GEOTECHNICAL INVESTIGATION

Mt Pleasant Sewer Lagoons

New Headworks Building

Sanpete County, Utah

Prepared for:
J-U-B Engineers

December 2024

RB&G ENGINEERING, INC.

GEOTECHNICAL INVESTIGATION

MT PLEASANT SEWER LAGOONS
NEW HEADWORKS BUILDING
MT PLEASANT, UTAH

1 INTRODUCTION

This report outlines the results of a geotechnical investigation conducted for the proposed new headworks building at the sewer lagoons located southwest of Mt Pleasant, in Sanpete County, Utah.

The information contained in this report is discussed under the following headings: Geological and Existing Site Conditions, Subsurface Soil and Water Conditions, Foundation Recommendations, and Site Preparation and Compacted Fill Requirements.

2 GEOLOGICAL AND EXISTING SITE CONDITIONS

Figure 1 is an aerial vicinity map showing the location of the site relative to the surrounding area. The sewer lagoon site is bordered on the north by 1000 S street and on the west by 1650 W street (both unpaved), and on the southeast by the former Denver and Rio Grande Western railroad alignment. The Mt Pleasant Airport, which closed in 2016, was located on the east side of the rail line across from the sewer lagoons. The site includes two active sewer lagoon ponds with surface areas of about 13 to 20 acres each, and two smaller ponds (less than about 12 acres) which appear (from aerial photos dating back to 1985) to store water less frequently. The proposed headworks building is located near the northeast corner of the site, near the location of an existing shed with plan dimensions in the order of 10 by 12 feet.

The natural soils mapped across the site and near vicinity consist of Holocene to Pliocene coalesced alluvial-fan deposits formed by overlapping and interfingering of adjacent alluvial fans.

This unit forms broad, low, sloping aprons at the foot of adjacent highlands and generally include silt, sand, granules, pebbles, cobbles, and sparse boulders. The thickness of this unit is uncertain, but may be as great as about 100 feet locally. The San Pitch River flows from northeast to southwest across the valley floor about two miles northwest of the site, with a broad floodplain (approx. one mile wide) mapped as Holocene stream alluvium. The hills about two miles west of the site are mapped as limestone and shale of the Green River Formation (Eocene), and the mountains east of Mt Pleasant include Flagstaff Limestone (Eocene and Paleocene) and fluvial rocks of the North Horn Formation (Paleocene to Upper Cretaceous). No faults are known to cross the site. The Fairview Diapiric(?) Fold is inferred on the geologic map as crossing through the valley about ¼-mile east of the site. This feature has been attributed to possible flexural failure of rocks along a fault lining the west flank of the Wasatch Plateau, which is theorized to have opened a conduit for the rise of confined and compressed salt which formed the fold.[1]

The nearest quaternary fault is the Gunnison fault, located about 9 miles west of the site near the base of the San Pitch Mountains. Several older faults are located within the Wasatch Plateau and associated mountain ranges east of Mt Pleasant.

The natural terrain at site is relatively flat with the ground sloping gently down to the west toward the San Pitch River. We have not reviewed existing plans nor topographic survey data for the sewer lagoons, but field observations estimate they may be excavated about 15 feet below the natural ground surface elevation at the site. Site vegetation consists of sparse weeds and grass cut down by grazing of livestock. At the time of our field investigations (November 14, 2024), the adjacent sewer lagoon was full and water was flowing in an irrigation ditch along the north side of 1000 S street, roughly 250 feet north of the proposed building site. A review of the Utah Division of Water Rights water well map server shows more than a dozen water wells have been drilled within a 5,000-foot radius of the site, and the well logs indicate water-bearing coarse grained soils (sands and gravels) were often encountered at various depths between about 30 and 100 feet below the ground surface.

No indications were noted of substandard foundation performance of the minor existing structure on the site. The aged concrete flatwork lining the north side of the existing headworks/inlet appears to have experienced some settlement.

[1] *Geologic Map of the Nephi 30' x 60' Quadrangle, Carbon, Emery, Juab, Sanpete, Utah, and Wasatch Counties, Utah*, 1991, by Irving J. Witkind and Malcolm P. Weiss, U.S. Geological Survey Map I-1937.

Other than the information provided above, no surface conditions appear to exist at the site which would adversely affect foundation performance.

3 SUBSURFACE SOIL AND WATER CONDITIONS

The characteristics of the subsurface material were evaluated by drilling and sampling in one geotechnical boring which extended to a depth of 35 feet below the existing ground surface. The approximate location of this boring is shown on Figure 2. The log for the boring is included in the appendix. The longitude and latitude shown on the log were estimated from hand-held equipment and/or online mapping tools and should be considered approximate.

Details of the methods, equipment, and tooling used in the subsurface exploration are included in the appendix to this report.

The soil profile encountered in Boring 24-1 included gravel with silt and sand (USCS symbol GP-GM) fill in the upper 8 inches, underlain by sandy lean clay (CL) to lean clay with sand to a depth of about 5 feet below the existing ground surface. This clay layer was very stiff, with an unconfined compressive strength of about 4,000 psf and a pocket penetrometer value greater than 4.5 tsf. The dry density of an intact sample of the clay was 101.2 pcf. This sample had a liquid limit of 30, a plasticity index of 13, and a moisture content of 15 percent. A gradation test conducted on the clay resulted in 1 percent gravel, 25 percent sand, and 74 percent finer than the 200 (0.075-mm) sieve.

The compressibility characteristics of the clay were evaluated by performing a consolidation test on the sample obtained between depths of 3 and 4 feet. The test results suggest the clay is normally consolidated, and subject to significant compression under loading exceeding about 1,000 psf. During the consolidation test, the sample was inundated and permitted to absorb water after the initial 0.25-tsf loading was applied to evaluate the effects of moisture on its compressibility characteristics. Expansive soils experience an increase in void ratio upon absorbing water, while collapsible soils exhibit an abrupt decrease in void ratio upon wetting. The sample tested for this project did not exhibit expansive nor collapsible characteristics.

The resistivity, pH, chloride content, sulfate content, and soluble salt content were tested to evaluate electrochemical characteristics of the lean clay obtained immediately beneath the surficial gravel fill. The tested sample tested had a pH of 8.0, resistivity of 2,500 ohm-cm, water-soluble chloride content of 42 mg/kg-dry (ppm), water-soluble sulfate content of 30 mg/kg-dry., and 2.82 percent soluble salts.

Below the clay, the boring encountered predominantly granular soils consisting of clayey sand (SC), silty sand (SM), silty clayey sand (SC-SM), and silty gravel (GM); however, a layer of sandy silt (ML) up to about 2 feet thick was noted between depths of 12 and 14 feet. Standard Penetration Test (SPT) blow counts in the sands and gravels suggest they are relatively dense to very dense. The sands and gravels were observed to be partially indurated (cemented or hardened) and bordering on weathered and/or crushed sandstone with increasing depth in the boring. Indications of possible cobbles and boulders were noted within this zone. Drilling and sampling observations made between depths of about 23 and 35 feet suggested sandstone bedrock may exist within these depths.

The sandy silt (ML) obtained at a depth of about 12.5 feet had a dry density of 110.3 pcf, a moisture content of 15.4 percent, and gradation characteristics of 1 percent gravel, 40 percent sand, and 59 percent non-plastic fines. An unconfined compression test was conducted on this sample to a failure load of 544 psf.

The partially indurated silty sand (SM) with gravel at 20 feet had a moisture content of 10.2 percent, with 34 percent gravel, 40 percent sand, and 26 percent finer than the No. 200 sieve. The silty clayey sand (SC-SM) with gravel (possible bedrock) at 30 feet had a moisture content of 7.2 percent, a liquid limit of 18, and a plasticity index of 4, and contained 36 percent gravel, 43 percent sand, and 21 percent fines.

The depth to groundwater was initially measured at about 20 feet below the ground surface upon completion of drilling, and had fallen below 33 feet before the geologist left the project site. A 1-inch diameter slotted PVC pipe was left in the boring to allow further verification of subsurface groundwater conditions if desired.

4 ENGINEERING ANALYSIS AND RECOMMENDATIONS

4.1 SEISMIC CONSIDERATIONS

The site is located at latitude 39.53121° North and longitude 111.47864° West. Based upon the site subsurface conditions and mapped geology, we recommend Site Class D be used for seismic design in accordance with ASCE 7.

The following table lists the calculated seismic design parameters in accordance with ASCE 7-16.[2]

Parameter	Value	Description
S_S:	0.591	MCE_R ground motion (period 0.2s)
S_1:	0.194	MCE_R ground motion (period 1.0s)
F_a:	1.328	Site Class D site amplification factor (period 0.2s)
F_v:	2.212	Site Class D site amplification factor (period 1.0s)
S_{MS}:	0.784	Site-modified spectral acceleration value (period 0.2s)
S_{M1}:	0.429	Site-modified spectral acceleration value (period 1.0s)
S_{DS}:	0.523	Numeric seismic design value at 0.2s SA
S_{D1}:	0.286	Numeric seismic design value at 1.0s SA
T_L:	8 seconds	Long-period transition period
PGA:	0.266	MCE_G peak ground acceleration
PGA_M:	0.354	Site-modified peak ground acceleration
F_{PGA}:	1.334	Site Class D site amplification factor for PGA

The site peak ground acceleration for liquefaction design (PGA_M) is greater than 0.30g, and could cause liquefaction in relatively loose saturated granular soils. However, the potential for significant liquefaction to affect seismic performance of the site facilities is low because (1) the granular soils within the depth investigated are relatively dense and/or indurated, and (2) groundwater does not appear to exist within 30 feet of the ground surface.

4.2 FOUNDATION TYPES AND BEARING CAPACITIES

We understand preliminary plans for this anticipate a concrete masonry block building with a slab on grade floor near the existing ground surface elevation and a rectangular footprint measuring approximately 40 feet by 20 feet. It has been assumed in preparing this report that wall foundation loads will not exceed 2 kips per foot.

We recommend all exterior foundations be located at a depth below finished grade sufficient to provide frost protection, which is about 2.5 feet in this area. We recommend all footings be at least 24 inches wide.

Although the sulfate content test performed for this investigation indicates low potential for sulfate attack in the native site soils, we recommend Type II Portland cement (or an equivalent blended cement with moderate sulfate resistance) be used for concrete that will be in contact with the site soils. The resistivity and chloride test results suggest the natural site soils are relatively non-aggressive in terms of corrosion; however, selection of piping and other potentially metallic project

[2] https://asce7hazardtool.online/

components should also consider potential corrosivity of wastewater, as well as the possible use of de-icing salts once the facility and site are developed further.

The native clay soils in the upper 5 feet of the soil profile are relatively stiff, with an allowable bearing capacity of about 4,000 psf. However, we recommend the allowable bearing capacity for design of structural foundations supported directly on the native clay soils be limited to 1,000 psf to minimize settlements of the footings. For the minimum recommended footing width of 2 feet, the allowable bearing capacity for design may be increased to 1,500 psf by placing a 12-inch thickness of structural fill beneath the footings, to 1,750 psf with 18 inches of structural fill, or to 2,000 psf with 24 inches of structural fill.

Recommended bearing resistance for foundations wider than 2 feet placed on structural fill can be provided if needed. The width of the over-excavation and structural fill section beneath the footing should extend at least 0.6*z* beyond the footing on all sides, where *z* is the thickness of the structural fill placed beneath the footing. For example, a 2-foot wide footing designed using an allowable bearing capacity of 1,500 psf should be placed on structural fill measuring at least 12 inches thick and at least 3.2 feet wide, with the footing centered over the structural fill.

The structural fill should consist of relatively well graded sandy gravel having a maximum size of 3 inches, with between 5 and 20 percent passing a No. 200 sieve. At least 70 percent should be finer than ¾ inch. The portion passing the No. 40 sieve should have a plasticity index less than 6. Structural fill should be placed in lifts no thicker than 12 inches (loose thickness), moisture conditioned to within 2 percent of the optimum moisture content, and compacted to an in-place dry unit weight of at least 95 percent of the maximum laboratory density as determined by ASTM D 1557.

4.3 LATERAL EARTH PRESSURES

It is not anticipated that earth-retaining structures will be required for the proposed facility. If earth-retaining structures are required, however, and if backfilling is performed using granular material and the backfill behind the wall is finished to be horizontal (i.e., no surcharge nor backslope behind the wall), we recommend that the earth pressures be calculated using the following equation, along with the earth pressure coefficient outlined below:

$$P = \tfrac{1}{2} \gamma K H^2$$

Where P = total lateral force on wall, plf
K = earth pressure coefficient
γ = unit weight of soil (130 pcf)
H = height of retained soil against wall

The earth pressure coefficient appropriate for use in designing retaining structures depends on whether the wall is free to move or restrained during backfilling operations. If the wall is free to move during backfilling operations and the backfill material is granular soil, we recommend an active earth pressure coefficient of 0.30 be used in the above equation to calculate the lateral earth pressures. If the wall is restrained from any movement during backfilling and the backfill material is granular soil, we recommend an at-rest earth pressure coefficient of 0.45 be used to calculate the lateral earth pressure. We recommend a passive earth pressure coefficient of 3.3 be used where the granular soil is used to restrain lateral movement.

For the design seismic event, the additional active earth pressure due to ground acceleration may be estimated using a coefficient of 0.12. The seismic ground motion will reduce the available passive resistance. This reduction may be accounted for as an earth pressure acting in the direction opposite the passive resistance and computed using a coefficient of 0.3. The pressure diagrams for these forces may be roughly approximated as inverted triangles, such that the resultant forces of the seismic components act at heights of approximately 2H/3 above the base of the wall.

It should be recognized that the pressures calculated by the above equation are earth pressures only and do not include hydrostatic pressures. Where hydrostatic pressures may exist behind a retaining structure, we recommend either the wall be designed to resist hydrostatic pressure, or a drainage system be placed behind the wall to prevent the development of hydrostatic pressures.

4.4 FLOOR SLABS

We recommend that at least 8 inches of granular fill be placed under the slab-on-grade floor. The granular fill should have a maximum size of 2 inches and non-plastic fines. If this recommendation is complied with, a free draining granular layer beneath floor slabs is not required. An alternative is to use site grading fill (see Section 5 below) for the bottom half of the granular layer and replace the top 4 inches with free draining gravel. The free draining layer should have a maximum size less than 1 inch and not more than 5 percent passing the No. 200 sieve. The free draining material should be densified using at least 4 passes of a smooth drum 5-ton vibratory roller or equivalent. If the above specifications are followed, the granular layer will prevent the accumulation of moisture beneath the floor slab and will also serve adequately as a base beneath the floor slabs.

If moisture-sensitive flooring, such as tile, is planned for slab-on-grade floors, a vapor retarder/barrier should be placed directly beneath the concrete floor slab in lieu of a free-draining granular layer. The vapor barrier should meet ASTM E 1745 Class A requirements.

A subgrade modulus of 150 pci may be assumed for design of slab-on-grade concrete floors.

5 SITE PREPARATION AND COMPACTED FILL REQUIREMENTS

We recommend the upper 8 inches be stripped from the area to remove excessive organic matter in the upper portion of the soil profile. Brush, including roots, should be removed where encountered within building area and adjacent areas to receive flatwork or pavement.

Project excavations should be performed with a smooth-edged bucket to avoid disturbing the subgrade clayey soils that will remain in place beneath structures and compacted fill. Where disturbance occurs or is observed at the foundation subgrade level during excavation, the foundation area should be over-excavated and replaced with structural fill. We recommend a qualified geotechnical engineer observe all footing excavations to help identify any disturbed soils that should be replaced before placement of foundation reinforcement and concrete. To help ensure uniform support exists across the bearing surface where over-excavation is performed, a uniform thickness of structural fill should be placed beneath the entire footprint of a given footing.

Imported fill used to establish final grade, if needed should consist of granular soil having a maximum size of 6 inches, with less than 20 percent passing the No. 200 sieve. At least 70 percent should be finer than ¾ inch. The material passing a No. 40 sieve should have a plasticity index less than 6. The fill should be compacted to an in-place density equal to at least 92 percent of the maximum density as determined by ASTM D 1557. Structural fill beneath foundations should meet requirements outlined in Section 4.2.

Grading around the structure should be performed in such a manner that all surface water will flow freely away from the building and no ponding will occur adjacent to the structure. Failure to comply with these recommendations will permit deep percolation into the foundation area which may contribute to settlement. Roof drains should extend well beyond the building lines, and sprinkler heads or any other sources of water located near the building should be directed away from the structure.

Backfilling around foundations and walls should be performed using granular materials densified to an in-place unit weight equal to at least 90% of the maximum laboratory density indicated above.

6 LIMITATIONS

The conclusions and recommendations presented in this report are based upon the results of field and laboratory tests which, in our opinion, define the characteristics of the subsurface material throughout the site in a satisfactory manner. It should be recognized that soil materials are inherently heterogeneous and that conditions may exist throughout this site which could not be defined during this investigation.

Since the bearing capacity of foundations will depend upon minimizing disturbance of the subgrade soils left in place, and upon adequate compaction of structural fill, it is recommended that we be given the opportunity to observe the foundation excavations and review compaction test records for any structural fill before the footings are placed.

If conditions are encountered during construction which appear to be different from those presented in this report, it is requested that we be advised so that appropriate action may be taken.

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FIELD AND LABORATORY TESTING PROCEDURES

The subsurface investigation was performed using a CME 55 rotary drill rig, with NW casing and a 2-15/16” rock bit (rotary wash method) used to advance the boring and water as the drilling fluid. During the subsurface investigation, sampling for the structure borings was performed at 3-foot depth intervals in the upper 15 feet and at 5-foot intervals thereafter.

Disturbed samples were obtained by driving a 2-inch OD or 2.5-inch OD split spoon sampling tube a depth of 18 inches, using a 140-pound weight dropped from a height of 30 inches by an automatic trip hammer. The number of blows required to drive the sampling spoon through each 6 inches of penetration is shown on the boring logs.

The sum of the last two blow counts, which represents the number of blows recorded while driving the sampling spoon through 12 inches, is defined as the standard penetration value. The standard penetration value, corrected for overburden soil pressure and hammer energy, provides a good indication of the in-place density of sandy material; however, it only provides an indication of the relative stiffness of the cohesive material, since the penetration resistance of materials of this type is a function of the moisture content.

Considerable care must be exercised in interpreting the standard penetration value in gravelly-type soils, particularly where the size of the gravel particle(s) exceeds the inside diameter of the sampling spoon. If the spoon can be driven through the full 18 inches with a reasonable sample recovery, the standard penetration value provides a good indication of the in-place density of gravelly-type material.

Relatively undisturbed samples were obtained at selected depths by pushing a thin-walled sampling tube into the soil using the hydraulic pressure on the drill rig. Depths at which undisturbed sampling was attempted are shown on the boring log.

Pocket penetrometer tests, which provide an indication of the unconfined compressive strength of cohesive soils were performed on selected samples of stiff cohesive materials. The results of these field strength tests are shown on the boring logs in units of tons per square foot.

Each sample obtained in the borings was classified in the laboratory according to the Unified Soil Classification System. Symbols designating the soil types according to this system are presented on the logs. A description of the Unified Soil Classification System is included in the appendix, and the meanings of the various symbols shown on the logs can be obtained from this figure.

Testing was performed following procedures outlined in the ASTM International standards.

FIGURES

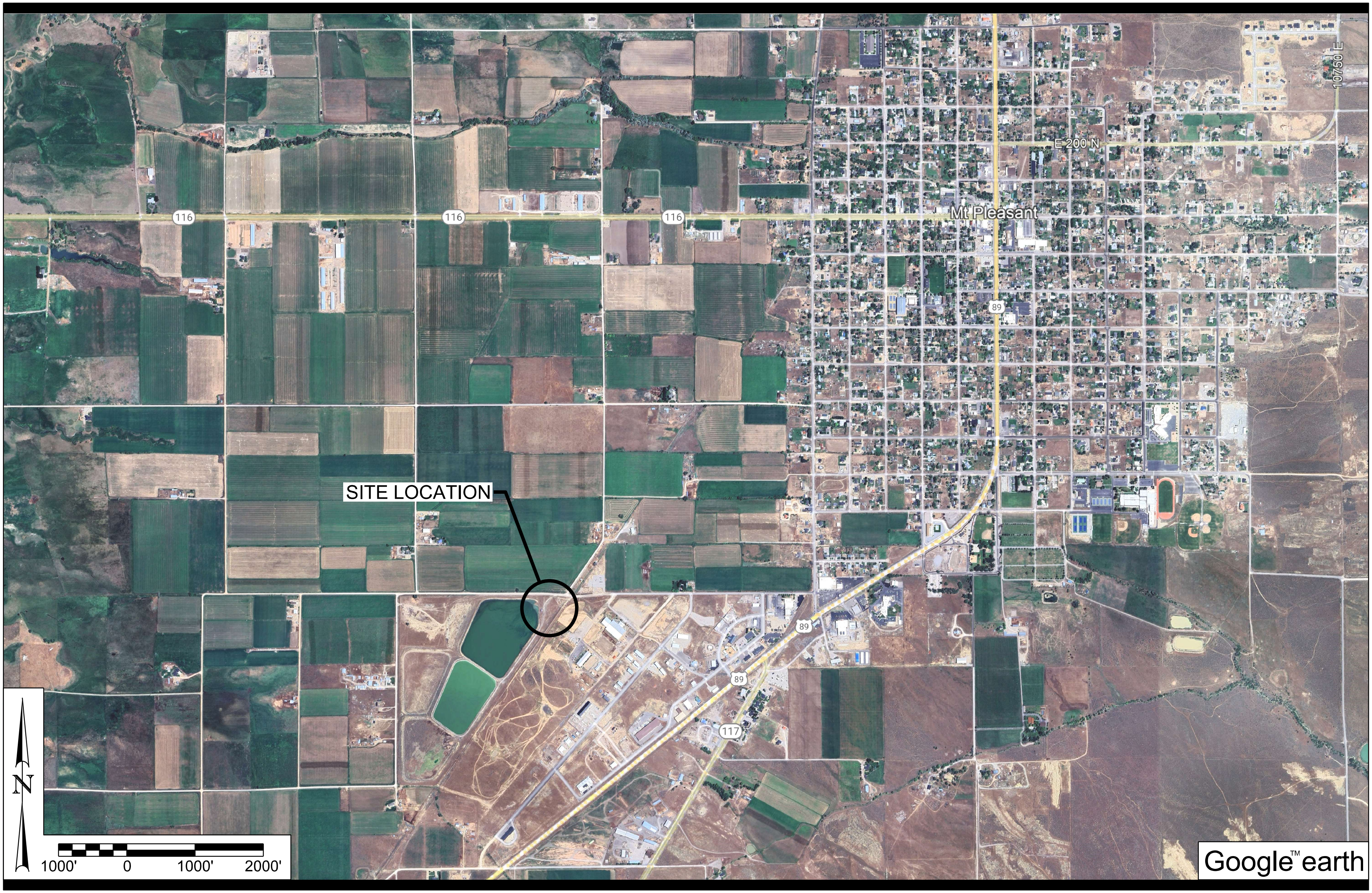

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Figure 1 VICINITY MAP

Mt. Pleasant Sewer Lagoons Headworks Building

Mt. Pleasant, Sanpete County, Utah

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Figure 2 SITE PLAN & TEST HOLE LOCATION

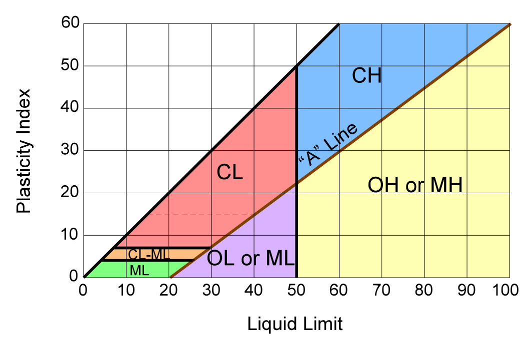
Mt. Pleasant Sewer Lagoons Headworks Building

Mt. Pleasant, Sanpete County, Utah

APPENDIX

Unified Soil Classification System

Major Divisions			Group Symbols		Typical Names	Laboratory Classification Criteria		
COARSE-GRAINED SOILS *more than half of material is larger than No. 200 sieve*	**Gravels** *more than half of coarse fraction is larger than No. 4 sieve size*	**Clean Gravels** *little or no fines*	**GW**		Well graded gravels, gravel-sand mixtures, little or no fines	*For laboratory classification of coarse-grained soils*	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines	Determine percentage of gravel and sand from grain-size curve.	Not meeting all gradation requirements for GW	
		Gravels With Fines *appreciable amount of fines*	**GM***	**d**	Silty gravels, poorly graded gravel-sand-silt mixtures	Depending on percentage of fines (*fraction smaller than No. 200 sieve size*), coarse-grained soils are classified as follows:	Atterberg limits below "A" line, or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring uses of dual symbols
				u				
			GC		Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, or PI greater	
	Sands *more than half of coarse fraction is smaller than No. 4 sieve size*	**Clean Sands** *little or no fines*	**SW**		Well graded sands, gravelly sands, little or no fines	**Less than 5%** GW, GP, SW, SP	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines	**More than 12%** GM, GC, SM, SC	Not meeting all gradation requirements for SW	
		Sands with Fines *appreciable amount of fines*	**SM***	**d**	Silty sands, poorly graded sand-silt mixtures	**5% to 12%** Borderline cases requiring use of dual symbols**	Atterberg limits below "A" line, or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring uses of dual symbols
				u				
			SC		Clayey sands, poorly graded sand-clay mixtures		Atterberg limits above "A" line, or PI greater	
FINE-GRAINED SOILS *more than half of material is smaller than No. 200 sieve*	**Silts and Clays** *liquid limit is less than 50*		**ML**		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	*For laboratory classification of fine-grained soils*		
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL		Organic silts and organic silt-clays of low plasticity			
	Silts and Clays *liquid limit is greater than 50*		**MH**		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
HIGHLY ORGANIC SOILS			**Pt**		Peat and other highly organic soils			



Plasticity Chart

*Division of GM and SM groups into subdivisions of d and u for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when liquid limit is 28 or less and the PI is 6 or less, the suffix u used when liquid limit is greater than 28.

***Borderline classification*: Soils possessing characteristics of two groups are designated by combinations of group symbols. (For example GW-GC, well graded gravel-sand mixture with clay biner.)

DRILL HOLE LOG

BORING NO. 24-1

SHEET 1 OF 1

PROJECT: MT. PLEASANT SEWER LAGOON HEADWORKS BUILDING

CLIENT: JUB ENGINEERS

LOCATION: APPROX. LAT: 39.53121° N, LONG: 111.47864° W

DRILLING METHOD: 20-CME-55 / NW CASING TO 20'

DRILLER: S.W., S.J.

DEPTH TO WATER - INITIAL: ∇ 33.2' **AFTER 24 HOURS:** ▼ N.M.

PROJECT NUMBER: 202401-043

DATE STARTED: 11/14/24

DATE COMPLETED: 11/14/24

GROUND ELEVATION:

LOGGED BY: M.N.H., J.B.

Elev. (ft)	Depth (ft)	Lithology	Sample: Type	Sample: Rec. (in)	Sample: See Legend	Sample: USCS (AASHTO)	Material Description	Dry Density (pcf)	Moisture Content (%)	Atter.: Liquid Limit	Atter.: Plast. Index	Gradation: Gravel (%)	Gradation: Sand (%)	Gradation: Silt/Clay (%)	Other Tests
	0		2.5" OD Split Spoon	14	12,12,9,(49)	GP-GM CL	lt. brown, sl. moist — GRAVEL W/SILT & SAND (fill) dk. brown, sl. moist, very stiff — SANDY LEAN CLAY								Chem.
			Thin-Walled Tube	8	Pushed *PP >4.5*	CL	brown, moist, very stiff — LEAN CLAY W/SAND trace organics	101.2	15.0	30	13	1	25	74	CT UC 4,080 psf
	5					SC	lt. brown, moist — CLAYEY SAND W/GRAVEL								
			2" OD Split Spoon (SPT)	14	9,39,50,(99+)	SM	lt. brown to yellow-brown, sl. moist, very dense — SILTY SAND W/GRAVEL TO SANDSTONE highly weathered, possible cobbles, boulders								
	10		2.5" OD Split Spoon	17	10,8,15,(40)	SM	lt. brown, moist, dense — SILTY SAND slightly plastic fines, white stringers, partially indurated								
						SM	lt. brown, moist								
			Thin-Walled Tube	9	Pushed	ML	brown, moist — SANDY SILT	110.3	15.4		NP	1	40	59	UC 544 psf
	15		Thin-Walled Tube	3	Pushed	GM	lt. brown, very moist — SILTY GRAVEL W/SAND possible cobbles, some partially indurated layers								
			2" OD Split Spoon (SPT)	9	21,50/3"	GM	lt. brown, very moist, very dense								
	20		2.5" OD Split Spoon	9	17,60/3"	SM	lt. brown, very moist, very dense — SILTY SAND W/GRAVEL possible cobbles, partially indurated		10.2		NP	34	40	26	
	25		2" OD Split Spoon (SPT)	4	60/4"	GM	yellow-brown, moist, very dense — SANDSTONE (SILTY GRAVEL W/SAND) possible cobbles, boulders, bedrock?								
	30		2.5" OD Split Spoon	15	31,44,57,(99+)	SC-SM	rusty-yellow-brown, moist, very dense — SILTY CLAYEY SAND W/GRAVEL possible cobbles, bedrock?		7.2	18	4	36	43	21	
∇															
	35			0	60/1"	-	no recovery BOTTOM OF HOLE								

DH_LOG_V8-2014-1 MP_SEWER_LAGOON.GPJ US EVAL.GDT 12/30/24

LEGEND:

- 2" OD Split Spoon (SPT)
- 2.5" OD Split Spoon
- 3" OD Split Spoon

Split Spoon Sample: 2,3,2,(6) — Blow Count per 6", $(N_1)_{60}$ Value; *T 0.45* — Torvane (tsf); *PP 2.0* — Pocket Penetrometer (tsf); * — With Liners

Thin-Walled Tube Sample: Pushed; *T 0.45* — Torvane (tsf); *PP 2.0* — Pocket Penetrometer (tsf)

OTHER TESTS
UC = Unconfined Compression
CT = Consolidation
DS = Direct Shear
UU = Unconsolidated, Undrained
CU = Consolidated, Undrained
Chem. = pH, Resistivity, Sulfate, Chloride, Soluble Salts
Hyd. = Hydrometer
DC = Dispersive Clay

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Table 1

SUMMARY OF TEST DATA

PROJECT	Mt Pleasant Sewer Lagoons Headworks	PROJECT NO.	202401-043
LOCATION	see site plan	FEATURE	Foundations

HOLE NO.	DEPTH BELOW GROUND SURFACE (ft)	IN-PLACE		UNCONFINED OR UU TRIAXIAL COMPRESSIVE STRENGTH (psf)	ATTERBERG LIMITS			MECHANICAL ANALYSIS			PERCENT FINER THAN 0.005 mm	UNIFIED SOIL CLASSIFICATION SYSTEM
		DRY UNIT WEIGHT (pcf)	MOISTURE (%)		LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	PERCENT GRAVEL	PERCENT SAND	PERCENT SILT & CLAY		
24-1	3-4.5	101.2	15.0	UC 4,080	30	17	13	1	25	74		CL
	12-13.5	110.3	15.4	UC 544			NP	1	40	59		ML
	20-21		10.2				NP	34	40	26		SM
	30-31.5		7.2		18	14	4	36	43	21		SC-SM

HOLE NO.	DEPTH (ft)	pH	RESISTIVITY (ohm-cm)	SULFATES (mg/kg-dry)	CHLORIDES (mg/kg-dry)	SOLUBLE SOLIDS (%)
24-1	0	8.0	2,500	30	42	2.82

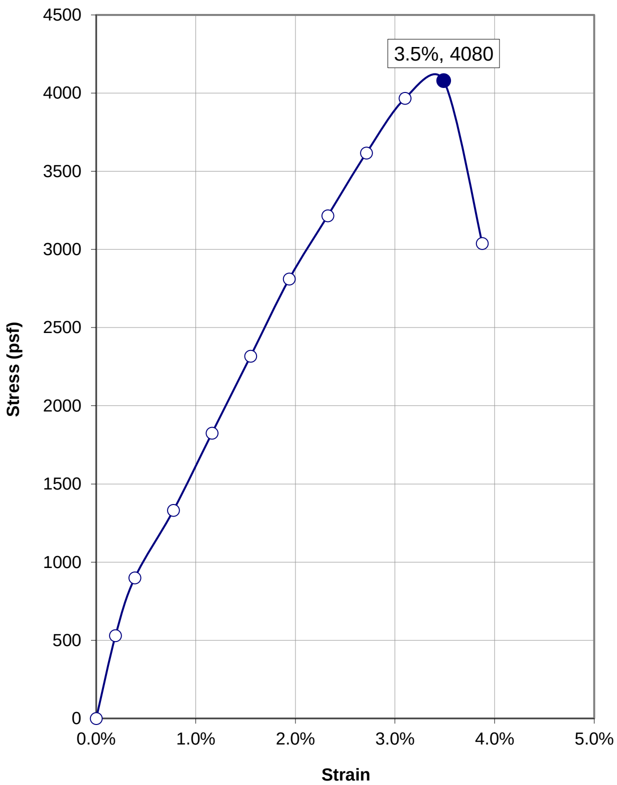
NP=Non-Plastic

RB&G ENGINEERING, INC.

UNCONFINED COMPRESSION TEST ON COHESIVE SOILS

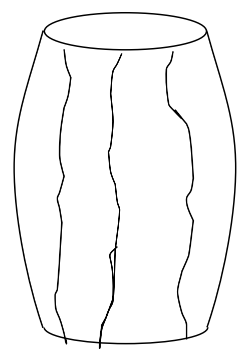
Project	Mt Pleasant Sewer Lagoon Headworks	**Boring No.**	24-1
Project No.	202401-043	**Sample**	1
Location	See Site Plan	**Depth / Elev. (ft)**	3-4.5'
Date	Friday, November 22, 2024	**Sample Description**	Lean Clay w/ Sand CL
Tested By	S Neil	**Sample Type**	Undisturbed (shelby)

Apparatus No.	UC-1	**Proving Ring No.**	5552



Total Strain	Stress (psf)
0.0%	0
0.2%	529
0.4%	900
0.8%	1331
1.2%	1826
1.6%	2318
1.9%	2812
2.3%	3216
2.7%	3618
3.1%	3967
3.5%	4080
3.9%	3039

Sketch of Specimen After Failure



Initial Sample Data

Initial height of specimen	L_o	5.16	(in)	**Liquid limit**	LL	30
Initial diameter of specimen	D_o	2.6	(in)	**Plastic index**	PI	13
Height-to-diameter ratio	L_o / D_o	1.98		**Moisture content***	w	15.0%
Dry unit weight	γ_d	100.1	(pcf)			

Test Results

Unconfined compressive strength	q_u	4080	(psf)
Shear strength	τ_f	2040	(psf)
Average strain rate to failure		1%	
Strain at failure		3.5%	

Remarks

*Moisture content obtained from cuttings and or excess material

H:\2024\043_Mt Pleasant Sewer Lagoon Headworks (JUB)\Lab Testing\24-1 @ 3-4.5' UC

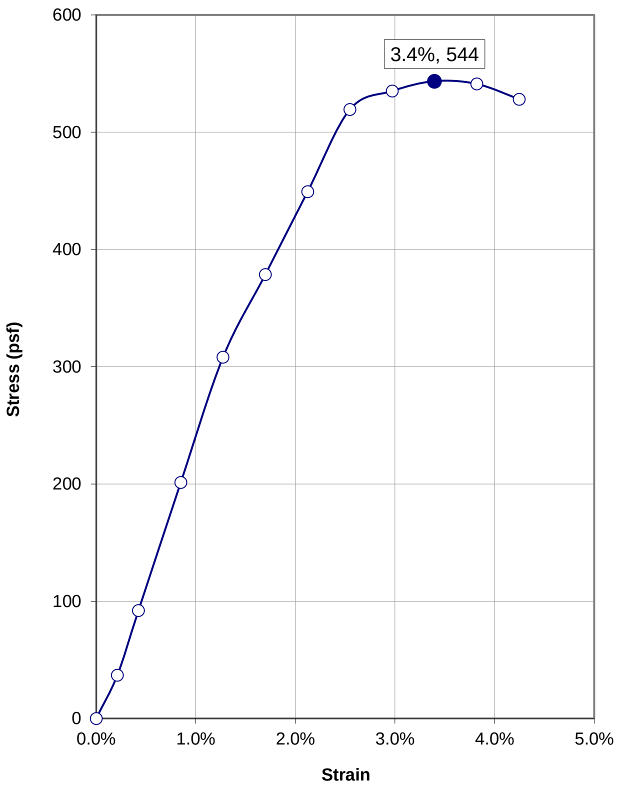
ASTM D 2166

RB&G ENGINEERING, INC.

UNCONFINED COMPRESSION TEST ON COHESIVE SOILS

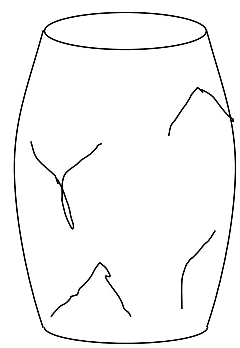
Project	Mt Pleasant Sewer Lagoon Headworks	**Boring No.**	24-1
Project No.	202401-043	**Sample**	1
Location	See Site Plan	**Depth / Elev. (ft)**	12-13.5'
Date	Friday, November 22, 2024	**Sample Description**	Sandy Silt ML
Tested By	S Neil	**Sample Type**	Undisturbed (shelby)

Apparatus No.	UC-1	**Proving Ring No.**	5552



Total Strain	Stress (psf)
0.0%	0
0.2%	37
0.4%	92
0.8%	201
1.3%	308
1.7%	379
2.1%	449
2.5%	519
3.0%	535
3.4%	544
3.8%	541
4.2%	528

Sketch of Specimen After Failure



Initial Sample Data

Initial height of specimen	L_o	4.71	(in)	**Liquid limit**	LL	NP
Initial diameter of specimen	D_o	2.5	(in)	**Plastic index**	PI	NP
Height-to-diameter ratio	L_o / D_o	1.88		**Moisture content***	w	15.4%
Dry unit weight	γ_d	110.3	(pcf)			

Test Results

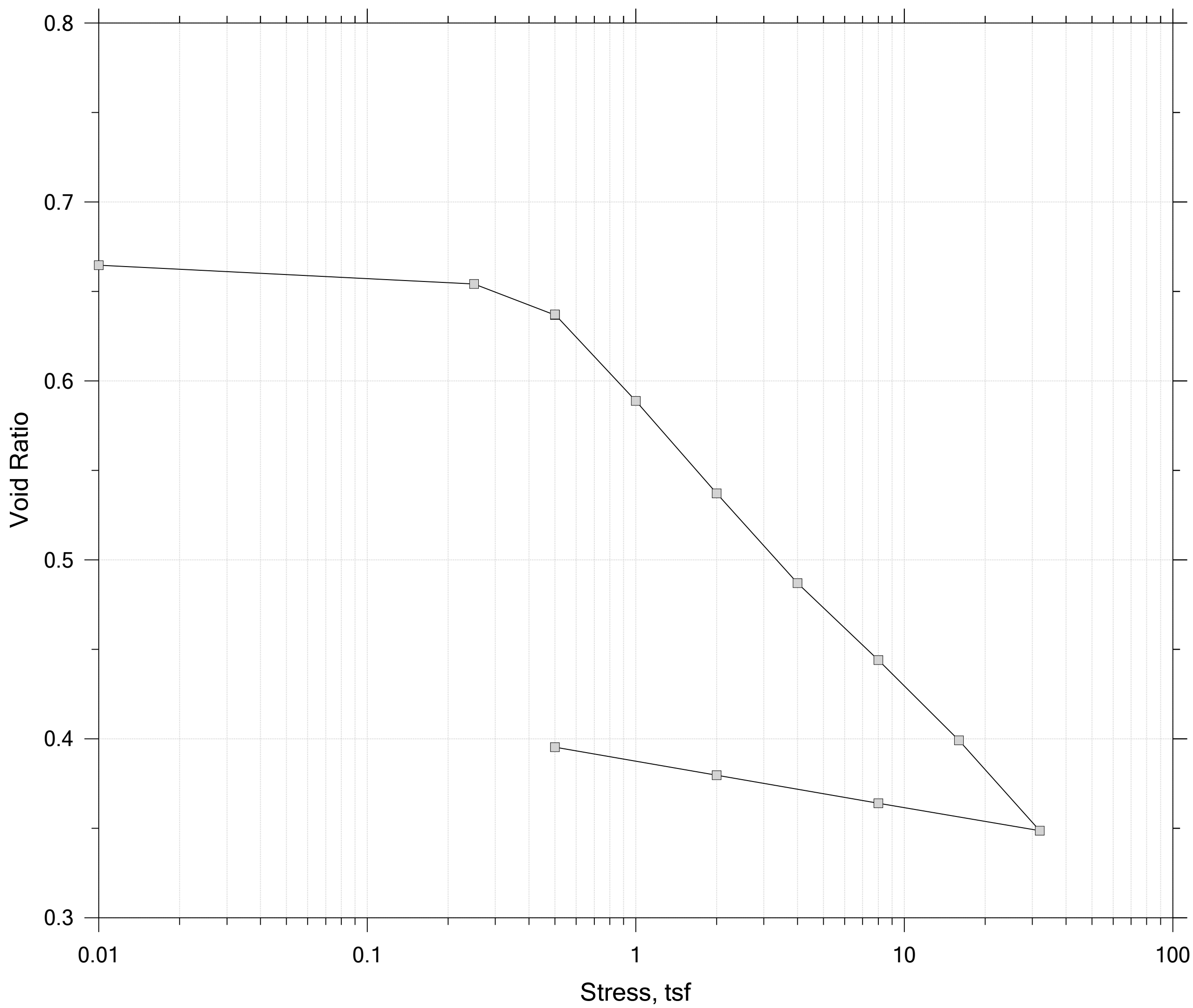
Unconfined compressive strength	q_u	544	(psf)
Shear strength	τ_f	272	(psf)
Average strain rate to failure		1%	
Strain at failure		3.4%	

Remarks Sample extruded out of shelby tube was smaller in diameter than the shelby tube.

*Moisture content obtained from cuttings and or excess material

Consolidation Test

Summary Report



					Before Test	After Test
Current Vertical Effective Stress, tsf: ---				Water Content, %	15.00	12.91
Preconsolidation Stress, tsf: ---				Dry Unit Weight, pcf	101.2	120.81
Compression Ratio: ---				Saturation, %	60.86	88.18
Specimen Diameter, in: 2.37		Specimen Height, in: 0.913		Void Ratio	0.67	0.40
LL: 30	PL: 17	PI: 13	GS: 2.70	Back Pressure, tsf	0.29952	0.29952

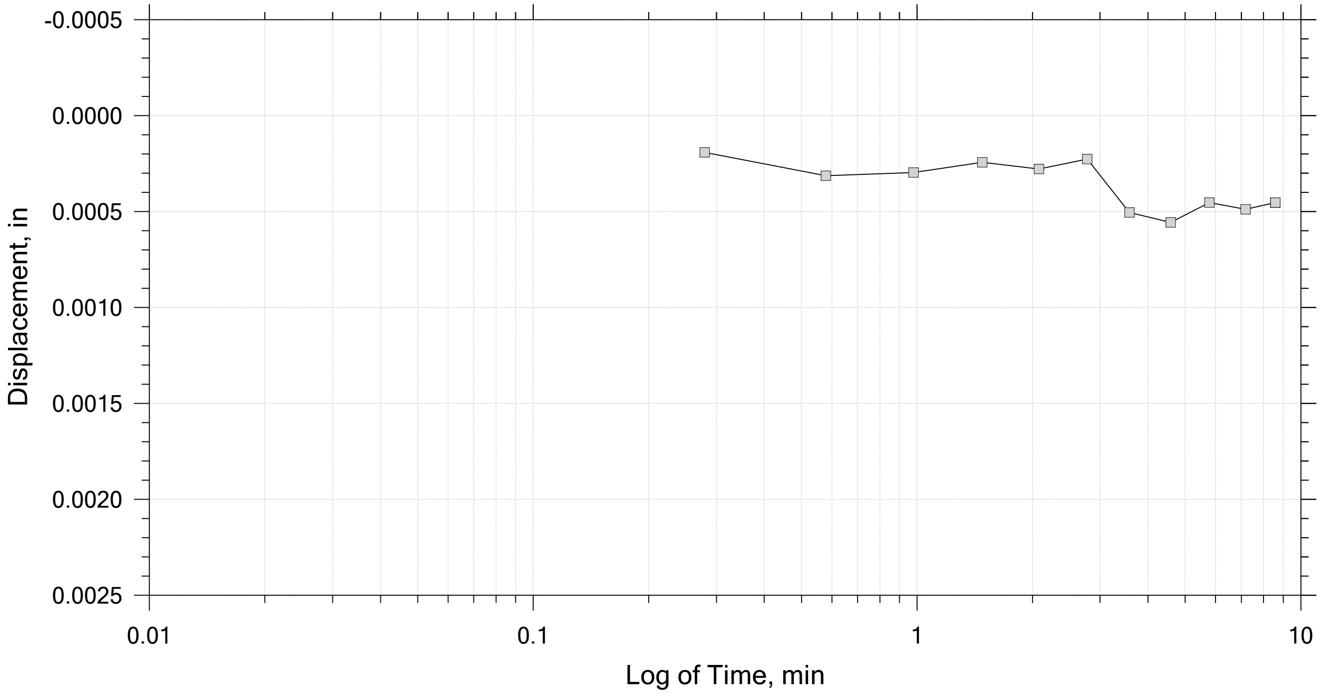
	Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
	Boring Number: 24-1	Tester: JB	Checker:
	Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
	Test Number: 1	Preparation: Shelby Tube	Elevation:
	Client:	Classification:	Group Symbol:
	Description: Lean Clay w/ Sand CL (A-6(8))		
	Remarks: Load Frame #68 collapse swell		
	Displacement at End of Increment		

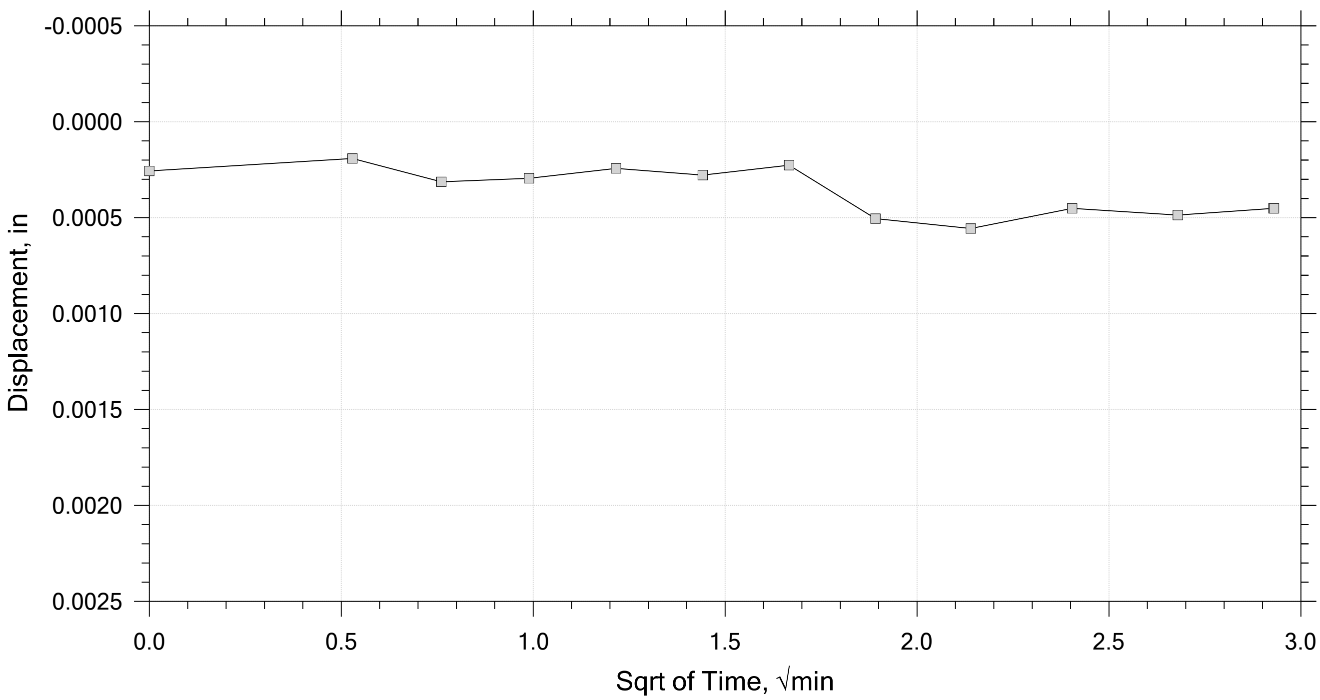
Consolidation Test

Time Curve 1 of 13

Constant Load Step

Stress: 0.01 tsf

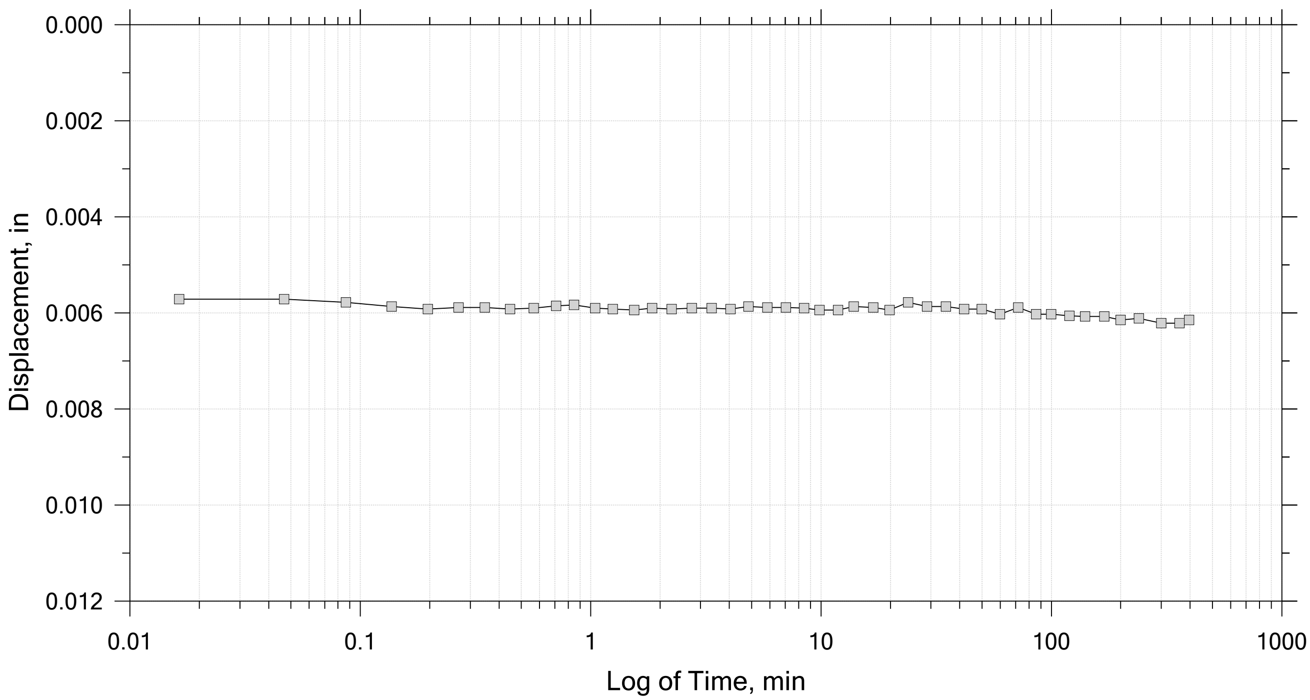


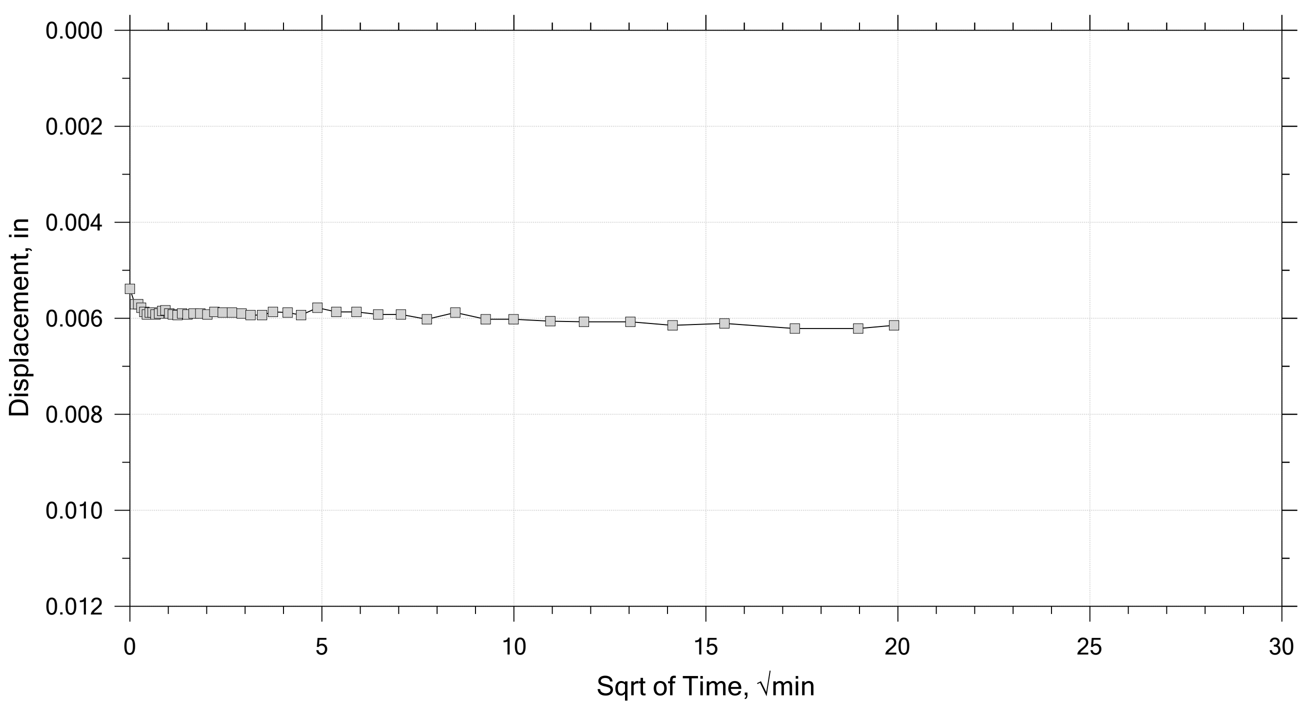


Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
Boring Number: 24-1	Tester: JB	Checker:
Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
Test Number: 1	Preparation: Shelby Tube	Elevation:
Client:	Classification:	Group Symbol:
Description: Lean Clay w/ Sand CL (A-6(8))		
Remarks: Load Frame #68 collapse swell		

Consolidation Test

Time Curve 2 of 13
Constant Load Step
Stress: 0.25 tsf





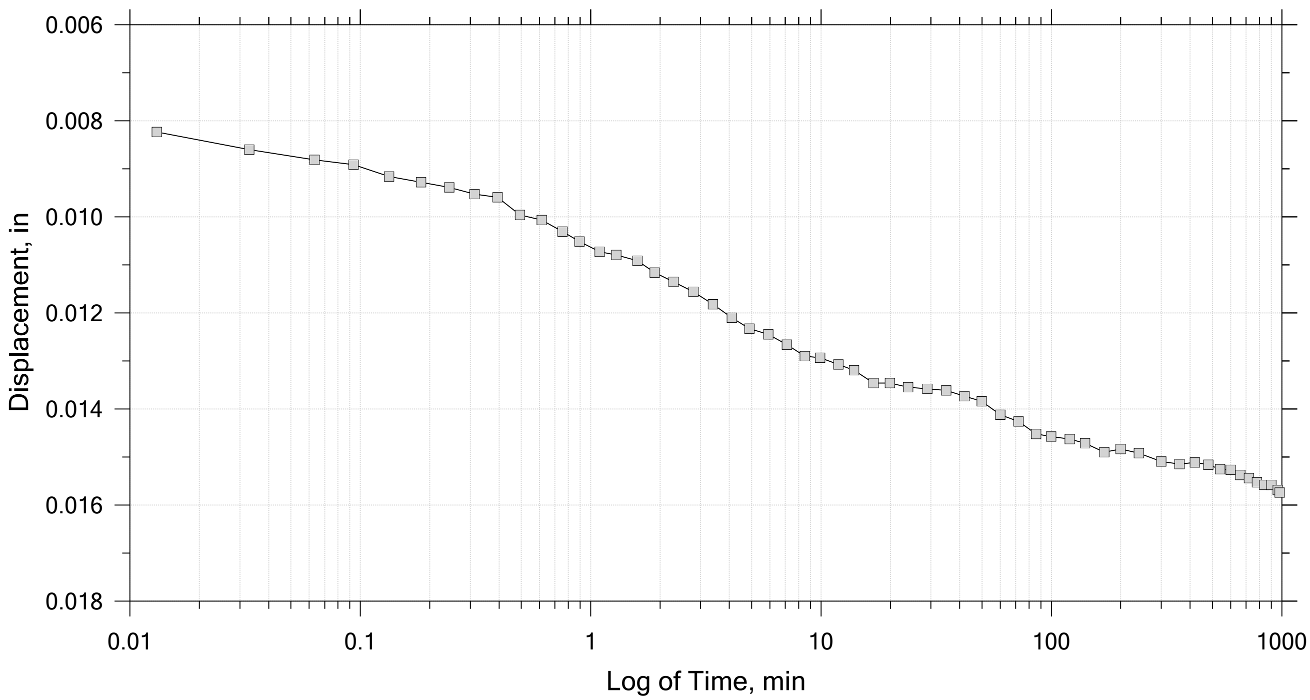
Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
Boring Number: 24-1	Tester: JB	Checker:
Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
Test Number: 1	Preparation: Shelby Tube	Elevation:
Client:	Classification:	Group Symbol:
Description: Lean Clay w/ Sand CL (A-6(8))		
Remarks: Load Frame #68 collapse swell		

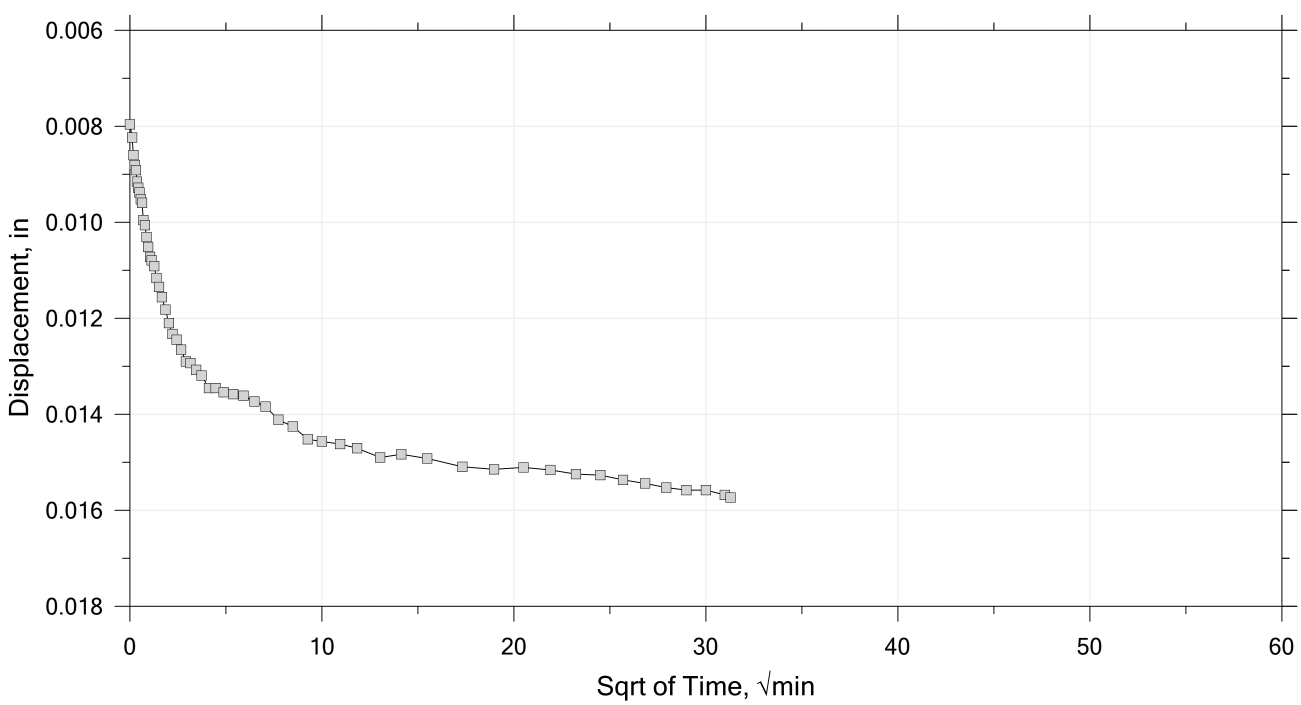
Consolidation Test

Time Curve 3 of 13

Constant Load Step

Stress: 0.5 tsf





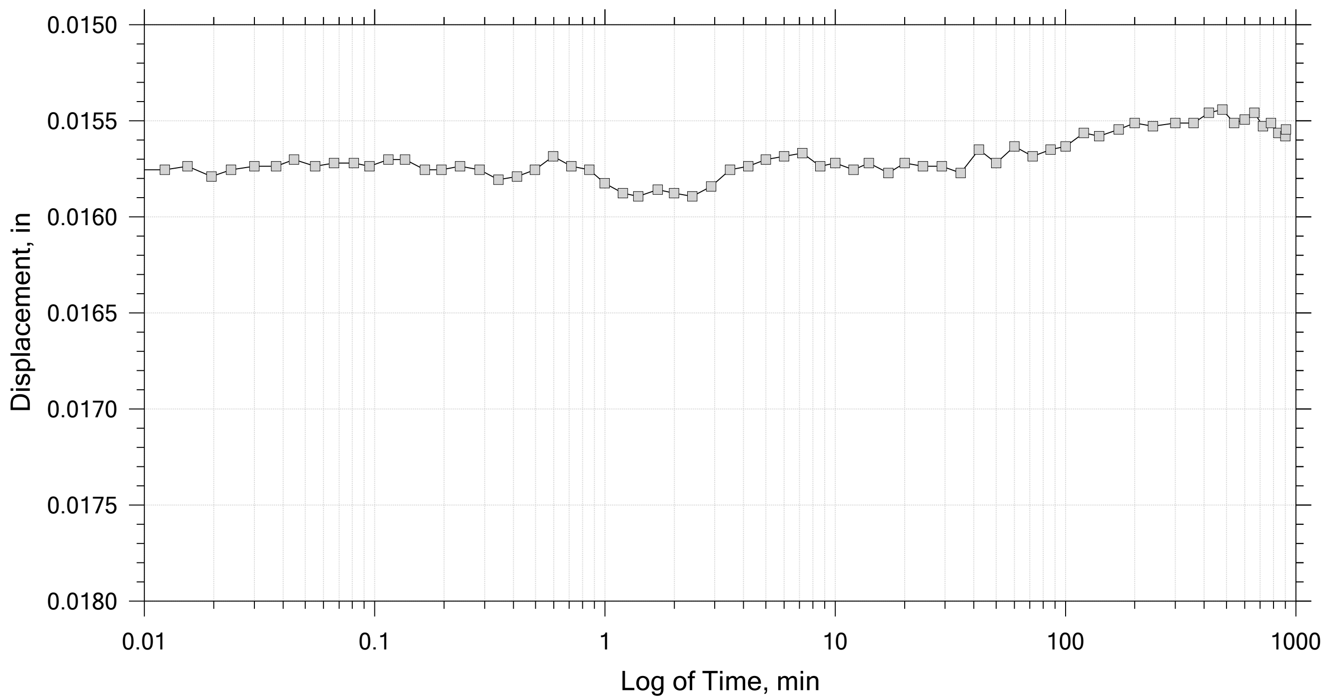
Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
Boring Number: 24-1	Tester: JB	Checker:
Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
Test Number: 1	Preparation: Shelby Tube	Elevation:
Client:	Classification:	Group Symbol:
Description: Lean Clay w/ Sand CL (A-6(8))		
Remarks: Load Frame #68 collapse swell		

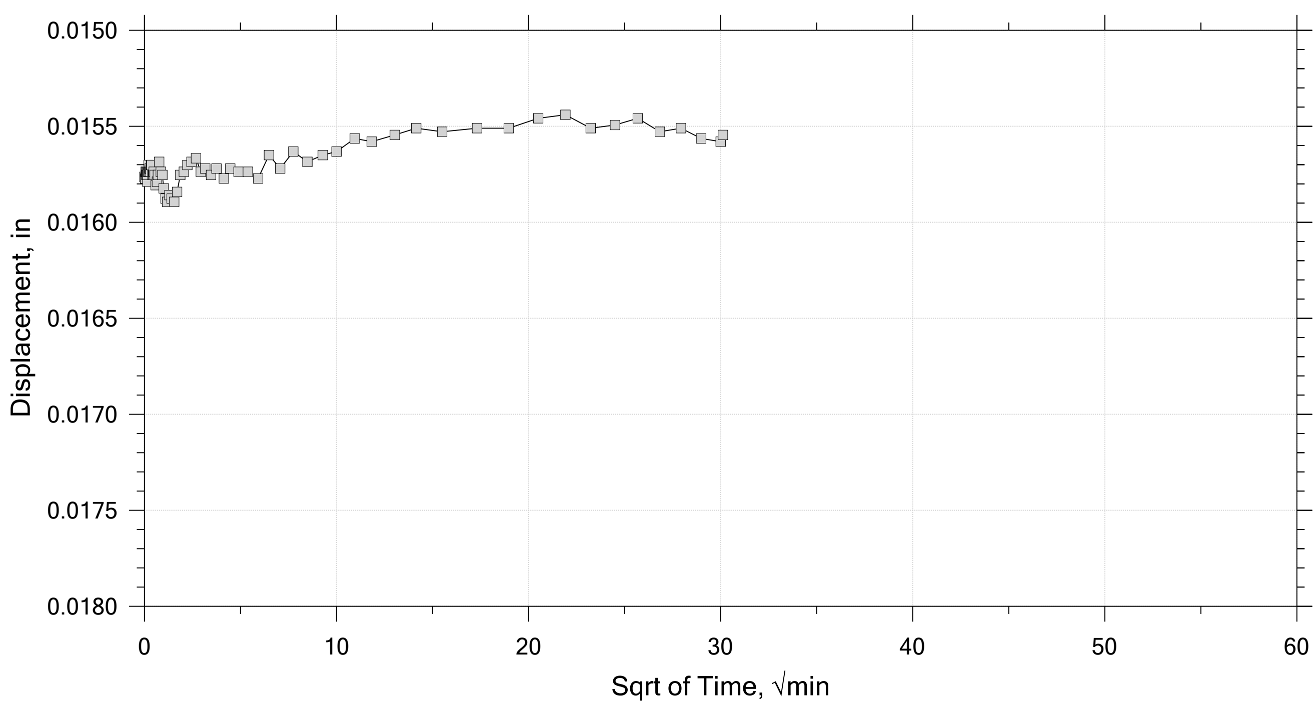
Consolidation Test

Time Curve 4 of 13

Constant Load Step

Stress: 0.5 tsf





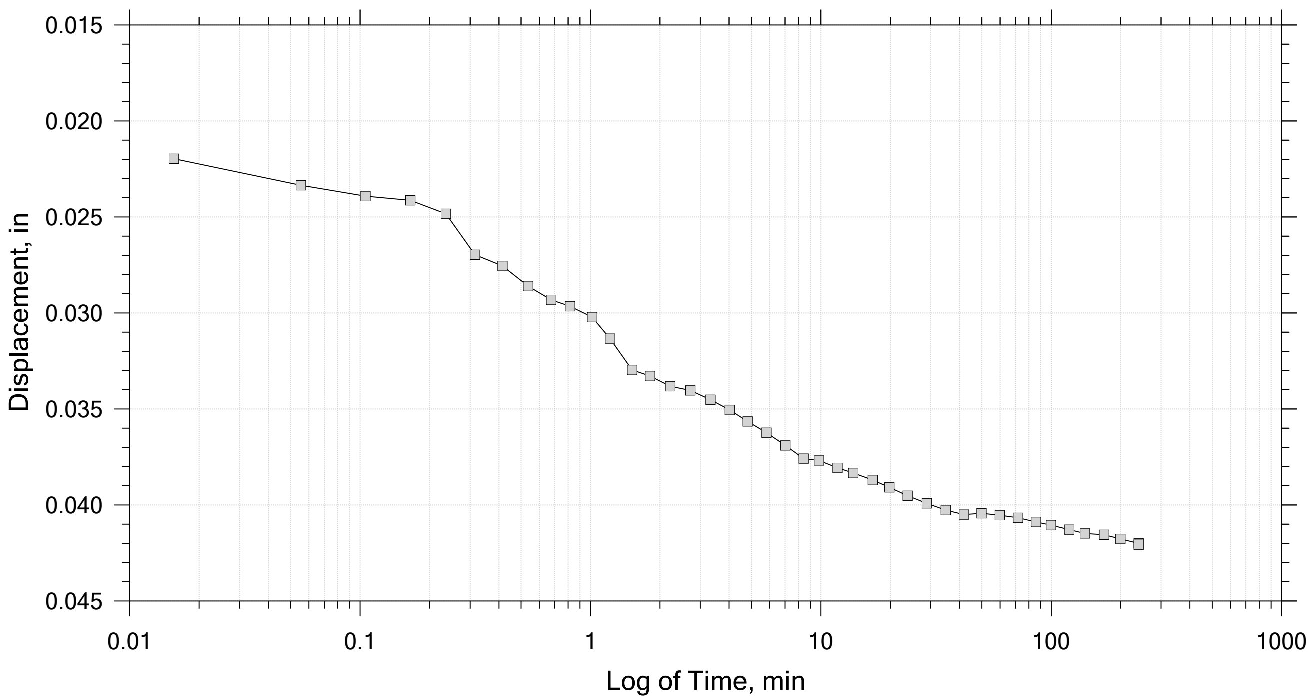
Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
Boring Number: 24-1	Tester: JB	Checker:
Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
Test Number: 1	Preparation: Shelby Tube	Elevation:
Client:	Classification:	Group Symbol:
Description: Lean Clay w/ Sand CL (A-6(8))		
Remarks: Load Frame #68 collapse swell		

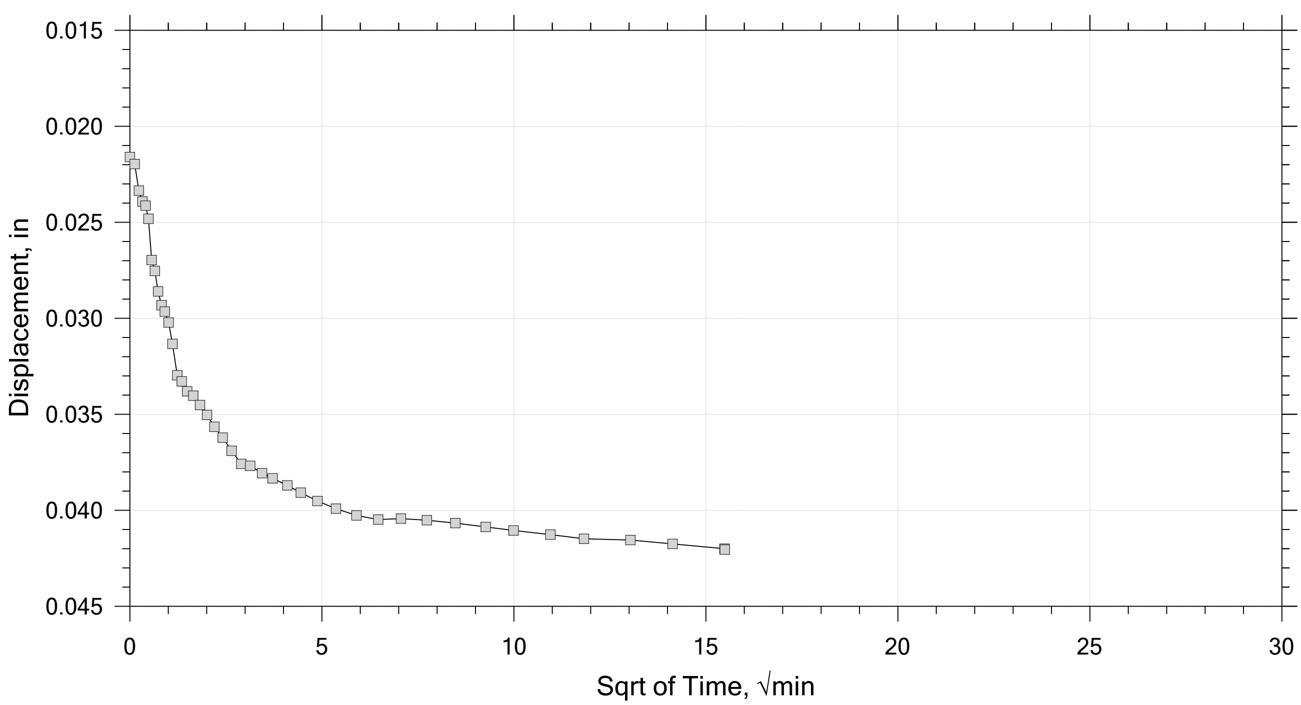
Consolidation Test

Time Curve 5 of 13

Constant Load Step

Stress: 1 tsf





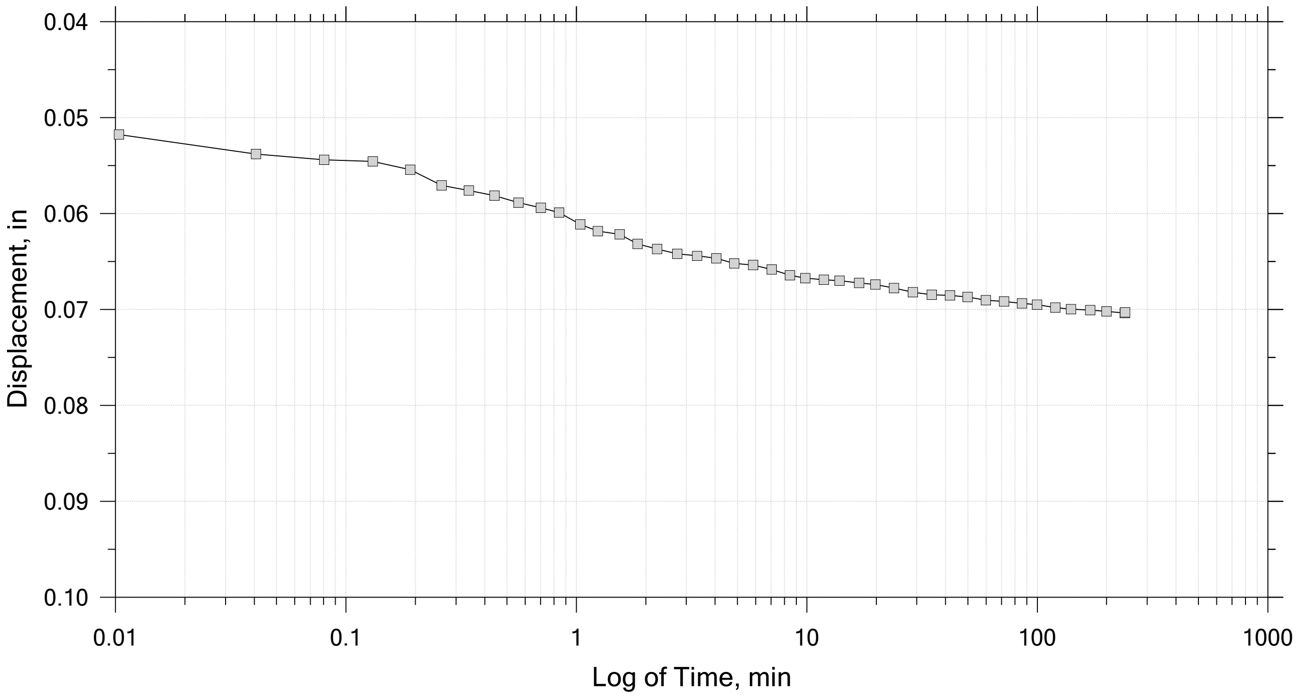
Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
Boring Number: 24-1	Tester: JB	Checker:
Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
Test Number: 1	Preparation: Shelby Tube	Elevation:
Client:	Classification:	Group Symbol:
Description: Lean Clay w/ Sand CL (A-6(8))		
Remarks: Load Frame #68 collapse swell		

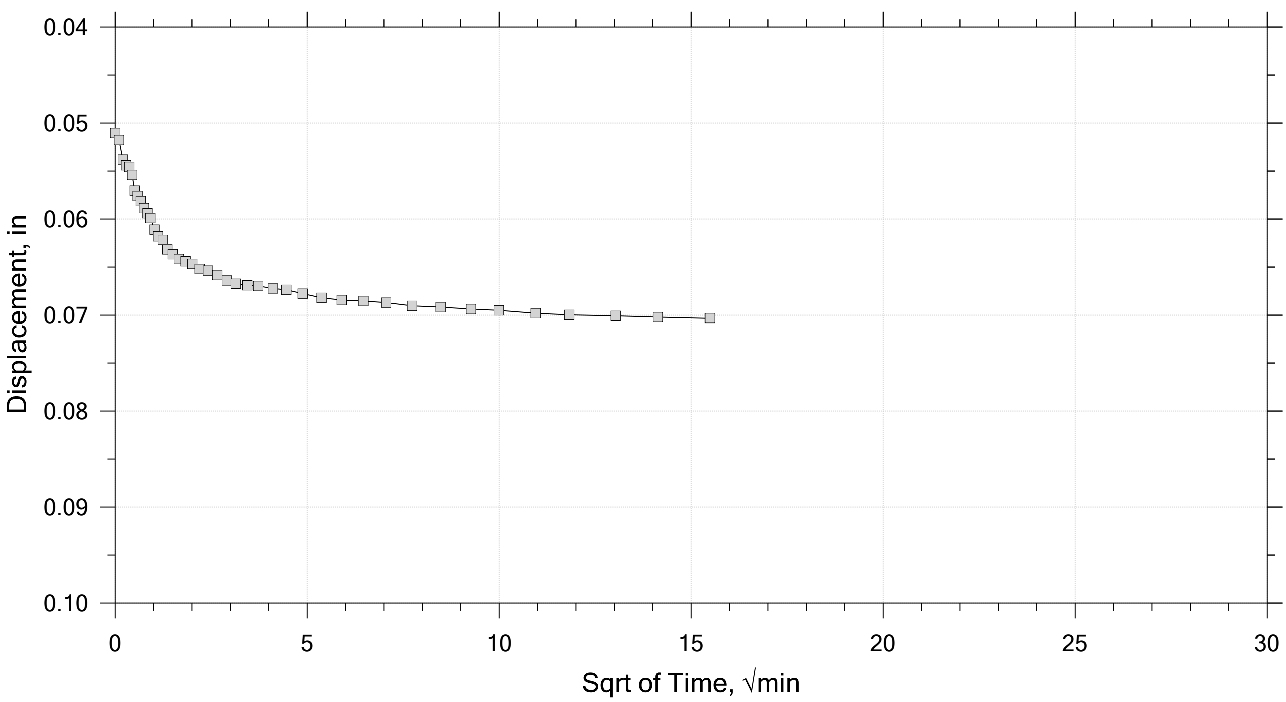
Consolidation Test

Time Curve 6 of 13

Constant Load Step

Stress: 2 tsf





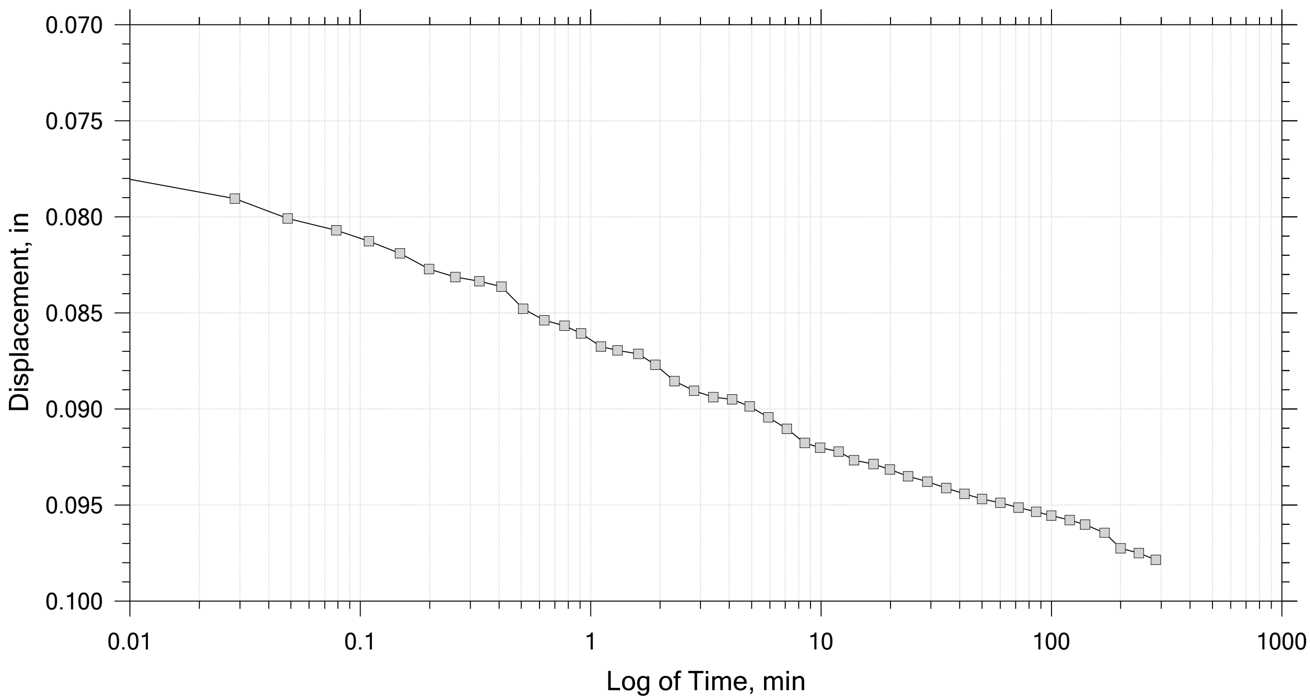
	Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
	Boring Number: 24-1	Tester: JB	Checker:
	Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
	Test Number: 1	Preparation: Shelby Tube	Elevation:
	Client:	Classification:	Group Symbol:
	Description: Lean Clay w/ Sand CL (A-6(8))		
	Remarks: Load Frame #68 collapse swell		

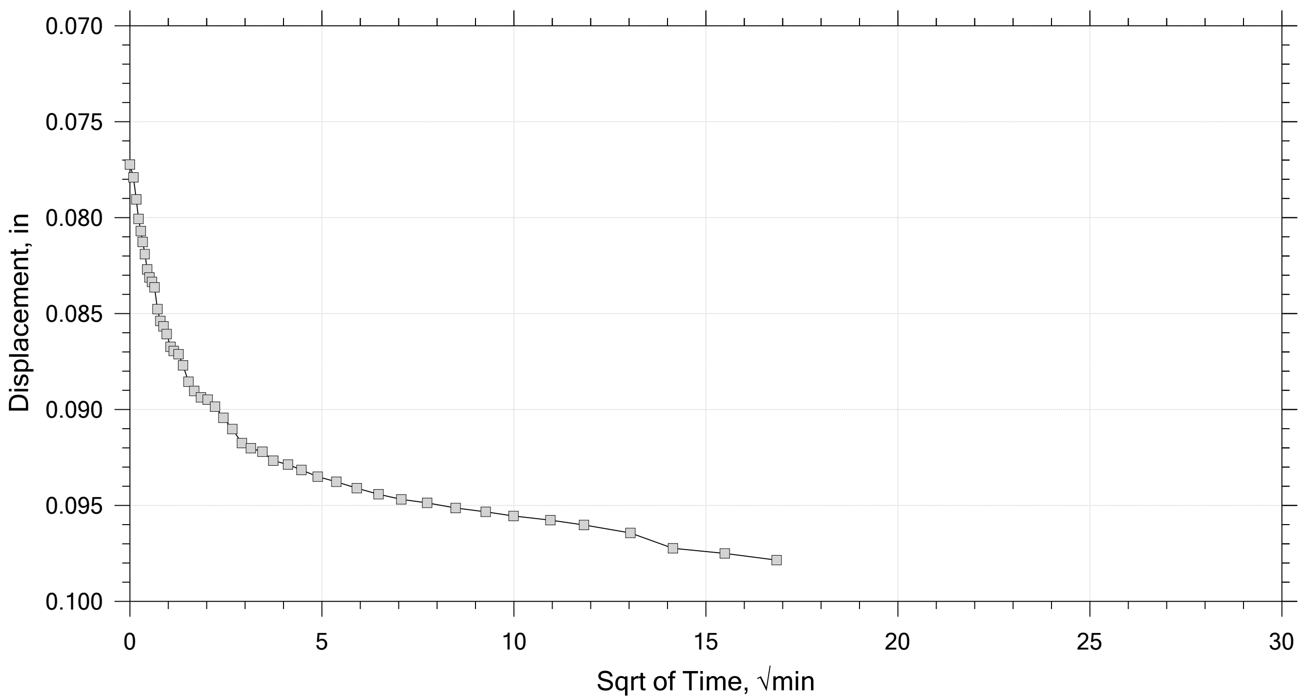
Consolidation Test

Time Curve 7 of 13

Constant Load Step

Stress: 4 tsf





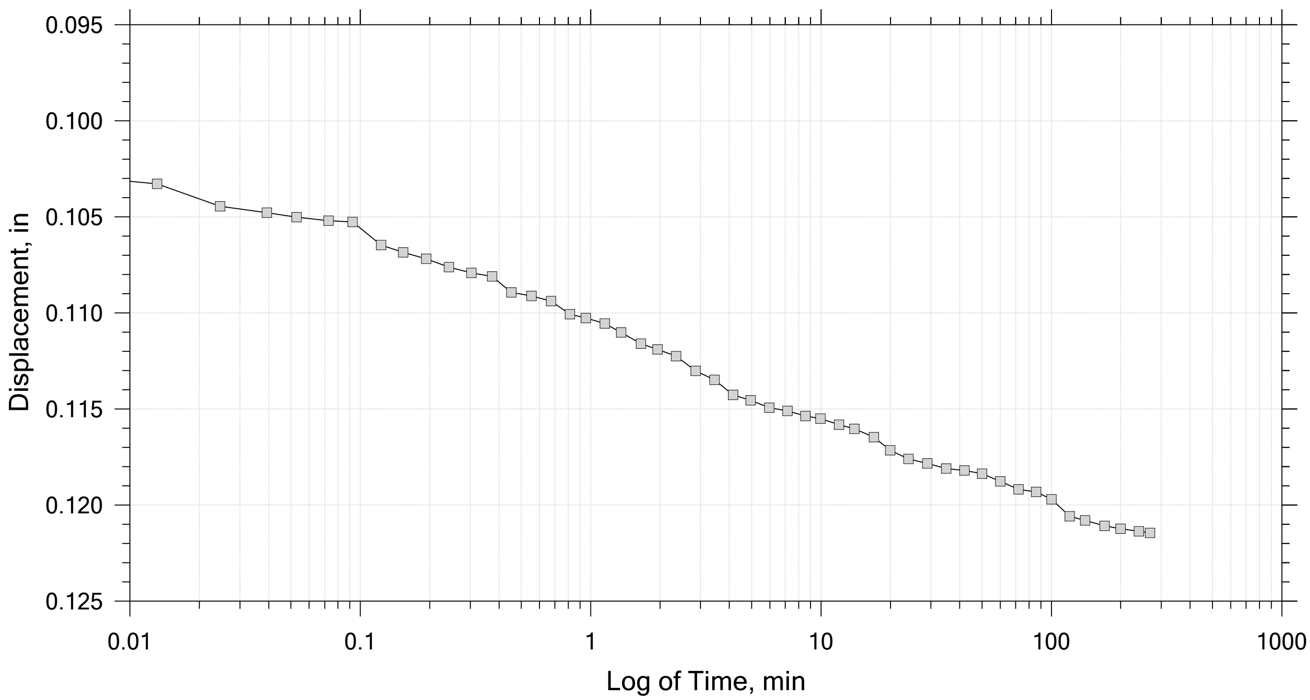
	Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
	Boring Number: 24-1	Tester: JB	Checker:
	Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
	Test Number: 1	Preparation: Shelby Tube	Elevation:
	Client:	Classification:	Group Symbol:
	Description: Lean Clay w/ Sand CL (A-6(8))		
	Remarks: Load Frame #68 collapse swell		

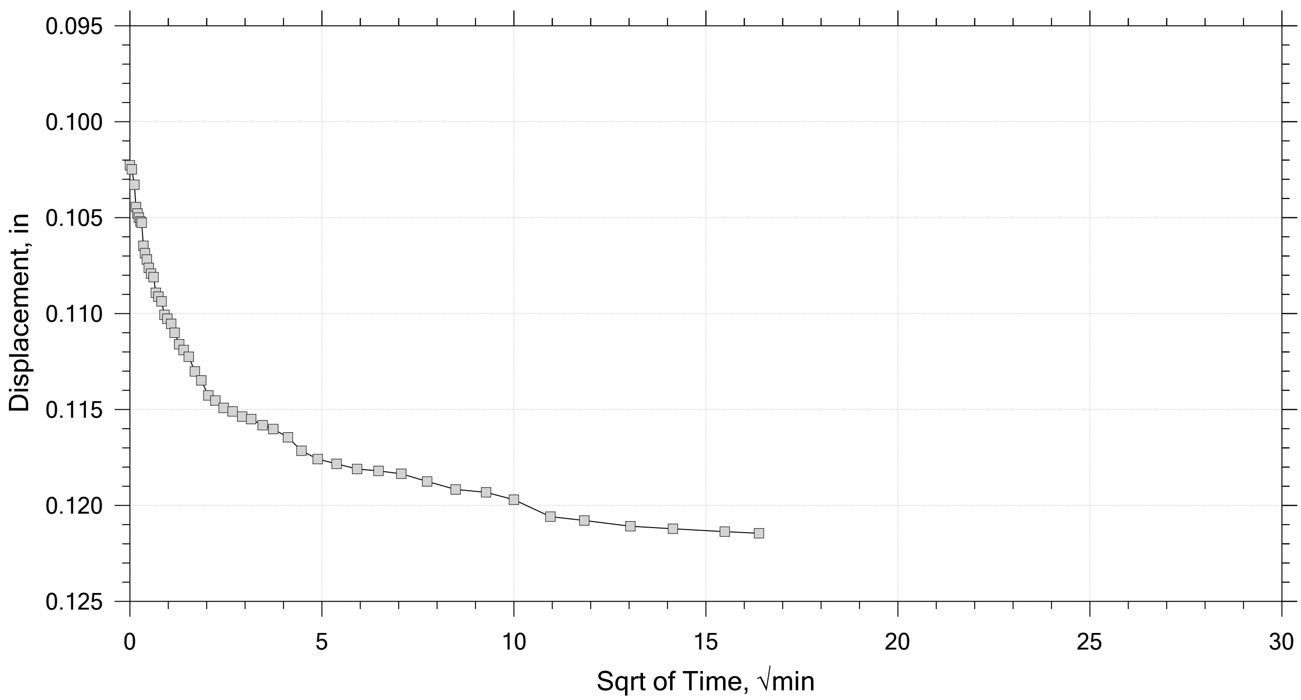
Consolidation Test

Time Curve 8 of 13

Constant Load Step

Stress: 8 tsf





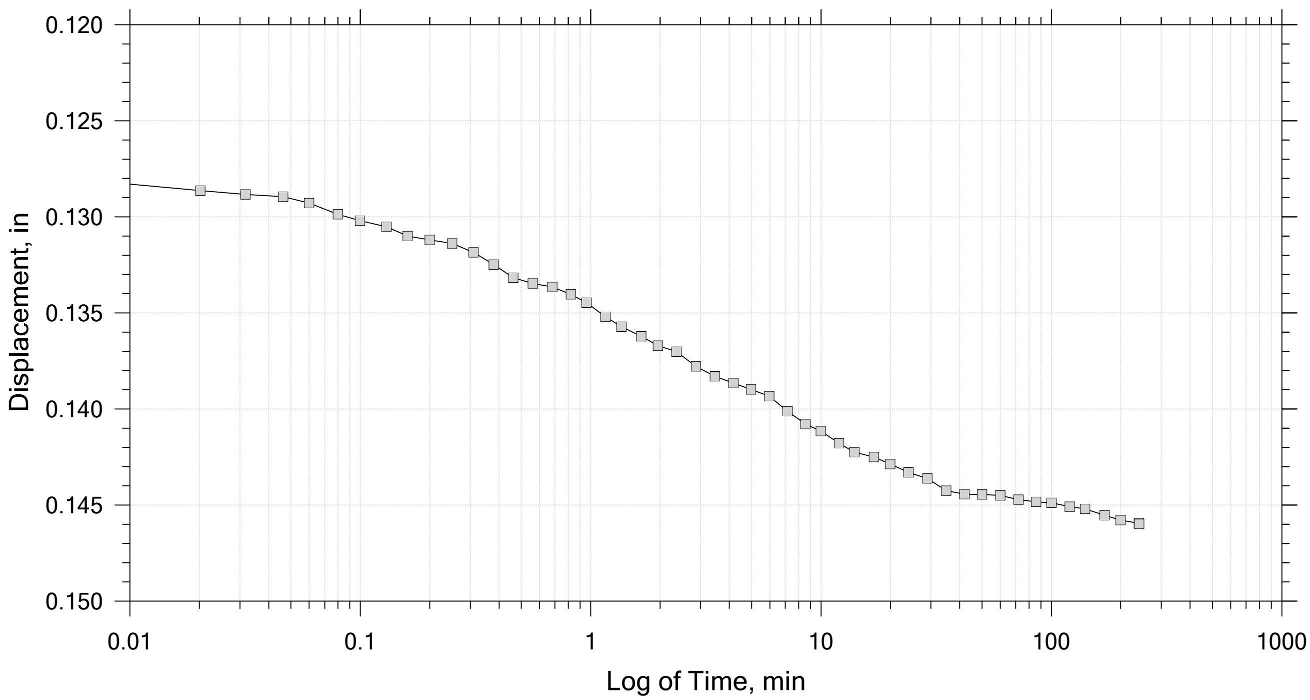
	Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
	Boring Number: 24-1	Tester: JB	Checker:
	Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
	Test Number: 1	Preparation: Shelby Tube	Elevation:
	Client:	Classification:	Group Symbol:
	Description: Lean Clay w/ Sand CL (A-6(8))		
	Remarks: Load Frame #68 collapse swell		

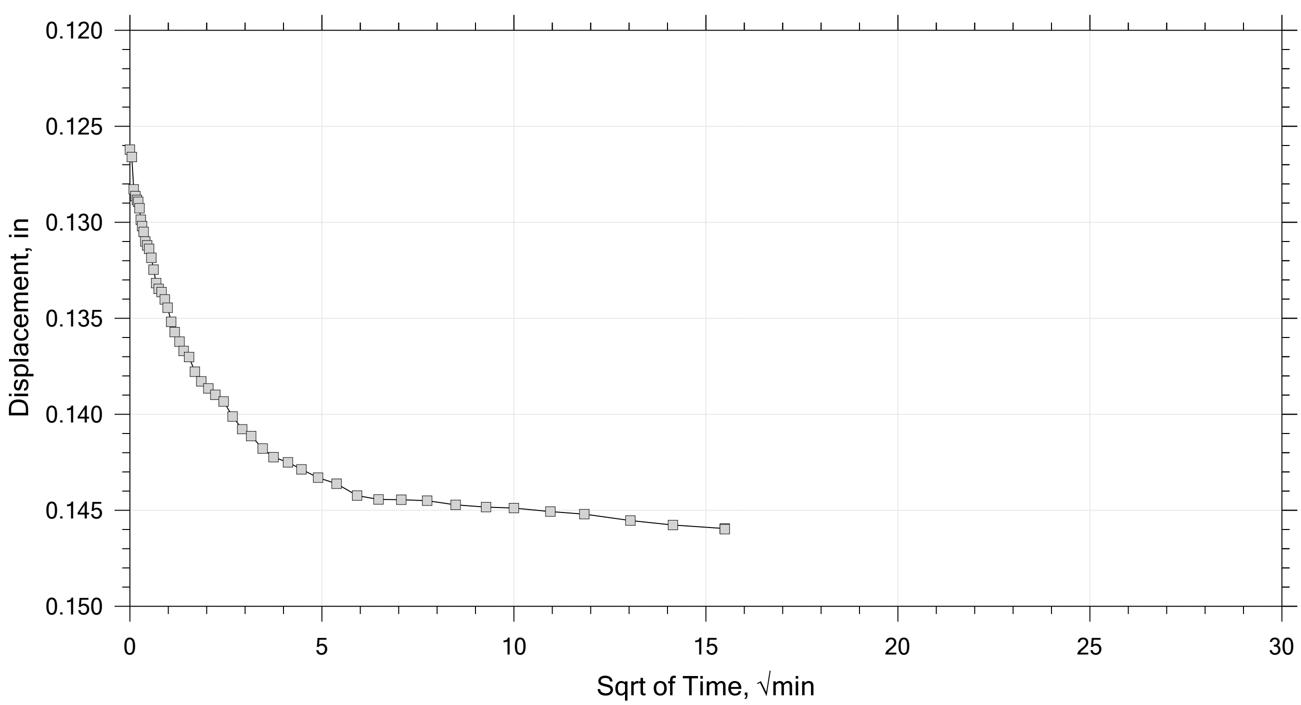
Consolidation Test

Time Curve 9 of 13

Constant Load Step

Stress: 16 tsf





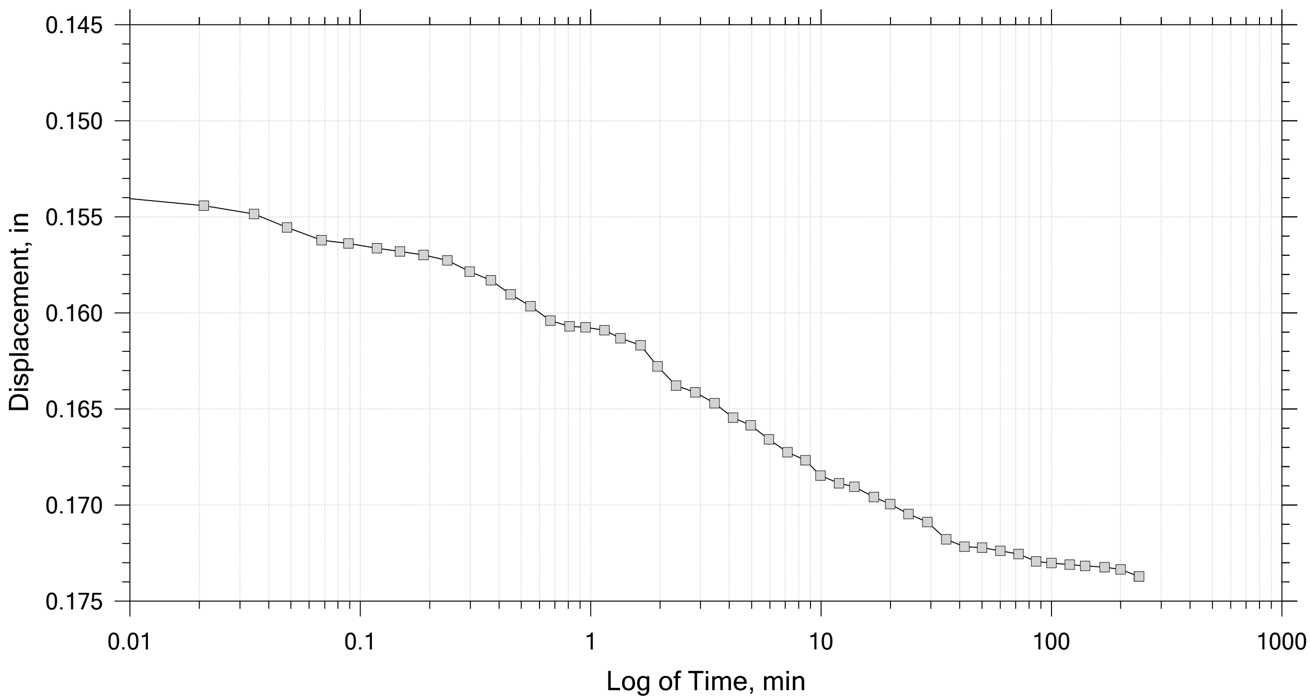
	Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
	Boring Number: 24-1	Tester: JB	Checker:
	Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
	Test Number: 1	Preparation: Shelby Tube	Elevation:
	Client:	Classification:	Group Symbol:
	Description: Lean Clay w/ Sand CL (A-6(8))		
	Remarks: Load Frame #68 collapse swell		

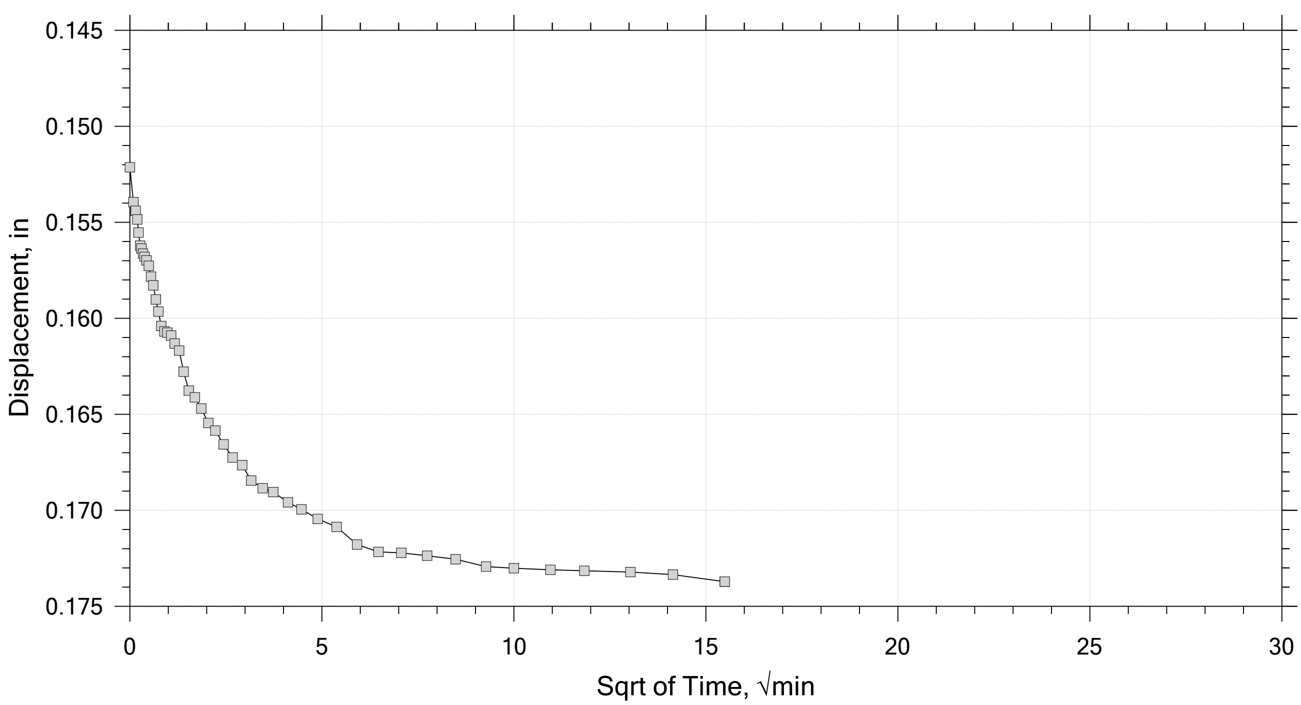
Consolidation Test

Time Curve 10 of 13

Constant Load Step

Stress: 32 tsf





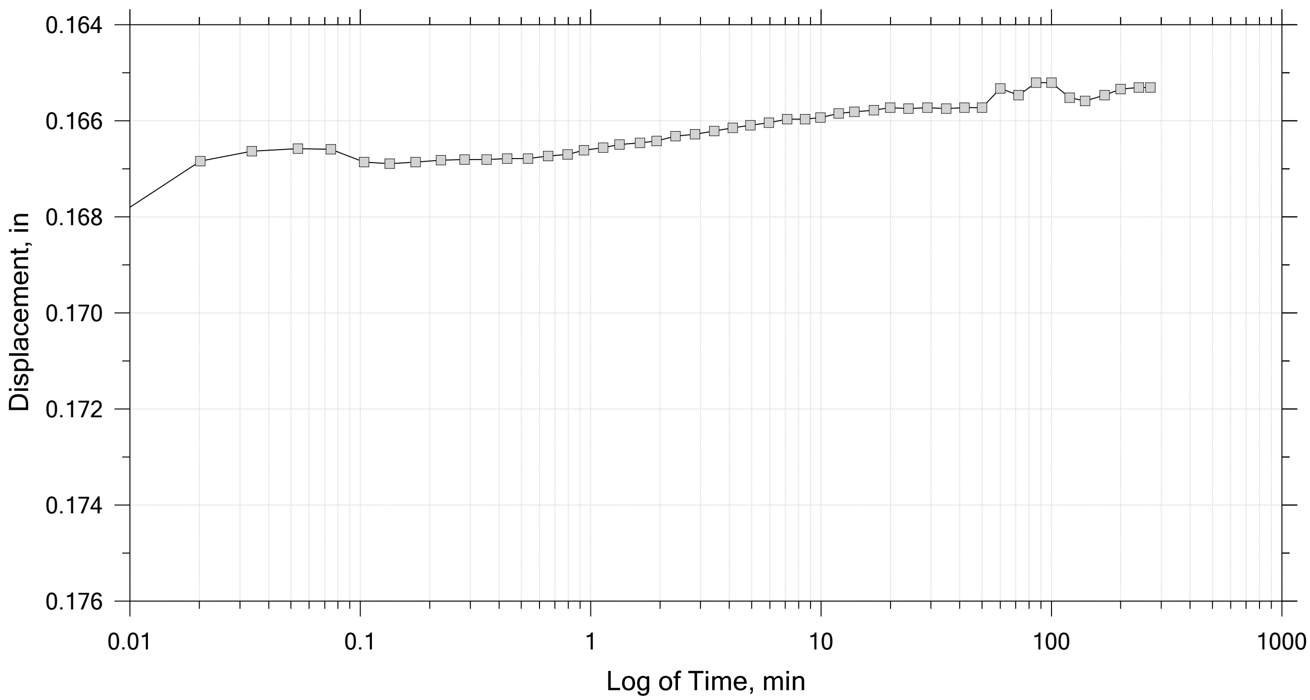
	Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
	Boring Number: 24-1	Tester: JB	Checker:
	Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
	Test Number: 1	Preparation: Shelby Tube	Elevation:
	Client:	Classification:	Group Symbol:
	Description: Lean Clay w/ Sand CL (A-6(8))		
	Remarks: Load Frame #68 collapse swell		

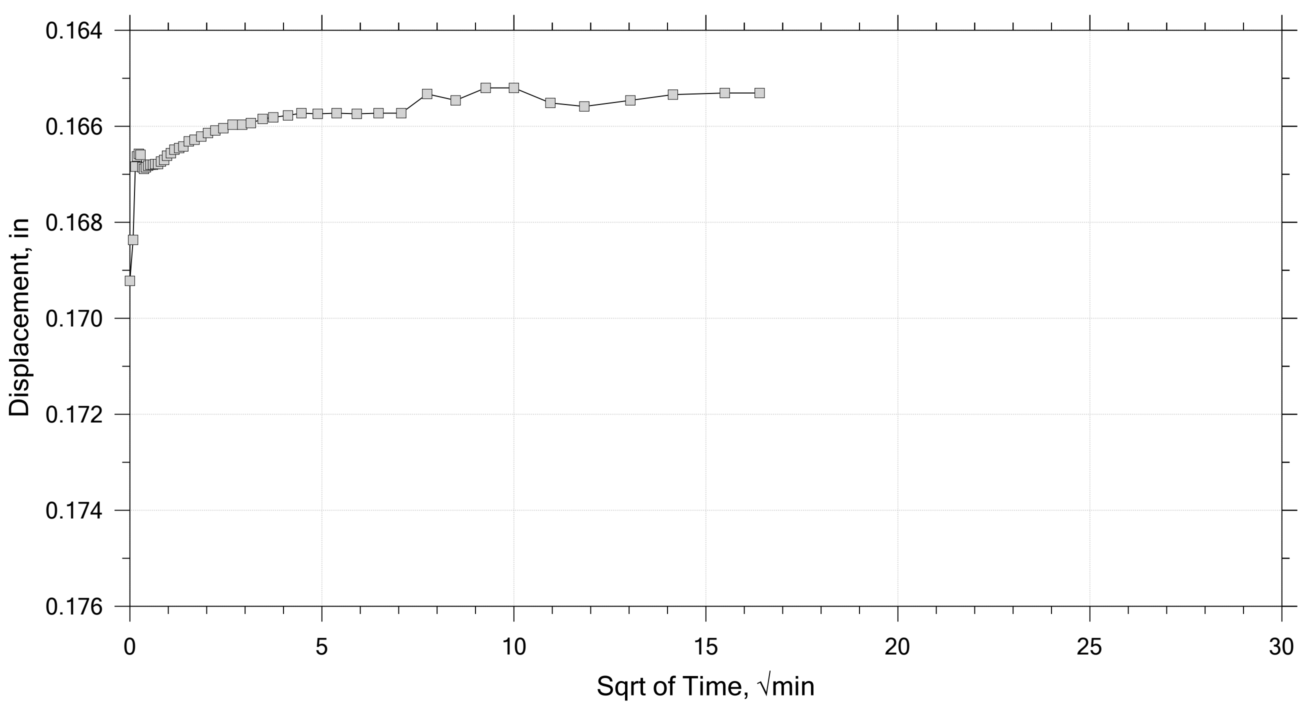
Consolidation Test

Time Curve 11 of 13

Constant Load Step

Stress: 8 tsf

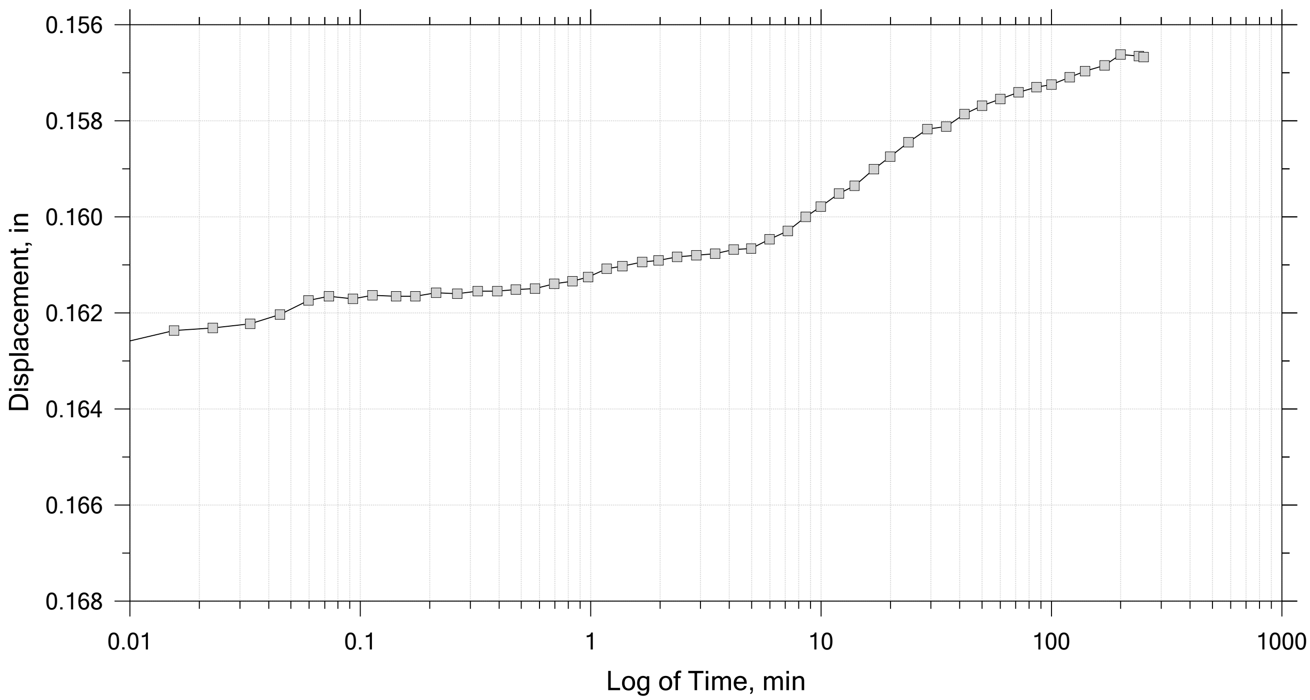


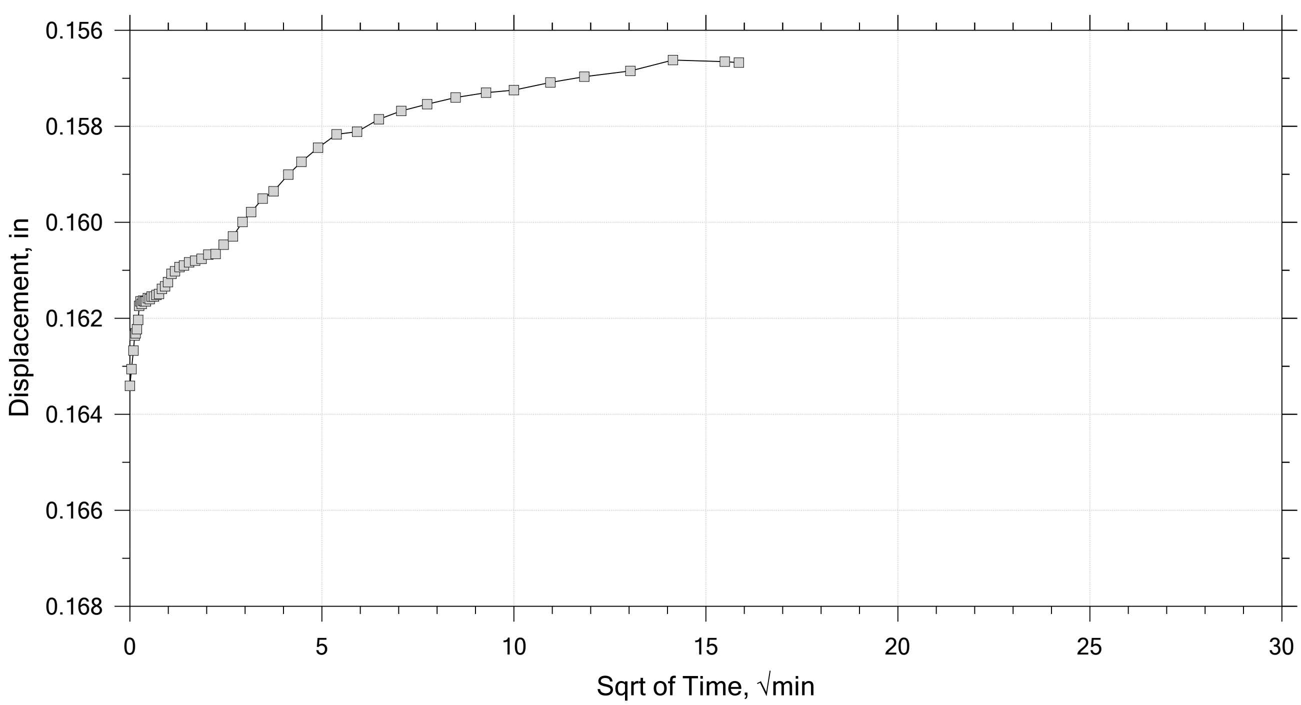


	Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
	Boring Number: 24-1	Tester: JB	Checker:
	Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
	Test Number: 1	Preparation: Shelby Tube	Elevation:
	Client:	Classification:	Group Symbol:
	Description: Lean Clay w/ Sand CL (A-6(8))		
	Remarks: Load Frame #68 collapse swell		

Consolidation Test

Time Curve 12 of 13
Constant Load Step
Stress: 2 tsf

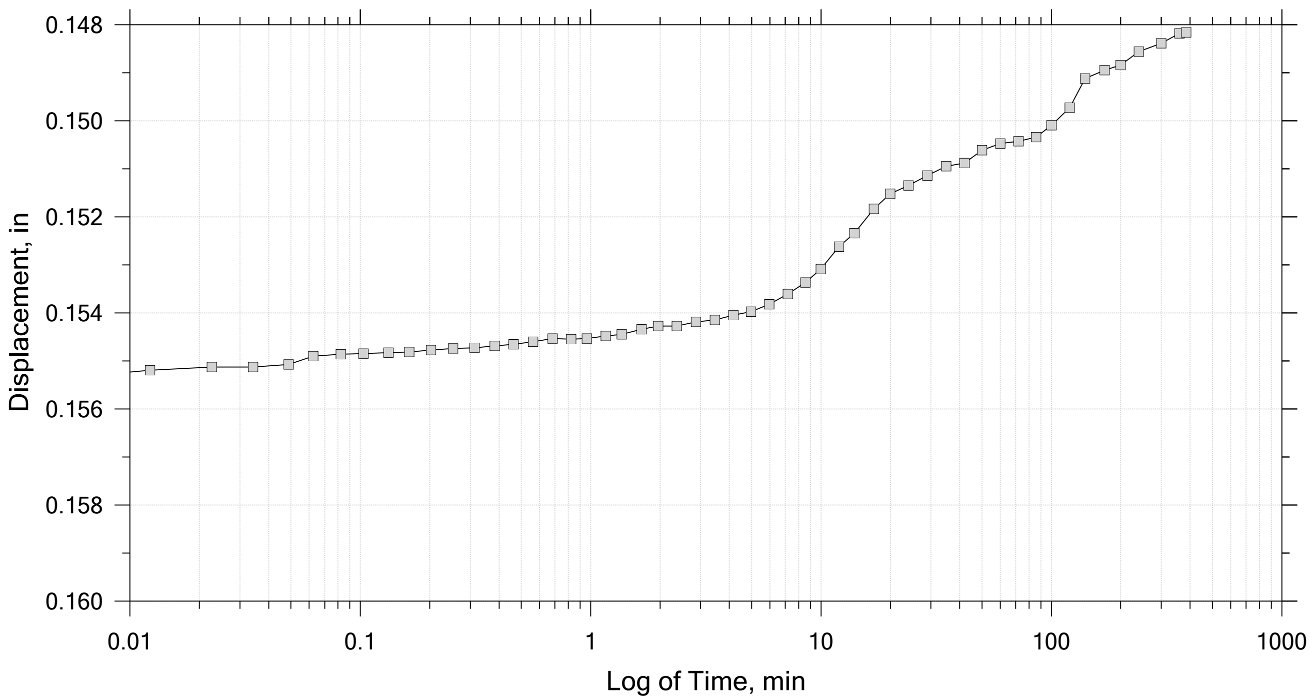


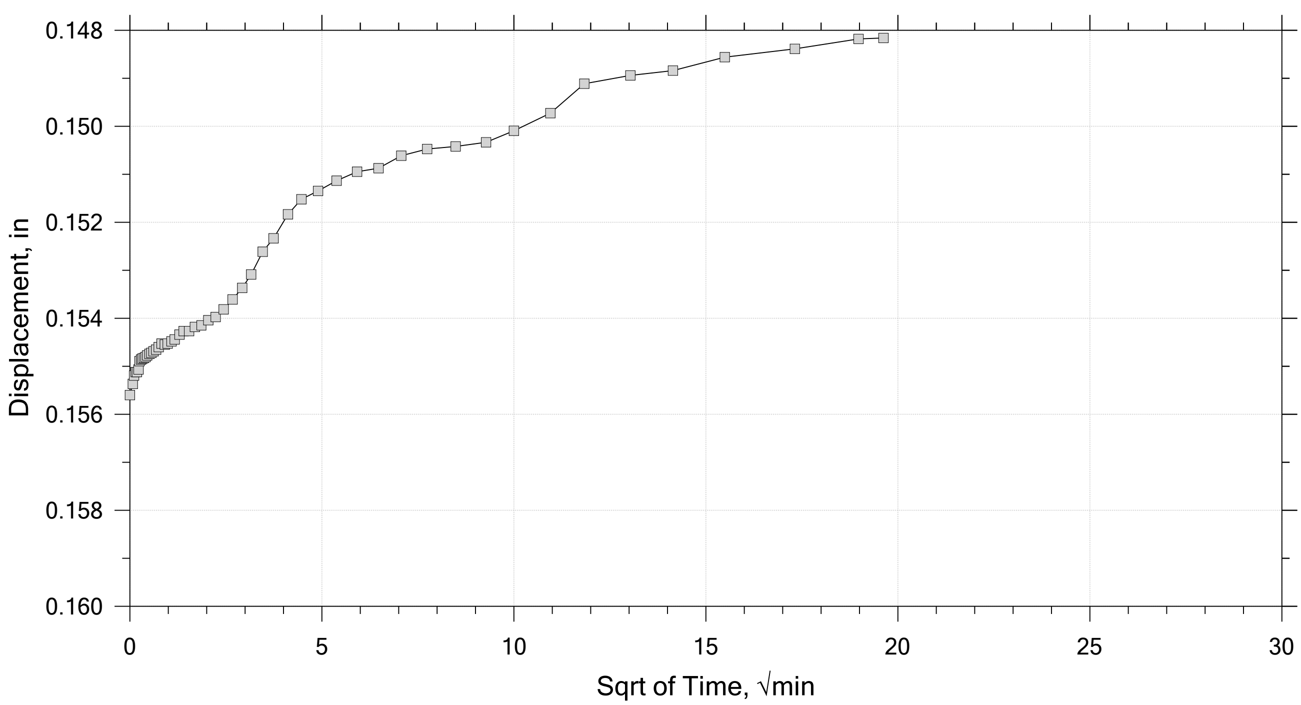


	Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
	Boring Number: 24-1	Tester: JB	Checker:
	Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
	Test Number: 1	Preparation: Shelby Tube	Elevation:
	Client:	Classification:	Group Symbol:
	Description: Lean Clay w/ Sand CL (A-6(8))		
	Remarks: Load Frame #68 collapse swell		

Consolidation Test

Time Curve 13 of 13
Constant Load Step
Stress: 0.5 tsf





	Project Name: MT. Pleasant Sewer Lagoon	Location: see site plan	Project Number: 202401-043
	Boring Number: 24-1	Tester: JB	Checker:
	Sample Number: 1	Test Date: 11-25-2024	Depth: 3-4.5'
	Test Number: 1	Preparation: Shelby Tube	Elevation:
	Client:	Classification:	Group Symbol:
	Description: Lean Clay w/ Sand CL (A-6(8))		
	Remarks: Load Frame #68 collapse swell		