GEOTECHNICAL REPORT

West Mountain Road over Kitchen Brook
Culvert Strengthening
C-10-024
February 2025
Cheshire, Massachusetts

Prepared for:

Town of Cheshire

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1. INTRODUCTION

1.1. Scope of Report

The purpose of this report is to summarize the geotechnical findings from the subsurface investigation and to provide recommendations for the reuse of the existing footings and the proposed wingwalls as part of the C-10-024 (AB2) culvert strengthening project in the Town of Cheshire, Massachusetts. The culvert strengthening is necessary due to the poor condition of the existing culvert. This report will evaluate the data from the subsurface exploration and provide the necessary parameters for designing the proposed wingwalls. All parameters provided will be in accordance with AASHTO LRFD 9th Edition Design Specifications and the 2024 LRFD MassDOT Bridge Manual. The report will also provide recommendations for the construction of the proposed wingwalls with guidance on minimizing potential construction issues.

1.2. Existing Structure and Site History

Bridge C-10-024 (AB2) is located on West Mountain Road and spans over Kitchen Brook as illustrated in Appendix 6.1 – Project Locus Map. The existing bridge is a corrugated steel arch with an open bottom on concrete footings. The clear opening of the existing culvert is 15'-8"± and the total length of the culvert is 42'-7"± .

The south headwall of the culvert has areas of settled stones, with the east half displaced up to 24". The corrugated pipe has areas of minor rusting with delamination and active leakage along the footings, with the entire arch rusted at the south end. The concrete footings are exposed up to 5-feet on either side. Both footings have a vertical crack above the utility box with a spall at the top of the east footing crack. There is localized scour on the downstream side (south end) of the utility box. At the southeast embankment there is a disconnected drain pipe causing moderate erosion. The wearing surface has several cracks and patches. There is minor collision damage to the southwest guardrail terminal end and last panel.

1.3. Site Description

West Mountain Road is oriented west-to-east and provides a single lane of traffic in either direction. It is classified as a Rural Local Road with an ADT of 350 as of 2020. The bridge location is surrounded by residential homes and some trees and vegetation. There is an underground utility vault located at the northwest corner of the project site that provides access to the 8" water main that runs along West Mountain Road.

At the project site, Kitchen Brook is lain with sandy soil and cobble stones. The slopes upstream and downstream of the bridge contain debris and are overgrown with low lying heavy vegetation. The stream site drainage area mostly consists of forest and grassland with some residential development.

2. SUBSURFACE CONDITIONS

2.1. Local Geology

According to the geologic map of the Cheshire quadrangle, the site geology contains a mix of sand, some silt, some clay with pebbles, cobble, boulder clasts, and some large surface boulders. The area has shallow bedrock with till generally less than 10 to 15 feet thick. See Appendix 6.2 for the map.

2.2. Subsurface Exploration

The subsurface exploration consisted of two (2) soil borings located just offset from the end of the existing culvert at both approaches (designated as B-1 and B-2). The borings were drilled using a 3 1/4-inch casing and a 1 3/8-inch split spoon sampler on September 25, 2023 and September 26, 2023 by New England Boring Contractors, Inc. of Derry, New Hampshire and observed by Gill Engineering Associates, Inc. (GEA) which included a visual and hands-on examination of the soil samples. See Appendix 6.3 for an as-drilled boring site plan and Appendix 6.4 for boring logs. GEA has prepared the boring logs presented in this report.

2.3. Subsurface Profile

2.3.1. East Boring B-1

The existing ground grade at B-1 is at 1122.9. The top 3 feet consist of dry, light brown-grey medium dense fine sand with some coarse sand. The layer from 5 feet to 7 feet consists of moist, light grey loose sand with some fine gravel. The layer below to 12 feet consists of moist, grey medium dense sand with fine and coarse gravel. The layer from 15 to 17 feet consists of wet, grey dense to very dense sand with gravel and traces of clay. The final layer from 20 feet to 22 feet consists of wet, grey silty-clay with some fine cobbles and coarse gravel. Refusal was met at a depth of 25 feet. Overall, the top 12 feet consists of medium dense granular material with SPT blow counts in the range of 10-30. Below 12 feet consists of very dense granular material with SPT blow counts exceeding 50. Bedrock was encountered at a depth of 28 feet.

2.3.2. West Boring B-2

The existing ground grade at B-2 is 1123.2. The top 3 feet consist of dry, loose to medium dense brown sand. The layer from 5 to 7 feet consists of moist, loose dark grey sand. The layer below to 12 feet consists of moist, medium dense dark grey sand with some fine cobbles. The layer from 15 to 17 feet consists of wet, brown-tan medium dense sand and stiff clay with traces of coarse sand. The next layer to 22 feet consists of wet, brown-tan medium to very stiff clay and medium dense fine sand. The final layer from 25 feet to 27 feet consists of wet, brown-tan with layers of light grey very stiff clay with traces of medium dense sand. A rock core was then taken at a depth of 30 feet. Overall, the top 12 feet consists of medium dense granular material with SPT blow counts in the range of 10-30. Below 12 feet consists of hard clay material with SPT blow counts exceeding 30.

2.3.3. Soil Testing

Due to the field identification of clay material within the depth of boring B-2, three separate samples were sent out for testing to determine the Atterberg Limits of the material. Samples S4, S5, and S6, with depths ranging from 15'-27' deep were tested. Refer to Appendix 6.5 for testing results. Generally, the samples were found to have relatively high consistency indexes, indicating hard material with high undrained shear strength.

2.3.4. Soil Parameters

See Table 1 for recommended soil parameters for design. See Appendix 6.6 for preliminary calculations.

Table 1: Recommended Soil Parameters (Drained Condition)

Location	Unit Weight γ (lb/ft^3)	Friction Angle Φ (DEG)	Cohesion (ksf)
Boring B-1	125	35	0
Boring B-2	125	0	3.25

1. Friction angle based upon SPT $N1_{60}$ Correlation and AASHTO LRFD Table 10.4.6.2.4-1.
2. Cohesion based upon SPT N60 and Consistency Index Correlation Table 3.6 from Section 3.15, Page 97 of Principles of Foundation Engineering, Eighth Edition by Braja M. Das.

2.4. Liquefaction Potential

Based on the soil conditions found at the bridge site, seismically induced settlement should not be significant; therefore, there is a low potential for liquefaction in the event of seismic activity. Additionally, the site has a low probability of having an event that would trigger liquefaction (Magnitude<6.0).

3. RECOMMENDATIONS

3.1. Reuse Existing Foundations

The existing culvert footings consist of gravity type concrete footings of unknown geometry. The latest inspection report categorizes them as being in satisfactory condition. The only real deficiencies noted occur directly over the concrete encased water line where both footings are cracked with minor spalls present near the top of footing. There are no other signs of distress on the existing footings and no signs of scour. The proposed strengthening concept proposes to pour a 12" thick concrete arch over the existing corrugated steel culvert. The proposed arch will be anchored into the top of the existing footings. The only change to the existing footings in terms of loading will be the result of replacing soil with concrete within the thickness of the arch itself (12”) and within the limits of the concrete headwalls. This amounts to an additional 25 pcf (weight of concrete less the weight of the soil, 150-125) within a small portion of the total fill area. As a result, it can be concluded that the culvert strengthening will have a negligible effect on the adequacy of the existing footings. This combined with the satisfactory current condition of the footings suggests they are a good candidate for reuse for the culvert strengthening.

3.2. Shallow Foundations for Southern Wingwall

The stiff soil will provide adequate bearing resistance to support a lightly loaded spread footing type wingwall foundation. Due to the relatively short height of wall required, conventional MSE walls, gabion faced MSE walls, gravity block walls, gabion walls, or concrete cantilever walls are all suitable types of walls for this location. Factored bearing resistance and settlement will vary depending on the wall type chosen and the resulting footing width.

A spread footing foundation shall be designed to the parameters in Table 1 and section 2.3.4 of this report. Embankment slopes may be constructed at 1.5:1 with added modified rock fill.

3.3. Deep Foundations

A deep foundation is not recommended for this site as it does not provide an economic advantage over a shallow foundation.

4. CONSTRUCTION CONSIDERATIONS

4.1. Water Table

Groundwater was not measured during the subsurface exploration, but it is assumed to be at the stream elevation which was measured at approximately 1112.2 at the North fascia and 1110.8 at the South fascia in May of 2024. Fluctuations with this elevation are expected with the seasonal flows of the stream. Work below this elevation or water elevations of the stream will require dewatering during construction in order to maintain construction in the dry. Discharge of pumped water should be performed in accordance with all federal, state and local regulations which may require a discharge permit.

4.2. Excavation

As required by OSHA regulations, lateral support is required for any excavation depth greater than four feet and where 1.5:1 slope cannot be maintained. Items for temporary earth support should be included in the contract documents. The design of any temporary support of earth (SOE) is the responsibility of the Contractor and should be designed in accordance with MassDOT and AASHTO requirements.

The proposed wingwall construction may require a water barrier system to maintain work in the dry and minimize impacts to the adjacent stream channel and/or wetlands. The water barrier system may consist of sheet pile, sandbags, or a porta dam. The top of the water barrier will need to be set above the 2-year high water elevation.

4.3. Temporary Bridge Foundation

The proposed staging concept consists of utilizing a temporary bridge in order to maintain a single 11’ lane of alternating traffic at all times during construction. The temporary bridge should be founded on abutment structures and shall have a maximum allowable bearing pressure of 4 ksf.

4.4. Obstructions

As is typical, there is the potential to encounter obstructions during excavation activities. The proposed wingwalls are to be located adjacent to the existing footing and run out parallel with the roadway. The exact geometry of the existing footings is unknown.

4.5. Protection of Adjacent Structures and Utilities

There is an 8” water main encased in concrete that runs underground along West Mountain Road. There is an access vault located underneath the roadway at the Northwest corner of the bridge approach. There are also overhead electrical lines that run along West Mountain Road on the North Side. All utilities shall be protected during construction. Coordination with the utility companies shall be performed to determine required construction clearances and to determine if any temporary measures to the utilities would be needed.

4.6. Foundation Preparation

The foundation shall be prepared, and a leveling pad shall be provided per the chosen walls manufacturer’s recommendations.

4.7. Sequence of Construction Activities

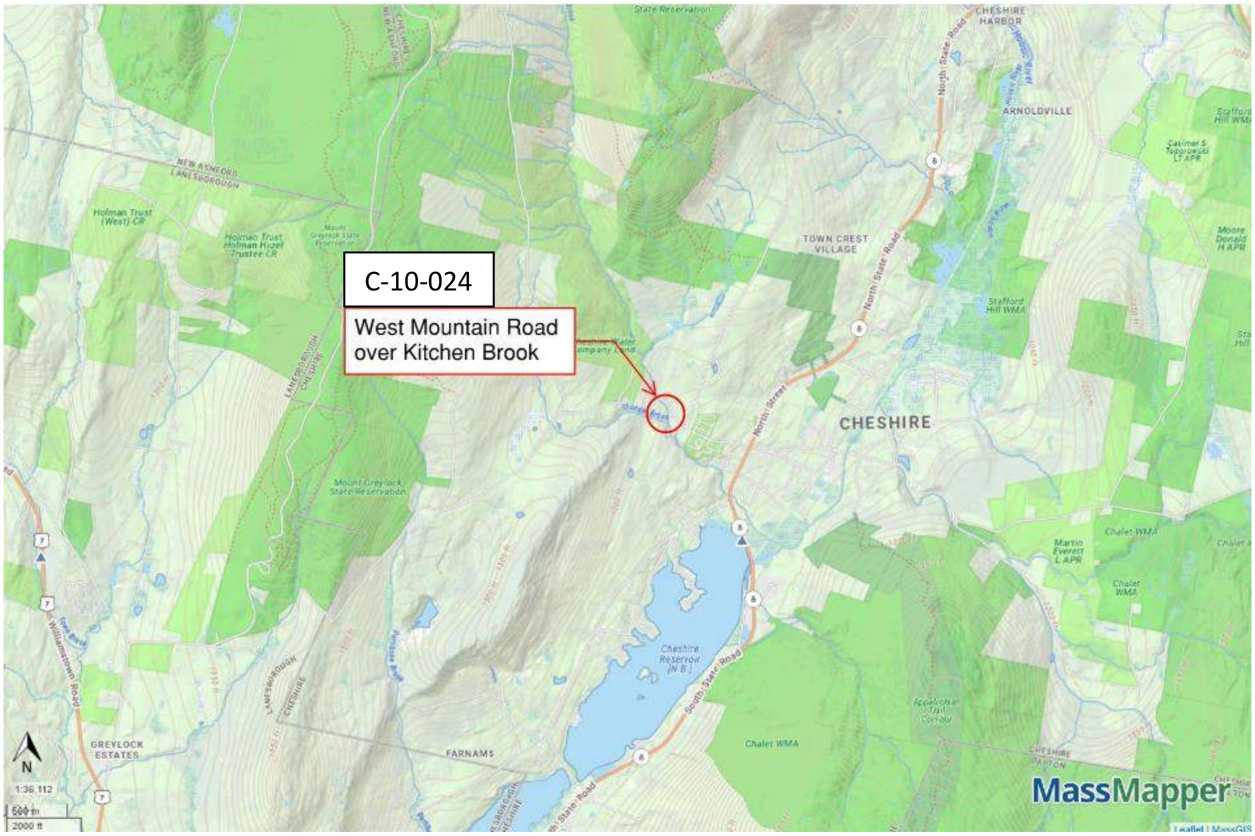
Construction shall be sequenced in order to maintain a single 11’-0” lane of alternating traffic at all times. This can be accommodated by the use of a temporary bridge that spans over the limits of the excavation.

5. CONCLUSION

The project site consists of relatively dense material with adequate bearing resistances to support a shallow foundation type wingwall system. Due to the short height of wall required, gravity type gabion/block walls or conventional/gabion faced MSE walls are all suitable and cost effective options.

APPENDIX

6.1. Project Locus Map


C-10-024
West Mountain Road over Kitchen Brook
CHESHIRE
TOWN CREST VILLAGE
ARNOLDVILLE
CHESHIRE HARBOR
GREYLOCK ESTATES
FARNAMS
North State Road
North Street
South State Road
Williamstown Road
Cheshire Reservoir (N B)
Stafford Hill WMA
Chalet WMA
Mount Greylock State Reservation
Holman Trust (West) CR
NEW ASHFORD
LANESBOROUGH
CHESHIRE
MassMapper

APPENDIX

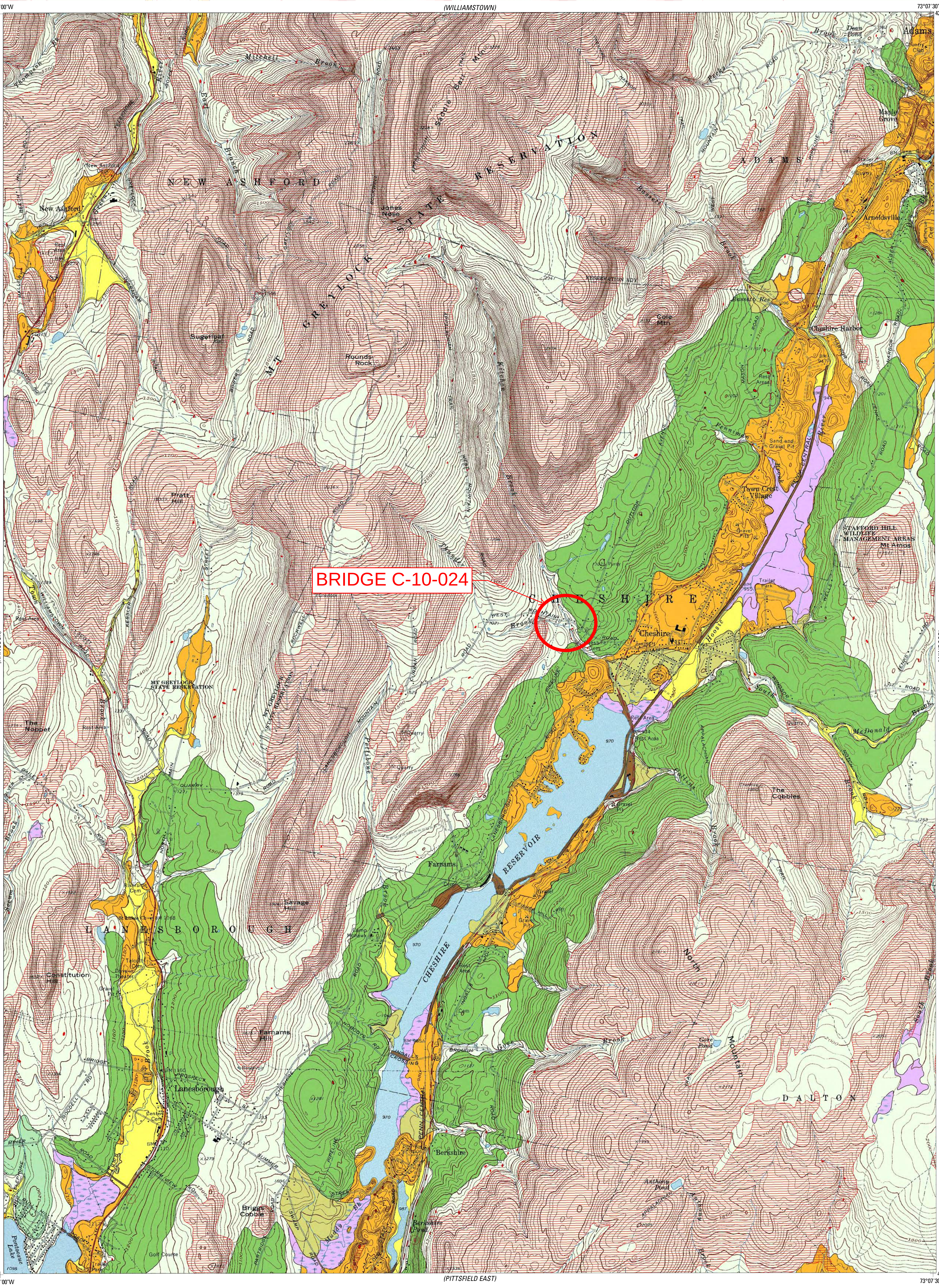
6.2. Geologic Maps

U.S. Department of the Interior
U.S. Geological Survey

Prepared in cooperation with the
Commonwealth of Massachusetts
Massachusetts Geological Survey
and Executive Office for Administration and Finance

Scientific Investigations Map 3402
Quadrangle 12—Cheshire

Discussion of surficial materials and description of map units available at https://doi.org/10.3133/sim3402



Base from U.S. Geological Survey, 1973

Map was scanned, processed, georeferenced, rectified, and cropped by the Massachusetts Geological Survey

Lambert Conformal Conic projection. North American Datum of 1983

Massachusetts state plane coordinate system, mainland zone

14°
TRUE NORTH
MAGNETIC NORTH
APPROXIMATE MEAN DECLINATION, 2018

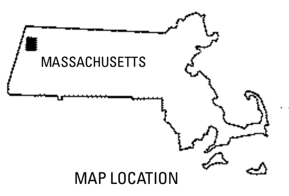
SCALE 1:24 000

1 0.5 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 0.5 0 1 KILOMETER

CONTOUR INTERVAL 20 FEET
NATIONAL GEODETIC VERTICAL DATUM OF 1929



MAP LOCATION

Map units were modified from Holmes (1968a) and from Warren, C.R., 1982, unpublished field map. Some bedrock outcrops are from Ratcliffe and others (1993) and Herz (1958). Shallow-bedrock areas were interpreted from analysis of topographic (lidar) and soils data. Some units were mapped or revised from recent field studies and from analysis of 2005 orthophoto images.

Surficial Materials Map of the Cheshire Quadrangle, Massachusetts

Compiled by
Byron D. Stone and Mary L. DiGiacomo-Cohen
2018

For sale by U.S. Geological Survey, Box 25286, Denver Federal Center, Denver, CO 80225; https://store.usgs.gov; 1–888–ASK–USGS (1–888–275–8747)

Suggested citation: Stone, B.D., and DiGiacomo-Cohen, M.L., comps., 2018, Surficial materials map of the Cheshire quadrangle, Massachusetts, quadrangle 12 *in* Stone, J.R., Stone, B.D., DiGiacomo-Cohen, M.L., and Mabee, S.B., comps., Surficial materials of Massachusetts—A 1:24,000-scale geologic map database: U.S. Geological Survey Scientific Investigations Map 3402, 1 sheet, scale 1:24,000, https://doi.org/10.3133/sim3402.

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USGS
science for a changing world

U.S. Department of the Interior
U.S. Geological Survey

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Scientific Investigations Map 3402
Index map of 7.5-minute quadrangles
Surficial materials maps of quadrangles 1–189 can be accessed individually at https://doi.org/10.3133/sim3402

DESCRIPTION OF MAP UNITS

[Full citations for references are given in the pamphlet. To locate physiographic features and major rivers mentioned in the map-unit descriptions, see figure 1 in the pamphlet]

POSTGLACIAL DEPOSITS

Artificial fill—Earth materials and manmade materials that have been artificially emplaced, primarily in highway and railroad embankments and in dams; unit may also include landfills, urban-development areas, and filled coastal wetlands

Cranberry bog deposits—Natural freshwater swamps or peat bogs overlain locally by artificially emplaced sand or other fill; these deposits occur primarily in southeastern Massachusetts and on Cape Cod. Commonly, cranberry bogs are also created by excavation into sand and gravel deposits that form the bed; peat and other organic material are then artificially emplaced over the bed, and water drainage pathways are diverted into the area to control seasonal flooding of the bog

Flood-plain alluvium—Sand, gravel, silt, and some organic material, stratified and well sorted to poorly sorted, beneath the flood plains of modern streams. The texture of alluvium commonly varies over short distances both laterally and vertically, and generally is similar to the texture of adjacent glacial deposits. Along smaller streams, alluvium is commonly less than 5 feet (ft) thick. The most extensive deposits of alluvium in Massachusetts are along the Housatonic, Deerfield, Westfield, Connecticut, Nashua, Merrimack, and Blackstone Rivers. Alluvium typically overlies thicker glacial stratified deposits

Swamp deposits—Organic muck and peat that contain minor amounts of sand, silt, and clay, are stratified and poorly sorted, and occur in swamps and freshwater marshes, in kettle depressions, or in poorly drained areas. Unit is shown only where deposits are estimated to be at least 3 ft thick; most deposits are less than 10 ft thick. Swamp deposits overlie glacial deposits or bedrock. They locally overlie glacial till even where they occur within thin glacial meltwater deposits

Salt-marsh and estuarine deposits—Peat and organic muck interbedded with sand and silt, deposited in saltwater or brackish-water environments of low wave energy along the coast and in river estuaries. Salt-marsh deposits are dominantly peat and muck, generally a few feet to 25 ft thick. In the major estuaries, these deposits locally overlie estuarine deposits (not mapped), which are sand and silt with minor organic material and are as much as 30 to 80 ft thick. Salt-marsh and estuarine deposits generally are underlain by adjacent glacial material, consisting of till, coarse stratified deposits, or glaciomarine fine deposits

Beach and dune deposits—Sand and fine gravel deposited along the shoreline by waves and currents, and by wind action. The texture of beach deposits varies over short distances and is generally controlled by the texture of nearby glacial materials exposed to wave action. Sand beach deposits are composed of moderately sorted, very coarse to fine sand, and are commonly laminated. Coarser layers may contain some fine gravel particles; finer layers may contain some very fine sand and silt. Gravel beach deposits are composed of granule- to cobble-size clasts in moderately sorted thin beds; deposits contain minor amounts of sand within gravel beds, and thin beds of sand as alternating layers. Beach deposits are rarely more than a few feet thick. Dune deposits are composed of moderately sorted to well-sorted, fine to medium sand, and are variably massive, laminated, and crossbedded. Dune deposits are as much as 100 ft thick. Unit includes artificial sand deposits in locally replenished beaches

EARLY POSTGLACIAL DEPOSITS

Alluvial-fan deposits—Generally coarse gravel and sand deposits on steep slopes where high-gradient streams entered lower gradient valleys. Alluvial fans in some places were graded to lowering levels of glacial lakes. Fans continue to form today at some locations in Massachusetts

Valley-floor fluvial deposits—Sand, gravel, and minor silt, stratified and moderately to poorly sorted, beneath flat floors of valleys, called furrows (Mather and others, 1942), that are eroded into glacial outwash plains. The texture of the fluvial deposits commonly varies over short distances both laterally and vertically, and generally is similar to the texture of adjacent glacial deposits. The fluvial deposits overlie thick glacial stratified deposits in the upper, dry reaches of the furrow valleys and probably are less than 20 ft thick. Swamp deposits and deformation of bedding related to melting of buried ice in kettles interrupt the fluvial deposits. The deposits probably extend beneath salt-marsh and estuarine deposits in coastal valley reaches. The most extensive valley-floor fluvial deposits are on upper Cape Cod along Quaker Run and the Coonamessett, Childs, and Quashnet Rivers, and on Martha's Vineyard in Quampache Bottom

Stream-terrace deposits—Sand, gravel, and silt deposited by meteoric water (locally distal meltwater) on terraces cut into glacial meltwater sediments along rivers and streams. These deposits are shown where they overlie glaciolacustrine deposits (fine deposits map unit) and glaciomarine fine deposits; elsewhere, stream-terrace deposits are included in the coarse deposits map unit. Most stream-terrace deposits are less than 10 ft thick and overlie thicker glacial deposits; textures commonly are similar to those of underlying glacial meltwater deposits. Many stream terraces in the Connecticut River valley are composed of fine to medium sand and overlie lake-bottom silt and clay

Marine regressive deposits—Sand and minor gravel deposited along former, higher shorelines in northeastern Massachusetts by waves and currents, and by wind action on beaches and spits. These deposits are shown where they overlie glaciomarine fine deposits. Regressive beach and nearshore deposits are composed of moderately sorted, very coarse to fine sand, commonly laminated. Coarser layers may contain some fine gravel particles; finer layers may contain some very fine sand and silt. Regressive beach and nearshore deposits are rarely more than a few feet thick. Regressive spit deposits are 10 to 30 ft thick

Inland-dune deposits—Fine to medium, well-sorted sand in transverse, parabolic, and hummocky dunes as much as 60 ft thick. Deposits occur mostly in the glacial Lake Hitchcock basin (in the Connecticut Valley lowland), where sand derived from extensive glacial-lake deltas that were not yet vegetated was deposited in dune forms by early postglacial winds. Dune sand is now fixed by vegetation except where disturbed by human activities

Talus deposits—Angular, loose blocks of basalt and diabase accumulated by rockfall and creep at the base of bedrock cliffs along linear traprock ridges in the Mesozoic lowland (Connecticut Valley lowland). Talus deposits form steep, unstable slopes. Generally less than 20 ft thick

GLACIAL STRATIFIED DEPOSITS

Sorted and stratified sediments composed of gravel, sand, silt, and clay (as defined in the particle-size diagram, below), deposited in layers by glacial meltwater. These sediments occur as four basic textural units: gravel deposits, sand and gravel deposits, sand deposits, and fine deposits. On this surficial geologic map, gravel deposits, sand and gravel deposits, and sand deposits are not differentiated and are shown as *Coarse Deposits* where they occur at the land surface. *Fine Deposits* also are shown where they occur at the land surface. Textural changes occur both areally and vertically; however, subsurface textural variations are not shown on this map.

PARTICLE DIAMETER										
10	2.5	.16	.08	.04	.02	.01	.005	.0025	.00015	inches
256	64	4	2	1	.5	.25	.125	.063	.004	mm
Boulders	Cobbles	Pebbles	Granules	Very coarse sand	Coarse sand	Medium sand	Fine sand	Very fine sand	Silt	Clay
GRAVEL PARTICLES				SAND PARTICLES				FINE PARTICLES		

Grain-size classification used in this report, modified from Wentworth (1922). Abbreviation: mm, millimeter.

Coarse deposits consist of *gravel deposits*, *sand and gravel deposits*, and *sand deposits*, not differentiated in this report. *Gravel deposits* are composed of at least 50 percent gravel-size clasts; cobbles and boulders predominate; minor amounts of sand occur within gravel beds, and sand comprises a few separate layers. Gravel layers generally are poorly sorted, and bedding commonly is distorted and faulted due to postdepositional collapse related to melting of ice. *Sand and gravel deposits* occur as mixtures of gravel and sand within individual layers and as layers of sand alternating with layers of gravel. Sand and gravel layers generally range between 25 and 50 percent gravel particles and between 50 and 75 percent sand particles. Layers are well sorted to poorly sorted; bedding may be distorted and faulted due to postdepositional collapse. *Sand deposits* are composed mainly of very coarse to fine sand, commonly in well-sorted layers. Coarser layers may contain up to 25 percent gravel particles, generally granules and pebbles; finer layers may contain some very fine sand, silt, and clay

Fine deposits include very fine sand, silt, and clay occurring as well-sorted, thin layers of alternating silt and clay (varves), or as thicker layers of very fine sand and silt. Very fine to fine sand commonly occurs at the surface of these lake-bottom deposits and grades downward into rhythmically bedded silt and clay varves. In some places on the lake-bottom surface of glacial Lake Hitchcock (in the Connecticut Valley lowland) and glacial Lake Narragansett (in southeastern Massachusetts), fine deposits are overlain by as much as 30 ft of fine to medium sand, deposited as the lake level lowered or the lake shallowed; this sand has not been mapped separately. Locally, this map unit may include areas underlain by fine sand

Glaciomarine fine deposits include clay, silty clay, fine sand, and some fine gravel deposited in a higher-level sea in environments of low wave energy along the coast and in river estuaries. Fine to very fine sand, massive and laminated, commonly is present at the surface and grades downward into interbedded very fine sand, silt, silty clay, and clay. The lower silty clay and clay is massive and thinly laminated. Total thickness is generally a few feet to 75 ft

Stagnant-ice deposits—Surface coarse sediments include scattered large surface boulders, gravel deposits, and sand and gravel deposits, totaling 5 to 30 ft thick, that overlie predominantly sand deposits. Sand deposits contain deltaic foreset bedding and interlayered beds of fine sand, silt, and a little clay. Sand and silty sand deposits extend downward to basal till and bedrock. Flowtill sediments are interlayered under ice-contact slopes. Stratification in surface and underlying sediments is generally distorted and faulted due to postdepositional collapse related to melting of buried ice. Stagnant-ice deposits are confined to irregular hummocky hills, bounded by ice-contact slopes, present on tops of till hills or extending more than 30 ft above the altitudes of adjacent meltwater morphosequences in lowlands. Deposits are aligned in belts parallel to the retreating ice margin

GLACIAL TILL AND MORAINE DEPOSITS

End moraine deposits—Composed predominantly of boulders and ablation-facies sandy upper till; lenses of stratified sand and gravel occur locally within the till. In the larger deposits on Cape Cod and Martha's Vineyard, the surface ablation till is as much as 30 ft thick and overlies sand, gravel, and silty sand meltwater deposits. Some end moraine deposits include thrusted sheets of glacial meltwater deposits resulting from readvance of the ice margin (Oldale and O'Hara, 1984). Stratification in underlying sediments may also be deformed, the result of postdepositional collapse caused by melting of buried ice. Surface boulders on end moraine deposits are generally more numerous than on adjacent till surfaces; dense concentrations of boulders are present in some places. Deposits occur as freestanding hummocky landforms, commonly in ridges that trend east-northeast to west-southwest, and range in height from 10 to 100 ft

Thrust moraine deposits—In western Martha's Vineyard, thrusted moraine deposits stand as high as 300 ft in altitude and are composed of allochthonous, ice-thrusted Cretaceous, Tertiary, and older Quaternary sediments, locally overlain by thin surface till and boulders. These coastal-plain beds are fossiliferous, semi-consolidated sand, gravel, and silty clay in tilted strata that were thrust up by glacial ice into positions well above the autochthonous coastal-plain surface, which lies below sea level. Numerous northeast-southwest-trending ridges within the thrust moraine unit mark the edges of these tilted and thrusted strata

Thin till—Nonsorted, nonstratified matrix of sand, some silt, and little clay containing scattered pebble, cobble, and boulder clasts; large surface boulders are common; unit was mapped where till is generally less than 10 to 15 ft thick including areas of shallow bedrock. Predominantly consists of upper till of the last glaciation; loose to moderately compact, generally sandy, commonly stony. Two facies are present in some places: a looser, coarser grained ablation facies, melted out from supraglacial position; and an underlying more compact, finer grained lodgement facies deposited subglacially. In general, both ablation and lodgement facies of upper till derived from fine-grained bedrock are finer grained, more compact, less stony and have fewer surface boulders than upper till derived from coarse-grained crystalline rocks. Across Massachusetts, fine-grained bedrock sources include the red Mesozoic sedimentary rocks of the Connecticut Valley lowland, marble in the western river valleys, and fine-grained schists in upland areas

Thick till—Nonsorted, nonstratified matrix of sand, some silt, and little clay containing scattered pebbles, cobbles, and boulders in the shallow subsurface; at greater depths consists of compact, nonsorted matrix of silt, very fine sand, and some clay containing scattered small gravel clasts. Mapped in areas where till is greater than 10 to 15 ft thick, mostly in drumlin landforms in which till thickness commonly exceeds 100 ft (maximum recorded thickness is 230 ft). Although upper till of late Wisconsinan age is the surface deposit, lower till of probable Illinoian age constitutes the bulk of the material in thick-till areas. Lower till is moderately to very compact and is commonly finer grained and less stony than upper till. An oxidized zone, the lower part of a soil profile formed during a period of interglacial weathering, is generally present in the upper part of the lower till. This zone commonly shows closely spaced joints that are stained with iron and manganese oxides

Glacially modified coastal-plain hill deposits—In the Marshfield Hills area (Scituate, Cohasset, Hanover, and Duxbury quadrangles) and in the Pine Hills area (Manomet quadrangle), very compact till and older glacial stratified deposits overlie thrusted blocks of Tertiary coastal-plain strata that are semi-consolidated dark clay layers. Miocene-age green sand deposits have also been reported at depth. These hills in many places were sculpted by the last ice sheet, but they are generally larger (3–4 miles [mi] long and 1–2 mi wide) than typical drumlins

Thick valley till and fine deposits—Composed of sandy surface till with boulders, 3 to 20 ft thick, overlying finer grained till, or fine sand, silt, or clay, local boulders, and local weathered limestone and dolostone bedrock; total thickness of all sediments is 6 to 135 ft, averaging 50 ft. Materials reported in drillers' records include four descriptions usually synonymous with till: hardpan with no boulders; boulders and clay; gravelly hardpan; and clay with few boulders. Unit includes materials probably defining glaciolacustrine fine sediments or various weathered carbonate bedrock materials, listed as follows: gray clay, gray and yellow clay, black soft rock, and weathered bedrock. The subsurface fine sediments are exposed only in fresh, temporary landslide slopes or shallow excavations, where silty-clayey fine sand typically appears to be sheared, deformed, or disaggregated. Original laminations are difficult to discern. Surface morphology of the thick valley till and fine deposits includes (1) a glacially smoothed surface without bedrock outcrops or any relief related to bedrock structure; (2) locally a streamlined shape similar to small drumlins composed of thick till in other parts of Massachusetts; (3) landslide scarps and stream-cut banks commonly having 5 to 10 ft of relief, locally as much as 50 ft; and (4) dry, meltwater-carved channels 3 to 10 ft deep. These deposits extend almost continuously along lower valley slopes in the Housatonic and Hoosic River valleys, and their tributary valleys, that are underlain by marble, dolostone, or limestone and shale bedrock (Zen and others, 1983). The deposits appear to extend beneath the edges of glacial meltwater deposits in the valley bottoms, but their extent beneath thick glacial deposits in the centers of the valleys is not known. Some of these deposits are present in north-draining upland valleys in areas that also contain thick till deposits in drumlins

BEDROCK AREAS

Bedrock outcrops and areas of abundant outcrop or shallow bedrock—Individual bedrock outcrops, and areas of shallow bedrock ***or*** areas where small outcrops are too numerous to map individually; in areas of shallow bedrock, surficial materials are less than 5 to 10 ft thick. These units were not mapped consistently among all quadrangles; see note at the beginning of appendix 1 in the pamphlet for information on bedrock outcrop mapping by quadrangle

Index Map of 7.5-Minute Quadrangles in Massachusetts Showing Distribution of Surficial Materials

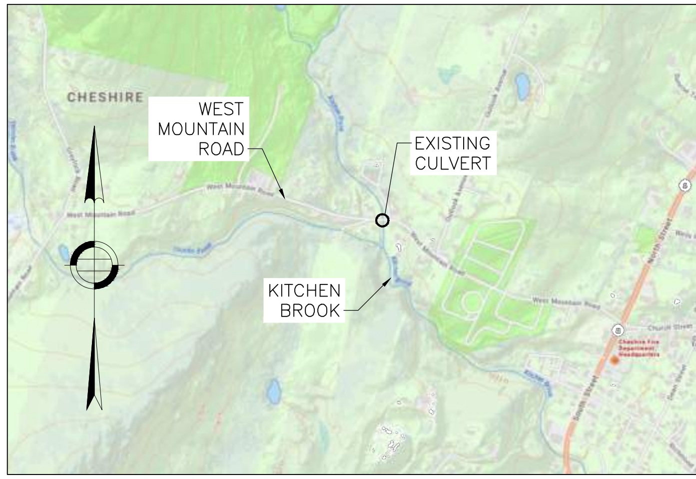
Surficial Materials of Massachusetts—A 1:24,000-Scale Geologic Map Database

Compiled by
Janet Radway Stone,[1] Byron D. Stone,[1] Mary L. DiGiacomo-Cohen,[1] and Stephen B. Mabee[2]
[1]U.S. Geological Survey, [2]Massachusetts Geological Survey

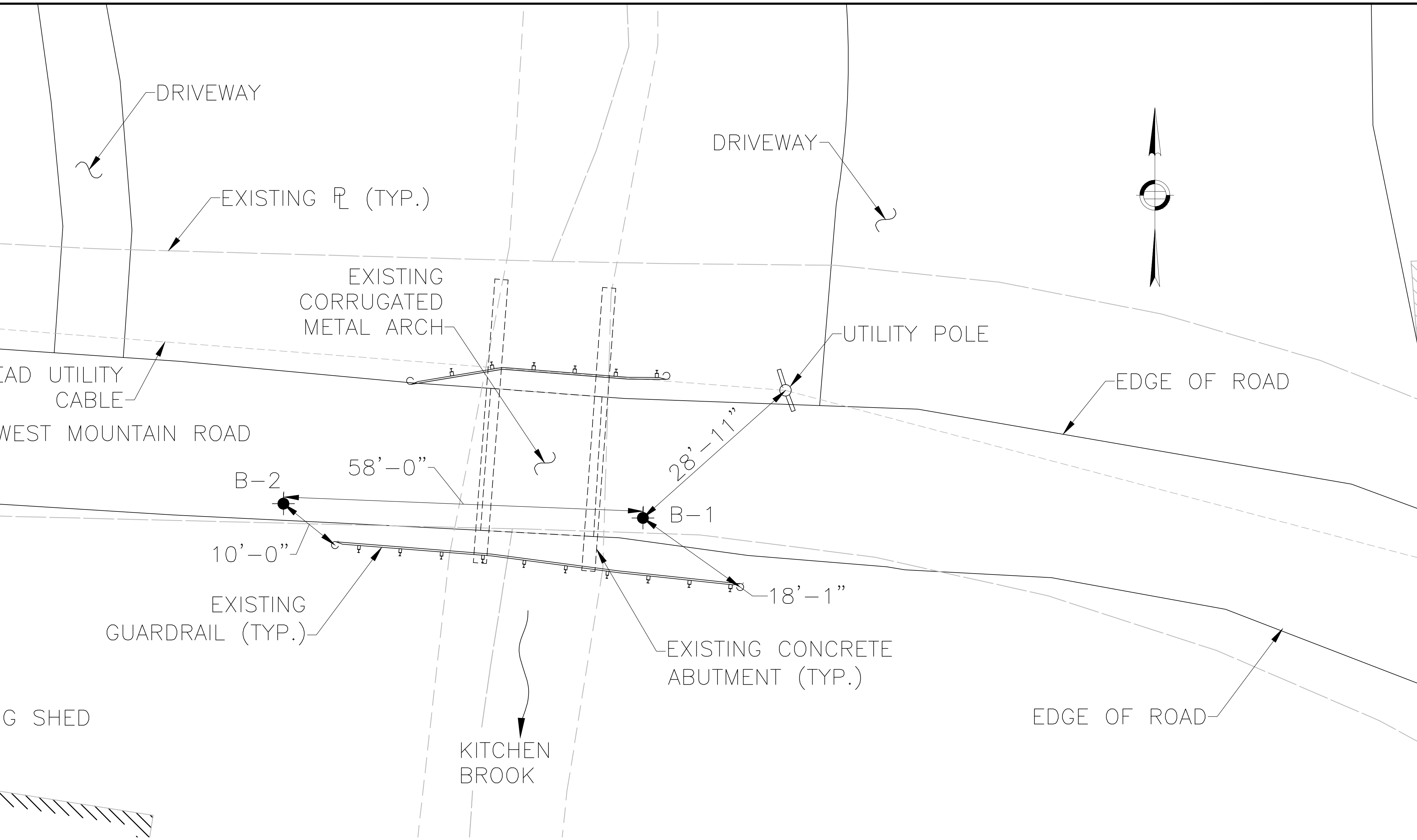
2018

APPENDIX

6.3. As-Drilled Boring Plan



LOCUS
NOT TO SCALE



BORING PLAN
NOT TO SCALE

NOTES:

1. LOCATION OF DRIVE SAMPLE BORINGS ARE SHOWN THUS:
2. BORING DEPTHS SHALL BE APPROXIMATELY 50 FEET.
3. BORING LOGS SHALL BE PREPARED AND SUBMITTED TO GILL ENGINEERING ASSOCIATES.
4. BORING LOCATIONS MAY BE ADJUSTED TO AVOID CONFLICTS WITH EXISTING UTILITIES.
5. SOIL SAMPLES SHALL BE PROVIDED TO THE ENGINEER ON-SITE FOR STORAGE AT GILL ENGINEERING.
6. THE BORING CONTRACTOR SHALL BE PREPARED TO OBTAIN ROCK CORES SHOULD ROCK BE ENCOUNTERED AT SHALLOW DEPTHS.

GILL ENGINEERING

BORING PLAN
TOWN OF CHESHIRE

BORING PLAN FOR CHESHIRE C10024
CULVERT OVER KITCHEN BROOK

PLAN VIEW
6/29/2023

SHEET 01 OF 01

Gill Engineering Associates, Inc. 63 Kendrick Street Needham, MA 02494 781-355-7100 www.gill-eng.com

APPENDIX

6.4. Boring Logs

GILL ENGINEERING	**Gill Engineering Associates, Inc.** **63 Kendrick Street** **Needham, MA 02494**	**Boring No. B-1** Page 1 of 1

City/Town: Cheshire	Bridge Number: C-10-024	Project File Number:	Contract Number:
Location: South Approach		Date & Time Started: 9/25/23 at 10:15am	Total Hours: 4.0
Groundwater Depth (Feet): 15	Date & Time: N/A	Date & Time Completed: 9/25/23 at 2:20pm	
Coordinates: N 61 °09' 10" E 71° 24' 24"		Driller's Company & Name: Richard Posa NEBC	
Ground Elevation (Feet): 1124+/-		Gill Representative: Kyle Coleman	

Depth (Feet)	Sample Number	Depth Range (Feet)	Blow Counts per 6 Inches / Coring Times Minutes per Foot	Recovery (inches)	Field Description	Strata Changes
-	S1	0-2	9-15-14-19	16	Dry, medium dense, light brown-grey FINE SAND with some coarse sand	
-						3.0
-						
-						
5	S2	5-7	15-8-4-12	8	Moist, loose, light grey SAND with some fine gravel	
-						
-						
-						
-						
10	S3	10-12	10-9-6-4	9	Moist, medium dense, grey SAND and fine and coarse GRAVEL	
-						
-						13.0
-						
-						
15	S4	15-17	41-36-50/3"	5	Wet, dense to very dense, grey SAND and GRAVEL with traces of clay	
-						
-						
-						
-						
20	S5	20-22	24-32-38-37	15	Wet, silty, grey CLAY with some fine cobbles and coarse gravel	
-						
-						
-						
-						25.0
25	S6	25-27	50/0" REFUSAL	0	No Recovery	
-						
-						
-		28	ROCK CORE TAKEN			
-						
30						

Remarks:	Arrow-Board: Signs: Cones:	Protective Device – Stand: Box: Well Depth: Solid Pipe: Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide				
Cohesionless Soils (Sands, Gravels)		Cohesive Soils (Silts, Clays)		Type of Drill Rig: B53
Relative Density	Penetration Resistance	Consistency	Penetration Resistance	Casing Type: FJ Size: 4"
Very Loose	0 – 4	Very Soft	0 – 2	Hammer Weight: 140lbs
Loose	4 – 10	Soft	2 – 4	Fall: 30"
Medium Dense	10 – 30	Medium Stiff	4 – 8	Depth:
Dense	30 – 50	Stiff	8 – 15	Sampler Type: SS Size: 1-3/8"
Very Dense	Over 50	Very Stiff	15 – 30	Automatic Hammer Weight: 140lbs
		Hard	Over 30	Safety Hammer Weight:
				Donut Hammer Weight:
N = Sum of Second and Third 6" Blow counts				Fall: 30"
Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less				Core Barrel Type: NQ Size: 1-7/8"

Gill Engineering Associates, Inc.
63 Kendrick Street
Needham, MA 02494

Boring No. B-2

City/Town: Cheshire	Bridge Number: C-10-024	Project File Number:	Contract Number:
Location: South Approach		Date & Time Started: 9/25/23 at 3:15pm	Total Hours: 4.5
Groundwater Depth (Feet): 15	Date & Time: N/A	Date & Time Completed: 9/26/23 at 2:30pm	
Coordinates: N 61 ° 09' 10" E 71° 24' 27"		Driller's Company & Name: Richard Posa NEBC	
Ground Elevation (Feet): 1123.2		Gill Representative: Kyle Coleman	

Depth (Feet)	Sample Number	Depth Range (Feet)	Blow Counts per 6 Inches / Coring Times Minutes per Foot	Recovery (inches)	Field Description	Strata Changes
-	S1	0-2	10-10-9-11	11	Dry, loose to medium dense, brown SAND	
-						
-						
-						
5	S2	5-7	10-7-7-10	1	Moist, loose, dark grey SAND	
-						
-						
-						
-						
10	S3	10-12	28-17-11-4	11	Moist, medium dense, dark grey SAND with Some fine cobbles	
-						
-						13.0
-						
-						
15	S4	15-17	9-14-17-19	7	Wet, medium dense, brown-tan SAND and stiff CLAY with traces of coarse sand	
-						
-						
-						
-						
20	S5	20-22	11-18-20-27	19	Wet, very stiff, brown-tan clay and medium dense fine SAND	
-						
-						
-						
-						
25	S6	25-27	13-16-26-26	7	Wet, very stiff, brown-tan with layers of light grey CLAY with traces of medium dense sand	
-						
-						
-						
-						
30		30	ROCK CORE TAKEN			

Remarks:	Arrow-Board: Signs: Cones:	Protective Device – Stand: Box: Well Depth: Solid Pipe: Stick Up Pipe: Screen Pipe:

Penetration Resistance (N) Guide

Cohesionless Soils (Sands, Gravels)		Cohesive Soils (Silts, Clays)	
Relative Density	Penetration Resistance	Consistency	Penetration Resistance
Very Loose	0 – 4	Very Soft	0 – 2
Loose	4 – 10	Soft	2 – 4
Medium Dense	10 – 30	Medium Stiff	4 – 8
Dense	30 – 50	Stiff	8 – 15
Very Dense	Over 50	Very Stiff	15 – 30
		Hard	Over 30

N = Sum of Second and Third 6" Blow counts

Terms Used for Second Entry of Descriptions: and = 40-50%, some = 10-40%, trace = 10% or less

Type of Drill Rig: B53

Casing Type: FJ Size: 4"
Hammer Weight: 140lbs
Fall: 30"
Depth:

Sampler Type: SS Size: 1-3/8"
Automatic Hammer Weight: 140lbs
Safety Hammer Weight:
Donut Hammer Weight:
Fall: 30"

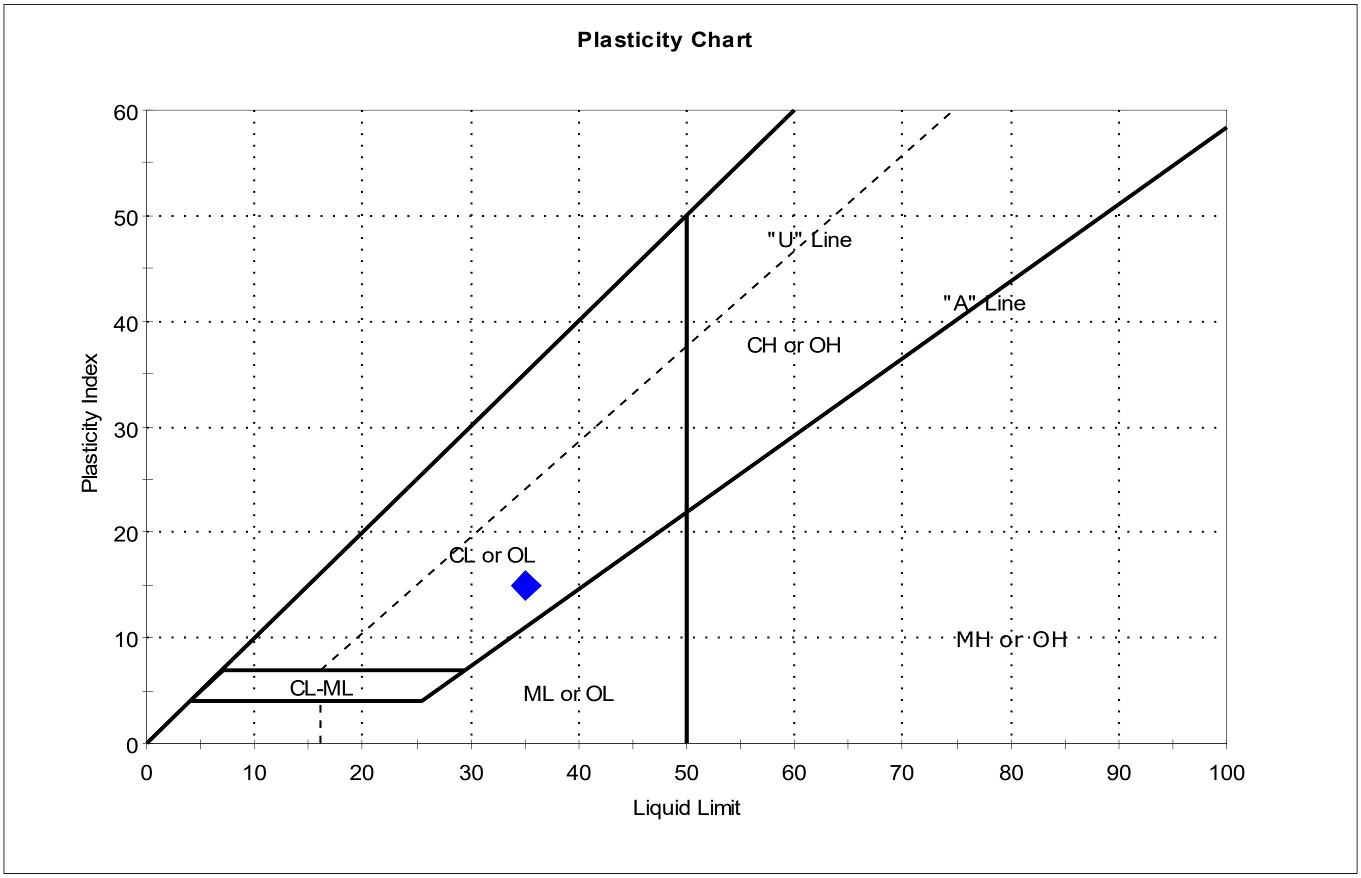
Core Barrel Type: NQ Size: 1-7/8"

APPENDIX

6.5. Clay Sample Testing Results

Client:	Gill Engineering Associates, Inc.		
Project:	C10024		
Location:	Cheshire, MA	Project No:	GTX-318007
Boring ID: B-2	Sample Type: tube	Tested By:	ckg
Sample ID: S4-5	Test Date: 10/25/23	Checked By:	ank
Depth : 15-22'	Test Id: 740200		
Test Comment:	---		
Visual Description:	Moist, reddish brown clay		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S4-5	B-2	15-22'	15	35	20	15	-0.3	

Sample Prepared using the WET method

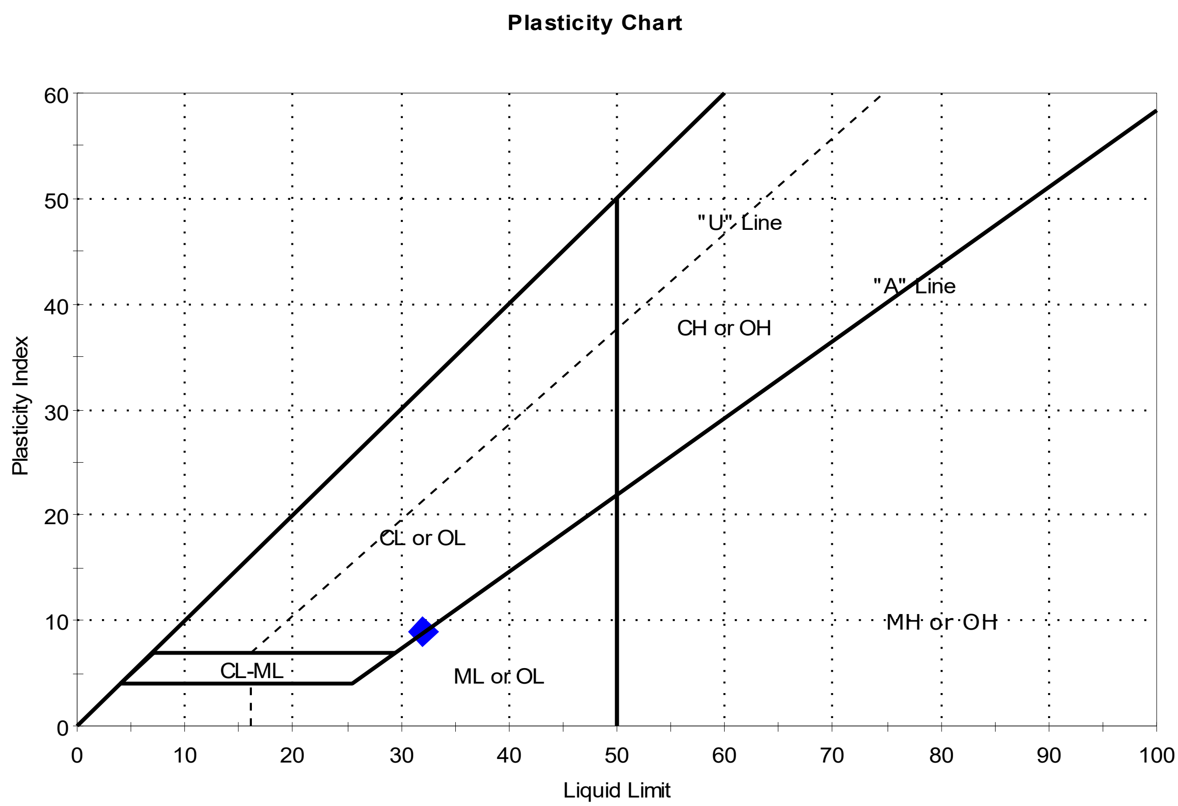
Dry Strength: HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Gill Engineering Associates, Inc.		
Project:	C10024		
Location:	Cheshire, MA	Project No:	GTX-318007
Boring ID: B-2	Sample Type: tube	Tested By:	ckg
Sample ID: S5	Test Date: 10/25/23	Checked By:	ank
Depth : 20-22'	Test Id: 740201		
Test Comment:	---		
Visual Description:	Moist, brown clay		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S5	B-2	20-22'	20	32	23	9	-0.3	

Sample Prepared using the WET method

Dry Strength: HIGH
Dilatancy: SLOW
Toughness: LOW

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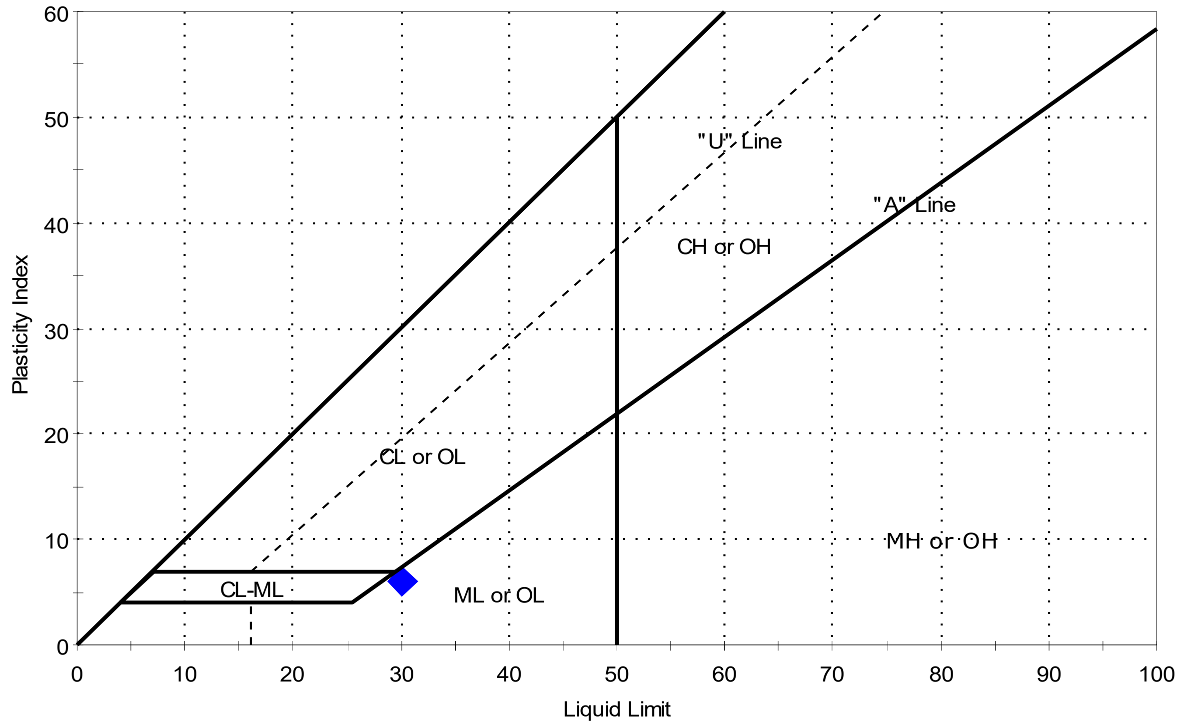
Client:	Gill Engineering Associates, Inc.				
Project:	C10024				
Location:	Cheshire, MA			Project No:	GTX-318007
Boring ID:	B-2	Sample Type:	tube	Tested By:	ckg
Sample ID:	S6	Test Date:	10/26/23	Checked By:	ank
Depth :	25-27'	Test Id:	740202		

Test Comment: ---
Visual Description: Moist, brown silt
Sample Comment: ---

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S6	B-2	25-27'	21	30	24	6	-0.5	

Sample Prepared using the WET method

Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

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APPENDIX

6.6. Preliminary Design Calculations

Geotechnical Calculations - Boring B-1 C-10-024

References:
1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020
2. MassDOT LRFD Bridge Manual, 2024

Calc Narrative:
For boring B-1, the majority of the soil was classified as sand and therefore the soil at B-1 will be treated as granular soil.

Soil Strength: (Ref 1 - 10.4.6.2)

Drained strength of granular soils, Ref 1 - 10.4.6.2.4

If SPT N values are used, they shall be corrected for the effects of overburden pressure determined as:

$N1 = C_N N$ (Ref 1 - Eq. 10.4.6.2.4-1)

Where $$C_N = \left[0.77\log_{10}\left(\frac{40}{\sigma'_v}\right)\right] < 2.0$$

σ'_v = vertical effective stress (ksf)
N = uncorrected SPT blow count (blows/ft)

SPT values should also be corrected for hammer efficiency, determined as:

$N_{60} = (ER/60\%)N$ (Ref 1 - Eq. 10.4.6.2.4-2)

Where

ER = hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used.

Therefore, when SPT blow counts have been corrected for both overburden effects and hammer efficiency effects, the resulting corrected blow count shall be denoted as $N1_{60}$, determined as:

$N1_{60} = C_N N_{60}$ (Ref 1 - Eq. 10.4.6.2.4-3)

The soil friction angle can then be determined based on the following table:

Ref 1 - Table 10.4.6.2.4-1		
$N1_{60}$	ϕ_f (low)	ϕ_f (high)
<4	25	30
4	27	32
10	30	35
30	35	40
50	38	43

Geotechnical Calculations - Boring B-1 C-10-024

Determine friction angle based on soil profile of closest boring:

Boring # =	B-1	Water Table Depth =	15.00 ft
Soil Unit Weight =	0.125 kcf	Ground Elevation =	1123.00 ft
Water Unit Weight =	0.0624 kcf		

Layer #	Depth to Bottom of Layer	Midpoint of Sample	h_1	h_2	Layer Description
1	4.00 ft	2.00 ft	2.00 ft	0.00 ft	Medium Dense Fine Sand
2	13.00 ft	8.50 ft	8.50 ft	0.00 ft	Loose-Medium Dense Sand
3	18.00 ft	15.50 ft	15.00 ft	0.50 ft	Dense-Very Dense Sand and Gravel
4	25.00 ft	21.50 ft	15.00 ft	6.50 ft	Hard Silty Clay
5	38.00 ft	33.00 ft	15.00 ft	18.00 ft	Bedrock

Layer #	σ'_v	C_N	N	Hammer Efficiency	N_{60}	$N1_{60}$
1	0.250 ksf	1.70	29	0.80	39	66
2	1.063 ksf	1.21	13.5	0.80	18	22
3	1.906 ksf	1.02	77	0.80	103	104
4	2.282 ksf	0.96	70	0.80	93	89
5	3.002 ksf	0.87	n/a	0.80	-	-

Layer #	ϕ_f (low)	ϕ_f (high)	ϕ_f (avg)	Start El.	End El.
1	38	43	40	1123.00 ft	1119.00 ft
2	33	38	35	1119.00 ft	1110.00 ft
3	38	43	40	1110.00 ft	1105.00 ft
4	38	43	40	1105.00 ft	1098.00 ft
5	-	-	-	1098.00 ft	1085.00 ft

Geotechnical Calculations - Southeast Wingwall Bearing Resistance C-10-024

References:
1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020
2. MassDOT LRFD Bridge Manual, 2024

The proposed wingwall at the southeast corner of the bridge will be founded on the existing soil defined in boring B-1. The following calculations provide the bearing resistance of the soil that the wingwall will be founded on.

Note that the proposed bottom of wall elevation for the wingwalls is 1109 ft +/- (14.5 ft below grade). The minimum soil friction angle calculated for boring B-1 was found to be 35, which represents dense material.

Bearing Resistance of Soil: (Ref 1 - 10.6.3.1)

Factored Resistance, $q_R = \phi_b q_n$ (Ref 1 - Eq. 10.6.3.1.1-1)

$\phi_b =$ 0.45 (Ref 1 - Table 10.5.5.2.2-1)

The nominal bearing resistance of a soil layer, in ksf, should be taken as:

$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$ (Ref 1 - Eq. 10.6.3.1.2a-1)

Where: $N_{cm} = N_c s_c i_c$ (Ref 1 - Eq. 10.6.3.1.2a-2)

$N_{qm} = N_q s_q d_q i_q$ (Ref 1 - Eq. 10.6.3.1.2a-3)

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ (Ref 1 - Eq. 10.6.3.1.2a-4)

Unit Wt. =	0.125 kcf	
$c =$	0.00 ksf	(cohesion, taken as undrained shear strength, 0 for granular soil)
$\phi_f =$	36 deg	(minimum value for Boring B-1)
$N_c =$	50.6	(Ref 1 - Table 10.6.3.1.2a-1)
$N_q =$	37.8	(Ref 1 - Table 10.6.3.1.2a-1)
$N_\gamma =$	56.3	(Ref 1 - Table 10.6.3.1.2a-1)
$D_f =$	2 ft	(minimum allowable embedment)
$B =$	11.0 ft	(nominal footing width)

Per Ref 1 - 10.6.3.1, where loads are eccentric, the effective footing dimensions, L' and B', as specified in Article 10.6.1.3, shall be used instead of the overall dimensions L and B in all equations, tables, and figures pertaining to bearing resistance.

$B' = B - 2e_B$ (Ref 1 - Eq. 10.6.1.3-1)

$L' = L - 2e_L$ (Ref 1 - Eq. 10.6.1.3-1)

Eccentricity in L direction can typically be neglected in abutment design, therefore L' = L

$e_B =$	3.67 ft	(1/3 B, maximum e allowed by code, Ref 1 - 10.6.3.3)
$B' =$	3.67 ft	
$L =$	22.00 ft	(approximate length of footing)

Geotechnical Calculations - Southeast Wingwall Bearing Resistance C-10-024

C_{wq} =	0.75	(Ref 1 - Table 10.6.3.1.2a-2)
$C_{w\gamma}$ =	0.5	(Ref 1 - Table 10.6.3.1.2a-2)
s_c =	1.12	(Ref 1 - Table 10.6.3.1.2a-3)
s_γ =	0.80	(Ref 1 - Table 10.6.3.1.2a-3)
s_q =	1.36	(Ref 1 - Table 10.6.3.1.2a-3)
d_q =	1.00	(Ref 1 - 10.6.3.1.2a)
i_c =	1.00	(Ref 1 - C10.6.3.1.2a, for footings with modest embedment, typically ignored)
i_q =	1.00	(Ref 1 - C10.6.3.1.2a, for footings with modest embedment, typically ignored)
i_γ =	1.00	(Ref 1 - C10.6.3.1.2a, for footings with modest embedment, typically ignored)

N_{cm} =	56.90 =	50.60 x	1.12 x	1.00	
N_{qm} =	51.53 =	37.80 x	1.36 x	1.00 x	1.00
$N_{\gamma m}$ =	45.04 =	56.30 x	0.80 x	1.00	

cN_{cm} =	0.00 =	0.00 ksf x	56.90			
$\gamma D_f N_{qm} C_{wq}$ =	9.66 =	0.125 kcf x	2 ft x	51.5 x	0.75	
$0.5\gamma B N_{\gamma m} C_{w\gamma}$ =	5.16 =	0.50 x	0.125 kcf x	3.67 ft x	45.04 x	0.5

q_n = 0.00 + 9.66 + 5.16 = = 14.82 ksf

q_u = 0.45 x 14.82 ksf = 6.7 ksf

Geotechnical Calculations - Southeast Wingwall Bearing Resistance C-10-024

Settlement Analyses: (Ref 1 - 10.6.2.4)

Total settlement, including elastic, consolidation, and secondary components may be taken as:

$S_t = S_e + S_c + S_s$ (Ref 1 - Eq. 10.6.2.4.1-1)

For cohesionless soils, only elastic settlement is typically considered, therefore $S_t = S_e$

Elastic settlement can be approximated using either the elastic half-space method or the empirical Hough method. Both methods are calculated here for reference.

Elastic Half-Space Method:

$$S_e = \frac{[q_o(1-v^2)\sqrt{A'}]}{144 E_s \beta_z}$$ (Ref 1 - Eq. 10.6.2.4.2-1)

Where:

q_o = applied vertical stress (ksf)

A' = effective area of footing (ft^2)

E_s = Young's modulus of soil taken from Article 10.4.6.3

B_z = shape factor taken as specified in table 10.6.2.4.2-1

v = Poisson's Ratio, taken as specified in Article 10.4.6.3

P = 37 kips (total vertical load on wingwall, conservatively use Strength I)

A' = B'L' = 81 ft² = 3.67 ft x 22.00 ft

q_o = P/A' = 0.5 ksf = 37 kips/ 81 ft²

E_s = 2.00 ksi (Ref 1 - Table C10.4.6.3-1, loose-medium dense sand)

v = 0.25

L/B = 6.00 = 22.00 ft/ 3.67 ft

Footing Type = Flexible (Typical spread footing)

βz = 1.258

Se = 0.01 ft = $\dfrac{0.5 \text{ ksf} \quad (1 - \quad 0.06) \text{ x} \quad 8.98 \text{ ft}}{(144 \text{ x} \quad 2.00 \text{ x} \quad 1.258)}$

Se = 0.13 in

Geotechnical Calculations - Southeast Wingwall Bearing Resistance C-10-024

Empirical Hough Method:

$$S_e = \sum_{i=1}^{n} \Delta H_i$$ (Ref 1 - Eq. 10.6.2.4.2-2)

Where: $$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$$ (Ref 1 - Eq. 10.6.2.4.2-3)

n = number of soil layers within zone of stress influence of the footing

H_c = initial height of each layer I (ft)

C' = bearing capacity index from Figure 10.6.2.4.2-1

σ'_o = initial vertical effective stress at the midpoint of layer I (ksf)

$\Delta\sigma_v$ = increase in vertical stress at the midpoint of layer I (ksf)

Depth of bottom of footing below grade, d = 15 ft

Increase in vertical stress at point directly below footing is equal to the applied vertical stress, q_o (ksf). Increase in vertical stress at a depth of z below the bottom of footing are assumed equal to applied load P divided by effective area at point of interest calculated assuming a 2:1 distribution slope, therefore A' = (B'+z)(L'+z)

Layer	Depth to Bottom of Layer	H_c	Midpoint Depth Below Footing, z	σ'_v	A'	$\Delta\sigma_v$ (P/A')
1	4.00 ft	4.00 ft	0.00 ft	0.250 ksf	81 ft²	0.5 ksf
2	13.00 ft	9.00 ft	0.00 ft	1.063 ksf	81 ft²	0.5 ksf
3	18.00 ft	5.00 ft	1.50 ft	1.906 ksf	121 ft²	0.3 ksf
4	25.00 ft	7.00 ft	6.50 ft	2.282 ksf	290 ft²	0.1 ksf
5	38.00 ft	13.00 ft	16.50 ft	3.002 ksf	776 ft²	0.0 ksf

Layer	$N1_{60}$	*C'	ΔH
1	66	170	-
2	22	70	-
3	104	200	0.00 ft
4	89	200	0.00 ft
5	-	-	-

Σ = 0.00 ft = 0.03 in

*In Figure 10.6.2.4.2-1 Clean well graded fine to coarse Sand was assumed

Geotechnical Calculations - Southwest Wingwall Bearing Resistance C-10-024

References:

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020
2. MassDOT LRFD Bridge Manual, 2024

The proposed wingwall at the southwest corner of the bridge will be founded on the existing soil defined in boring B-2. The following calculations provide the bearing resistance of the soil that the wingwall will be founded on.

Note that the proposed bottom of wall elevation for the wingwalls is 1109 ft +/- (14.5 ft below grade). The majority of material found below this depth at boring B-2 was identified (and tested) as hard clay. As such, a conservative value was used for the cohesion (undrained shear strength) with a corresponding friction angle of 0 for determining bearing resistance.

Bearing Resistance of Soil: (Ref 1 - 10.6.3.1)

Factored Resistance, $q_R = \phi_b q_n$ (Ref 1 - Eq. 10.6.3.1.1-1)

$\phi_b =$ 0.50 (Ref 1 - Table 10.5.5.2.2-1)

The nominal bearing resistance of a soil layer, in ksf, should be taken as:

$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$ (Ref 1 - Eq. 10.6.3.1.2a-1)

Where: $N_{cm} = N_c s_c i_c$ (Ref 1 - Eq. 10.6.3.1.2a-2)

$N_{qm} = N_q s_q d_q i_q$ (Ref 1 - Eq. 10.6.3.1.2a-3)

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ (Ref 1 - Eq. 10.6.3.1.2a-4)

Unit Wt. = 0.125 kcf

To determine the undrained shear strength, a conservative assumption will be necessary based on the SPT blow counts found in the field. From the Text Book "Principles of Foundation Engineering" Eigth Edition, by Braja M. Das, there is a correlation table of SPT blow counts (N60) and estimated consistency of the cohesive soil vs empirical values to be used for the unconfined compressive strength. It should be noted that the undrained shear strength is equal to 1/2 the unconfined compressive strength.

Table 3.6 Approximate Correlation between CI, N_{60}, and q_u

Standard penetration number, N_{60}	Consistency	CI	Unconfined compression strength, q_u (kN/m^2)	(lb/ft^2)
<2	Very soft	<0.5	<25	500
2–8	Soft to medium	0.5–0.75	25–80	500–1700
8–15	Stiff	0.75–1.0	80–150	1700–3100
15–30	Very stiff	1.0–1.5	150–400	3100–8400
>30	Hard	>1.5	>400	8400

Suc = 6.50 ksf (SPT counts >32, Hard Clay, CI around 1.33)

c = 3.25 ksf (cohesion, taken as undrained shear strength, 0 for granular soil, Suc/2 for cohesive soil)

$\phi_f =$ 0 deg (from above)

N_c =	5.1	(Ref 1 - Table 10.6.3.1.2a-1)
N_q =	1.0	(Ref 1 - Table 10.6.3.1.2a-1)
N_γ =	0.0	(Ref 1 - Table 10.6.3.1.2a-1)
D_f =	2 ft	(minimum allowable embedment)
B =	11.0 ft	(nominal footing width)

Per Ref 1 - 10.6.3.1, where loads are eccentric, the effective footing dimensions, L' and B', as specified in Article 10.6.1.3, shall be used instead of the overall dimensions L and B in all equations, tables, and figures pertaining to bearing resistance.

$B' = B - 2e_B$ (Ref 1 - Eq. 10.6.1.3-1)

$L' = L - 2e_L$ (Ref 1 - Eq. 10.6.1.3-1)

Eccentricity in L direction can typically be neglected in abutment design, therefore L' = L

e_B =	3.67 ft	(1/3 B, maximum e allowed by code, Ref 1 - 10.6.3.3)
B' =	3.67 ft	
L =	22.00 ft	(approximate length of footing)
C_{wq} =	0.75	(Ref 1 - Table 10.6.3.1.2a-2)
$C_{w\gamma}$ =	0.5	(Ref 1 - Table 10.6.3.1.2a-2)
s_c =	1.10	(Ref 1 - Table 10.6.3.1.2a-3)
s_γ =	1.00	(Ref 1 - Table 10.6.3.1.2a-3)
s_q =	1.00	(Ref 1 - Table 10.6.3.1.2a-3)
d_q =	1.00	(Ref 1 - 10.6.3.1.2a)
i_c =	1.00	(Ref 1 - C10.6.3.1.2a, for footings with modest embedment, typically ignored)
i_q =	1.00	(Ref 1 - C10.6.3.1.2a, for footings with modest embedment, typically ignored)
i_γ =	1.00	(Ref 1 - C10.6.3.1.2a, for footings with modest embedment, typically ignored)

N_{cm} =	5.65 =	5.14 x	1.10 x	1.00	
N_{qm} =	1.00 =	1.00 x	1.00 x	1.00 x	1.00
$N_{\gamma m}$ =	0.00 =	0.00 x	1.00 x	1.00	

cN_{cm} =	18.38 =	3.25 ksf x	5.65			
$\gamma D_f N_{qm} C_{wq}$ =	0.19 =	0.125 kcf x	2 ft x	1.0 x	0.75	
$0.5\gamma B N_{\gamma m} C_{w\gamma}$ =	0.00 =	0.50 x	0.125 kcf x	3.67 ft x	0.00 x	0.5

q_n = 18.38 + 0.19 + 0.00 = = 18.56 ksf

q_u = 0.50 x 18.56 ksf = 9.3 ksf

Settlement Analyses:

Since the clay that is present at the elevation of the proposed bottom of footing and below is hard (blow counts >30) the clay is classified as overconsolidated and therefore settlement is not a concern. This is further confirmed by the fact that this is for a wingwall footing that is lightly loaded.

References:

1. MassDOT LRFD Bridge Manual 2024
2. AASHTO LRFD Bridge Design Specifications, 9th Edition 2020, with current Interims

Calculation Narrative:

The following calculations determine the overall geometry for the MSE wingwalls along the south side of the roadway and the adequacy for external stability. Final design, including internal stability of the MSE wall system will be submitted by the MSE wall designer chosen by the general contractor.

General Input:

Min. Embedment Depth =	2.00 ft
Curb Reveal =	0.50 ft
Safety Curb Chamfer =	0.17 ft

Profile Information:

Grade Break Station 1 =	1015.00 ft
Grade Break Elevation 1 =	1124.25 ft
Grade 1 =	0.64%
Grade Break Station 2 =	1050.00 ft
Grade Break Elevation 2 =	1124.48 ft
Grade 2 =	-0.51%

Wall Heights:

Southwest Wall

Wall Length =	20.00 ft
Start Station of SW Wall =	1020.55 ft
End Station of SW Wall =	1040.56 ft
Roadway Cross Slope =	2.00%
Offset =	12.00 ft
Top of Start of SW Wall =	1124.38 ft
Top of End of SW Wall =	1124.51 ft
Use =	1124.50 ft
Grade at Start =	1124.50 ft
Grade at End =	1111.00 ft
Bottom of Wall at Start =	1120.50 ft
Bottom of Wall at End =	1109.00 ft
Height of Wall at Start =	3.88 ft
Height of Wall at End =	15.51 ft

Southeast Wall

Wall Length =	20.00 ft
Start Station of SE Wall =	1059.01 ft
End Station of SE Wall =	1079.00 ft
Roadway Cross Slope =	2.00%
Offset =	12.00 ft
Top of Start of SE Wall =	1124.53 ft
Top of End of SE Wall =	1124.43 ft
Use =	1124.50 ft
Grade at Start =	1111.00 ft
Grade at End =	1124.50 ft
Bottom of Wall at Start =	1109.00 ft
Bottom of Wall at End =	1120.50 ft
Height of Wall at Start =	15.53 ft
Height of Wall at End =	3.93 ft

MSE Wall External Stability - Max Height C-10-024

References:
1. MassDOT LRFD Bridge Manual 2024
2. AASHTO LRFD Bridge Design Specifications, 9th Edition 2020, with current Interims

Calculation Narrative:

The following calculations determine the overall geometry for the MSE wingwalls along the south side of the roadway and the adequacy for external stability. Final design, including internal stability of the MSE wall system will be submitted by the MSE wall designer chosen by the general contractor.

General Input:

Wall Length, L =	20.00 ft				
Wall Height, H =	15.53 ft				
Base Width, B =	11.00 ft	>		0.70H	OK
Soil Unit Weight =	125 pcf				
Retained Soil Friction Angle =	35 deg =	0.61 rad	(minimum value)		
Reinforced Soil Friction Angle =	35 deg =	0.61 rad	(gravel borrow)		
Foundation Soil Friction Angle =	35 deg =	0.61 rad	(minimum value)		
Wall Friction Angle =	23 deg =	0.41 rad			

Applied Loads:

Soil Mass:

Base Width =	11.00 ft			
Wall Height =	15.53 ft			
Soil Mass Area =	171 ft² =	11.00 ft x	15.53 ft	
Soil Mass Weight =	21.35 klf =	171 ft² x	125 pcf/	1000

MSE Wall External Stability C-10-024

Lateral Earth Pressure:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.27$$

R =	4.08 klf =	0.50 x	0.27 x	0.125 kcf x	(15.53 ft)²
Y =	5.18 ft =	15.53 ft/	3		
H =	3.75 klf =	4.08 klf x	0.92		
V =	1.62 klf =	4.08 klf x	0.40		
X =	-5.50 ft				

Live Load Surcharge:

The increase in horizontal pressure due to live load surcharge may be estimated as:

$$\Delta_p = k\gamma_s h_{eq} \qquad (3.11.6.4\text{-}1)$$

where:

Δ_p = constant horizontal earth pressure due to live load surcharge (ksf)
γ_s = total unit weight of soil (kcf)
k = coefficient of lateral earth pressure
h_{eq} = equivalent height of soil for vehicular load (ft)

Equivalent heights of soil, h_{eq}, for highway loadings on abutments and retaining walls may be taken from Tables 3.11.6.4-1 and 3.11.6.4-2. Linear interpolation shall be used for intermediate wall heights.

The wall height shall be taken as the distance between the surface of the backfill and the bottom of the footing along the pressure surface being considered.

Table 3.11.6.4-2—Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic

Retaining Wall Height (ft)	h_{eq} (ft) Distance from wall backface to edge of traffic: 0.0 ft	h_{eq} (ft) Distance from wall backface to edge of traffic: 1.0 ft or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

The load factor for both vertical and horizontal components of live load surcharge shall be taken as specified in Table 3.4.1-1 for live load surcharge.

heq = 2.00 ft

∆ =	1.05 klf =	0.27 x	0.125 kcf x	2.00 ft x	15.53 ft
Y =	7.76 ft =	15.53 ft/	2		
H =	0.97 klf =	1.05 klf x	0.92		
V =	0.42 klf =	1.05 klf x	0.40		
X =	-5.50 ft				

MSE Wall External Stability C-10-024

For bearing evaluation, also include vertical component of live load surcharge.

P = 2.75 klf = 0.125 kcf x 2.00 ft x 11.00 ft

Vehicular Impact:

P =	23.0 k	(Ref 1 - Table 3.3.2-1, TL 2)
H =	2.00 ft	(Ref 1 - Table 3.3.2-1, TL 2)
L =	20.00 ft	(Ref 1 - 3.3.2.4, entire wall length can be used to resist impact)

P = 1.15 klf = 23.0 k/ 20.00 ft
Y = 17.53 ft = 15.53 ft + 2.00 ft

Load Factors:

	Strength I		Extreme II
	Bearing	Sliding/Ecc.	
EV	1.35	1.00	1.00
EH	1.50	1.50	1.00
LS	1.75	1.75	0.50
CT	0.00	0.00	1.00

Strength 1, Bearing Evaluation:

Loads:

	Unfactored	Load Factor	Factored	V or H
EV	21.35 klf	1.35	28.82 klf	V
EHh	3.75 klf	1.50	5.62 klf	H
EHv	1.62 klf	1.50	2.43 klf	V
LSh	0.97 klf	1.75	1.69 klf	H
LSv	0.42 klf	1.75	0.73 klf	V
LS-Vert	2.75 klf	1.75	4.81 klf	V
CT	1.15 klf	0.00	0.00 klf	H

MSE Wall External Stability C-10-024

	H	Y	V	X	M
EV	0.00 klf	-	28.82 klf	0.00 ft	0.00 k-ft
EHh	5.62 klf	5.18 ft	0.00 klf	-	29.11 k-ft
EHv	-	-	2.43 klf	-5.50 ft	-13.34 k-ft
LSh	1.69 klf	7.76 ft	0.00 klf	-	13.12 k-ft
LSv	-	-	0.73 klf	-5.50 ft	-4.01 k-ft
LS-Vert	0.00 klf	-	4.81 klf	-	0.00 k-ft
CT	0.00 klf	17.53 ft	0.00 klf	-	0.00 k-ft
	7.31 klf		36.79 klf		24.88 k-ft

Eccentricity, e = 0.68 ft = 24.88 k-ft/ 36.79 klf

Effective Width, B' = 9.65 ft = 11.00 ft - 2 x 0.68 ft

Applied Pressure = 3.81 ksf = 36.79 klf/ 9.65 ft

Ultimate Bearing Capacity = 6.67 ksf (see bearing capacity calculations, minimum from B-1/B-2 used)

Applied	<	Allowable
3.81 ksf	<	6.67 ksf
	OK	

MSE Wall External Stability C-10-024

Strength 1, Sliding and Eccentricity Evaluation:

Loads:

	Unfactored	Load Factor	Factored	V or H
EV	21.35 klf	1.00	21.35 klf	V
EHh	3.75 klf	1.50	5.62 klf	H
EHv	1.62 klf	1.50	2.43 klf	V
LSh	0.97 klf	1.75	1.69 klf	H
LSv	0.42 klf	1.75	0.73 klf	V
LS-Vert	2.75 klf	0.00	0.00 klf	V
CT	1.15 klf	0.00	0.00 klf	H

	H	Y	V	X	M
EV	0.00 klf	-	21.35 klf	0.00 ft	0.00 k-ft
EHh	5.62 klf	5.18 ft	0.00 klf	-	29.11 k-ft
EHv	-	-	2.43 klf	-5.50 ft	-13.34 k-ft
LSh	1.69 klf	7.76 ft	0.00 klf	-	13.12 k-ft
LSv	-	-	0.73 klf	-5.50 ft	-4.01 k-ft
LS-Vert	0.00 klf	-	0.00 klf	-	0.00 k-ft
CT	0.00 klf	17.53 ft	0.00 klf	-	0.00 k-ft
	7.31 klf		24.51 klf		24.88 k-ft

Eccentricity Check:

Eccentricity, e = 1.02 ft = 24.88 k-ft/ 24.51 klf

Per Ref 1 - 11.6.3.3, for foundations on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds of the base width.

Allowable e = 3.67 ft > 1.02 ft OK

MSE Wall External Stability C-10-024

Sliding Check:

10.6.3.4—Failure by Sliding

Failure by sliding shall be investigated for footings that support horizontal or inclined load and/or are founded on slopes.

For foundations on clay soils, the possible presence of a shrinkage gap between the soil and the foundation shall be considered. If passive resistance is included as part of the shear resistance required for resisting sliding, consideration shall also be given to possible future removal of the soil in front of the foundation.

The factored resistance against failure by sliding, in kips, shall be taken as:

$$R_R = \varphi R_n = \varphi_\tau R_\tau + \varphi_{ep} R_{ep} \quad (10.6.3.4\text{-}1)$$

where:

- φ = resistance factor (dim)
- R_n = nominal sliding resistance against failure by sliding (kips)
- φ_τ = resistance factor for shear resistance between soil and foundation specified in Table 10.5.5.2.2-1
- R_τ = nominal sliding resistance between soil and foundation (kips)
- φ_{ep} = resistance factor for passive resistance specified in Table 10.5.5.2.2-1
- R_{ep} = nominal passive resistance of soil available throughout the design life of the structure (kips)

If the soil beneath the footing is cohesionless, the nominal sliding resistance between soil and foundation shall be taken as:

$$R_\tau = CV \tan \phi_f \quad (10.6.3.4\text{-}2)$$

for which:

- C = 1.0 for concrete cast against soil
- = 0.8 for precast concrete footing

where:

- ϕ_f = internal friction angle of drained soil (degrees)
- V = total vertical force (kips)

Min. ∅ = 35.00 deg

tan (∅) = 0.70

C = 1

V = 24.51 klf

R = 17.16 klf = 1 x 24.51 klf x 0.70

∅ = 0.9 (Ref 1 - Table 10.5.5.2.2-1, soil on soil)

∅Rn = 15.44 klf = 0.90 x 17.16 klf

Applied H = 7.31 klf < 15.44 klf OK

MSE Wall External Stability C-10-024

Extreme 2 Evaluation:

Loads:

	Unfactored	Load Factor	Factored	V or H
EV	21.35 klf	1.00	21.35 klf	V
EHh	3.75 klf	1.00	3.75 klf	H
EHv	1.62 klf	1.00	1.62 klf	V
LSh	0.97 klf	0.50	0.48 klf	H
LSv	0.42 klf	0.50	0.21 klf	V
LS-Vert	2.75 klf	0.00	0.00 klf	V
CT	1.15 klf	1.00	1.15 klf	H

	H	Y	V	X	M
EV	0.00 klf	-	21.35 klf	0.00 ft	0.00 k-ft
EHh	3.75 klf	5.18 ft	0.00 klf	-	19.41 k-ft
EHv	-	-	1.62 klf	-5.50 ft	-8.90 k-ft
LSh	0.48 klf	7.76 ft	0.00 klf	-	3.75 k-ft
LSv	-	-	0.21 klf	-5.50 ft	-1.15 k-ft
LS-Vert	0.00 klf	-	0.00 klf	-	0.00 k-ft
CT	1.15 klf	17.53 ft	0.00 klf	-	20.16 k-ft
	5.38 klf		23.18 klf		33.27 k-ft

Eccentricity Check:

Eccentricity, e = 1.44 ft = 33.27 k-ft/ 23.18 klf

Per Ref 1 - 11.6.5.1, for foundations on soil and rock, the location of the resultant of the reaction forces shall be within the middle eight-tenths of the base width for Extreme Event.

Allowable e = 4.40 ft > 1.44 ft OK

MSE Wall External Stability C-10-024

Sliding Check:

Min. ∅ =	35.00 deg			
tan (∅) =	0.70			
C =	1			
V =	23.18 klf			
R =	16.23 klf =	1 x	23.18 klf x	0.70
∅ =	1	(Ref 1 - Extreme Event)		
∅Rn =	16.23 klf =	1.00 x	16.23 klf	
Applied H =	5.38 klf	<	16.23 klf	OK

MSE Wall External Stability - Min Height — C-10-024

References:
1. MassDOT LRFD Bridge Manual 2024
2. AASHTO LRFD Bridge Design Specifications, 9th Edition 2020, with current Interims

Calculation Narrative:

The following calculations determine the overall geometry for the MSE wingwalls along the south side of the roadway and the adequacy for external stability. Final design, including internal stability of the MSE wall system will be submitted by the MSE wall designer chosen by the general contractor.

General Input:

Wall Length, L =	20.00 ft				
Wall Height, H =	4.00 ft				
Base Width, B =	6.00 ft	>	0.70H		OK
Soil Unit Weight =	125 pcf				
Retained Soil Friction Angle =	35 deg =	0.61 rad	(minimum value)		
Reinforced Soil Friction Angle =	35 deg =	0.61 rad	(gravel borrow)		
Foundation Soil Friction Angle =	35 deg =	0.61 rad	(minimum value)		
Wall Friction Angle =	23 deg =	0.41 rad	(2/3 * ∅)		

Applied Loads:

Soil Mass:

Total Width =	6.00 ft				
Wall Height =	4.00 ft				
Soil Mass Area =	24 ft² =	6.00 ft x	4.00 ft		
Soil Mass Weight =	3.00 klf =	24 ft² x	125 pcf/	1000	

MSE Wall External Stability - Min Height C-10-024

Lateral Earth Pressure:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.27$$

R = 0.27 klf = 0.50 x 0.27 x 0.125 kcf x $(4.00\text{ ft})^2$
Y = 1.33 ft = 4.00 ft/ 3

H = 0.25 klf = 0.27 klf x 0.92
V = 0.11 klf = 0.27 klf x 0.40
X = -3.00 ft

Live Load Surcharge:

The increase in horizontal pressure due to live load surcharge may be estimated as:

$$\Delta_p = k\gamma_s h_{eq} \qquad (3.11.6.4\text{-}1)$$

where:

Δ_p = constant horizontal earth pressure due to live load surcharge (ksf)
γ_s = total unit weight of soil (kcf)
k = coefficient of lateral earth pressure
h_{eq} = equivalent height of soil for vehicular load (ft)

Equivalent heights of soil, h_{eq}, for highway loadings on abutments and retaining walls may be taken from Tables 3.11.6.4-1 and 3.11.6.4-2. Linear interpolation shall be used for intermediate wall heights.

The wall height shall be taken as the distance between the surface of the backfill and the bottom of the footing along the pressure surface being considered.

Table 3.11.6.4-2—Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic

Retaining Wall Height (ft)	h_{eq} (ft) Distance from wall backface to edge of traffic	
	0.0 ft	1.0 ft or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

The load factor for both vertical and horizontal components of live load surcharge shall be taken as specified in Table 3.4.1-1 for live load surcharge.

heq = 2.00 ft

Δ = 0.27 klf = 0.27 x 0.125 kcf x 2.00 ft x 4.00 ft
Y = 2.00 ft = 4.00 ft/ 2

H = 0.25 klf = 0.27 klf x 0.92
V = 0.11 klf = 0.27 klf x 0.40
X = -3.00 ft

MSE Wall External Stability - Min Height C-10-024

For bearing evaluation, also include vertical component of live load surcharge.

P = 1.50 klf = 0.125 kcf x 2.00 ft x 6.00 ft

Vehicular Impact:

P =	23.0 k	(Ref 1 - Table 3.3.2-1, TL 2)
H =	2.00 ft	(Ref 1 - Table 3.3.2-1, TL 2)
L =	20.00 ft	(Ref 1 - 3.3.2.4, entire wall length can be used to resist impact)

P = 1.15 klf = 23.0 k/ 20.00 ft

Y = 6.00 ft = 4.00 ft + 2.00 ft

Load Factors:

	Strength I		Extreme II
	Bearing	Sliding/Ecc.	
EV	1.35	1.00	1.00
EH	1.50	1.50	1.00
LS	1.75	1.75	0.50
CT	0.00	0.00	1.00

Strength 1, Bearing Evaluation:

Loads:

	Unfactored	Load Factor	Factored	V or H
EV	3.00 klf	1.35	4.05 klf	V
EHh	0.25 klf	1.50	0.37 klf	H
EHv	0.11 klf	1.50	0.16 klf	V
LSh	0.25 klf	1.75	0.44 klf	H
LSv	0.11 klf	1.75	0.19 klf	V
LS-Vert	1.50 klf	1.75	2.63 klf	V
CT	1.15 klf	0.00	0.00 klf	H

MSE Wall External Stability - Min Height C-10-024

	H	Y	V	X	M
EV	0.00 klf	-	4.05 klf	0.00 ft	0.00 k-ft
EHh	0.37 klf	1.33 ft	0.00 klf	-	0.50 k-ft
EHv	-	-	0.16 klf	-3.00 ft	-0.48 k-ft
LSh	0.44 klf	2.00 ft	0.00 klf	-	0.87 k-ft
LSv	-	-	0.19 klf	-3.00 ft	-0.56 k-ft
LS-Vert	0.00 klf	-	2.63 klf	-	0.00 k-ft
CT	0.00 klf	6.00 ft	0.00 klf	-	0.00 k-ft
	0.81 klf		7.02 klf		0.32 k-ft

Eccentricity, e = 0.05 ft = 0.32 k-ft/ 7.02 klf

Effective Width, B' = 5.91 ft = 6.00 ft - 2 x 0.05 ft

Applied Pressure = 1.19 ksf = 7.02 klf/ 5.91 ft

Ultimate Bearing Capacity = 6.67 ksf (see bearing capacity calculations, minimum from B-1/B-2 used)

Applied	<	Allowable
1.19 ksf	<	6.67 ksf
	OK	

MSE Wall External Stability - Min Height C-10-024

Strength I, Sliding and Eccentricity Evaluation:

Loads:

	Unfactored	Load Factor	Factored	V or H
EV	3.00 klf	1.00	3.00 klf	V
EHh	0.25 klf	1.50	0.37 klf	H
EHv	0.11 klf	1.50	0.16 klf	V
LSh	0.25 klf	1.75	0.44 klf	H
LSv	0.11 klf	1.75	0.19 klf	V
LS-Vert	1.50 klf	0.00	0.00 klf	V
CT	1.15 klf	0.00	0.00 klf	H

	H	Y	V	X	M
EV	0.00 klf	-	3.00 klf	0.00 ft	0.00 k-ft
EHh	0.37 klf	1.33 ft	0.00 klf	-	0.50 k-ft
EHv	-	-	0.16 klf	-3.00 ft	-0.48 k-ft
LSh	0.44 klf	2.00 ft	0.00 klf	-	0.87 k-ft
LSv	-	-	0.19 klf	-3.00 ft	-0.56 k-ft
LS-Vert	0.00 klf	-	0.00 klf	-	0.00 k-ft
CT	0.00 klf	6.00 ft	0.00 klf	-	0.00 k-ft
	0.81 klf		3.35 klf		0.32 k-ft

Eccentricity Check:

Eccentricity, e = 0.10 ft = 0.32 k-ft/ 3.35 klf

Per Ref 1 - 11.6.3.3, for foundations on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds of the base width.

Allowable e = 2.00 ft > 0.10 ft OK

Sliding Check:

10.6.3.4—Failure by Sliding

Failure by sliding shall be investigated for footings that support horizontal or inclined load and/or are founded on slopes.

For foundations on clay soils, the possible presence of a shrinkage gap between the soil and the foundation shall be considered. If passive resistance is included as part of the shear resistance required for resisting sliding, consideration shall also be given to possible future removal of the soil in front of the foundation.

The factored resistance against failure by sliding, in kips, shall be taken as:

$$R_R = \varphi R_n = \varphi_\tau R_\tau + \varphi_{ep} R_{ep} \qquad (10.6.3.4\text{-}1)$$

where:

- φ = resistance factor (dim)
- R_n = nominal sliding resistance against failure by sliding (kips)
- φ_τ = resistance factor for shear resistance between soil and foundation specified in Table 10.5.5.2.2-1
- R_τ = nominal sliding resistance between soil and foundation (kips)
- φ_{ep} = resistance factor for passive resistance specified in Table 10.5.5.2.2-1
- R_{ep} = nominal passive resistance of soil available throughout the design life of the structure (kips)

If the soil beneath the footing is cohesionless, the nominal sliding resistance between soil and foundation shall be taken as:

$$R_\tau = CV \tan \phi_f \qquad (10.6.3.4\text{-}2)$$

for which:

- C = 1.0 for concrete cast against soil
- = 0.8 for precast concrete footing

where:

- ϕ_f = internal friction angle of drained soil (degrees)
- V = total vertical force (kips)

Min. ∅ = 35.00 deg

tan (∅) = 0.70

C = 1

V = 3.35 klf

R = 2.34 klf = 1 x 3.35 klf x 0.70

∅ = 0.9 (Ref 1 - Table 10.5.5.2.2-1, soil on soil)

∅Rn = 2.11 klf = 0.90 x 2.34 klf

Applied H = 0.81 klf < 2.11 klf OK

MSE Wall External Stability - Min Height C-10-024

Extreme 2 Evaluation:

Loads:

	Unfactored	Load Factor	Factored	V or H
EV	3.00 klf	1.00	3.00 klf	V
EHh	0.25 klf	1.00	0.25 klf	H
EHv	0.11 klf	1.00	0.11 klf	V
LSh	0.25 klf	0.50	0.12 klf	H
LSv	0.11 klf	0.50	0.05 klf	V
LS-Vert	1.50 klf	0.00	0.00 klf	V
CT	1.15 klf	1.00	1.15 klf	H

	H	Y	V	X	M
EV	0.00 klf	-	3.00 klf	0.00 ft	0.00 k-ft
EHh	0.25 klf	1.33 ft	0.00 klf	-	0.33 k-ft
EHv	-	-	0.11 klf	-3.00 ft	-0.32 k-ft
LSh	0.12 klf	2.00 ft	0.00 klf	-	0.25 k-ft
LSv	-	-	0.05 klf	-3.00 ft	-0.16 k-ft
LS-Vert	0.00 klf	-	0.00 klf	-	0.00 k-ft
CT	1.15 klf	6.00 ft	0.00 klf	-	6.90 k-ft
	1.52 klf		3.16 klf		7.00 k-ft

Eccentricity Check:

Eccentricity, e = 2.21 ft = 7.00 k-ft/ 3.16 klf

Per Ref 1 - 11.6.5.1, for foundations on soil and rock, the location of the resultant of the reaction forces shall be within the middle eight-tenths of the base width for Extreme Event.

Allowable e = 2.40 ft > 2.21 ft OK

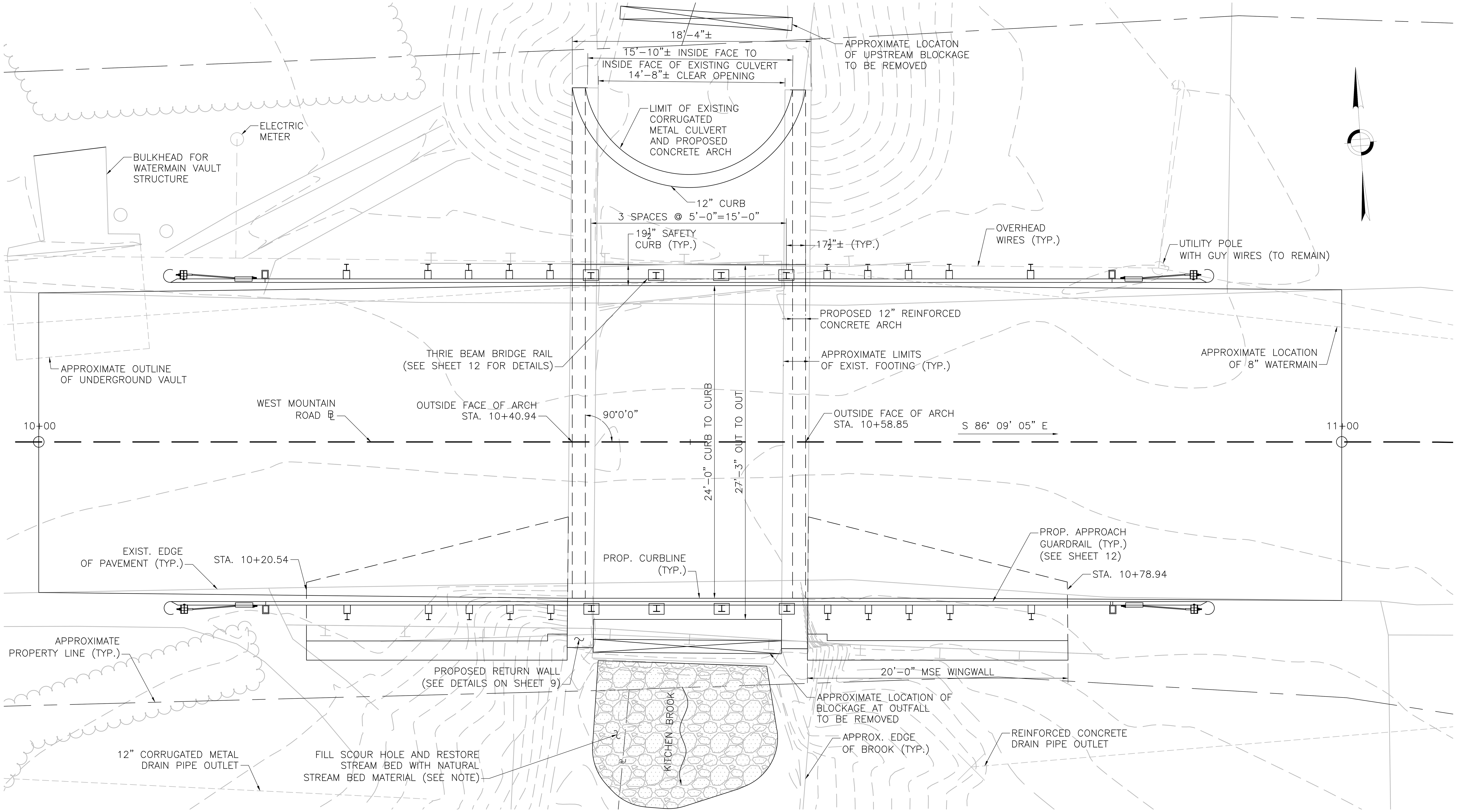
MSE Wall External Stability - Min Height C-10-024

Sliding Check:

Min. ∅ =	35.00 deg			
tan (∅) =	0.70			
C =	1			
V =	3.16 klf			
R =	2.21 klf =	1 x	3.16 klf x	0.70
∅ =	1	(Ref 1 - Extreme Event)		
∅Rn =	2.21 klf =	1.00 x	2.21 klf	
Applied H =	1.52 klf	<	2.21 klf	OK

APPENDIX

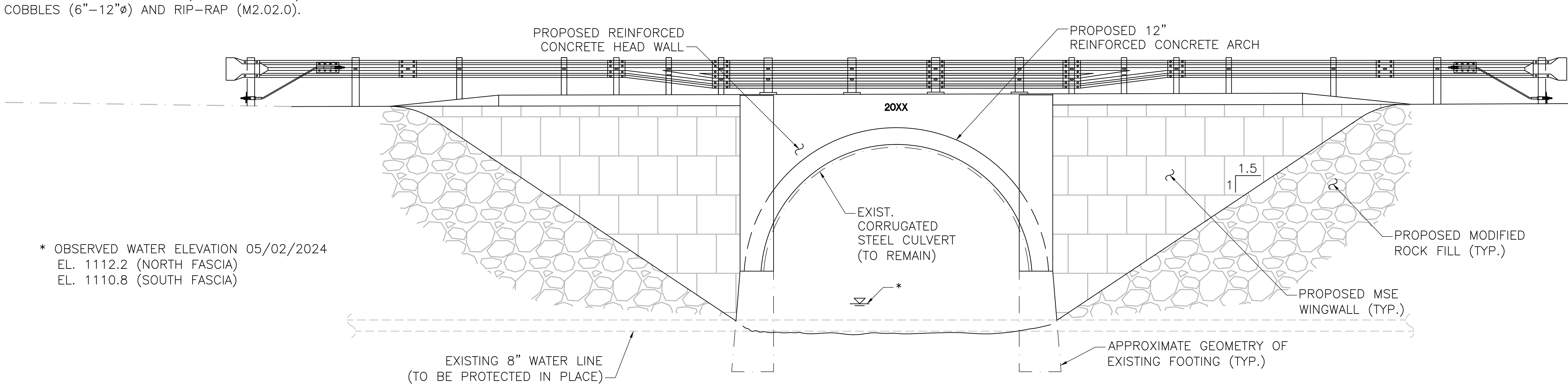
6.7. Preliminary Plans



GENERAL PLAN

SCALE: ¼" = 1'-0"

NOTE:
NATURAL STREAM CHANNEL SHALL CONSIST OF WELL GRADED MIX (COMPACTED AND CONSOLIDATED) CONSISTING OF GRAVEL MATERIAL (M1.03.0 TYPE A) OVER COBBLES (6"-12"ø) AND RIP-RAP (M2.02.0).



* OBSERVED WATER ELEVATION 05/02/2024
EL. 1112.2 (NORTH FASCIA)
EL. 1110.8 (SOUTH FASCIA)

SOUTH ELEVATION

SCALE: ¼" = 1'-0"

COMMONWEALTH OF MASSACHUSETTS
MassDOT, Highway Division
APPROVED UNDER PROVISIONS OF MASS. GEN. LAWS CH 85 S 35

STATE BRIDGE ENGINEER | DATE

GILL ENGINEERING
63 KENDRICK STREET
NEEDHAM, MA 02494
781-355-7100
781-355-7101 (FAX)

Plotted on 20-Nov-2024 3:56 PM
C10024_BR4-6.dwg

DATE	DRW. BY	CALC. BY	APPRV. BY	DESCRIPTION
11/20/24	MMS	MMS	JEP	CHAPTER 85 REVIEW

REGISTERED PROFESSIONAL ENGINEER | DATE

CULVERT STRENGTHENING
TOWN OF CHESHIRE
CULVERT STRENGTHENING FOR CHESHIRE
C-10-024 (AB2)
WEST MOUNTAIN ROAD OVER KITCHEN BROOK

GENERAL PLAN AND ELEVATION