420**	**Job Number**	**: 4000-001-00**
Latitude	**: 42.836778**	**Excavator Supplier**	**: Precision Underground, LLC**	**Client**	**: HDR, Inc.**
Longitude	**: -112.644854**	**Logged By**	**: JAH**	**Project**	**: Bannock Pumping Plant Modernization**
Elevation	**: 4461(ft)**	**Reviewed By**	**: MSH**	**Location**	**: Fort Hall Reservation, ID, USA**
Total Depth	**: 14 ft**	**Date**	**: 08/07/2024**	**Loc Comment**	**: BPP Central**

Elevation (ft)	Depth (ft)	Sample Number	Sample Type	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content	Fines Content	Remarks
4461							Light brown SILT with sand and gravel, weak to moderate cementation, strong HCl reaction, and construction debris (stiff to hard, dry) (Fill)			Rootlets to 8 inches bgs
		S-1	GRAB			ML				
4460										
4459							Light brown SILT with sand (stiff to very stiff, dry) (Loess)			
		S-2	GRAB			ML				
4458							Light brown SILT with pinhole structure and weakly cemented nodules (medium stiff to stiff, dry to moist)			
		S-3	GRAB							
4457										
4456										
		S-4	SHELBY	CONSOL				15.8		Dry Unit Weight: 93.1 pcf
4455										
		S-5	GRAB	CHEM						Chemistry Suite Results: pH: 8.45 | Resistivity: 3,403 Ohm-cm | Sulfates: 37 ppm
4454										
4453										
		S-6	GRAB			ML				
4452										
4451										
4450										Increased pinhole diameter
4449										
4448										
		S-7	GRAB							
4447							**CHE-TP2-24 Terminated at 14ft**			

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Figure A-3

CIANI&HATCH ENGINEERING

Testpit No: CHE-TP3-24

UTM	: 12T	**Excavator**	: CAT 420	**Job Number**	: 4000-001-00
Latitude	: 42.836335	**Excavator Supplier**	: Precision Underground, LLC	**Client**	: HDR, Inc.
Longitude	: -112.644675	**Logged By**	: JAH	**Project**	: Bannock Pumping Plant Modernization
Elevation	: 4465(ft)	**Reviewed By**	: MSH	**Location**	: Fort Hall Reservation, ID, USA
Total Depth	: 13 ft	**Date**	: 08/07/2024	**Loc Comment**	: BPP SE

Elevation (ft)	Depth (ft)	Sample Number	Sample Type	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content	Fines Content	Remarks
4465						ML	Light brown SILT with sand and weak to moderate cementation (very stiff, dry) (Loess)			
4464							Light brown SILT with pinhole structure and no to weak cementation (medium stiff, dry)			
4463										
4462		S-1	GRAB	CHEM		ML				Chemistry Suite Results: pH: 8.73 | Resistivity: 5,790 Ohm-cm | Sulfates: Non-Detectable ppm
4461										
4460										
4459		S-2	GRAB	MC			Light brown SILT with pinhole structure and no to weak cementation (stiff, dry)	11.9		
4458						ML				
4457										
4456		S-3	GRAB				Light brown SILT with pinhole structure and no to weak cementation (medium stiff, dry)			
4455						ML				
4454		S-4	GRAB				Light brown SILT with pinhole structure and no to weak cementation (soft to medium stiff, dry)			
4453						ML				
4452							**CHE-TP3-24 Terminated at 13ft**			
4451										

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

Figure A-4

CIANI&HATCH ENGINEERING

Testpit No: CHE-TP4-24

UTM	: 12T	**Excavator**	: CAT 420	**Job Number**	: 4000-001-00
Latitude	: 42.837874	**Excavator Supplier**	: Precision Underground, LLC	**Client**	: HDR, Inc.
Longitude	: -112.639205	**Logged By**	: JAH	**Project**	: Bannock Pumping Plant Modernization
Elevation	: 4458(ft)	**Reviewed By**	: MSH	**Location**	: Fort Hall Reservation, ID, USA
Total Depth	: 15 ft	**Date**	: 08/07/2024	**Loc Comment**	: Catwalk Landing

Elevation (ft)	Depth (ft)	Sample Type	Sample Number	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content	Fines Content	Remarks
4458						GM	Light to medium brown, silty, GRAVEL with sand, with occasional slag pieces (medium dense to dense, dry) (Fill)			
4457										
		GRAB	S-1							
4456										
4455						ML	Brown SILT with sand (medium stiff, moist) (Loess)			
		GRAB	S-2							
4454										
		SHELBY	S-3							
4453						ML	Dark brown with orange SILT with fine sand and wood/organic debris (very soft to medium stiff, moist) (Alluvium)			Organic debris (wood fibers, twigs) encountered below 5' bgs
4452										
4451										
		GRAB	S-4	MC						
4450						CL	Brown with orange CLAY with fine sand and wood/organic debris (soft to medium stiff, moist)			
4449										

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Figure A-5

CIANI&HATCH ENGINEERING

Testpit No: CHE-TP4-24

UTM	: 12T	**Excavator**	: CAT 420	**Job Number**	: 4000-001-00
Latitude	: 42.837874	**Excavator Supplier**	: Precision Underground, LLC	**Client**	: HDR, Inc.
Longitude	: -112.639205	**Logged By**	: JAH	**Project**	: Bannock Pumping Plant Modernization
Elevation	: 4458(ft)	**Reviewed By**	: MSH	**Location**	: Fort Hall Reservation, ID, USA
Total Depth	: 15 ft	**Date**	: 08/07/2024	**Loc Comment**	: Catwalk Landing

Elevation (ft)	Depth (ft)	Sample Type	Sample Number	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content	Fines Content	Remarks
4448		GRAB	S-5	MC, AL		CL	Brown with orange CLAY with fine sand and wood/organic debris (soft to medium stiff, moist)	31.9		
4447										
4446										
4445										
4444		GRAB	S-6							
4443							**CHE-TP4-24 Terminated at 15ft**			
4442										
4441										
4440										
4439										

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Figure A-5

CIANI&HATCH ENGINEERING

Testpit No: CHE-TP5-24

UTM	: 12T	**Excavator**	: CAT 420	**Job Number**	: 4000-001-00
Latitude	: 42.838109	**Excavator Supplier**	: Precision Underground, LLC	**Client**	: HDR, Inc.
Longitude	: -113.639225	**Logged By**	: JAH	**Project**	: Bannock Pumping Plant Modernization
Elevation	: 4456(ft)	**Reviewed By**	: MSH	**Location**	: Fort Hall Reservation, ID, USA
Total Depth	: 12.5 ft	**Date**	: 08/08/2024	**Loc Comment**	: Siphon W

Elevation (ft)	Depth (ft)	Sample Number	Sample Type	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content	Fines Content	Remarks
4456						ML	Light brown SILT with gravel and occasional cobbles (very stiff to hard, dry) (Fill)			
4455										
4454		S-1	GRAB			ML	Light brown SILT with occasional cobbles and weakly cemented nodules (stiff to very stiff, dry)			6-inch diameter cobble encountered at 2' bgs
4453										
4452		S-2	GRAB			ML	Brown SILT with sand and organic debris (medium stiff to stiff, moist) (Loess)			
4451						CL-ML	Brown, silty, lean CLAY with sand, with organic debris (medium stiff to stiff, moist) (Alluvium)			Organic debris (wood fibers, twigs) encountered below 5' bgs
4450			GRAB							
4449										
4448										
4447										
4446		S-4	GRAB			CL-ML	Brown, silty, lean CLAY with sand, with organic debris (medium stiff to stiff, moist)			
4445		S-4	GRAB	AL		CL	Olive-gray to brown, lean CLAY with sand, with organic debris (medium stiff to stiff, moist)			
4444										
4443							**CHE-TP5-24 Terminated at 12.5ft**			
4442										

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

Figure A-6

CIANI&HATCH ENGINEERING

Testpit No: CHE-TP6-24

UTM	: 12T	**Excavator**	: CAT 420	**Job Number**	: 4000-001-00
Latitude	: 42.838151	**Excavator Supplier**	: Precision Underground, LLC	**Client**	: HDR, Inc.
Longitude	: -113.638905	**Logged By**	: JAH	**Project**	: Bannock Pumping Plant Modernization
Elevation	: 4459(ft)	**Reviewed By**	: MSH	**Location**	: Fort Hall Reservation, ID, USA
Total Depth	: 15 ft	**Date**	: 08/08/2024	**Loc Comment**	: Siphon E

Elevation (ft)	Depth (ft)	Sample Number	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content	Fines Content	Remarks
4459					ML	Light brown SILT with sand and gravel, with occasional cobbles (very stiff, dry) (Fill)			
4458									
4457									
4456					ML	Light to medium brown SILT (medium stiff to very stiff, moist) (Loess)			
		S-1							
4455									
		S-2							
4454									
4453					ML	Light to dark brown SILT, with pinhole structure (medium stiff to stiff, dry)			
4452									
4451									
4450					ML	Light to dark brown SILT, with pinhole structure (medium stiff to stiff, moist)			
		S-3							

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Figure A-7

CIANI&HATCH ENGINEERING

Testpit No: CHE-TP6-24

UTM	: 12T	**Excavator**	: CAT 420	**Job Number**	: 4000-001-00
Latitude	: 42.838151	**Excavator Supplier**	: Precision Underground, LLC	**Client**	: HDR, Inc.
Longitude	: -113.638905	**Logged By**	: JAH	**Project**	: Bannock Pumping Plant Modernization
Elevation	: 4459(ft)	**Reviewed By**	: MSH	**Location**	: Fort Hall Reservation, ID, USA
Total Depth	: 15 ft	**Date**	: 08/08/2024	**Loc Comment**	: Siphon E

Elevation (ft)	Depth (ft)	Sample Number	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content	Fines Content	Remarks
4449						Light to dark brown SILT, with pinhole structure (medium stiff to stiff, moist)			
4448									
4447					ML				
4446									
		S-4							
4445						Light brown SILT, with pinhole structure and slight plasticity (medium stiff to stiff, moist)			
		S-5			ML				
4444						**CHE-TP6-24 Terminated at 15ft**			
4443									
4442									
4441									
4440									

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

Figure A-7

CIANI&HATCH ENGINEERING

Testpit No: CHE-TP7-24

UTM	**: 12T**	**Excavator**	**: CAT 420**	**Job Number**	**: 4000-001-00**
Latitude	**: 42.838229**	**Excavator Supplier**	**: Precision Underground, LLC**	**Client**	**: HDR, Inc.**
Longitude	**: -113.639153**	**Logged By**	**: JAH**	**Project**	**: Bannock Pumping Plant Modernization**
Elevation	**: 4456(ft)**	**Reviewed By**	**: MSH**	**Location**	**: Fort Hall Reservation, ID, USA**
Total Depth	**: 12 ft**	**Date**	**: 08/08/2024**	**Loc Comment**	**: Siphon N**

Elevation (ft)	Depth (ft)	Sample Number	Sample Type	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content	Fines Content	Remarks
4456						ML	Brown, SILT with gravel (very stiff to hard, dry to moist) (Fill)			
4455										
		S-1	GRAB	AL						
4454						ML	Brown SILT (medium stiff to very stiff, moist) (Loess)			
4453		S-2	GRAB							
4452										
4451							Dark brown SILT, with organic debris (soft to medium stiff, moist) (Alluvium)			Organic debris (wood fibers, twigs) encountered below 5' bgs
4450		S-3	GRAB	CHEM						Chemistry Suite Results | pH: 8.5 | Resistivity: 2,195 Ohm-cm | Sulfates: 45 ppm
4449										
4448		S-4	GRAB			ML				
4447										
4446										
4445		S-5	GRAB							
4444							**CHE-TP7-24 Terminated at 12ft**			
4443										
4442										

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

Figure A-8

CIANI&HATCH ENGINEERING

Testpit No: CHE-TP8-24

UTM	: 12T	**Excavator**	: CAT 420	**Job Number**	: 4000-001-00
Latitude	: 42.838020	**Excavator Supplier**	: Precision Underground, LLC	**Client**	: HDR, Inc.
Longitude	: -112.639471	**Logged By**	: JAH	**Project**	: Bannock Pumping Plant Modernization
Elevation	: 4457(ft)	**Reviewed By**	: MSH	**Location**	: Fort Hall Reservation, ID, USA
Total Depth	: 3 ft	**Date**	: 08/07/2024	**Loc Comment**	: Shallow TP - Siphon W

Elevation (ft)	Depth (ft)	Sample Number	Sample Type	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content	Remarks
4457		S-1	GRAB			ML	Brown SILT with sand and gravel (very soft to very stiff, dry) (Fill)		Test pit advanced at slope to avoid utilities
4456		S-2	GRAB						
4455		S-3	GRAB						DCP testing at 3' bgs encounters refusal blowcounts at 4' bgs - hammer rebound & no advancement - suspect concrete debris/structure
4454							**CHE-TP8-24 Terminated at 3ft**		
4453									
4452									
4451									
4450									
4449									
4448									

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

Figure A-9

Appendix B.

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Soil samples obtained from the explorations were transported to the STRATA geotechnical laboratory in Boise, Idaho. STRATA examined samples for natural moisture content, Atterberg Limits (plasticity indices), collapse-consolidation characteristics, and Idaho R-Value. The results of the laboratory testing program are provided in the appended Summary of Laboratory Test Results dated September 3, 2024, and December 13, 2024. A brief description of each laboratory test performed is provided in Table B-1 below.

Table B-1. Laboratory Test Description

Test Method	Description	Test Purpose
ASTM D2216	Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass	Determine the *in-situ* moisture content of the soil sample
ASTM D2937	Standard Test Methods for Density of Soil in Place by the Drive-Cylinder Method	Determine the *in-situ* density (unit weight) of the sample
ASTM D4318	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	Determine the plasticity characteristics of the soil sample
ASTM D5333	Standard Test Method for Measurement of Collapse Potential of Soils	Estimate the collapse consolidation settlement of a soil sample
ASTM G51	Standard Test Method for Measuring pH of Soil for Use in Corrosion Evaluations	Determine the pH of a soil sample for determination of corrosivity
ASTM G57	Standard Test Method for Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	Determine the resistivity of a soil sample for determination of corrosivity
EPA 300.0	Determination Of Inorganic Anions by Ion Chromatography	Determine the sulfate concentration of a soil sample for determination of corrosivity
Idaho T-8 Method	Idaho Standard method of Test for Resistance R-Value and Expansion Pressure of Compacted Soils and Aggregates	Determinate the Idaho R-Value of a soil sample for use in pavement section design

STRATA

Summary of Laboratory Test Results

Project: Bannock Pumping Plant Modernization
Client: Ciani & Hatch Engineering

Project Number: BO24262T
Date: 9/3/2024

Sample Source	Depth (Feet)	Lab Number	Dry Unit Weight (pcf) [1]	In Situ Moisture, % [2]	Atterberg Limits [3] LL	PI	pH [4]	Resistivity [5] ohm-cm	Sulfates, [6] ppm	Fines Class.
CHE-TP1-24	3.0-4.0	55239	101.8	15.1						
CHE-TP1-24	4.0-4.5	55240		14.6						
CHE-TP2-24	5.0-6.0	55241	93.1	15.8						
CHE-TP2-24	6.0-7.0	55242					8.45	3403	37	
CHE-TP3-24	3.0-4.0	55243					8.73	5790	ND	
CHE-TP3-24	6.0-7.0	55244		11.9						
CHE-TP4-24	7.0-8.0	55245		29.1						
CHE-TP4-24	10.0-11.0	55246		31.9	33	13				CL
CHE-TP5-24	11.0-12.0	55247			47	28				CL
CHE-TP7-24	1.5-2.0	55248					8.5	2195	45	
CHE-TP7-24	6.0-7.0	55249		14.3						

Reviewed By: Keith Wildman
Keith Wildman
Laboratoy Services Coordinator

Notes:
1- Dry Unit Weight tested in accordance with ASTM D2937
2- In-Situ Moisture content testing performed in accordance with ASTM D2216
3- Atterberg Limits testing performed in accordance with ASTM D4318
4- pH testing performed in accordance with ASTM G51
5- Resistivity testing performed in accordance with ASTM G57
6- Sulfates testing performed in accordance with EPA 300.0

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8653 West Hackamore Drive, Boise, Idaho 83709 Phone 208.376.8200 Fax 208.376.8201
www.stratageotech.com

ASTM D5333

Report Date: 9/03/2024

Boise
8653 W. Hackamore Dr
Boise, ID 83709
Phone: 208.376.8200

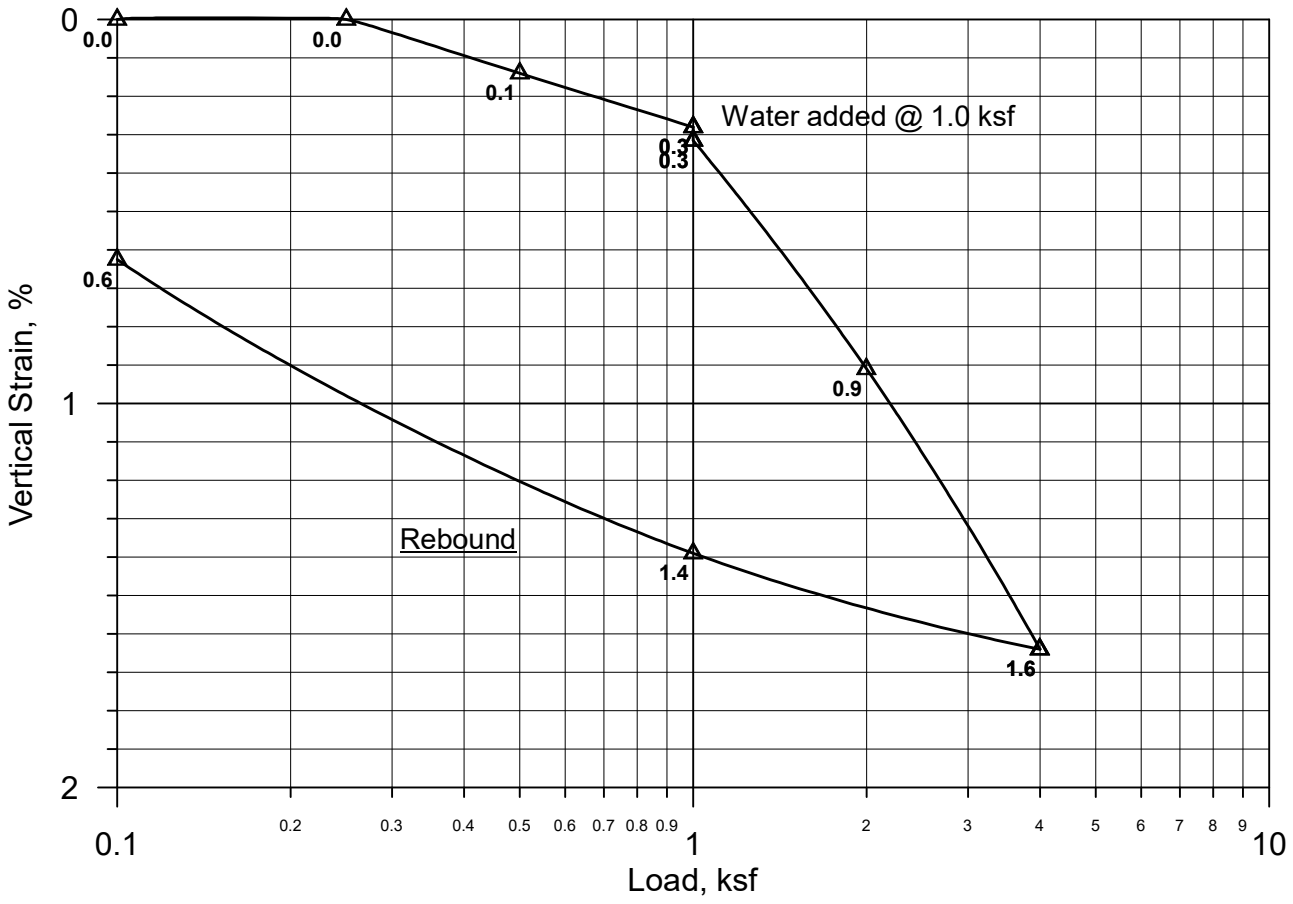
Client
Ciani & Hatch Engineering

Project
BO24262T
Bannock Pumping Plant Modernization

SAMPLE INFORMATION

SAMPLE No.:	55239	**SAMPLE DATE:**	8/7/2024	**TEST PIT No.:**	CHE-TP1-24
SPECIMEN SIZE:	Shelby Tube	**TEST DATE:**	8/26/2024	**DEPTH (FT):**	3.0-4.0
USCS CLASSIFICATION:	Silt with Sand (ML)	**SAMPLED BY:**	Client	**SAMPLE CONDITION:**	Good
		TESTED BY:	V. Barinaga		

APPARATUS:	Humboldt Load Frame	**INITIAL MOISTURE CONTENT:**	15.1%
SAMPLE PREP:	Intact	**FINAL MOISTURE CONTENT:**	20.6%
TEST WATER:	Tap	**DRY UNIT WEIGHT:**	101.8 pcf



REMARKS: PP Reading - 4.25

Reviewed by Keith Wildman
Laboratory Services Coordinator

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ASTM D5333

Report Date: 9/03/2024

Boise
8653 W. Hackamore Dr
Boise, ID 83709
Phone: 208.376.8200

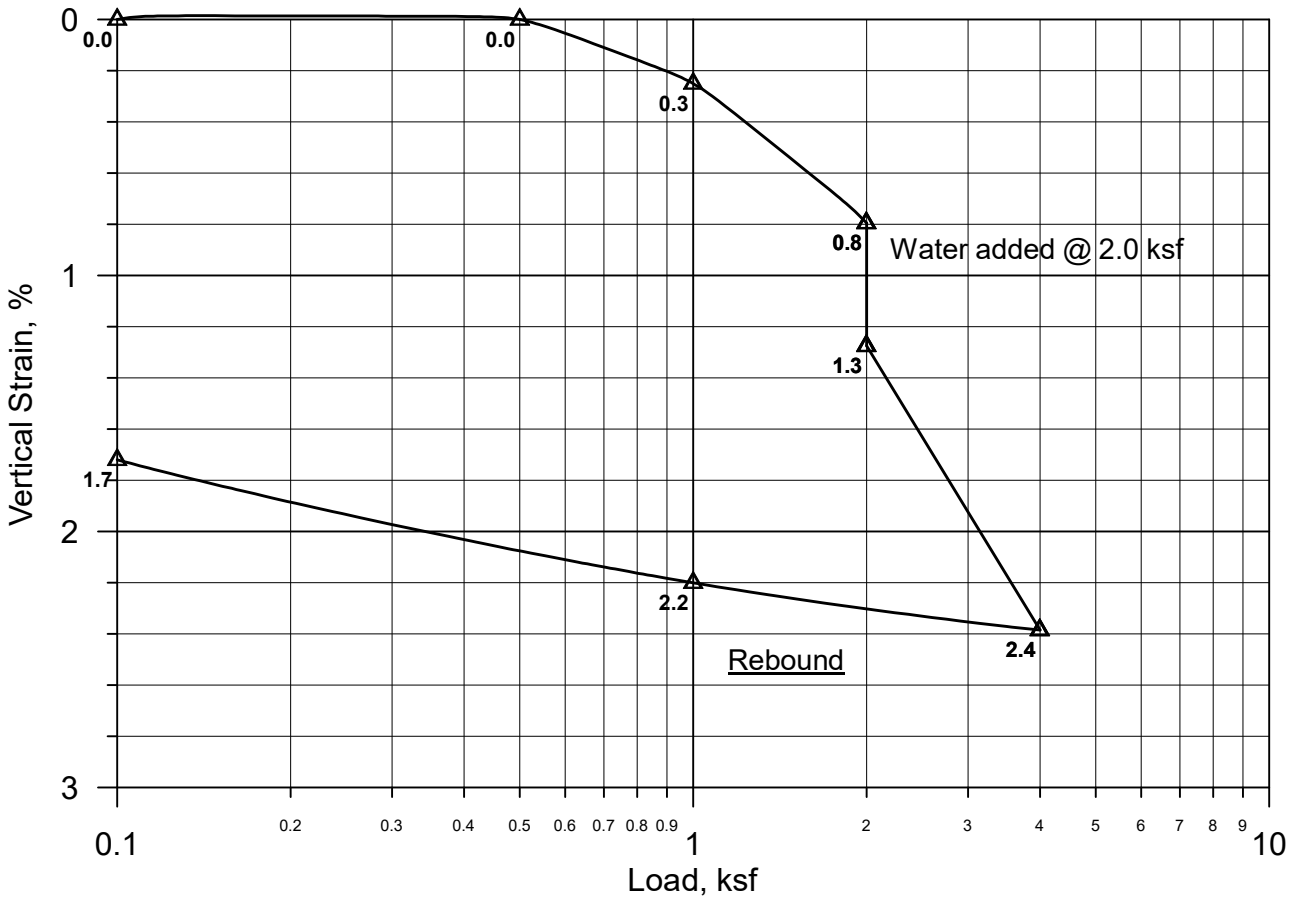
Client
Ciani & Hatch Engineering

Project
BO24262T
Bannock Pumping Plant Modernization

SAMPLE INFORMATION

SAMPLE No.:	55241	**SAMPLE DATE:**	8/07/2024	**TEST PIT No.:**	CHE-TP2-24
SPECIMEN SIZE:	Shelby Tube	**TEST DATE:**	8/26/2024	**DEPTH (FT):**	5.0-6.0
USCS CLASSIFICATION:	Silt with Sand (ML)	**SAMPLED BY:**	Client	**SAMPLE CONDITION:**	Good
		TESTED BY:	V. Barinaga		

APPARATUS:	Humboldt Load Frame	**INITIAL MOISTURE CONTENT:**	15.8%
SAMPLE PREP:	Intact	**FINAL MOISTURE CONTENT:**	26.0%
TEST WATER:	Tap	**DRY UNIT WEIGHT:**	93.1 pcf



REMARKS: PP Reading - >4.5

Reviewed by Keith Wildman
Laboratory Services Coordinator

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Boise
8653 W. Hackamore Drive
Boise, ID 83709
Phone: 208.376.8200

Idaho T-8

Report Date: 12/13/2024

Client
Ciani & Hatch Engineering

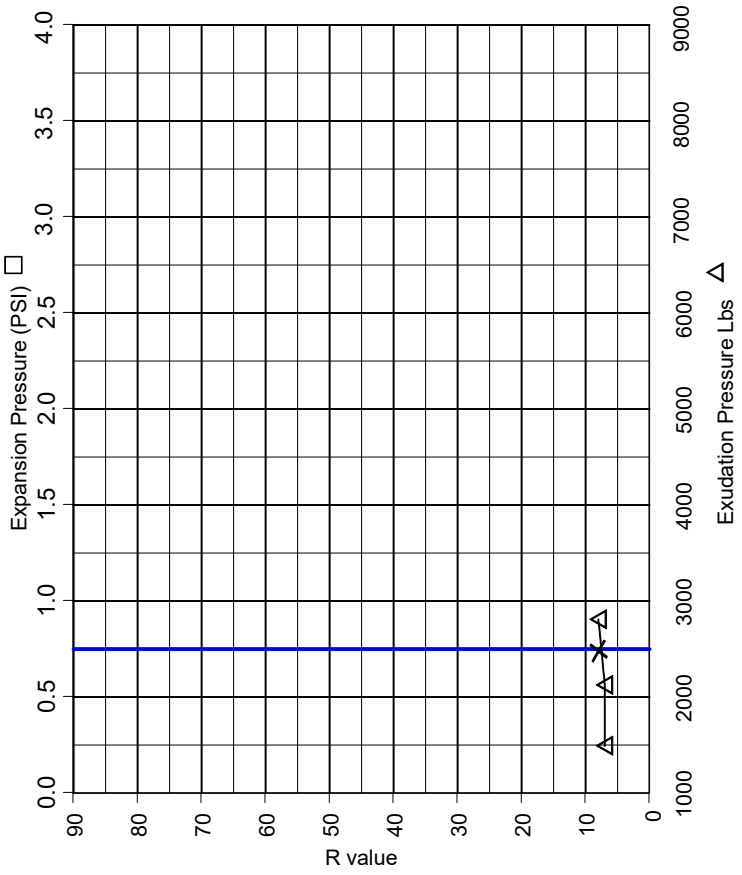
Project
BO24262T
Bannock Pumping Plant Modernization

SAMPLE INFORMATION

SAMPLE No.:	57803	**SAMPLE DATE:**	8/7/2024	**LOCATION.:**	Roadway
SPECIMEN SIZE:	Bulk	**TEST DATE:**	12/12/2024	**DEPTH (FT):**	0.25-1.0
USCS CLASSIFICATION:	Sandy Silt (ML)	**SAMPLED BY:**	Client		
		TESTED BY:	V. Barinaga		

METHOD: Bulk Exploration

R-VALUE: 8



R VALUE DATA

Percolation: Fast	Point 1	Point 2	Point 3
Exudation, PSI	119	169	223
Dry Density, PCF	93.2	95.8	95.7
Moisture Content, %	25.6	24.4	23.1
Exp. Pressure, PSI	0.00	0.34	0.49

GRADATION: AASHTO T-11, T27

SCREEN SIZE	AS RECEIVED % PASSING	AS TESTED % PASSING
4"		
3"		
2"		
1"		
3/4"		
1/2"		
3/8"		
No. 4	100	100
No. 8		
No. 16		
No. 30		
No. 50		
No. 100		
No. 200		

Reviewed by Keith Wildman
Laboratory Services Coordinator

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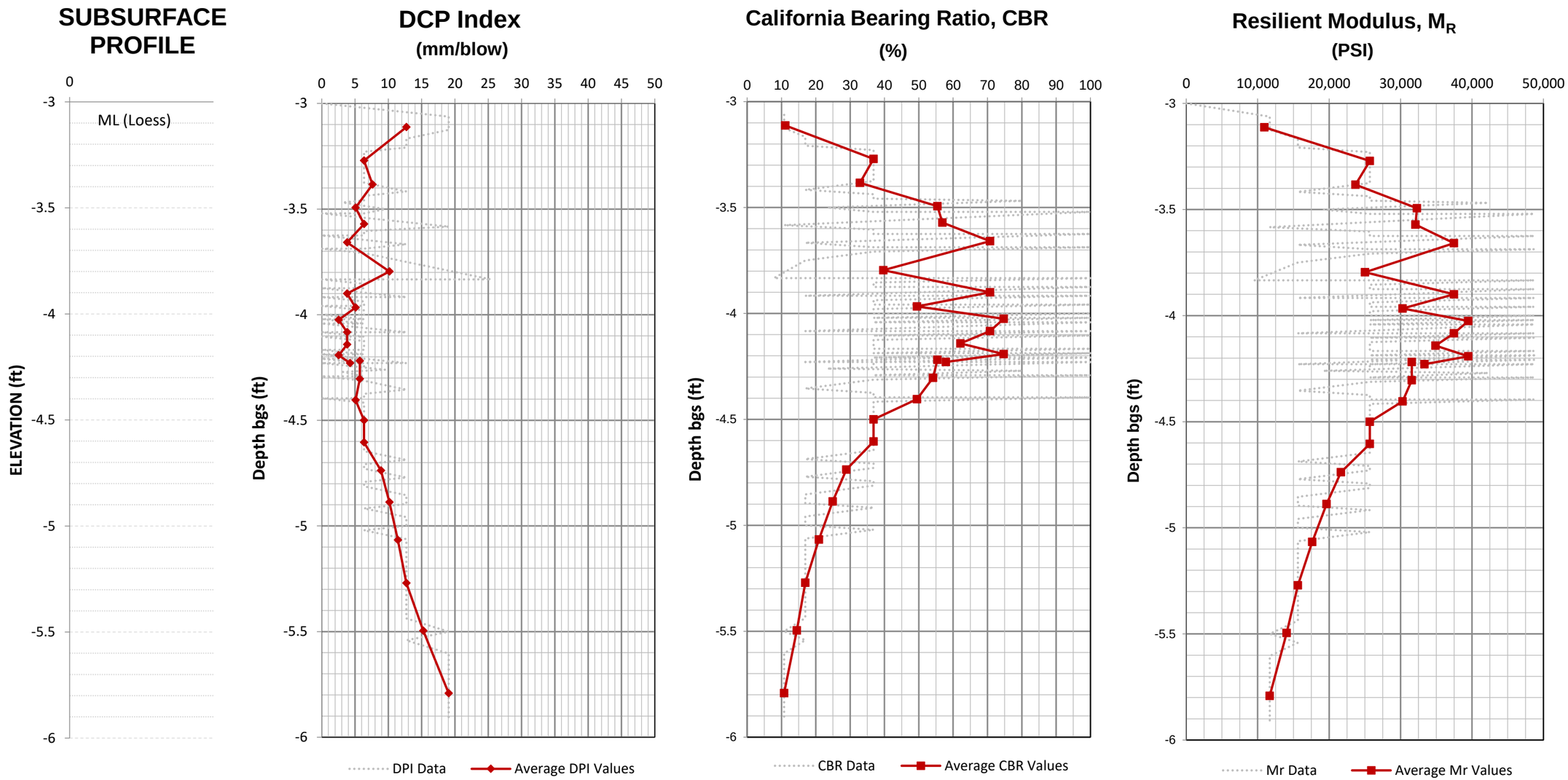
Appendix C.

DYNAMIC CONE PENETROMETER ANALYSIS

APPENDIX C

DYNAMIC CONE PENETROMETER ANALYSIS

Dynamic Cone Penetrometer (DCP) testing was performed as part of our subsurface exploration program in all test pits except CHE -TP3-24. DCP testing was performed in general accordance with ASTM D6951. The results of the DCP testing are presented on Figures C-1 through C-7 and include penetration index (DPI) and estimated values for California Bearing Ratio (CBR) and Resilience Modulus (M_R) based on correlations provided in ASTM D6951.



Notes:

1. California Bearing Ratio (CBR) and the Resilient Modulus (M_R) is correlated in accordance with ASTM D6951 and NCHRP 1-37A (2004).
2. Ciani & Hatch Engineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.

CIANI & HATCH ENGINEERING

Dynamic Cone Penetrometer Results

CHE-TP1-24

Fort Hall Irrigation Improvement Project

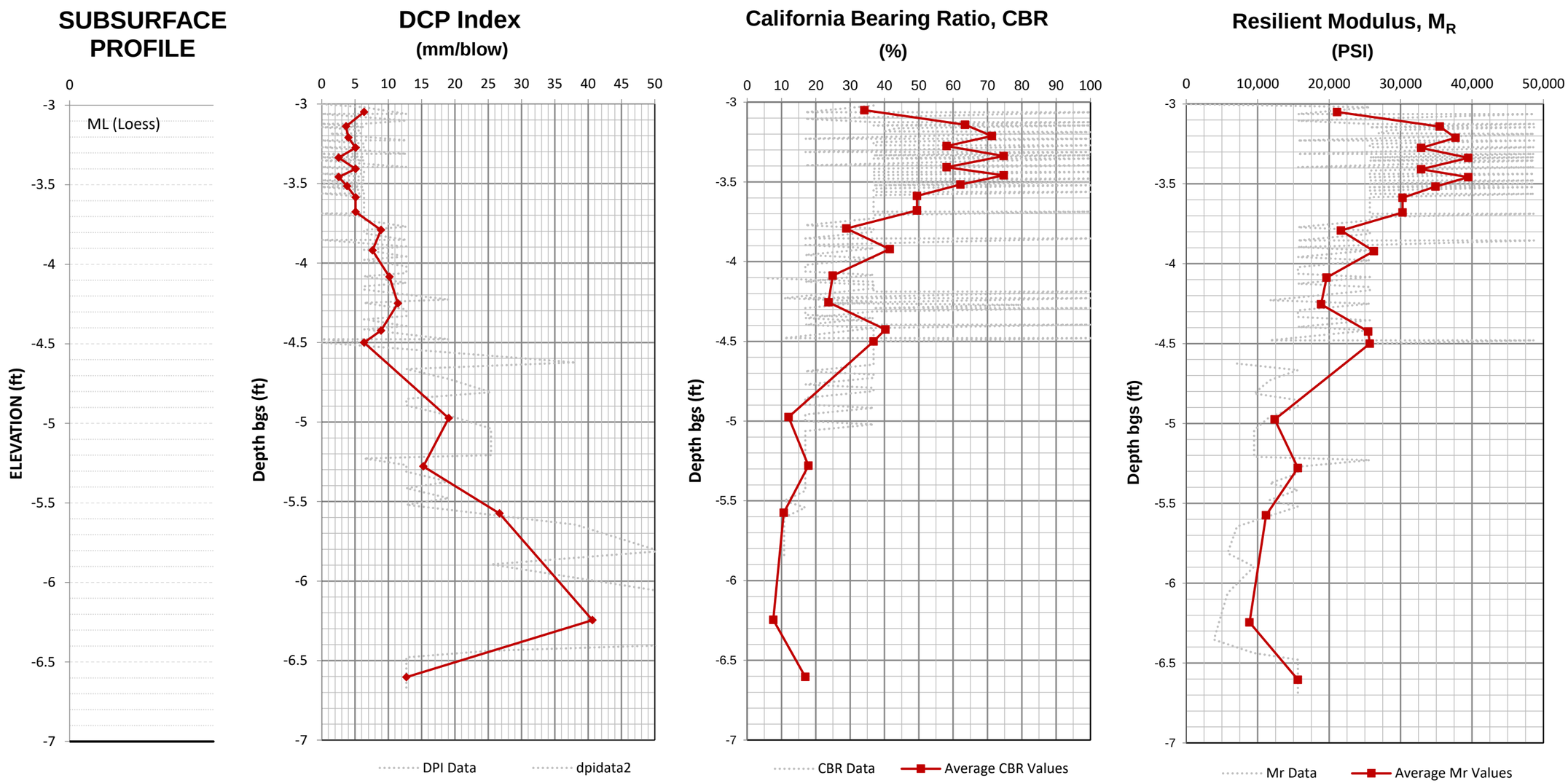
4000-001-00

Client: HDR, Inc.

FIGURE

C-1

Project Number: 4000-001-00 Date Exported: 09/06/2024



Notes:

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CIANI & HATCH ENGINEERING

Dynamic Cone Penetrometer Results

CHE-TP2-24

Fort Hall Irrigation Improvement Project

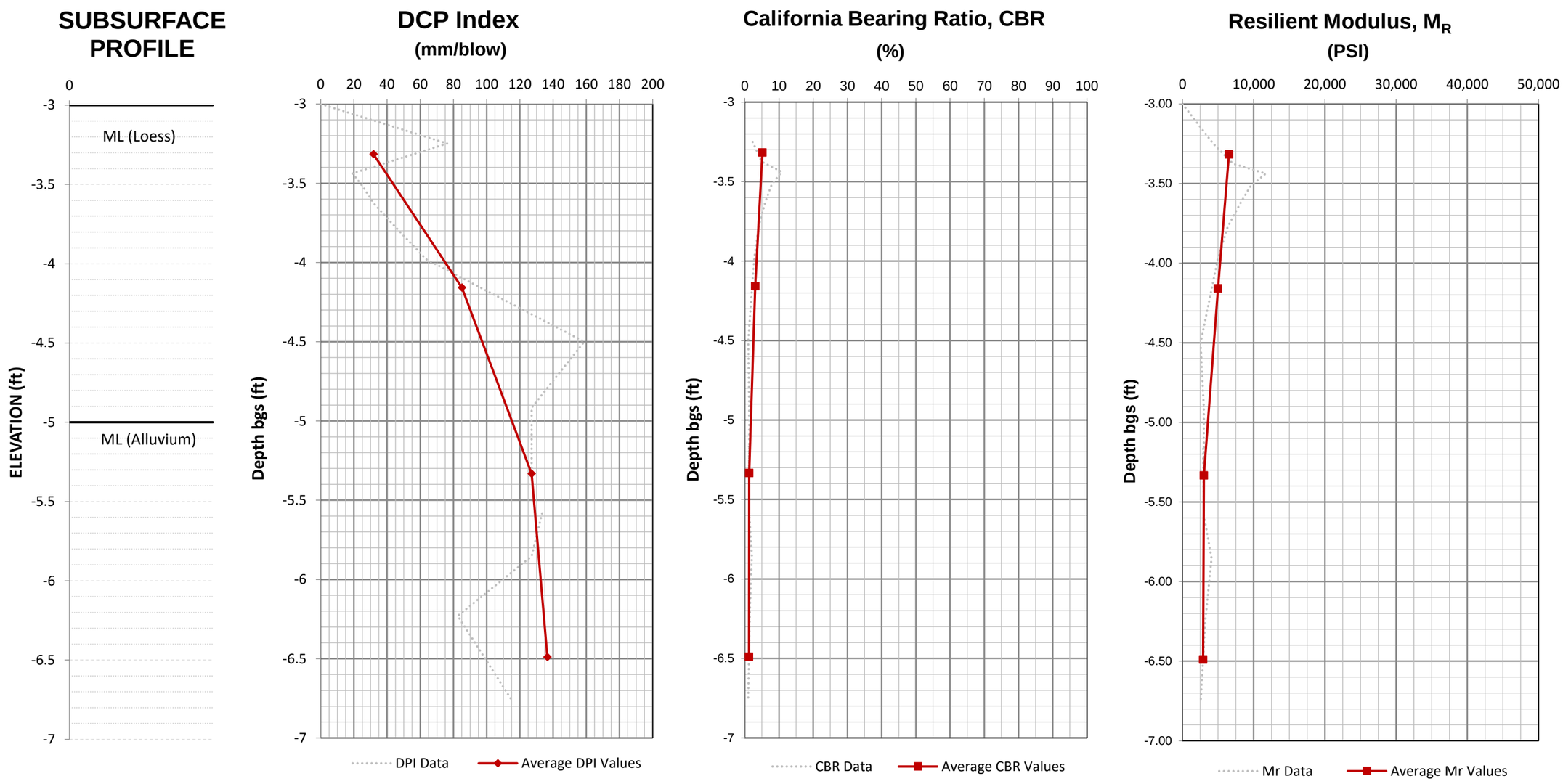
4000-001-00

Client: HDR, Inc.

FIGURE

C-2

Project Number: 4000-001-00 Date Exported: 09/06/2024



Notes:

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CIANI & HATCH ENGINEERING

Dynamic Cone Penetrometer Results

CHE-TP4-24

Fort Hall Irrigation Improvement Project

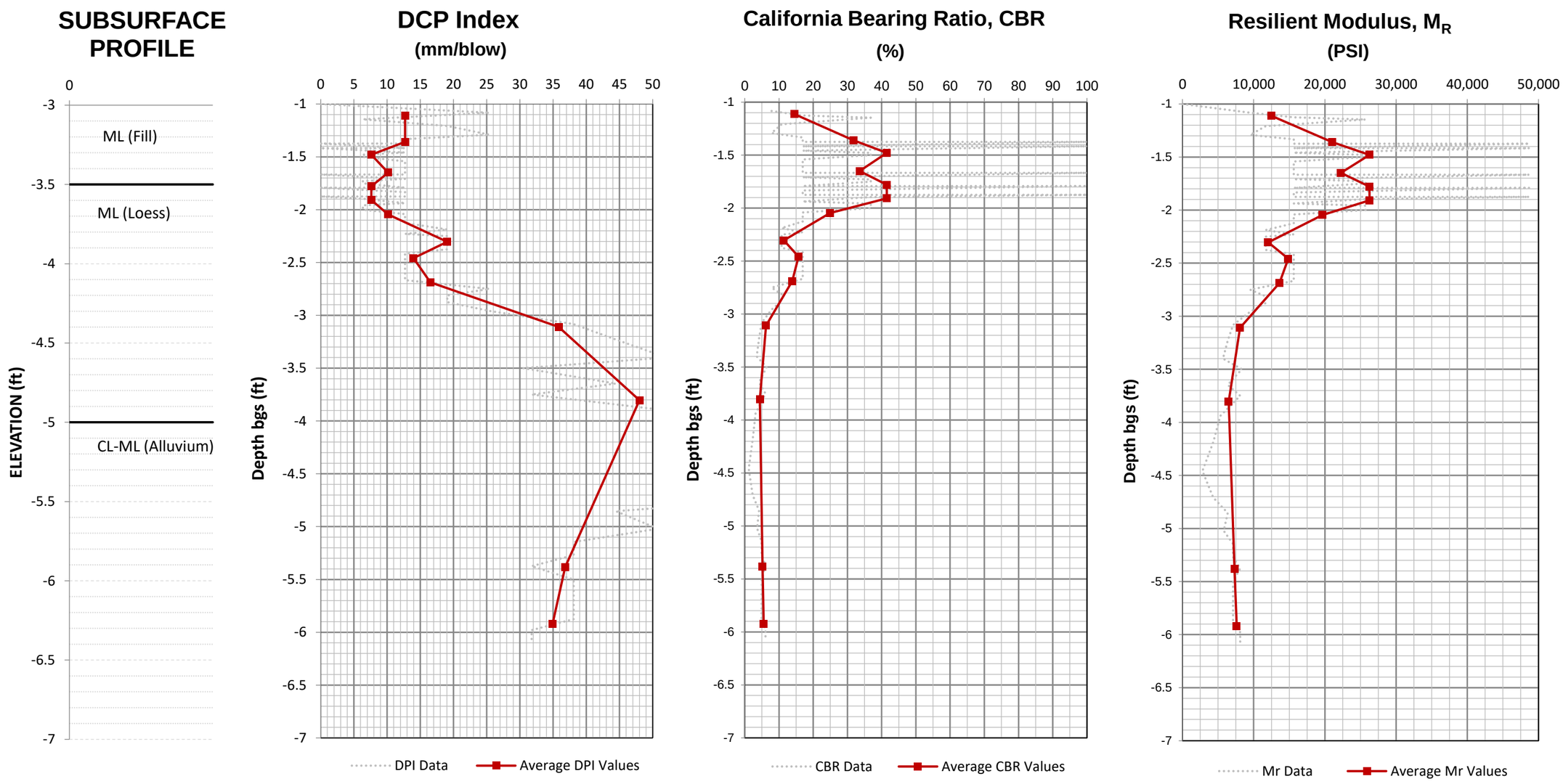
4000-001-00

Client: HDR, Inc.

FIGURE

C-3

Project Number: 4000-001-00 Date Exported: 09/06/2024



Notes:

1. California Bearing Ratio (CBR) and the Resilient Modulus (M_R) is correlated in accordance with ASTM D6951 and NCHRP 1-37A (2004).
2. Ciani & Hatch Engineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.

CIANI & HATCH ENGINEERING

Dynamic Cone Penetrometer Results

CHE-TP5-24

Fort Hall Irrigation Improvement Project

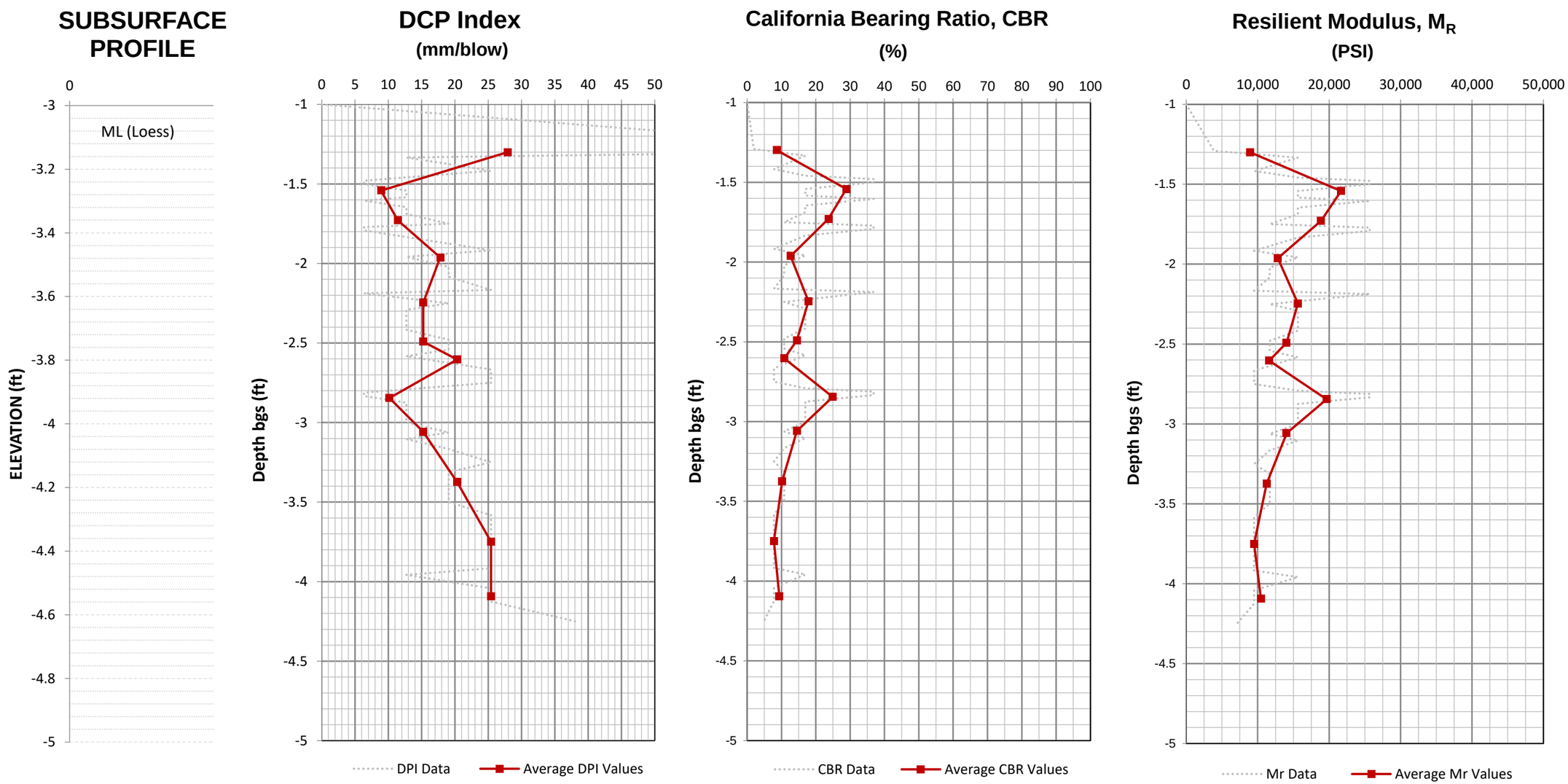
4000-001-00

Client: HDR, Inc.

FIGURE

C-4

Project Number: 4000-001-00 Date Exported: 09/06/2024



Notes:

1. California Bearing Ratio (CBR) and the Resilient Modulus (M_R) is correlated in accordance with ASTM D6951 and NCHRP 1-37A (2004).
2. Ciani & Hatch Engineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.

CIANI & HATCH ENGINEERING

Dynamic Cone Penetrometer Results

CHE-TP6-24

Fort Hall Irrigation Improvement Project

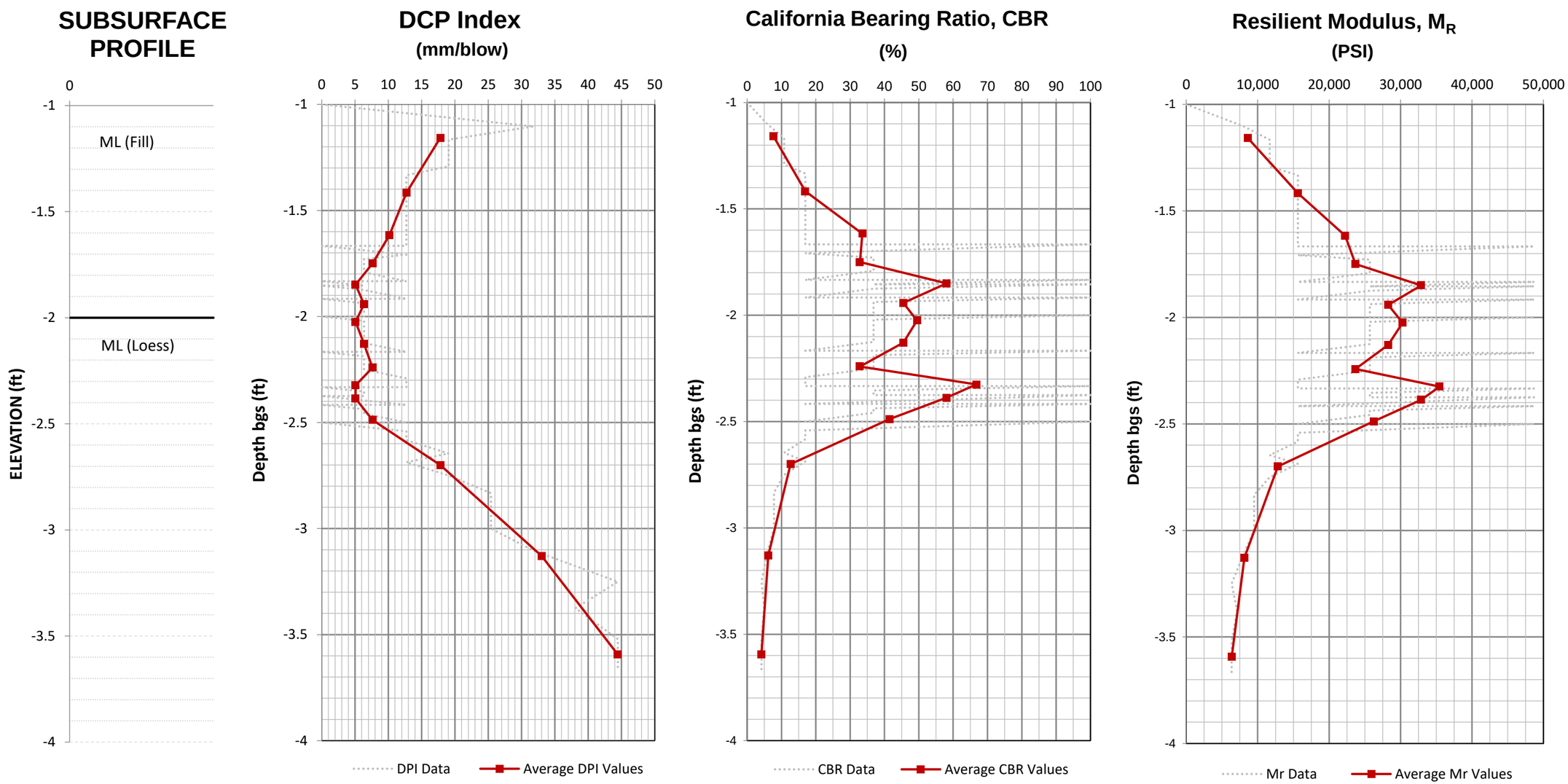
4000-001-00

Client: HDR, Inc.

FIGURE

C-5

Project Number: 4000-001-00 Date Exported: 09/06/2024



Notes:

1. California Bearing Ratio (CBR) and the Resilient Modulus (M_R) is correlated in accordance with ASTM D6951 and NCHRP 1-37A (2004).
2. Ciani & Hatch Engineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.

CIANI & HATCH ENGINEERING

Dynamic Cone Penetrometer Results

CHE-TP7-24

Fort Hall Irrigation Improvement Project

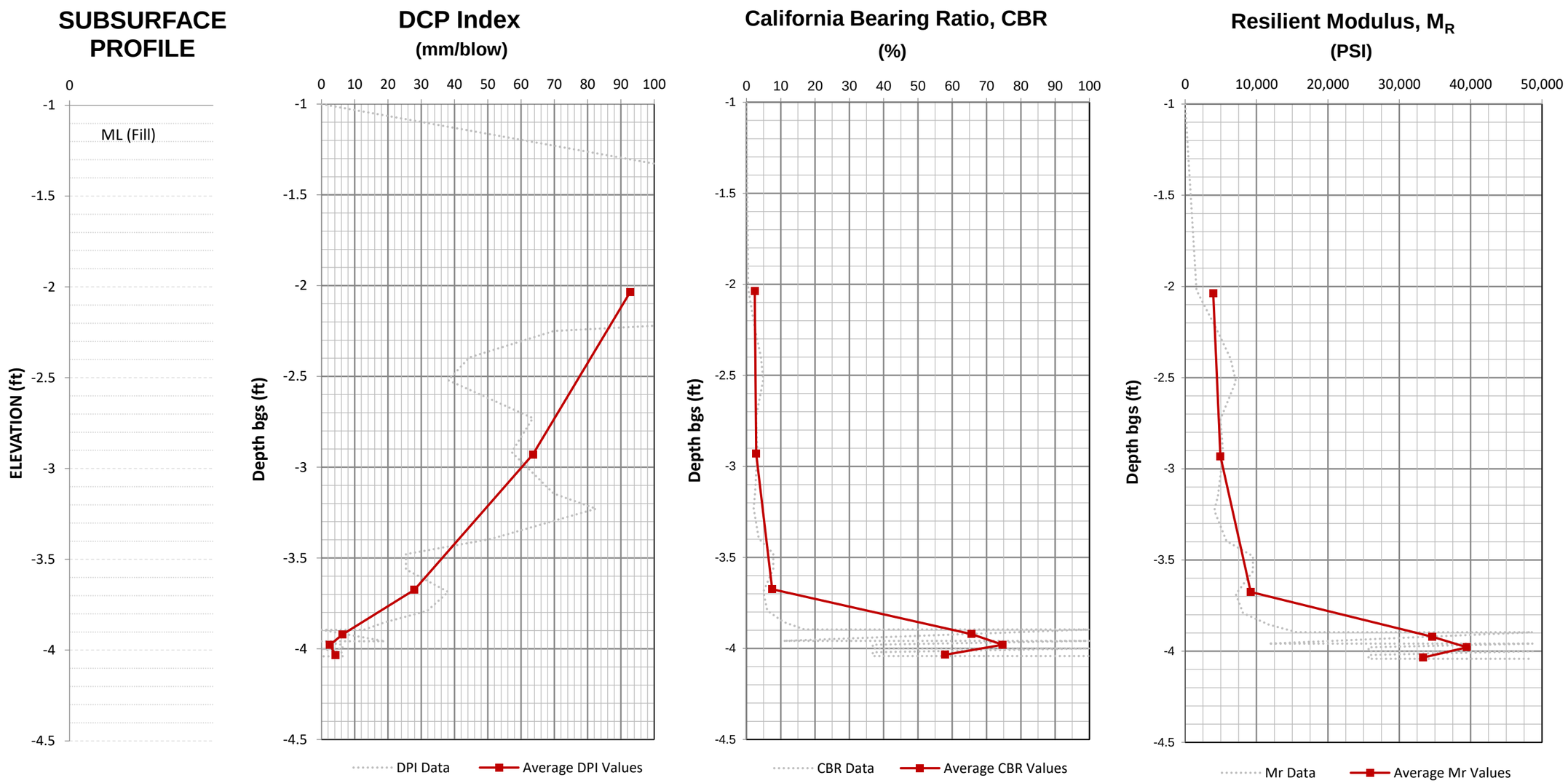
4000-001-00

Client: HDR, Inc.

FIGURE

C-6

Project Number: 4000-001-00 Date Exported: 09/06/2024



Notes:
1. California Bearing Ratio (CBR) and the Resilient Modulus (M_R) is correlated in accordance with ASTM D6951 and NCHRP 1-37A (2004).
2. Ciani & Hatch Engineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.

CIANI & HATCH ENGINEERING

Dynamic Cone Penetrometer Results

CHE-TP8-24

Fort Hall Irrigation Improvement Project

4000-001-00

Client: HDR, Inc.

FIGURE

C-7

Project Number: 4000-001-00 Date Exported: 09/06/2024