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PRELIMINARY GEOTECHNICAL EVALUATION REPORT

DRP/ DAVISON/ STAGE 7 DUE DILIGENCE
BURTON, MICHIGAN

SME Project Number: 100047.00
August 27, 2025

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August 27, 2025

Ms. Shannon R. Selby
Detroit Regional Partnership
1001 Woodward Avenue, Suite 800
Detroit, Michigan 48226

Via E-mail: Shannon.Selby@DetroitRegionalPartnership.com

RE: Preliminary Geotechnical Engineering Report
DRP/ Davison/ Stage 7 Due Diligence
Burton, Michigan
SME Project No. 100047.00

Dear Ms. Selby:

We have completed the preliminary geotechnical evaluation for the parcel located south of Davison Road and east of North Center Road, in Burton, Genesee County, Michigan. This report presents the results of our observations and analyses, and our preliminary comments regarding subgrade preparation and earthwork, slabs-on-grade and pavements, below-grade walls, and foundations. SME's environmental team is submitting their environmental findings under separate cover.

We appreciate the opportunity to be of service. If you have questions or require additional information, please contact us.

Sincerely,

SME

Laurel M. Johnson, PE
Principal Consultant

Enclosure: SME Geotechnical Evaluation Report; Dated August 27, 2025

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1. INTRODUCTION

This report presents the results of the preliminary geotechnical evaluation performed by SME for the RACER Trust Davison Road Industrial Land in Burton, Michigan. This evaluation was conducted in general accordance with the scope of services outlined in SME Proposal No. P01585.25, dated May 27, 2025. Our services for this evaluation were authorized by Shannon Selby of the Detroit Regional Partnership (DRP) on June 5, 2025.

NOTE: SME is also providing surveying and environmental services, presented under separate cover.

SME completed six borings (B1 through B6) at the project site on August 4 and 5, 2025. Refer to the boring logs included in Appendix A for the specific depth of each boring. We returned to the project site on August 20, 2025 to perform six additional test pit excavations (TP5 through TP10) in the general vicinity of borings B5 and B6. The approximate as-drilled boring locations and approximate test pit locations are depicted on the Boring and Test Pit Location Plan located in Appendix A (Figure No. 1). Soil descriptions and the field and laboratory test results are represented on the boring logs. Exploration and laboratory testing procedures are presented in Section 4.

This report was prepared for due diligence services only and should be considered preliminary until a proposed site layout, grading plan, and estimated building and traffic loads are provided to SME for review. Do not base final site plans solely on the content of this preliminary document.

1.1 SITE CONDITIONS

The project site includes 56.21 acres of vacant, wooded land located south of Davison Road, east of North Center Road, in the City of Burton, Genesee County, Michigan. The property is more specifically identified as parcel identification numbers (PIDs) 59-10-100-031 and 59-10-100-030 with no recorded property address. Northgate – Davison is located to the west of the project site and the new General Motors Davison Road processing center is located to the east of the site. Commercial developments separate the property from Davison Road to the north. The southern boundary consists of Gilkey Creek and a railroad easement. Presently, the site is undeveloped with a paved road accessed at the northwest corner of the property, which spans the site from east to west. An existing metal building is also located in the northwest corner of the property. The remainder of the site is either heavily wooded or surfaced with tall grass. Based on Monument Engineering Group Associates' *Topographic Survey* drawing dated September 12, 2012, site grades range from approximate elevation 750 to 768 feet NAVD'88. In general, the property is sloped towards Gilkey Creek. Refer to Image No. 1 for a recent aerial image of the project site.

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IMAGE NO. 1: Google earth aerial image (Image Date April 29, 2023)

Most recently, a surficial gravel layer was placed at the project site to provide stable and relatively level areas for temporary production vehicle storage. These areas are visible in Image No. 2, which shows construction equipment spreading the fill material. Due to the lack of tree growth on most of the site, we assume that the vegetation on the property is maintained on a seasonal basis. The aggregate surfaced areas were overgrown during our field investigation as indicated in Image No. 1. Reviewing additional aerial imagery indicates that prior to 1957 the property was used for agricultural purposes. In 1957, the first indication of a roadway system for a development is present. The project site appears to never have been fully developed, although sanitary sewer easements for the county and city are present within the boundaries of the project site. Following the construction of the General Motors facility (currently Northgate) to the west of the project site, the development plans appear to have been abandoned, and used by General Motors for various industrial purposes, including wasting of construction debris and industrial materials. More detailed information regarding site history and usage can be obtained from the environmental documentation. Refer to Appendix A for additional aerial photography and topographic drawings.

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IMAGE NO. 2: Google earth aerial image (Image Date May 17, 2023)

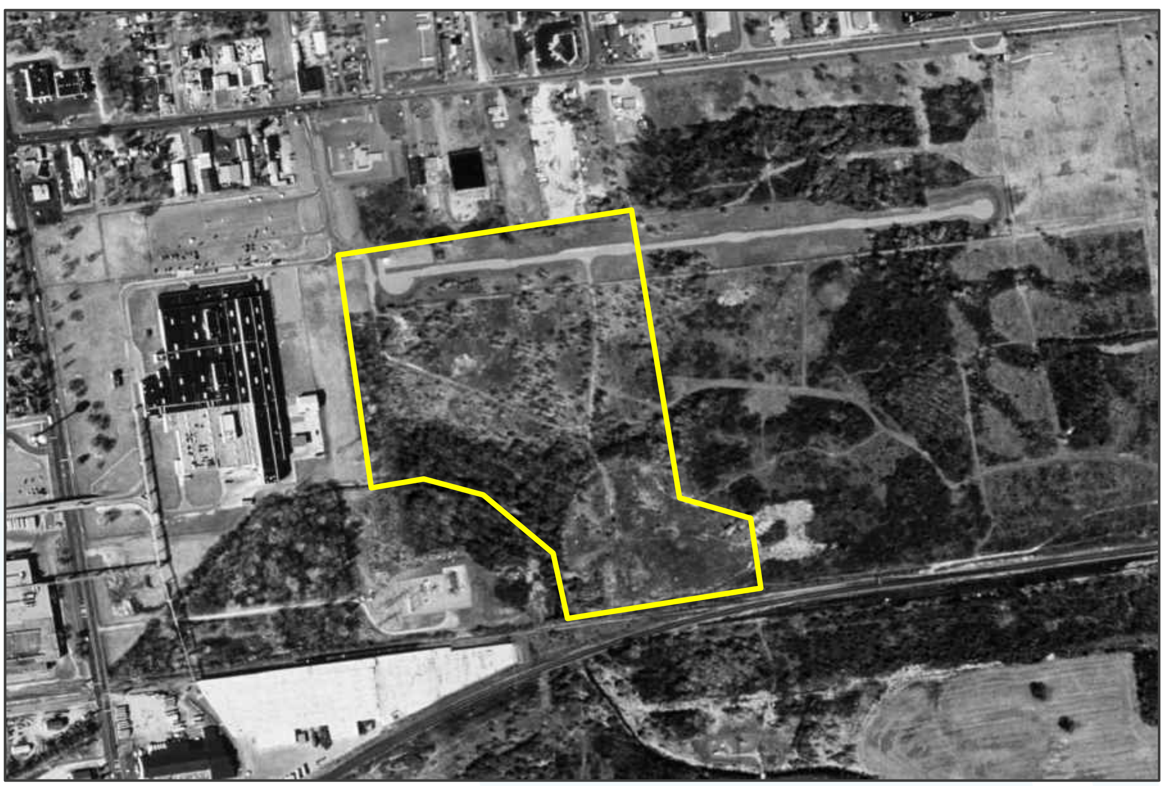

IMAGE NO. 3: Google earth aerial image (Image Date April 24, 2000)

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1.2 PROJECT DESCRIPTION

There are no specific plans to redevelop the project site. However, given the redevelopment funds and targeted programs, the site will be added to the Detroit Regional Partnership's available Brownfield Inventory. The inventory is generally targeted for industrial type operations. We understand the site is currently zoned for light industrial.

1.3 GEOLOGY AND SUBSURFACE CONDITIONS

1.3.1 GENERAL GEOLOGY

In general, Burton is located within the "Saginaw Lowlands", which is characterized by relatively thick glacial drift consisting of mixed sands, silt, clays, and gravel over an intermediate depth overconsolidated layer of extremely dense granular soils and hard clayey soils. Drift thickness (including the heavily overconsolidated materials) typically exceed 200 feet. Post-glacial periods resulted in a complex combination of lacustrine and alluvial deposits. This leads to the deposition of interbedded sands, silts, and clays overlying relatively dense glacial tills. The glacial till in this area is generally underlain by weathered limestone, shale, and/or sandstone bedrock, which often contain coal deposits. A nearby abandoned coal mine- the "What Cheer Mine" underlies Interstate I-69, near Lapeer Road and Court Street. The mine operated until about 1920, when it was flooded by Gilkey Creek.

1.3.2 FLOOD RISK

Refer to the *National Flood Hazard Layer FIRMette* in Appendix A.

At least portions of the site, generally those directly adjacent to Gilkey Creek, are located in a regulatory floodway and "special flood hazard areas", designated by the Federal Emergency Management Agency (FEMA) as "Zone AE". Based on the referenced FIRMette, flood elevations along the perimeter of the site range from about elevation 756.8 to 757.0 feet.

1.3.3 SOIL AND GROUNDWATER CONDITIONS

Refer to the boring logs in Appendix A for the soil conditions at specific boring locations. In general, we encountered fill soils at the ground surface with minimal topsoil that was discernable from the existing fill. Sand fill extended from about 1.0 to 9 feet below the ground surface at the boring locations and 2.0 to 12.5 feet below the ground surface at the test pit locations. We obtained Standard Penetration Test (SPT) resistances, adjusted for drill rig hammer efficiency, (N_{60}-values) of 3 to 39 blows per foot (bpf) in the sand fill, indicating a very loose to medium dense condition. At least some of the sand fill appeared to be foundry sand and was observed to have magnetic properties during visual classification of the soils in the laboratory, indicative of the presence of metal or potentially steel furnace slag.

Below the fill soils, we encountered natural sands, silts, and clays extending to the explored depths of the borings and test pits. However, test pit TP8 was terminated at a depth of 12.5 feet due to encountering groundwater without encountering natural soils. N_{60}-values of 3 to 65 bpf were encountered in the native sands and silts, indicating very loose to very dense condition. We encountered native clays with shear strengths ranging from 0.5 to greater than 4.5 kips per square-foot (ksf) with moisture contents ranging from 10 to 25 percent, indicating a soft to hard condition. It should be noted that the natural soil conditions at the project site are highly variable. Due to the variable soil conditions, boring B2 was extended deeper than initially planned until elevated blow counts consistent with glacial till were encountered. The terminal blow count at about 39 feet exceeded 50 blows per 6-inch increment.

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We encountered groundwater during drilling between the depths of 9 and 19 feet below the ground surface at the boring locations. Groundwater was not encountered after the completion of drilling at the boring locations. Groundwater was encountered during the excavation of test pit TP8 at a depth of 12.5 feet. In cohesive soils (clays and silts), a long time may be required for the groundwater level in the borehole to reach an equilibrium position. Therefore, the use of groundwater observation wells (piezometers) is necessary to accurately determine the hydrostatic groundwater level within cohesive soils as observed at this site. Due to the predominantly granular near-surface soils, groundwater levels encountered during drilling are judged representative of static groundwater at the time of drilling. Expect perched groundwater in silty/clayey seams within the sands or in sand layers above lower-permeability silts and clays. We did not obtain long-term groundwater measurements.

Refer to the Field-Testing Procedures in Appendix B for additional information about groundwater level measurements. If more information regarding groundwater levels at this site is required, then we recommend installing piezometers or wells.

1.3.4 BURIED CONCRETE AND DEBRIS

Test pits were performed in the southern portion of the site to better characterize the fills soils after a significant amount of industrial materials and construction debris were encountered at borings B5 and B6. Buried concrete was encountered at test pit TP6. We were unable to determine whether or not the concrete was part of a buried structure. However, the concrete contained reinforcing steel. Other building debris such as masonry blocks and bricks were encountered in the test pits.

Both test pits TP6 and TP8 had fill material primarily composed of metal, plastic or dried epoxy, spark plugs, and other industrial debris. All of the test pit locations had at least a thin layer of spark plugs present in the excavations. Refer to the test pit photolog in Appendix A for pictures of the test pit excavations and the materials encountered. Based on the depth of fill encountered at the test pit locations, it appears that deeper deposits of fill materials are present adjacent to Gilkey Creek. We assume the fill was placed in previous low lying areas.

2. ANALYSIS AND PRELIMINARY RECOMMENDATIONS

This geotechnical report does not consider costs/feasibility of import/export, vapor mitigation systems, or other environmental considerations/remediations not specifically addressed in the following sections. In some cases, we expect environmental/regulatory requirements could influence planning and/or design.

Further, this evaluation was prepared to provide preliminary data for future development. Do not base final project design or cost estimates solely on the content of this report. Additional soil borings and/or test pits will be required once a development plan is in place.

2.1 DEVELOPMENT CHALLENGES & OPPORTUNITIES

Based on the information discussed in the previous sections, there will be site-specific challenges to developing the project site for industrial use. We summarize our comments regarding challenges and opportunities related to developing the property in the following comments:

- Unlike many Brownfield redevelopment projects, there is a lack of existing structures present on the project site. This will allow for ground improvement methods to be used relatively efficiently and will reduce the amount of demolition and removal of abandoned below-grade structures.
- Based on our limited subsurface investigation, the fill soils on most of the project site appear to be limited in thickness and generally contain minor amounts of debris, except in isolated areas as described elsewhere in this report. Again, this supports the efficient implementation of ground improvement methods to support proposed structures.

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- During the test pit evaluation, we encountered significant deposits of debris on the southern end of the project site. This debris was composed of concrete, metal, and other forms of industrial/automotive materials. Ground improvement, while feasible, would be more challenging in this area due to possible obstructions. A proposed development plan may want to avoid building structures in these areas. If construction is planned in this area, a more extensive evaluation of the fill materials in the area should be performed to delineate the deeper debris deposits. Removal and replacement may be necessary or pre-excavation for ground improvement could also be considered.
- Impacted fill soils and groundwater are present on the property. Exporting excess fill soils or industrial debris will be more costly than greenfield sites, as excess materials that cannot remain on site will need to be exported to (a) landfill(s). It will be necessary to follow an environmental management plan regarding the handling of impacted soil and groundwater.
- Due to the presence of impacted fill, environmental considerations may govern certain aspects of design and construction, such as stormwater management and slab support and/or vapor mitigation systems.
- The project site is eligible for funding to assist with certain remediation activities and to support added costs associated with redevelopment of the project site. The successful developer will need to work with a firm experienced with obtaining funding and with design and construction-phase testing and documentation of eligible activities.
- The near-surface natural soils are highly variable at the project site. A comprehensive geotechnical evaluation will need to be performed once development plans are available. Very loose to loose natural sands and silts are present throughout the project site, as are isolated areas of softer native clays. For typical industrial loads of 350 kips or less and floor loads of 1,500 psf or less, we expect a practical ground improvement method would consist of Rammed Aggregate Piers (RAPs), extending through the fill and weaker near-surface soils to bear on dense/hard glacial till estimated at depths of 35 to 45 feet below the ground surface. We note only a single deeper boring was advanced for purposed of identifying this denser layer, and the elevation at which glacial till is encountered is likely to vary across the site and will need to be profiled during the final geotechnical phase.
- Due to the very loose to loose granular soils, increasing site grades significantly may cause greater than typical settlement. However, settlement in granular soils occurs relatively quickly and a surcharge/pre-loading plan could be implemented in areas where site grades would be raised significantly. Plans to increase site grades should be evaluated as part of future geotechnical explorations.
- Groundwater levels do not appear to be particularly high, which will reduce the need to manage groundwater levels during construction; however, Gilkey Creek appears to have a relatively large floodplain that extends onto the property. At-grade and below-grade structures should be installed with finish grades above Zone AE flood elevations. If LOMR/CLOMRs are planned, flood resistant construction methods, such as using low-permeability clay fill to raise site grades, must be implemented. Refer to the FEMA FIRMette in Appendix A for additional information.
- We did not identify organic deposits, such as peat during our geotechnical exploration. However, organic deposits may be present within or adjacent to the floodplain of Gilkey Creek and/or within the heavily wooded southwest portion of the site, which was not evaluated for this project.
- Development near Gilkey Creek may be limited due to the possible presence of wetlands. Existing wetlands should be delineated prior to developing a proposed site layout.

2.2 SUPPLEMENTAL EVALUATIONS

Based on the results of the soil borings and the environmental services provided to date, supplemental evaluations will be required to provide engineering recommendations for final design. Based on SME's experience with Brownfield redevelopment, the developer may consider the following:

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1. A main phase geotechnical exploration will be necessary once development plans are available. Additional borings will need to be advanced in proposed building footprints, equipment footprints, and paved areas. Utilize this report (along with environmental reports) as a basis for preliminary development considerations.
2. An evaluation of the near-surface soils for the support of proposed pavements.
3. An extensive test pit program should be implemented to evaluate and delineate the fill along Gilkey Creek.
4. Michigan has developed low impact development (LID) recommendations that include specific requirements for on-site stormwater management. This site is environmentally impacted, retention/detention basins should be lined with either clay or a membrane-type product. We recommend assuming a minimum permeability rate of 1×10^{-6} to 1×10^{-7} cm/hour if native clays are used as liner materials. Generally, environmentally impacted sites are not suitable for groundwater infiltration. If infiltration will be relied upon for stormwater management, additional environmental analysis may be needed to verify the pond footprint(s) are free of impacted soil and groundwater. In-situ infiltration testing will be required for stormwater improvements relying on groundwater infiltration.
5. No evaluation has been considered within the heavily-wooded portion of the site. Once the site has been cleared, additional borings and test pits in that area will be needed.
6. At least some of the existing fill contain materials typically indicative of soils with an elevated risk for corrosion, such as cinders, metal products, and organics. Corrosion analysis should be considered if sensitive and/or metal utilities are planned.
7. The upper soils are highly variable; a final geotechnical evaluation should include in-situ and/or laboratory analysis for settlement, including consolidation testing of softer clays. As noted previously, this information will be required in areas where site grades are raised and significant settlement may be expected.

2.3 SUBGRADE PREPARATION AND EARTHWORK CONSIDERATIONS

2.3.1 UTILITY ABANDONMENT

Based on the limited history of development on the project site, it is unlikely that significant quantities of existing utilities will be encountered on-site. Further, we would expect the existing easement to remain unless negotiated with the City/County. It may be necessary to reroute sewer lines to accommodate development plans at the project site. We do not recommend construction of buildings over utility easements, even if allowed by the governing entity. Where encountered, we generally recommend utilities 8-inches or less in diameter may remain in-place within structural areas. Larger diameter utilities should either be removed entirely from within proposed building footprints and beneath equipment foundations or be fully grouted.

Within pavement areas, we recommend removing utilities to a minimum of 2.5 feet below final design subgrade elevations. This will provide a buffer between the bottom of pavement and top of utility that will reduce the risk of formation of hardspots.

We recommend backfilling excavations associated with utility removal with granular backfill. Granular backfill may consist of existing sands (including clayey or silty sands) or manufactured MDOT Class II, MDOT 6A crushed concrete or crushed limestone, 1-to-3-inch diameter crushed concrete, and/or MDOT 21AA crushed concrete or crushed limestone. Where 12 inches or more MDOT 6A crushed or 1-to-3-inch diameter crushed concrete/aggregate is used to backfill excavations, “choke” the surface of the open-graded material with a minimum of 3 inches of MDOT 21AA. This will reduce the risk of migration of fines into void spaces within the MDOT 6A or 1-to-3-inch diameter crushed concrete.

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2.4 GROUND IMPROVEMENT

Based on the limited subsurface information, rammed aggregate piers (RAPs) would be feasible to support slabs-on-grade and light to moderate foundation loads. RAPs would reduce the risk of settlement related to building on undocumented fill soils as well as improve the very loose to loose native sands and silts. Displacement installation methods generally appear feasible and could be used to limit the generation of spoils that would need to be disposed of offsite. Displacement piers are advanced by reverse-augering or with a vibratory probe such that auger cuttings or excavation spoils are not generated. Some pre-excavation where buried concrete and other debris are encountered should be expected. Design bearing pressures of 3,000 to 4,000 psf appear feasible for shallow foundations constructed over RAPs. Rigid inclusions could also be considered to support greater foundation loads.

RAPs and rigid inclusions are proprietary design, and the design will be by the supplier/specialty geotechnical contractor. Regarding RAPs, include a contingency for mass removal of large debris or obstructions (e.g., concrete slabs or mats) where installation machinery cannot progress beyond such items where encountered.

If fill within proposed building footprints are found to be free of organics, consideration may be given to improving existing fill in-place and constructing conventional slabs-on-grade. However, if the developer has a low risk tolerance for long-term settlement, the existing undocumented fill will need to be removed and replaced, or slabs will need to be either structural slabs or slabs will need to be supported on a grid of RAPs. Support of slabs on grids of RAPs are common for similar Brownfield sites in Southern Michigan.

2.5 SUBGRADE PREPARATION

Utilities and buried structures should be removed to a minimum depth of 2.5 feet below final subgrade elevations to reduce the risk of formation of hardspots in paved areas and below slabs-on-grade. Any remaining voids (such as abandoned pipes) should be completely filled with a flowable fill or grout within the building footprints, or the utilities removed and replaced with engineered fill.

Where utilities will be abandoned in place, a specialty grout mix may be required where impacted soil and groundwater are encountered. Backfill excavations resulting from removal of substructures with granular engineered fill (imported MDOT Class II or MDOT 6A).

Prior to beginning engineered fill placement, strip surface topsoil and existing pavements to expose the soils below and compact the underlying soils prior to placing additional fill or implementing subgrade improvement. Using a large steel drum roller, perform a minimum of five passes per unit roller width in each of two perpendicular directions. Proofroll the resulting subgrade using a loaded tandem axle dump truck. Where unsuitably loose/soft/yielding subgrade is encountered, improve the subgrade by thoroughly compacting (charging) a layer of 1-to-3-inch diameter or MDOT 6A crushed concrete or crushed limestone into the subgrade using a steel drum roller or excavator-mounted hoe pac. Alternatively, conventional undercuts or in-place chemical modification (cement-lime stabilization) could be considered. **Environmental considerations may govern design.** For example, where impacted subgrade does not pass a proofroll, consider improving the subgrade by charging open-graded crushed concrete into the subgrade, rather than undercut the soil and replace with engineered fill.

Isolated areas of localized improvements will be required, and we recommend including contingency funds for subgrade improvement.

Note: Subgrade improvement within the building footprint(s) may not be required if either structural slabs or RAPs/rigid inclusions are used to support new structures.

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2.6 ENGINEERED FILL

The fill needs to be free of frozen soil, significant organics and construction debris, and particle sizes hindering compaction. If the proposed engineered fill soils contain more than four percent organics, or debris larger than 6 inches in nominal diameter, do not use soils for engineered fill. Also, if debris material is significantly variable in nature, suspect in origin, or greater than about five percent of the soil (by weight or volume), do not use the soils for engineered fill.

Spread fill in level layers with a loose thickness appropriate for the type of equipment used to obtain compaction, but not exceeding 9 inches in thickness for clay fill. Thinner lifts will be required in confined spaces and where compaction is achieved with walk-behind rollers/compactors or plate compactors mounted on a backhoe or excavator. Compact the fill to a minimum of 95 percent of the maximum dry density as determined in accordance with the Modified Proctor test. Compact sand fill with a smooth drum vibratory roller or vibratory plate compactors. To reduce the risk of settlement using large vibratory rollers, dead roll engineered fill adjacent to existing structures and utilities that will remain active. Compact clay fill (including clayey sands) by overlapping passes with a sheepsfoot or padfoot roller at a moisture content between the two percent above the optimum and two percent below the optimum.

Based on the results of our field and laboratory testing, some, but not all, of the existing inorganic natural sand and clay soils are considered suitable for reuse as general engineered fill in areas where compaction is achieved with large equipment. The lean clays (CL) and silty sands (SM) are considered moisture-sensitive and could be difficult to compact, particularly when moisture contents are above the optimum moisture content. The contractor should moisture-condition soils as-needed by aerating and drying overly wet soils or adding water to overly dry soils so that the required density can be achieved during compaction. Unsuitable soils may be placed in non-structural areas of the site or remove them from the site.

Silts (USCS symbol “ML”) should not be reused as engineered fill. Silts are easily disturbed under dynamic loading and are susceptible to frost heave.

The successful reuse of the on-site sands and clays for engineered fill will depend on the time of year and the care the earthwork contractor uses during construction. During cold and wet periods of the year, the subgrade soils can become overly wet and disturbed, and the clayey sands and lean clays can be difficult to dry. Earthwork at sites with clayey subgrades is more successfully performed during the warmer and drier months when the likelihood of subgrade disturbance is lower, and the soils are more easily moisture conditioned. Imported fill needs to be sampled at the source, tested, and approved prior to importing to the site. We recommend imported fill consist of material meeting the gradation requirements of MDOT Class II sands.

Clays and clayey sands will be difficult to compact with smaller compaction equipment in confined areas such as utility trenches or as foundation excavation backfill. In confined areas, in areas where drainage is required, and in areas where compaction is accomplished primarily by hand-operated equipment, use an approved granular material, such as MDOT Class II granular material as backfill. Based on the borings, cleaner sands classified as (SP) or (SP-SM) may meet MDOT Class II gradational requirements and could be reused onsite. SME or another third-party agency should verify the suitability of soils proposed prior to reuse.

Coarse crushed aggregate used to backfill undercuts or to stabilize subgrades need to consist of a well graded crushed natural aggregate or clean crushed concrete ranging from 1 to 3 inches in size with no more than seven percent by weight passing the No. 200 sieve. Minus 1½-inch diameter crushed material (MDOT 6A) may also be used for this purpose. Spread the crushed material in layers not exceeding 12 inches and compact with a vibratory compactor until no further densification is observed. In cases where granular engineered fill will be placed over the crushed aggregate, choke the surface of the coarse crushed material with a layer of at least 6 inches of MDOT 21AA dense-graded aggregate and cover the surface with a suitable non-woven geotextile fabric to prevent migration of the granular engineered fill into the coarser crushed material.

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2.7 SLAB-ON-GRADE RECOMMENDATIONS

Slabs-on-grade could be considered for lightly loaded structures where existing fill can be properly prepared as described elsewhere in this report. If the developer has a low risk of tolerance for settlement of slabs-on-grade, support the slabs on RAPs or remove the fill entirely from beneath the slabs-on-grade.

Protect the slab-on-grade subgrade soils from frost action during winter construction. Any frozen soils have to be thawed and compacted or removed and replaced prior to slab-on-grade construction.

2.8 FOUNDATION RECOMMENDATIONS

2.8.1 SHALLOW FOUNDATIONS

As previously discussed, light to moderately loaded structures should be supported by shallow foundations bearing on either RAPs or rigid inclusions. For foundations bearing above RAPs or rigid inclusions, we expect allowable bearing pressures of 3,000 to 4,000 psf are feasible, with the final design bearing pressure based on the aggregate pier performance design requirements and final RAPs/rigid inclusion design, including aggregate or concrete load transfer platforms (LTPs).

Foundation design should also consider long-term settlement associated with fill placed to establish final design site grades. Once RAPs or rigid inclusions (RIs) are in place, foundations are constructed conventionally, bearing directly on the RAPs/RIs, or on the LTP overlying the RAPs/RIs.

Extend foundations a minimum of 42 inches below final design site grades for frost protection for typical winters. Place interior foundations constructed during winter months at shallower depths only if the foundations and subgrade are protected from freezing during construction.

To verify suitable subgrade at the bearing surface of foundation excavations, verify the tops of RAPs/rigid inclusions are exposed at the base of the foundation excavations and the surface of the pier properly compacted/prepared per the requirements of the aggregate pier designer. Unless undercuts are required for environmental purposes, foundation undercuts within the building footprint are generally not expected for foundations bearing on rigid inclusions or RAPs.

Remove caved soils from the foundation bearing surfaces before placing concrete. The subgrade soils are susceptible to disturbance, especially when exposed to water and trafficked. Remove disturbed soils immediately prior to foundation concrete placement. Place a working surface of either crushed aggregate or crushed concrete in excavation areas where groundwater accumulates to protect the exposed surface from disturbance.

For frost heave considerations, excavate trench foundations in a vertical manner and do not allow them to "mushroom out" near the top. Clay subgrade may be suitable for bank-formed foundations; however, where granular fill or loose native sands or silts are encountered and verticality of excavation sidewalls cannot be maintained, over-excavate and form foundations. For constructability purposes, establish minimum foundation sizes of 18 inches for continuous (wall) foundations and 30 inches for spread (column) foundations.

For foundations and foundation walls also functioning as retaining walls (such as truck wells), preliminary designs may be based on a friction factor of 0.4, limited to a maximum allowable adhesion of 1,000 psf on the base of the foundation to resist sliding. Size foundations such that the resultant vertical load is within the central third of the foundation.

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Depending upon soil and groundwater constituents special concrete mixes could be required for foundations and below-grade walls extending into existing impacted site soils. The final geotechnical analysis should include corrosion analysis of site soils including fill with industrial debris, environmentally impacted existing site soils, native inorganic clays and silts, and any organic soils encountered that may remain in place. If the site is built up and foundations are constructed in imported sand or clay fill, no special considerations would be needed.

2.8.2 DEEP FOUNDATIONS

Based on the soil conditions encountered at the project site, deep foundations are not expected to be required. A higher RAP/RI capacity could be obtained by extending the units deeper to bear on very dense to extremely dense glacial till. Based on the single boring advanced to depth, higher capacity RAPs/RIs will likely bear at depths of 35 feet or greater. For applications where high loads are present that cannot be supported on RAPs/RIs, driven pipe piles, straight-shaft drilled piers, and/or augered cast-in-place piers could be considered. The final geotechnical evaluation will need to include borings that extend at least 20 feet into extremely dense glacial till to evaluate both end-bearing and friction capacities.

2.9 SEISMIC SITE CLASS

According to the limited information obtained from the borings, we designate the subgrade soils at this site as seismic site Class D in determining seismic design forces for this project in accordance with the 2015 Michigan Building Code (MBC) referencing Table 20.3-1 in ASCE Standard ASCE/SEI 7-10. A design category A applies. Pending additional deep borings, a seismic site Class C also appears feasible.

2.10 RETAINING/ BELOW-GRADE WALLS AND TEMPORARY EARTH RETENTION

Low height cast-in-place concrete or mechanically stabilized earthen (MSE) retaining walls could be needed at truck docks. For conventional concrete or mechanically stabilized earthen (MSE), or segmental block retaining walls, walls should be backfilled with MDOT Class II granular material. Provide positive surface drainage and install edge drains at the base of retaining walls, tied to nearby storm structures to prevent water from accumulating behind walls.

Wall backfill that will support floor slabs and pavements should be compacted to a minimum of 95 percent of the maximum dry density as determined by the Modified Proctor test. As a minimum, backfill that will not be used for structural support of pavements, floor slabs or sidewalks should be compacted to the degree where it is stable under construction equipment. Exercise care during backfilling operations to avoid overstressing the walls.

For a drained granular backfill and a level finish surface behind the wall, we recommend an active equivalent fluid pressure of 40 pounds per cubic foot (pcf) for design. This earth pressure is based on the wall being flexible enough to permit the active earth pressure condition to be reached. An outward movement away from the backfill equal to approximately 0.001 times the height of the wall is generally required to achieve the active earth pressure condition for granular backfill. If the wall is restrained or is rigid enough so that it does not rotate sufficiently to reach the active earth condition, a higher lateral earth pressure (at-rest condition) should be used for design. For rigid walls backfilled with a free-draining granular material and a level finish surface behind the wall, we recommend an at-rest fluid pressure of 55 pcf for design. Also, any additional lateral pressures due to surcharge loading, such as adjacent floor or traffic loads, sloping ground, or parking loads, should be added to lateral earth pressures for design.

The earth pressures presented above are for a drained backfill. To reduce the potential for the build-up of hydrostatic pressure behind below-grade walls, we recommend permanent edge drains be installed along the base of the perimeter of the below-grade walls, if feasible.

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The perimeter edge drains should consist of a minimum 6-inch-diameter geotextile-wrapped perforated plastic edge drain, surrounded by 6 inches of a geotextile-wrapped filter material, such as MDOT 6A. As indicated above, the walls should be backfilled with MDOT Class II granular material above the edge drain.

As building usage is currently unknown, heavy industrial usage is possible, including the potential for press pits and other localized below-grade areas. Shallow pits can be designed as water-tight structures, provided pit walls are designed to resist the additional lateral pressure associated with hydrostatic pressure, and the walls and foundations are waterproofed with a sheet-applied membrane.

The base slabs/mats may need to be designed to resist hydrostatic uplift. Pits and basements should be established above base flood elevations (BFEs).

The following parameters for evaluating the stability of the retaining walls assume the base of the wall bears directly on suitably prepared existing site soils, with walls backfilled with MDOT Class II sand.

Sliding resistance may be determined by using a recommended ultimate sliding coefficient of 0.40 for foundations constructed over crushed concrete or newly placed engineered fill. Passive, active and at-rest earth pressure coefficients of 3.0, 0.33 and 0.50, respectively, may be used in combination with a unit weight of backfill of 120 pcf. This assumes a granular backfill will be in contact with the wall on the back side and the toe of the wall will be backfilled with engineered fill or constructed in direct contact with existing site soils (bank-formed or "neat trench").

In addition to checking sliding stability of the retaining walls, the safety factor from overturning, location of the resultant force at the base, mass stability, and contact pressure at the base should also be evaluated.

Temporary earth retention could be required for installation of utilities. For most utility installations, we expect excavations can be adequately sloped or conventional trench boxes will be used for excavation support. However, an engineer-designed temporary earth retention system (TERS) will likely be required for deeper utilities. TERS may be designed based on the earth pressures provided in the previous paragraphs. Dewatering could be required to install deep utilities extending more than about 1 foot below static water levels, unless steel sheet piles can be installed, sealed into the native clays, and bulk-headed such that a temporary cofferdam-type structure is installed). Stabilization at the base of the excavation, consisting of either crushed concrete or a lean concrete mudmat could be required to maintain a dry and reasonably firm work surface/bedding layer. This will be especially crucial where excavations extend into native silts, which are sensitive to dynamic loading and can become unstable under even light foot traffic.

NOTE: Dewatering restrictions and/or special handing requirements could apply where impacted soil and/or groundwater are encountered. Do not release groundwater into the creek, wetlands, or storm or sanitary sewers.

2.11 SLOPE CONSIDERATIONS

Transitions along Gilkey Creek are currently unknown, but for maintenance purposes, we recommend a minimum slope of 1V:2H, and flatter slopes may be required if granular soils are used to raise site grades/are exposed on final slopes.

While slopes constructed at the minimum slopes recommended are not suscepible to slope instability, where non-homogenous fill (varying layers of sands and clays) are exposed at the face of the slope, localized shallow instability, or "sloughs" can occur. Therefore, use only one engineered fill type for slope construction, and place engineered fill in uniform horizontal layers (rather than placing fill on the slope). Place the fill first and then cut the design slope, rather than placing fill in sloping layers. See Image No. 4 for additional information.

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While beyond our current scopes of services, if construction is planned that will extend fill out on or adjacent to the top of slope, slope stability must be considered.

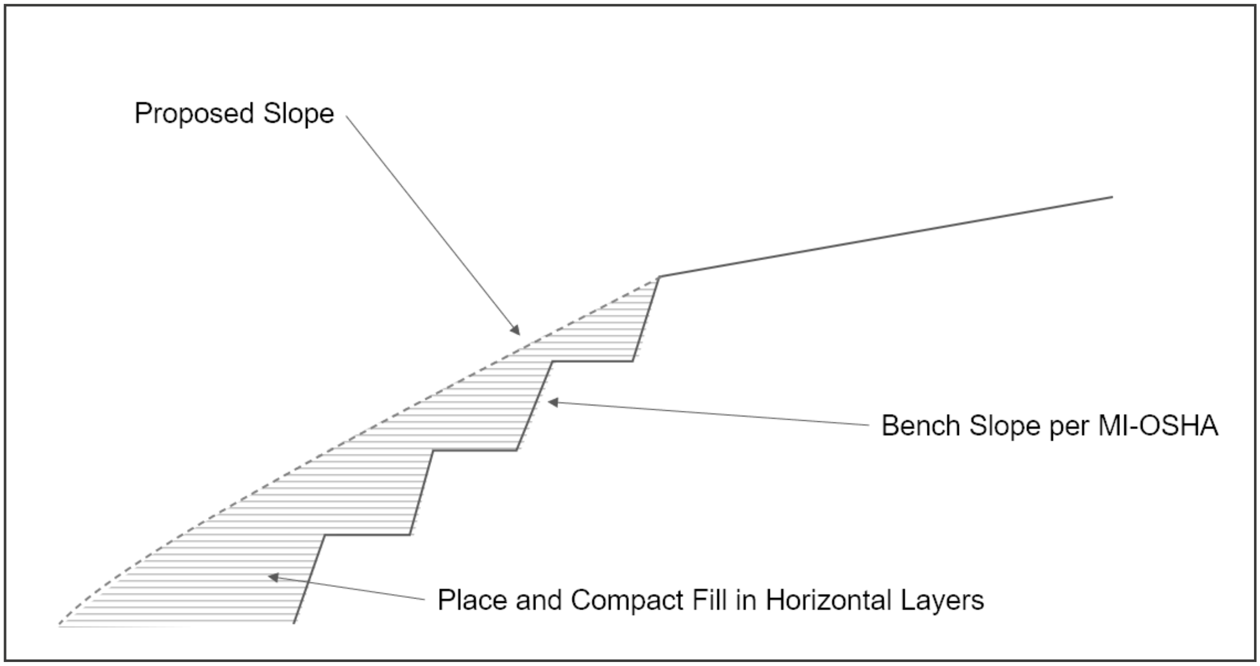

IMAGE NO. 4: Engineered Fill Placement on Slopes

3. EXCAVATION AND GROUNDWATER MANAGEMENT

Due to the nature of the upper fills and impacted soil and groundwater, utilities in at least some of these materials are likely at risk for corrosion. Install sensitive utilities in concrete duct banks or use materials that are not susceptible to corrosion. Refer to previous comments regarding corrosion analysis recommended for a final geotechnical phase evaluation. Bed all other utilities and backfill them with clean imported MDOT Class II sand and install them either completely above or completely below groundwater levels where feasible.

Regarding groundwater, contractors should anticipate perched water at varying depths. For most excavations, we anticipate standard sump pit and pumping procedures will be adequate to control these accumulations, on a localized basis. Further, due to potentially impacted soils and groundwater, we expect the contractors will not be allowed to discharge into existing storm sewers or into the creek and/or wetlands, although direct discharge onto the ground away from work areas could be acceptable. If not, it may be necessary to discharge into portable tanks for later off-site disposal or for storage prior to on-site treatment. An environmental consultant should determine if additional restrictions on groundwater management will be required.

The contractors should be made aware of the site history, including the potential to encounter deeper areas of undocumented fill, below-grade obstructions, and environmentally impacted soils. Provide adequate contingencies in construction budgets and include unit rates for removal and replacement of impacted soils, obstructions, and undercuts in contract documents.

Where Brownfield or other incentives are in place, it is necessary to carefully record, track, and verify eligible activities.

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When redeveloping impacted sites, conventional construction phase testing may not always be feasible and/or practical. We recommend informing the Architect, municipal building authority, and other design team engineers that variations from typical specifications will be required. Third party testing should be performed by a consultant experienced with Brownfield redevelopment. The third party consultant must use engineering judgement to determine when such deviations may be required, and deviations from conventional activities adequately reported.

Typical variations may include, but are not limited to, performing proofrolls on shallow fill and/or highly variable fill in lieu of density testing; verifying suitable compaction using dynamic cone penetrometers (DCPs) and/or wet density comparisons, rather than conventional dry density nuclear density gauge readings; "charging" open-graded crushed aggregate/crushed concrete into loose/soft subgrade in lieu of undercutting impacted soils; and/or chemically modifying soft or high moisture content clays instead of undercutting.

The exposed subgrade soils are prone to disturbance due to weather and activity on-site. Undercut disturbed soils and replace them with engineered fill or otherwise stabilize in place. For near-surface clayey soils, chemical modification using lime and/or cement may be feasible and less costly than conventional undercutting, especially where impacted soils are encountered.

In heavily trafficked areas and/or under adverse weather conditions, protect areas of exposed subgrade at the site by placement of crushed concrete or crushed aggregate on the exposed subgrade.

Regarding subgrade protection, install designated haul roads and do not allow construction traffic to randomly traffic the subgrade. Take care to avoid contaminating relatively clean areas of the site with soils from more heavily contaminated areas.

We expect truck clean off areas or other methods of reducing tracking of potentially impacted soils onto surrounding roadways will be required during construction.

The contractor must protect existing structures and utilities to remain in place during construction. Do not extend temporary excavations below the bottom of existing adjacent improvements (e.g., foundations, utilities, etc.) without bracing, shoring or underpinning of the existing improvement. Exercise care during the excavating and compacting operations so that excessive vibrations do not cause settlement of existing structures and pavements, or utilities are not damaged or undermined.

Vibrations from construction operations can be felt at levels much lower than those required to cause structural distress or damage. Consider pre- and post-construction condition assessments of adjacent properties, possibly in combination with vibration monitoring during construction operations. Implementing these steps can reduce the risk of potentially nuisance damage claims.

Regarding excavations, properly slope or brace all excavations in accordance with MI-OSHA requirements. Based on MIOSHA-STD-1306 (3/18/13) Part 9. Excavation, Trenching, and Shoring, the maximum angle of repose for excavations deeper than 5 feet (unsupported) for the soil and groundwater conditions encountered at the site are as follows:

TABLE NO. 4.13.1: MAXIMUM ANGLE OF REPOSE FOR EXCAVATIONS DEEPER THAN 5 FEET

SOIL TYPE	ANGLE OF REPOSE[1]
Natural Sands Above Groundwater Table	1H:1V (45 °)
Natural Sands Below Groundwater Table	2H:1V (26 °)
Engineered Sand Fill	1H:1V (45 °)
Stiff Clay (minimum of 2.5 tsf[2])	1/2H:1V (63°)
Firm Clay (minimum of 1.5 tsf[2])	2/3H:1V (56 °)

NOTES:
1. Conditions encountered during construction may require flatter slopes and/or a flat working space at the top of the slope. Also, flatter slopes would be required in clay with sand seams or partings that could potentially develop slide planes. As with any temporary slopes, weather conditions and surface runoff can adversely affect the slope condition.
2. Strength values are unconfirmed compressive strength based on a hand penetrometer.

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Supplemental geotechnical services will be required to provide final engineering recommendations for the projects. This evaluation was provided by SME specifically for the Detroit Regional Partnership, and should not be relied upon by outside parties, other than for purposes of developing preliminary development plans. Do not base final design solely on the contents of this report.

4. EVALUATION PROCEDURES

4.1 BORINGS

The proposed number, locations, and depths of the borings were determined by the Detroit Regional Partnership, RACER Trust, and SME. Some borings were extended deeper due to loose/soft/variable upper zone soil conditions. SME located the borings in field and obtained the existing ground surface elevations at the boring locations using our hand-held global positioning system (GPS).

A truck-mounted drill rig was mobilized to the project site on August 4 and 5, 2025. Boring B1 through B6 were advanced using a rotary-type drill rig. The borings included soil sampling based on Split-Barrel Sampling procedures. Each boring was advanced to depths ranging from 25 feet to 39 feet. Borings B1 through B3 were advanced deeper than the initially planned depths of 25 feet due to encountering loose soils at the termination depth.

Groundwater level measurements were recorded during and immediately after completion of each boring. After completion of drilling and obtaining groundwater level measurements, the boreholes were backfilled with auger cuttings, EPCO hole plugs, and asphalt cold patch. Natural auger cuttings were used to backfill in the natural soil layers and fill soil cuttings were used to backfill in the fill soil layers, with a native clay layer placed near the surface over the fill.

The Field-Testing Procedures in Appendix B provides more detailed descriptions of field tests typically performed by SME and referenced in this report. Soil samples recovered from the field exploration were taken to our laboratory for further observations and testing.

Upon completion of the laboratory testing, boring logs were prepared and include the soil descriptions, penetration resistances, pertinent field observations, and the results of the laboratory testing. Each log also includes the estimated existing ground surface elevation. Explanations of symbols and terms used on the boring logs are provided on the Boring Log Terminology sheet included in Appendix A.

Soil samples are normally retained in our laboratory for 60 days and are then disposed, unless instructed otherwise.

4.2 TEST PITS

We mobilized an earthwork contractor to the project site on August 20, 2025 to excavate test pits at boring locations B5 and B6 (TP5 and TP6) to further evaluate the fill materials at these locations. An excavator was used to excavate a total of six test pits on the project site. The four additional test pits beyond TP5 and TP6 were selected to assist in understanding the distribution of the fill deposits. In general, the test pits were excavated until natural materials or groundwater were encountered. After the completion of the test pits, the excavated spoils (existing fill materials) were placed back in the excavation in lifts in the reverse order excavated and tamped with the excavator bucket.

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4.3 LABORATORY TESTING

The laboratory testing program consisted of visually classifying the recovered samples in accordance with ASTM D-2488. Based on the laboratory testing, we prepared soil descriptions and assigned a group symbol for the various soil strata observed based on the Unified Soil Classification System (USCS). In addition, moisture content tests, Torvane tests, and hand penetrometer tests were performed on portions of cohesive samples obtained.

The Laboratory Testing Procedures in Appendix B provides descriptions of the laboratory tests.

5. SIGNATURES

PREPARED BY:

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Kyle P. Areaux, PE
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REVIEWED BY:

Laurel M. Johnson

Laurel M. Johnson, PE
Principal Consultant

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APPENDIX A

BORING AND TEST PIT LOCATION PLAN (FIGURE NO. 1)
BORING LOG TERMINOLOGY
BORING LOGS (B1 THROUGH B6)
TEST PIT LOGS (TP7 THROUGH TP10)
TEST PIT PHOTOLOG
SEISMIC DATA (OSHDA)
FEMA FIRMETTE
HISTORICAL AERIAL IMAGERY (PREPARED BY ERIS DATED JUNE 12, 2025)
HISTORICAL TOPOGRAPHIC MAPS (PREPARED BY ERIS DATED JUNE 8, 2025)

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LEGEND

- APPROX. BORING LOCATIONS
- APPROX. TEST PIT LOCATIONS

NORTH

NOTE:
BORING AND TEST PIT LOCATIONS ARE SHOWN AS APPROXIMATE.. THE BACKGROUND IMAGE IS FROM GOOGLE EARTH PRO, DATED APRIL 29, 2025.

SME

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Project

DRP/ DAVISON/ STAGE 7 DUE DILIGENCE

Project Location

DONEGAL STREET BURTON, MICHIGAN

Sheet Name

BORING AND TEST PIT LOCATION PLAN

No.	Revision Date

Date: 08-25-2025

CADD: -

Designer: KPA

Scale: N/A

Project: 100047.00

Figure No.: 1

DRAWING NOTE: SCALE DEPICTED IS MEANT FOR 11" X 17" AND WILL SCALE INCORRECTLY IF PRINTED ON ANY OTHER SIZE MEDIA

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BORING LOG TERMINOLOGY

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOIL
(more than 50% of material is larger than No. 200 sieve size.)

GRAVEL More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravel (Less than 5% fines)	
	GW	Well-graded gravel; gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravel; gravel-sand mixtures, little or no fines
	Gravel with fines (More than 12% fines)	
	GM	Silty gravel; gravel-sand-silt mixtures
	GC	Clayey gravel; gravel-sand-clay mixtures
SAND 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sand (Less than 5% fines)	
	SW	Well-graded sand; sand-gravel mixtures, little or no fines
	SP	Poorly graded sand; sand-gravel mixtures, little or no fines
	Sand with fines (More than 12% fines)	
	SM	Silty sand; sand-silt-gravel mixtures
	SC	Clayey sand; sand–clay-gravel mixtures

FINE-GRAINED SOIL
(50% or more of material is smaller than No. 200 sieve size)

SILT AND CLAY Liquid limit less than 50%	ML	Inorganic silt; sandy silt or gravelly silt with slight plasticity
	CL	Inorganic clay of low plasticity; lean clay, sandy clay, gravelly clay
	OL	Organic silt and organic clay of low plasticity
SILT AND CLAY Liquid limit 50% or greater	MH	Inorganic silt of high plasticity, elastic silt
	CH	Inorganic clay of high plasticity, fat clay
	OH	Organic silt and organic clay of high plasticity
HIGHLY ORGANIC SOIL	PT	Peat and other highly organic soil

OTHER MATERIAL SYMBOLS

Topsoil
Void
Sandstone
Asphalt Concrete
Glacial Till
Siltstone
Aggregate Base
Coal
Limestone
Portland Cement Concrete
Shale
Fill

LABORATORY CLASSIFICATION CRITERIA

GW	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{D_{30}^{\ 2}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with PI greater than 7	
SW	$C_U = \frac{D_{60}}{D_{10}}$ greater than 6; $C_C = \frac{D_{30}^{\ 2}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for SW	
SM	Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
SC	Atterberg limits above "A" line with PI greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent……………………….……..GW, GP, SW, SP
More than 12 percent…………………….……..GM, GC, SM, SC
5 to 12 percent…………….……..Cases requiring dual symbols

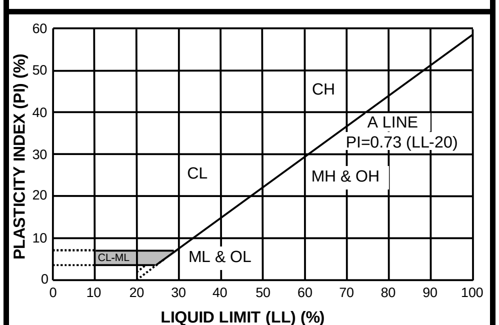
- SP-SM or SW-SM (SAND with Silt or SAND with Silt and Gravel)
- SP-SC or SW-SC (SAND with Clay or SAND with Clay and Gravel)
- GP-GM or GW-GM (GRAVEL with Silt or GRAVEL with Silt and Sand)
- GP-GC or GW-GC (GRAVEL with Clay or GRAVEL with Clay and Sand)

If the fines are CL-ML:
- SC-SM (SILTY CLAYEY SAND or SILTY CLAYEY SAND with Gravel)
- SM-SC (CLAYEY SILTY SAND or CLAYEY SILTY SAND with Gravel)
- GC-GM (SILTY CLAYEY GRAVEL or SILTY CLAYEY GRAVEL with Sand)

PARTICLE SIZES

Boulders	-	Greater than 12 inches
Cobbles	-	3 inches to 12 inches
Gravel- Coarse	-	3/4 inches to 3 inches
Fine	-	No. 4 to 3/4 inches
Sand- Coarse	-	No. 10 to No. 4
Medium	-	No. 40 to No. 10
Fine	-	No. 200 to No. 40
Silt and Clay	-	Less than (0.074 mm)

PLASTICITY CHART



VISUAL MANUAL PROCEDURE

When laboratory tests are not performed to confirm the classification of soils exhibiting borderline classifications, the two possible classifications would be separated with a slash, as follows:

For soils where it is difficult to distinguish if it is a coarse or fine-grained soil:

- SC/CL (CLAYEY SAND to Sandy LEAN CLAY)
- SM/ML (SILTY SAND to SANDY SILT)
- GC/CL (CLAYEY GRAVEL to Gravelly LEAN CLAY)
- GM/ML (SILTY GRAVEL to Gravelly SILT)

For soils where it is difficult to distinguish if it is sand or gravel, poorly or well-graded sand or gravel; silt or clay; or plastic or non-plastic silt or clay:

- SP/GP or SW/GW (SAND with Gravel to GRAVEL with Sand)
- SC/GC (CLAYEY SAND with Gravel to CLAYEY GRAVEL with Sand)
- SM/GM (SILTY SAND with Gravel to SILTY GRAVEL with Sand)
- SW/SP (SAND or SAND with Gravel)
- GP/GW (GRAVEL or GRAVEL with Sand)
- SC/SM (CLAYEY to SILTY SAND)
- GM/GC (SILTY to CLAYEY GRAVEL)
- CL/ML (SILTY CLAY)
- ML/CL (CLAYEY SILT)
- CH/MH (FAT CLAY to ELASTIC SILT)
- CL/CH (LEAN to FAT CLAY)
- MH/ML (ELASTIC SILT to SILT)

DRILLING AND SAMPLING ABBREVIATIONS

2ST	–	Shelby Tube – 2" O.D.
3ST	–	Shelby Tube – 3" O.D.
AS	–	Auger Sample
GS	–	Grab Sample
LS	–	Liner Sample
NR	–	No Recovery
PM	–	Pressuremeter
RC	–	Rock Core diamond bit. NX size, except where noted
SB	–	Split Barrel Sample 1-3/8" I.D., 2" O.D., except where noted
VS	–	Vane Shear
WS	–	Wash Sample

OTHER ABBREVIATIONS

WOH	–	Weight of Hammer
WOR	–	Weight of Rods
SP	–	Soil Probe
PID	–	Photo Ionization Device
FID	–	Flame Ionization Device

DEPOSITIONAL FEATURES

Parting	–	as much as 1/16 inch thick
Seam	–	1/16 inch to 1/2 inch thick
Layer	–	1/2 inch to 12 inches thick
Stratum	–	greater than 12 inches thick
Pocket	–	deposit of limited lateral extent
Lens	–	lenticular deposit
Hardpan/Till	–	an unstratified, consolidated or cemented mixture of clay, silt, sand and/or gravel, the size/shape of the constituents vary widely
Lacustrine	–	soil deposited by lake water
Mottled	–	soil irregularly marked with spots of different colors that vary in number and size
Varved	–	alternating partings or seams of silt and/or clay
Occasional	–	one or less per foot of thickness
Frequent	–	more than one per foot of thickness
Interbedded	–	strata of soil or beds of rock lying between or alternating with other strata of a different nature

DESCRIPTION OF RELATIVE QUANTITIES

The visual-manual procedure uses the following terms to describe the relative quantities of notable foreign materials, gravel, sand or fines:

Trace	–	particles are present but estimated to be less than 5%
Few	–	5 to 10%
Little	–	15 to 25%
Some	–	30 to 45%
Mostly	–	50 to 100%

CLASSIFICATION TERMINOLOGY AND CORRELATIONS

Cohesionless Soils

Relative Density	N_{60} (N-Value) (Blows per foot)
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	51 to 80
Extremely Dense	Over 81

Cohesive Soils

Consistency	N_{60} (N-Value) (Blows per foot)	Undrained Shear Strength (kips/ft^2)
Very Soft	<2	0.25 or less
Soft	2 - 4	> 0.25 to 0.50
Medium	5 - 8	> 0.50 to 1.0
Stiff	9 - 15	> 1.0 to 2.0
Very Stiff	16 - 30	> 2.0 to 4.0
Hard	> 30	> 4.0 or greater

Standard Penetration 'N-Value' = Blows per foot of a 140-pound hammer falling 30 inches on a 2-inch O.D. split barrel sampler, except where noted. N60 values as reported on boring logs represent raw N-values corrected for hammer efficiency only.

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BORING B 1

BORING DEPTH: 30 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence **PROJECT NUMBER:** 100047.00
CLIENT: Detroit Regional Partnership **PROJECT LOCATION:** Burton, Michigan

DATE STARTED: 8/5/25 **COMPLETED:** 8/5/25 **BORING METHOD:** Hollow-stem Auger
DRILLER: WN **RIG NO.:** 552 (CME 55) **LOGGED BY:** RP **CHECKED BY:** KPA

LATITUDE: 43.03031
LONGITUDE: -83.62872
ELEVATION: 760 FT

HAMMER EFFICIENCY: 78%
DATE: 5/13/2025
N_{60} -- O

DRY DENSITY (pcf) -- ■ (90 100 110 120)
MOISTURE & ATTERBERG LIMITS (%) PL MC LL (10 20 30 40)

HAND PENE. / TORVANE SHEAR / UNC. COMP. / VANE SHEAR (PK) / VANE SHEAR (REM) / TRIAXIAL (UU)
SHEAR STRENGTH (KSF) 1 2 3 4

ELEVATION (FEET)	DEPTH (FEET)	PROFILE DESCRIPTION	Bottom Elev.
760	0	0.5 FILL- Fine SAND with Brick- Occasional Silt Seams- Black- Moist- Loose (SP)	759.5
		FILL- Fine SAND with Brick- Trace Organics- Occasional Silt Seams- Black- Moist- Loose to Medium Dense (SP)	
	8.8	Sandy SILT- Occasional Clay Seams- Gray- Moist to Wet- Loose to Medium Dense (ML)	751.3
	23.0	Sandy LEAN CLAY- Gray- Hard (CL)	737.0
730	30.0	END OF BORING AT 30.0 FEET.	730.0

SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60}	MOISTURE (%)	SHEAR STRENGTH (KSF)	REMARKS
SB1	20	1 3 3 5	8			
SB2	21	3 5 7 9	16			
SB3	24	5 5 3 4	10			
SB4	20	4 7 6 5	17	18		
SB5	18	2 3 3	8	22		
SB6	18	4 7 7	18	23		
SB7	18	7 14 13	35	10	4.5+	
SB8	18	15 35 28	82	11		

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
∇ **DURING BORING:**	9.0	751.0
▼ **AT END OF BORING:**	None	

BACKFILL METHOD: Auger Cuttings capped with EPCO Hole Plug

NOTES: 1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

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SME

BORING B 2

BORING DEPTH: 38.92 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence **PROJECT NUMBER:** 100047.00

CLIENT: Detroit Regional Partnership **PROJECT LOCATION:** Burton, Michigan

DATE STARTED: 8/5/25 **COMPLETED:** 8/5/25 **BORING METHOD:** Hollow-stem Auger

DRILLER: WN **RIG NO.:** 552 (CME 55) **LOGGED BY:** RP **CHECKED BY:** KPA

LATITUDE: 43.03037
LONGITUDE: -83.62589
ELEVATION: 762 FT

HAMMER EFFICIENCY: 78%
DATE: 5/13/2025

DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60}
0–3.0	FILL- Fine to Medium SILTY SAND- Trace Wood Chips- Occasional Clay Layers- Black- Moist- Medium Dense (SM)	759.0	SB1	14	5 7 9 4	21
			SB2	24	6 6 6 7	16
3.0–9.5	Fine SILTY SAND- Brown- Moist- Medium Dense to Very Loose (SM)	752.5	SB3	24	6 6 5 5	14
			SB4	24	1 1 2 3	4
9.5–30+	Fine to Medium SAND- Occasional Silt Seams- Brown- Wet- Loose to Medium Dense (SP)		SB5	18	2 3 7	13
			SB6	18	2 3 4	9
			SB7	18	2 3 6	12
			SB8	18	2 3 5	10

Column headers also include: DRY DENSITY (pcf) -- ■ (90 100 110 120); MOISTURE & ATTERBERG LIMITS (%) PL MC LL (10 20 30 40); HAND PENE.; TORVANE SHEAR; UNC. COMP.; VANE SHEAR (PK); VANE SHEAR (REM); TRIAXIAL (UU); SHEAR STRENGTH (KSF) (1 2 3 4); REMARKS.

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
DURING BORING:	10.0	752.0
AT END OF BORING:	None	

BACKFILL METHOD: Auger Cuttings capped with EPCO Hole Plug

NOTES: 1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

(Continued Next Page)

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SME®

BORING B 2

BORING DEPTH: 38.92 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence
PROJECT NUMBER: 100047.00
CLIENT: Detroit Regional Partnership
PROJECT LOCATION: Burton, Michigan

LATITUDE: 43.03037
LONGITUDE: -83.62589
ELEVATION: 762 FT

HAMMER EFFICIENCY: 78%
DATE: 5/13/2025
N_{60} -- O

DRY DENSITY (pcf) -- ■ (90, 100, 110, 120)
MOISTURE & ATTERBERG LIMITS (%) PL MC LL (10, 20, 30, 40)

SHEAR STRENGTH (KSF) (1, 2, 3, 4): HAND PENE., TORVANE SHEAR, UNC. COMP., VANE SHEAR (PK), VANE SHEAR (REM), TRIAXIAL (UU)

DEPTH (FEET)	ELEVATION (FEET)	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60}	REMARKS
30.0–38.0	to 724.0	Fine to Medium SAND- Occasional Silt Seams- Brown- Wet- Loose to Medium Dense (SP) *(continued)*	SB9	18	9, 10, 11	27	
38.0–38.9	724.0–723.1	Fine to Coarse SAND with Gravel- Occasional Silt Seams- Brown- Moist- Extremely Dense (SP)	SB10	5	50/5"	65+	

END OF BORING AT 38.9 FEET.

Depth scale: 30–70 ft; Elevation scale: 730–695 ft.

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SME

BORING B 3

BORING DEPTH: 30 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence **PROJECT NUMBER:** 100047.00
CLIENT: Detroit Regional Partnership **PROJECT LOCATION:** Burton, Michigan

DATE STARTED: 8/4/25 **COMPLETED:** 8/4/25 **BORING METHOD:** Hollow-stem Auger
DRILLER: WN **RIG NO.:** 552 (CME 55) **LOGGED BY:** RP **CHECKED BY:** KPA

LATITUDE: 43.02925
LONGITUDE: -83.62824
ELEVATION: 762 FT

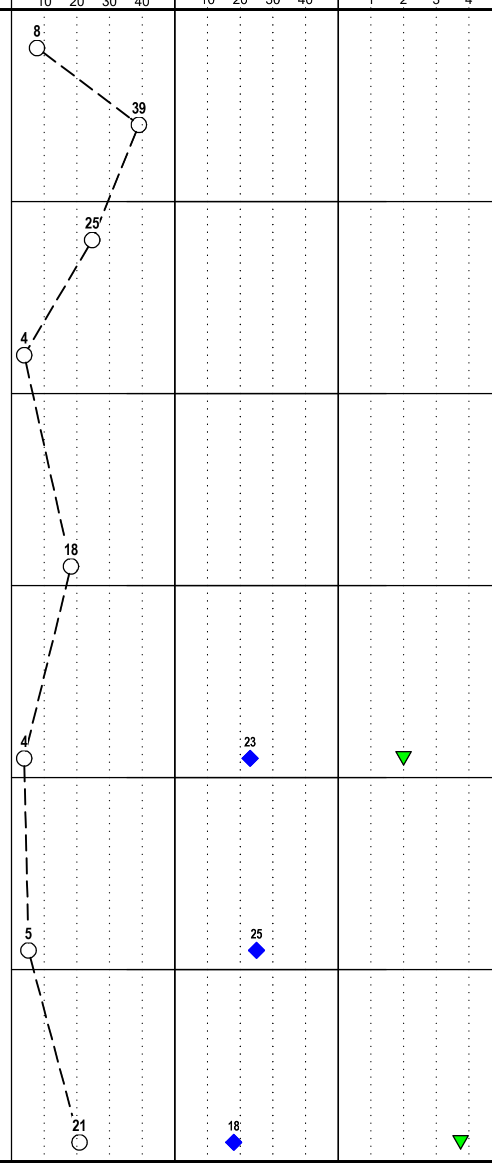
HAMMER EFFICIENCY: 78%
DATE: 5/13/2025
N_{60} -- ○

DRY DENSITY (pcf) -- ■ (90, 100, 110, 120)
MOISTURE & ATTERBERG LIMITS (%) PL, MC, LL (10, 20, 30, 40)

▼ HAND PENE.
🟥 TORVANE SHEAR
● UNC. COMP.
⊡ VANE SHEAR (PK)
✕ VANE SHEAR (REM)
⊕ TRIAXIAL (UU)
SHEAR STRENGTH (KSF) (1, 2, 3, 4)

DEPTH (FEET)	ELEVATION (FEET)	PROFILE DESCRIPTION
0 – 8.5	762 – 753.5	FILL- Fine SILTY SAND- Some Organics- Trace Brick, Concrete, Coal Ash, & Wood Debris- Black- Moist- Loose to Dense (SM)
8.5 – 18.5	753.5 – 743.5	Fine SILTY SAND- Occasional Clay Seams- Gray- Moist to Wet- Very Loose to Medium Dense (SM)
18.5 – 28.5	743.5 – 733.5	Clayey SILT- Frequent Clay Layers- Gray- Wet- Very Loose to Loose (ML)
28.5 – 29.7	733.5 – 732.3	Fine to Medium SAND- Gray- Wet- Medium Dense (SP)

SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60}	MOISTURE (%)
SB1	22	3 3 3 3	8	
SB2	24	9 15 15 17	39	
SB3	20	10 12 7 9	25	
SB4	18	2 2 1 2	4	
SB5	18	5 7 7	18	
SB6	18	1 1 2	4	23
SB7	18	1 2 2	5	25
SB8	18	6 8 8	21	18



REMARKS

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
DURING BORING:	10.0	752.0
AT END OF BORING:	None	

BACKFILL METHOD: Auger Cuttings capped with EPCO Hole Plug

NOTES: 1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

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SME

BORING B 3

BORING DEPTH: 30 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence **PROJECT NUMBER:** 100047.00

CLIENT: Detroit Regional Partnership **PROJECT LOCATION:** Burton, Michigan

LATITUDE: 43.02925
LONGITUDE: -83.62824
ELEVATION: 762 FT

HAMMER EFFICIENCY: 78%
DATE: 5/13/2025
N_{60} -- O

DRY DENSITY (pcf) -- ■ (90, 100, 110, 120)
MOISTURE & ATTERBERG LIMITS (%) PL MC LL (10, 20, 30, 40)

HAND PENE. / TORVANE SHEAR / UNC. COMP. / VANE SHEAR (PK) / VANE SHEAR (REM) / TRIAXIAL (UU)
SHEAR STRENGTH (KSF) (1, 2, 3, 4)

ELEVATION (FEET)	DEPTH (FEET)	SYMBOLIC PROFILE	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60} (10 20 30 40)	MOISTURE & ATTERBERG LIMITS (10 20 30 40)	SHEAR STRENGTH (KSF) (1 2 3 4)	REMARKS
	30		30.0 Sandy LEAN CLAY- Gray- Very Stiff (CL) 732.0							
			END OF BORING AT 30.0 FEET.							
730										
	35									
725										
	40									
720										
	45									
715										
	50									
710										
	55									
705										
	60									
700										
	65									
695										
	70									

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SME

BORING B 4

BORING DEPTH: 25 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence **PROJECT NUMBER:** 100047.00

CLIENT: Detroit Regional Partnership **PROJECT LOCATION:** Burton, Michigan

DATE STARTED: 8/4/25 **COMPLETED:** 8/4/25 **BORING METHOD:** Hollow-stem Auger

DRILLER: WN **RIG NO.:** 552 (CME 55) **LOGGED BY:** RP **CHECKED BY:** KPA

LATITUDE: 43.02872
LONGITUDE: -83.62611
ELEVATION: 761 FT

HAMMER EFFICIENCY: 78%
DATE: 5/13/2025
N_{60} -- O

DRY DENSITY (pcf) -- ■ (90, 100, 110, 120)

MOISTURE & ATTERBERG LIMITS (%): PL, MC, LL (10, 20, 30, 40)

HAND PENE. / TORVANE SHEAR / UNC. COMP. / VANE SHEAR (PK) / VANE SHEAR (REM) / TRIAXIAL (UU)

SHEAR STRENGTH (KSF) (1, 2, 3, 4)

DEPTH (FEET)	ELEVATION (FEET)	PROFILE DESCRIPTION
0.0 – 1.0	761.0 – 760.0	FILL- Fine SILTY SAND- Black- Moist- Loose (SM)
1.0 – 8.0	760.0 – 753.0	LEAN CLAY with SAND- Brown to Gray- Hard to Soft (CL)
8.0 – 13.5	753.0 – 747.5	Sandy SILT- Gray- Moist- Medium Dense (ML)
13.5 – 25.0	747.5 – 736.0	LEAN CLAY with SAND- Frequent Sand & Silt Seams- Gray- Very Stiff to Hard (CL)

END OF BORING AT 25.0 FEET.

SAMPLE TYPE/NO.	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60}	MOISTURE (%)	SHEAR STRENGTH (KSF)	REMARKS
SB1	18	2-2-2-2	5	12	4.5+ (Hand Pene.)	
SB2	24	1-2-1-2	4	18	0.5 (Torvane Shear)	
SB3	20	2-3-3-5	8	20	(Hand Pene.)	
SB4	18	5-5-3-3	10	22		
SB5	18	2-3-5	10	16	(Hand Pene.)	
SB6	18	7-13-6	25	13	4.5+ (Hand Pene.)	
SB7	18	8-11-11	29	14	4.5+ (Hand Pene.)	

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▽ DURING BORING:	19.0	742.0
▼ AT END OF BORING:	None	

BACKFILL METHOD: Auger Cuttings capped with EPCO Hole Plug

NOTES: 1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

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SME

BORING B 5/ TP 5

BORING DEPTH: 25 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence **PROJECT NUMBER:** 100047.00

CLIENT: Detroit Regional Partnership **PROJECT LOCATION:** Burton, Michigan

DATE STARTED: 8/4/25 **COMPLETED:** 8/4/25 **BORING METHOD:** Hollow-stem Auger

DRILLER: WN **RIG NO.:** 552 (CME 55) **LOGGED BY:** RP **CHECKED BY:** KPA

LATITUDE: 43.02739
LONGITUDE: -83.62641
ELEVATION: 760.5 FT

HAMMER EFFICIENCY: 78%
DATE: 5/13/2025
N_{60} -- O

DRY DENSITY (pcf) -- ■ (90, 100, 110, 120)
MOISTURE & ATTERBERG LIMITS (%) (PL, MC, LL) (10, 20, 30, 40)

SHEAR STRENGTH (KSF) (1, 2, 3, 4): HAND PENE.; TORVANE SHEAR; UNC. COMP.; VANE SHEAR (PK); VANE SHEAR (REM); TRIAXIAL (UU)

PROFILE DESCRIPTION

Depth (ft)	Elevation (ft)	Profile Description
0 – 1.5	760.5 – 759.0	FILL- Sandy TOPSOIL- Some Gravel- Black- Moist
1.5 – 2.5	759.0 – 758.0	FILL- Fine SILTY SAND with Slag, & Coal Ash- Black- Moist- Medium Dense (SM)
2.5 – 3.0	758.0 – 757.5	FILL- Fine SILTY SAND with Spark Plugs- Gray- Moist- Medium Dense (SM)
3.0 – 6.5	757.5 – 754.0	Fine to Medium SAND- Some Silt & Clay- Brown- Moist- Medium Dense (SP)
6.5 – 7.0	754.0 – 753.5	Fine SILTY SAND- Gray- Moist- Medium Dense (SM)
7.0 – 9.5	753.5 – 751.0	Sandy SILT- Some Clay- Gray- Moist- Very Loose (ML)
9.5 – 18.5	751.0 – 742.0	Silty CLAY- Some Sand- Gray- Soft to Stiff (CL)
18.5 – 25.0	742.0 – 735.5	Fine SILTY SAND- Occasional Clay Seams- Gray- Wet- Loose to Medium Dense (SM)

END OF BORING AT 25.0 FEET.

Sample Type/No.	Recovery Length (inches)	SPT Blows per Six Inches	N_{60}	Moisture Content (%)	Shear Strength (KSF)	Remarks
SB1	14	8-10-7-8	22			
SB2	20	5-7-8-7	20			
SB3	19	6-7-7-8	18			
SB4	17	1-1-1-2	3	22, 19	0.5 (Torvane)	
SB5	17	2-2-2	5	22	Hand Pene.	
SB6	10	2-3-2	7			
SB7	16	5-9-12	27			

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
DURING BORING:	13.5	747.0
AT END OF BORING:	None	

BACKFILL METHOD: Auger Cuttings capped with EPCO Hole Plug

NOTES:
1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.
3. Boring log was updated with information from the test pit excavation, which was performed at the same location as the boring.

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SME

BORING B 6 / TP 6

BORING DEPTH: 25 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence **PROJECT NUMBER:** 100047.00

CLIENT: Detroit Regional Partnership **PROJECT LOCATION:** Burton, Michigan

DATE STARTED: 8/4/25 **COMPLETED:** 8/4/25 **BORING METHOD:** Hollow-stem Auger/Solid-stem Augers

DRILLER: WN **RIG NO.:** 552 (CME 55) **LOGGED BY:** RP **CHECKED BY:** KPA

LATITUDE: 43.02682
LONGITUDE: -83.62542
ELEVATION: 763 FT

HAMMER EFFICIENCY: 78%
DATE: 5/13/2025
N_{60} -- O

DRY DENSITY (pcf) -- ■ (90, 100, 110, 120)

MOISTURE & ATTERBERG LIMITS (%): PL, MC, LL (10, 20, 30, 40)

SHEAR STRENGTH (KSF) (1, 2, 3, 4): ▼ HAND PENE.; ☒ TORVANE SHEAR; ● UNC. COMP.; ⊡ VANE SHEAR (PK); ✕ VANE SHEAR (REM); ⊕ TRIAXIAL (UU)

PROFILE DESCRIPTION

Depth (ft)	Profile Description	Elevation (ft)
0 – 1.0	12 inches of Clayey TOPSOIL	762.0
1.0 – 2.0	FILL- Fine SAND with Silt- Trace Concrete Debris- Brown- Moist- Medium Dense (SP-SM)	761.0
2.0 – 6.5	FILL- Fine to Medium SILTY SAND with Concrete, Brick, Masonry Block, & Spark Plug Debris- Black, White, & Gray- Moist- Very Loose to Extremely Dense (SM)	756.5
6.5 – 8.0	FILL- Plastic & Concrete	755.0
8.0 – 13.5	Fine SILTY SAND- Occasional Clay Seams- Brown- Moist- Very Loose (SM)	749.5
13.5 – 18.5	LEAN CLAY- Gray- Medium (CL)	744.5
18.5 – 24.0	Sandy LEAN CLAY- Gray- Very Stiff (CL)	739.0
24.0 – 25.0	Clayey SILT with Sand- Gray- Wet- Medium Dense (ML)	738.0

END OF BORING AT 25.0 FEET.

Sample Type/No.	Recovery Length (inches)	SPT Blows per Six Inches	N_{60}	MC (%)	Shear Strength (KSF)	Remarks
SB1	18	4-5-6-18	14			
NR2	0	15-21-50	92			
SB3	18	1-1-1-6	3	9	4.5+ (Hand Pene.)	
SB4	18	2-1-2-4	4			
SB5	18	1-2-3	7	25	0.75 (Torvane Shear)	
SB6	16	2-2-3	7	20	▼ (Hand Pene.)	
SB7	18	9-10-12	29	18	4.5+ (Hand Pene.)	

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▽ DURING BORING:	9.0	754.0
▼ AT END OF BORING:	None	

BACKFILL METHOD: Auger Cuttings capped with EPCO Hole Plug

NOTES:
1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.
3. Boring log was updated with information from the test pit excavation, which was performed at the same location as the boring.

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BORING TP 7

BORING DEPTH: 4 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence **PROJECT NUMBER:** 100047.00

CLIENT: Detroit Regional Partnership **PROJECT LOCATION:** Burton, Michigan

DATE STARTED: 8/20/25 **COMPLETED:** 8/20/25 **BORING METHOD:** Test Pit

FIELD REPRESENTATIVE: KPA **EQUIPMENT:** CAT Excavator **LOGGED BY:** KPA **CHECKED BY:** LMJ

DEPTH (FEET)	SYMBOLIC PROFILE	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	BLOWS PER SIX INCHES	DYNAMIC CONE PENETROMETER (DCP) -- ○ (10 20 30 40)	DRY DENSITY (pcf) -- ■ (90 100 110 120) / MOISTURE & ATTERBERG LIMITS (%) PL MC LL (10 20 30 40)	HAND PENE. / TORVANE SHEAR / UNC. COMP. / VANE SHEAR (PK) / VANE SHEAR (REM) / TRIAXIAL (UU) / SHEAR STRENGTH (KSF) (1 2 3 4)	REMARKS
0 – 3.5		FILL- Fine to Medium SILTY SAND with Brick Debris and Spark Plugs - Black, Gray, and White- Moist (SM)						
3.5 – 4.0		Fine SILTY SAND- Brown and Gray- Moist (SM)						
4.0		END OF BORING AT 4.0 FEET.						

Depth scale: 0, 5, 10, 15, 20, 25, 30

GROUNDWATER & BACKFILL INFORMATION

GROUNDWATER WAS NOT ENCOUNTERED

BACKFILL METHOD: Excavation Spoils

NOTES: 1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

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SME

BORING TP 8

BORING DEPTH: 12.5 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence **PROJECT NUMBER:** 100047.00

CLIENT: Detroit Regional Partnership **PROJECT LOCATION:** Burton, Michigan

DATE STARTED: 8/20/25 **COMPLETED:** 8/20/25 **BORING METHOD:** Test Pit

FIELD REPRESENTATIVE: KPA **EQUIPMENT:** CAT Excavator **LOGGED BY:** KPA **CHECKED BY:** LMJ

DEPTH (FEET)	SYMBOLIC PROFILE	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	BLOWS PER SIX INCHES	DYNAMIC CONE PENETROMETER (DCP) -- ○ (10 20 30 40)	DRY DENSITY (pcf) -- ■ (90 100 110 120) / MOISTURE & ATTERBERG LIMITS (%) PL MC LL (10 20 30 40)	HAND PENE. / TORVANE SHEAR / UNC. COMP. / VANE SHEAR (PK) / VANE SHEAR (REM) / TRIAXIAL (UU) SHEAR STRENGTH (KSF) (1 2 3 4)	REMARKS
0–1.0	1.0	Crushed Concrete AGGREGATE						
1.0–12.5	12.5	FILL- Fine to Medium SILTY SAND with Brick, Concrete, Plastic, Shingles and Spark Plugs- Black, Gray, and White- Moist (SM)						
12.5		END OF BORING AT 12.5 FEET.						

Depth scale: 0, 5, 10, 15, 20, 25, 30

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)
∇ DURING BORING:	12.5
▼ AT END OF BORING:	12.5

BACKFILL METHOD: Excavation Spoils

NOTES: 1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

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SME

BORING TP 9

PAGE 1 OF 1

BORING DEPTH: 3 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence
PROJECT NUMBER: 100047.00
CLIENT: Detroit Regional Partnership
PROJECT LOCATION: Burton, Michigan
DATE STARTED: 8/20/25
COMPLETED: 8/20/25
BORING METHOD: Test Pit
FIELD REPRESENTATIVE: KPA
EQUIPMENT: CAT Excavator
LOGGED BY: KPA
CHECKED BY: LMJ

DEPTH (FEET)	SYMBOLIC PROFILE	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	BLOWS PER SIX INCHES	DYNAMIC CONE PENETROMETER (DCP) -- ○ (10 20 30 40)	DRY DENSITY (pcf) -- ■ (90 100 110 120); MOISTURE & ATTERBERG LIMITS (%) PL MC LL (10 20 30 40)	▼ HAND PENE. ☒ TORVANE SHEAR ● UNC. COMP. ⊡ VANE SHEAR (PK) × VANE SHEAR (REM) ◈ TRIAXIAL (UU) SHEAR STRENGTH (KSF) (1 2 3 4)	REMARKS
0		Crushed Concrete AGGREGATE						
	1.5	FILL- Fine to Medium SILTY SAND						
	2.0	with Spark Plugs- White- Moist (SM)						
	3.0	Fine Silty Sand- Brown- Moist (SM)						
		END OF BORING AT 3.0 FEET.						
5								
10								
15								
20								
25								
30								

GROUNDWATER & BACKFILL INFORMATION

GROUNDWATER WAS NOT ENCOUNTERED

BACKFILL METHOD: Excavation Spoils

NOTES: 1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

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SME

BORING TP10

BORING DEPTH: 10 FEET

PROJECT NAME: DRP/Davison Road, Burton/Stage 7 Due Diligence **PROJECT NUMBER:** 100047.00

CLIENT: Detroit Regional Partnership **PROJECT LOCATION:** Burton, Michigan

DATE STARTED: 8/20/25 **COMPLETED:** 8/20/25 **BORING METHOD:** Test Pit

FIELD REPRESENTATIVE: KPA **EQUIPMENT:** CAT Excavator **LOGGED BY:** KPA **CHECKED BY:** LMJ

DEPTH (FEET)	SYMBOLIC PROFILE	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	BLOWS PER SIX INCHES	DYNAMIC CONE PENETROMETER (DCP) -- O (10 20 30 40)	DRY DENSITY (pcf) -- ■ (90 100 110 120) / MOISTURE & ATTERBERG LIMITS (%) PL MC LL (10 20 30 40)	HAND PENE. / TORVANE SHEAR / UNC. COMP. / VANE SHEAR (PK) / VANE SHEAR (REM) / TRIAXIAL (UU) / SHEAR STRENGTH (KSF) (1 2 3 4)	REMARKS
0	1.0	Crushed Concrete AGGREGATE						
5	8.5	FILL- Fine to Medium SILTY SAND with Brick, Concrete, Metal Debris and Spark Plugs- Black and Brown- Moist (SM)						
10	10.0	Fine SILTY SAND- Gray- Moist (SM)						
		END OF BORING AT 10.0 FEET.						
15								
20								
25								
30								

GROUNDWATER & BACKFILL INFORMATION

GROUNDWATER WAS NOT ENCOUNTERED

BACKFILL METHOD: Excavation Spoils

NOTES: 1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

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PHOTO NO. 1: Test pit TP-5 with fill soils and spark plugs.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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PHOTO NO. 2: Test Pit TP-5 picture of excavated spoils.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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PHOTO NO. 3: Test Pit TP-6 partially excavated with concrete debris.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

PHOTO NO. 4: Test Pit TP-6 with large concrete debris.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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PHOTO NO. 5: Test Pit TP-6 excavated spoils.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

PHOTO NO. 6: Test Pit TP-6 picture of excavated test pit to nearly full depth.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

PHOTO NO. 7: Test Pit TP-7 general soil conditions.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

PHOTO NO. 8: Test Pit TP-8 general soil and fill conditions.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

PHOTO NO. 9: Test Pit TP-8 picture of excavated spoils.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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PHOTO NO. 10: Test Pit TP-9 general soil conditions.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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PHOTO NO. 11: Test Pit TP-10 general soil conditions.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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PHOTO NO. 12: Test Pit TP-10 excavated spoils.

SME Project No.:	100047.00
Photographs by:	Kyle P. Areaux, PE
Date:	August 26, 2025
Project:	DRP/ Davison/ Stage 7 Due Diligence
Location:	Burton, Michigan

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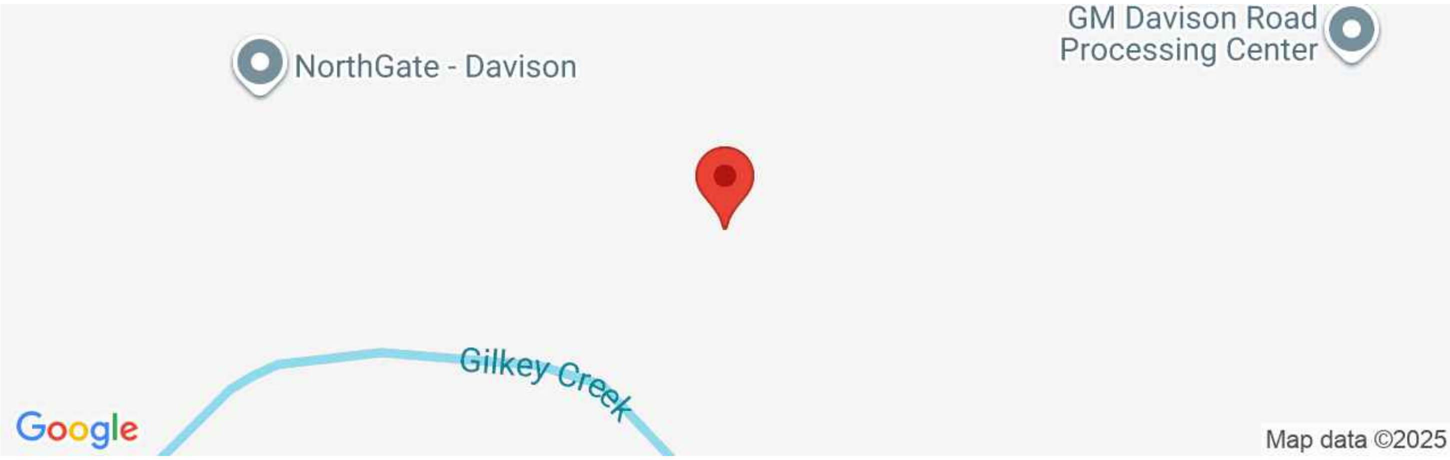
Announcement
ASCE 7-22 is now available.

Latitude, Longitude: 43.02913904, -83.62750729



Date	8/27/2025, 8:05:25 AM
Design Code Reference Document	ASCE7-22
Risk Category	II
Site Class	D

Type	Value	Description (Data)
S_S	0.089	The MCE_R spectral response acceleration at 0.2 seconds for Site Class BC, in units of g.
S_1	0.046	The MCE_R spectral response acceleration at 1 second for Site Class BC, in units of g.
S_{MS}	0.12	S_{MS} = 1.5 x S_{DS}, the Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration for short periods (of the two-period spectrum) and the user-specified Site Class.
S_{M1}	0.1	S_{M1} = 1.5 x S_{D1}, the MCE_R spectral response acceleration for 1 second (of the two-period spectrum) and the user-specified Site Class.
S_{DS}	0.079	The design spectral response acceleration for short periods (of the two-period spectrum) and the user-specified Site Class, in units of g.
S_{D1}	0.067	The design spectral response acceleration for 1 second (of the two-period spectrum) and the user-specified Site Class, in units of g

Type	Value	Description (Data Contd.)
SDC	A	Seismic design category
PGA_M	0.046	PGA_M, the Geometric-Mean Maximum Considered Earthquake (MCE_G) peak ground acceleration for the user-specified Site Class, in units of g
T_S	0.839	$T_S = S_{D1}/S_{DS}$, in seconds, for construction of the two-period design spectrum
T_0	0.168	T_0 = 0.2 x T_S, in seconds, for construction of the two-period design response spectrum
T_L	12	T_L, the long-period transition period, in seconds, for construction of the two-period design response spectrum

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Type	Value	Description (Underlying Data and Metadata)
PGA_{uh}	0.046	Probabilistic uniform-hazard (2%-in-50-years), geometric-mean peak ground acceleration, in units of g.
PGA_{84th}		Deterministic 84th-percentile, geometric-mean peak ground acceleration (without deterministic lower limit), in units of g.
V_{S30}	260	The shear-wave velocity used for the user-specified Site Class, in units of m/s
Spatial Interpolation Method	linearloglinear	Identifier for spatial interpolation method used to obtain values for location of interest from underlying gridded values: "linearloglinear" for bilinear of natural logarithm of values.
PGA_{dFloor}	0.53	Deterministic lower limit peak ground acceleration (PGA_G) for the user-specified Site Class, in units of g.
riskTargetedSpectrum		Probabilistic risk-targeted, maximum direction response spectrum (for 1%-in-50-years collapse risk)
eightyFourthSpectrum		Deterministic 84^{th}-percentile, maximum-direction response spectrum (without deterministic lower limit)

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

National Flood Hazard Layer FIRMette

FEMA

83°37'58"W 43°1'58"N

83°37'20"W 43°1'32"N

0 250 500 1,000 1,500 2,000 Feet

1:6,000

Basemap Imagery Source: USGS National Map 2023

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- B 20.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- 8 Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 8/25/2025 at 10:58 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

ERIS

HISTORICAL AERIALS

Project Property: Davidson Road Industrial Land
Davidson Road
Burton MI 48509

Project No: 100047.00.002

Requested By: SME-USA

Order No: 25060600551

Date Completed: June 12,2025

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Environmental Risk Information Services
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1.866.517.5204 | info@erisinfo.com | erisinfo.com

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

Date	Source	Scale	Comments
2022	United States Department of Agriculture	1" = 500'	
2020	United States Department of Agriculture	1" = 500'	
2018	United States Department of Agriculture	1" = 500'	
2016	United States Department of Agriculture	1" = 500'	
2014	United States Department of Agriculture	1" = 500'	Best Copy Available
2012	United States Department of Agriculture	1" = 500'	
2010	United States Department of Agriculture	1" = 500'	
2009	United States Department of Agriculture	1" = 500'	
2006	United States Department of Agriculture	1" = 500'	
2005	United States Department of Agriculture	1" = 500'	
1999	United States Geological Survey	1" = 500'	
1993	United States Geological Survey	1" = 500'	Best Copy Available
1988	United States Geological Survey	1" = 500'	
1982	United States Geological Survey	1" = 500'	
1975	United States Geological Survey	1" = 500'	
1973	United States Geological Survey	1" = 500'	Best Copy Available
1964	Agricultural Stabilization & Conserv. Service	1" = 500'	
1957	Agricultural Stabilization & Conserv. Service	1" = 500'	
1950	Agricultural Stabilization & Conserv. Service	1" = 500'	
1941	Agricultural Stabilization & Conserv. Service	1" = 500'	
1937	Agricultural Stabilization & Conserv. Service	1" = 500'	

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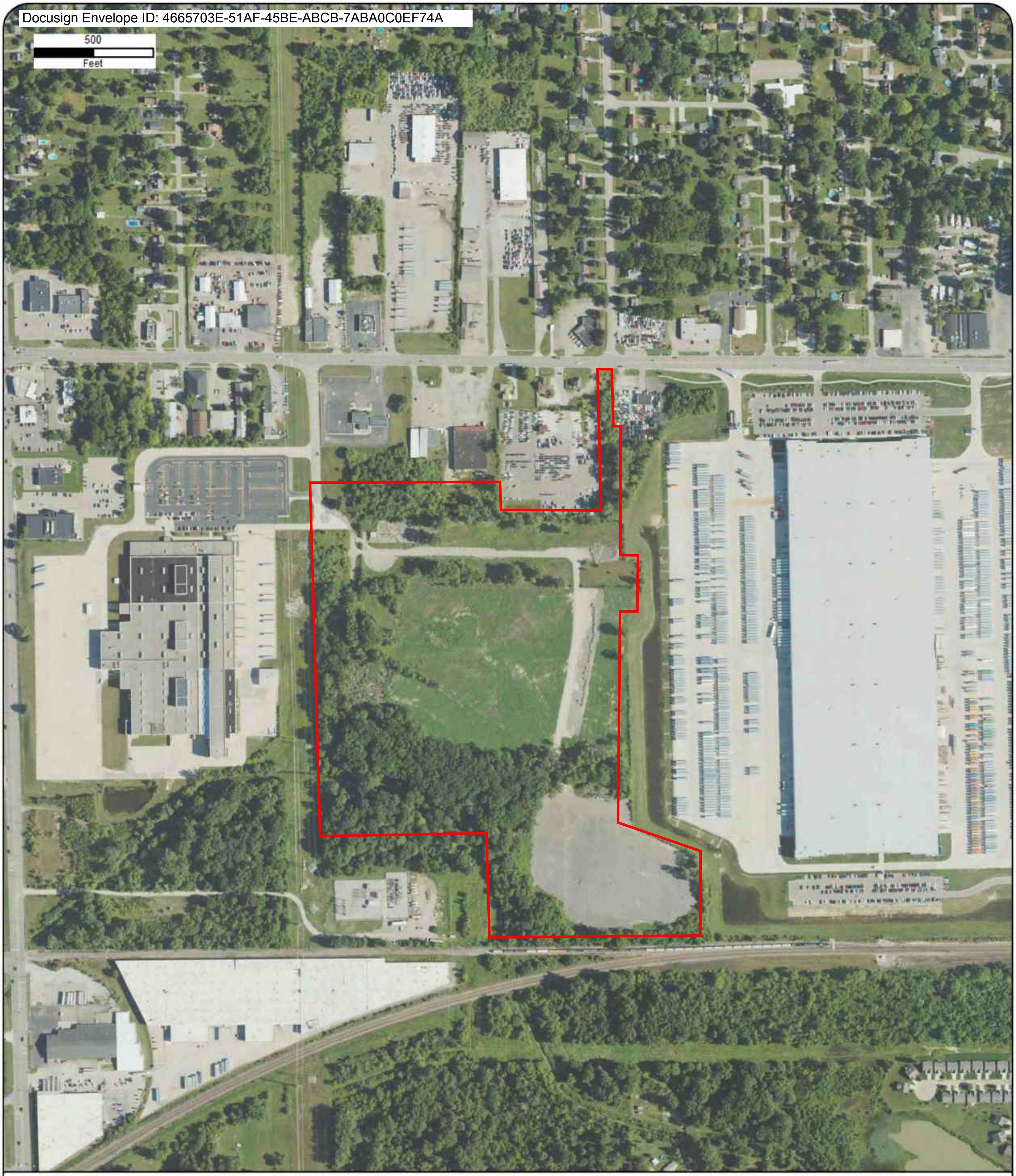


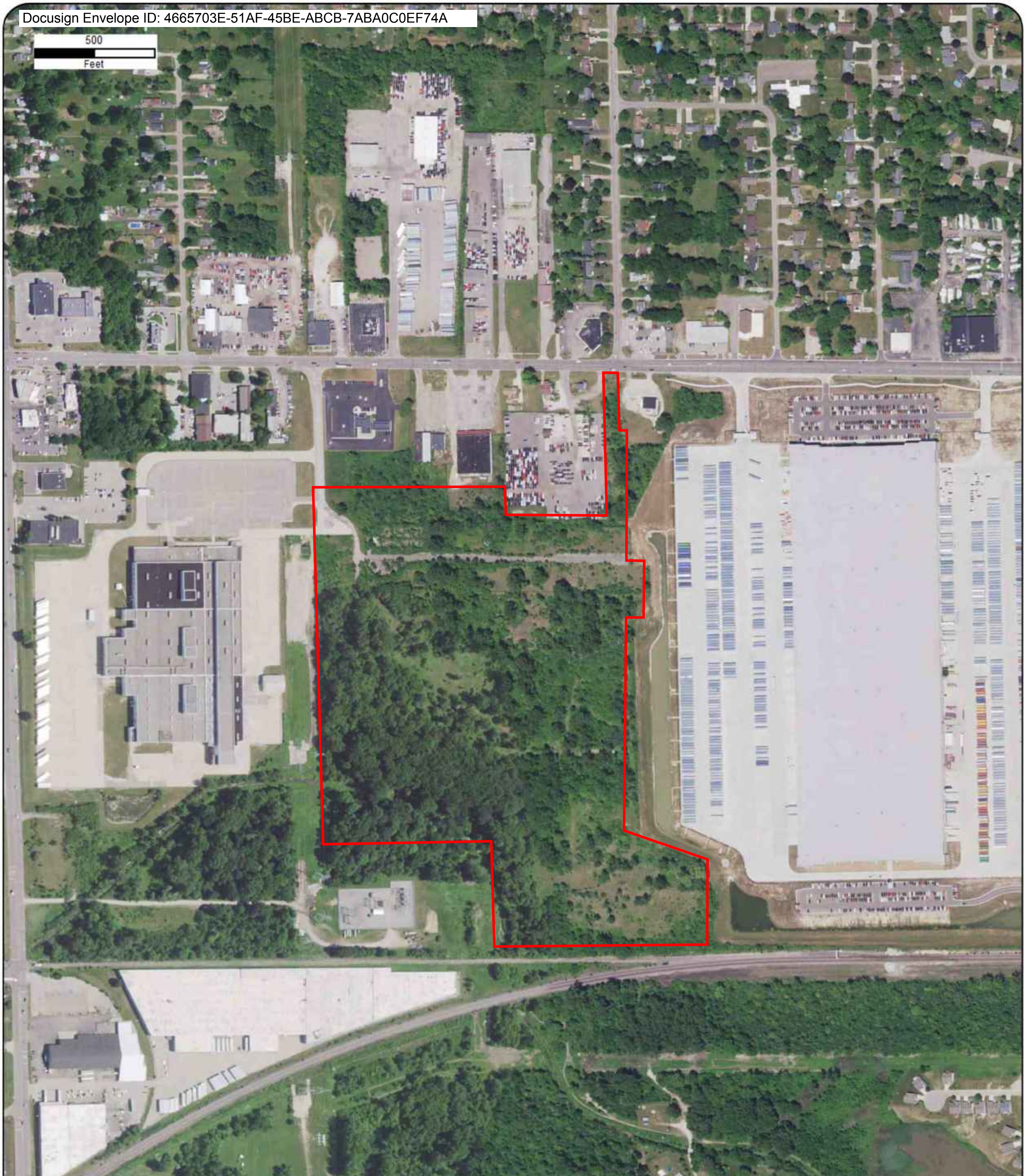
Year: 2022
Source: USDA
Scale: 1" = 500'
Comment:

Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

ERIS

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A



Year: 2020
Source: USDA
Scale: 1" = 500'
Comment:

Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

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Year: 2018
Source: USDA
Scale: 1" = 500'
Comment:

Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

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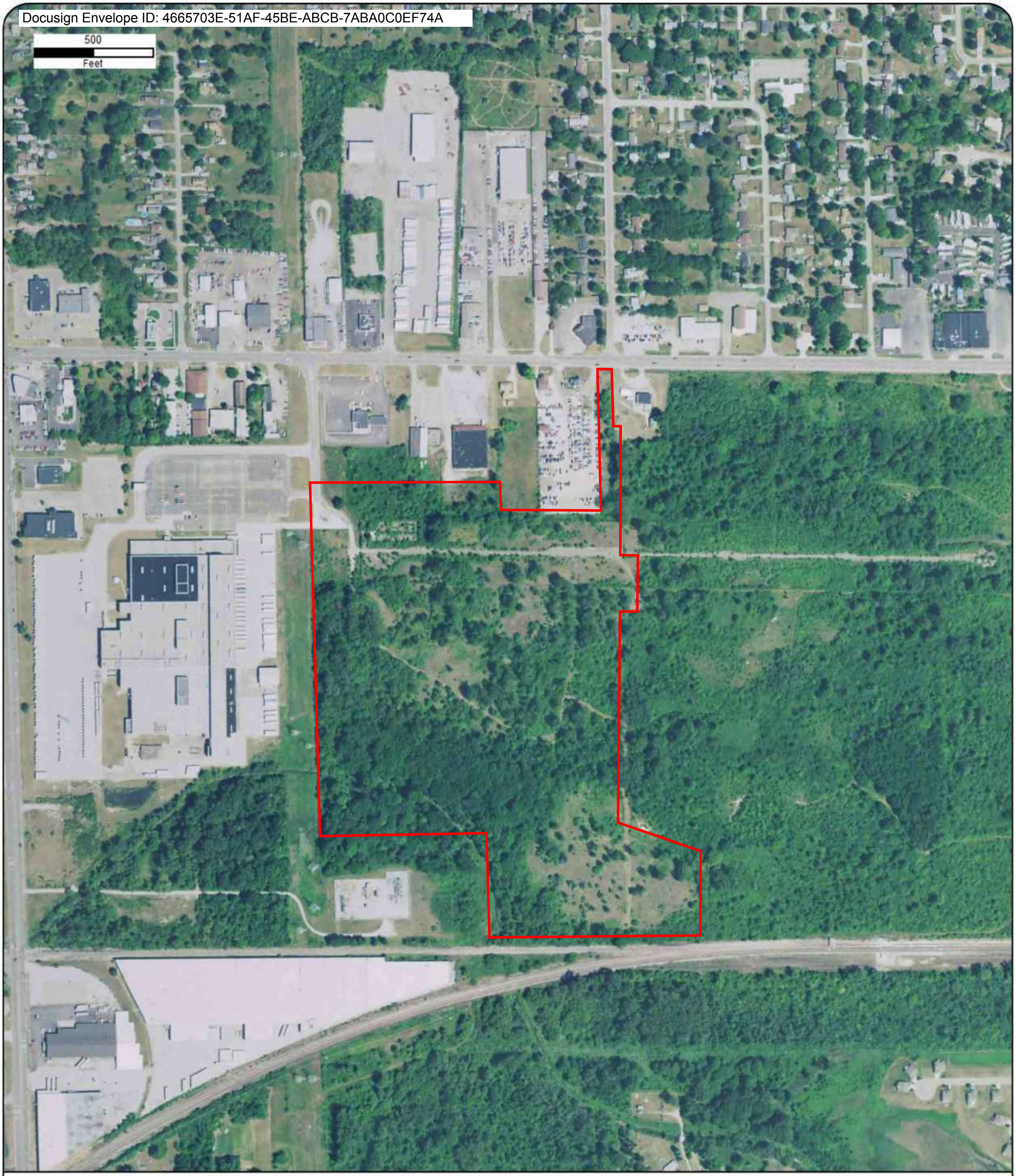

Year: 2016
Source: USDA
Scale: 1" = 500'
Comment:

Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

ERIS

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Year: 2014
Source: USDA
Scale: 1" = 500'
Comment: Best Copy Available

Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

ERIS

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

Year: 2012
Source: USDA
Scale: 1" = 500'
Comment:

Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

ERIS

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

Year: 2010
Source: USDA
Scale: 1" = 500'
Comment:

Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

ERIS

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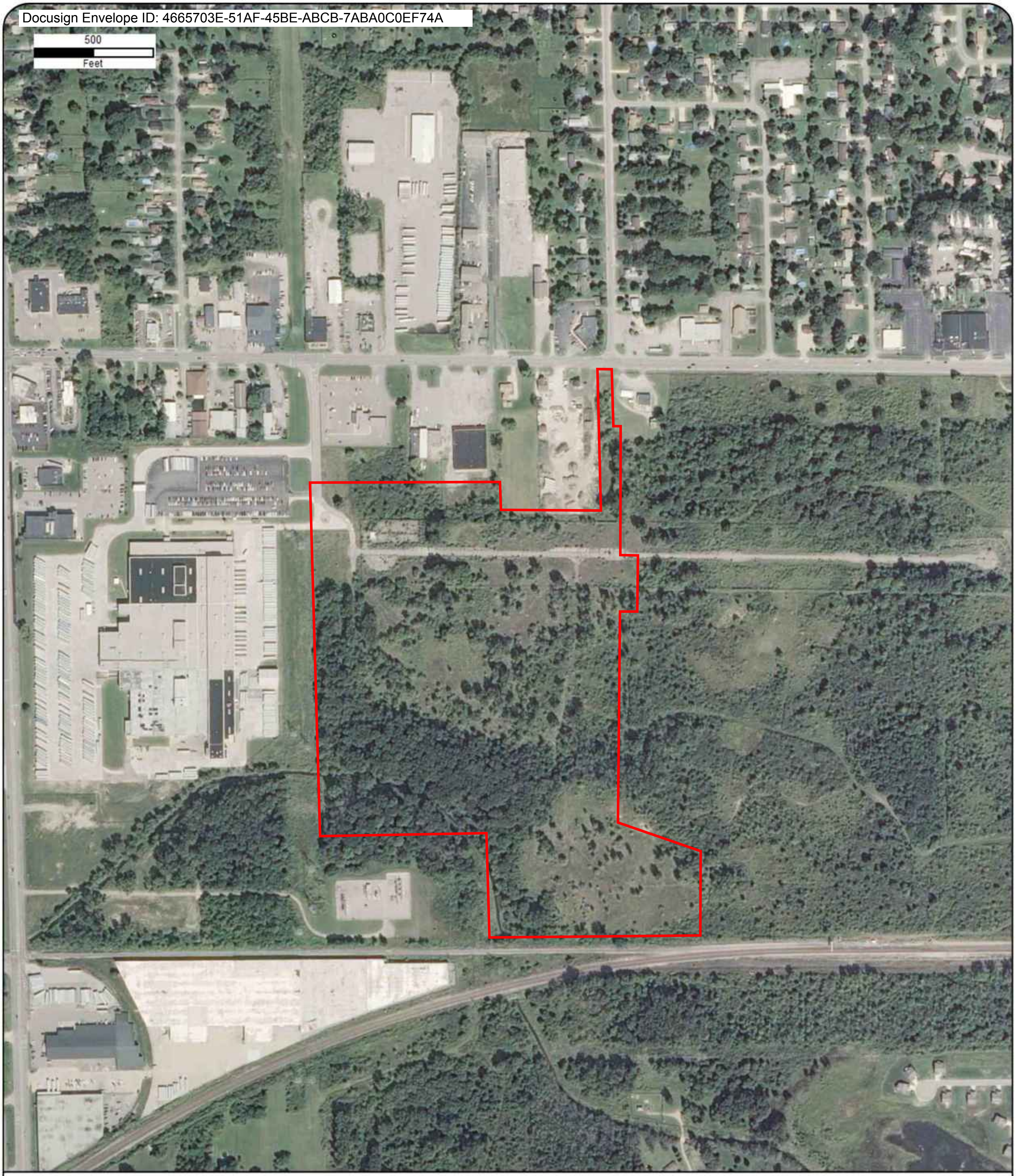

Year: 2009
Source: USDA
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Year: 2006
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Comment:

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Order No: 25060600551

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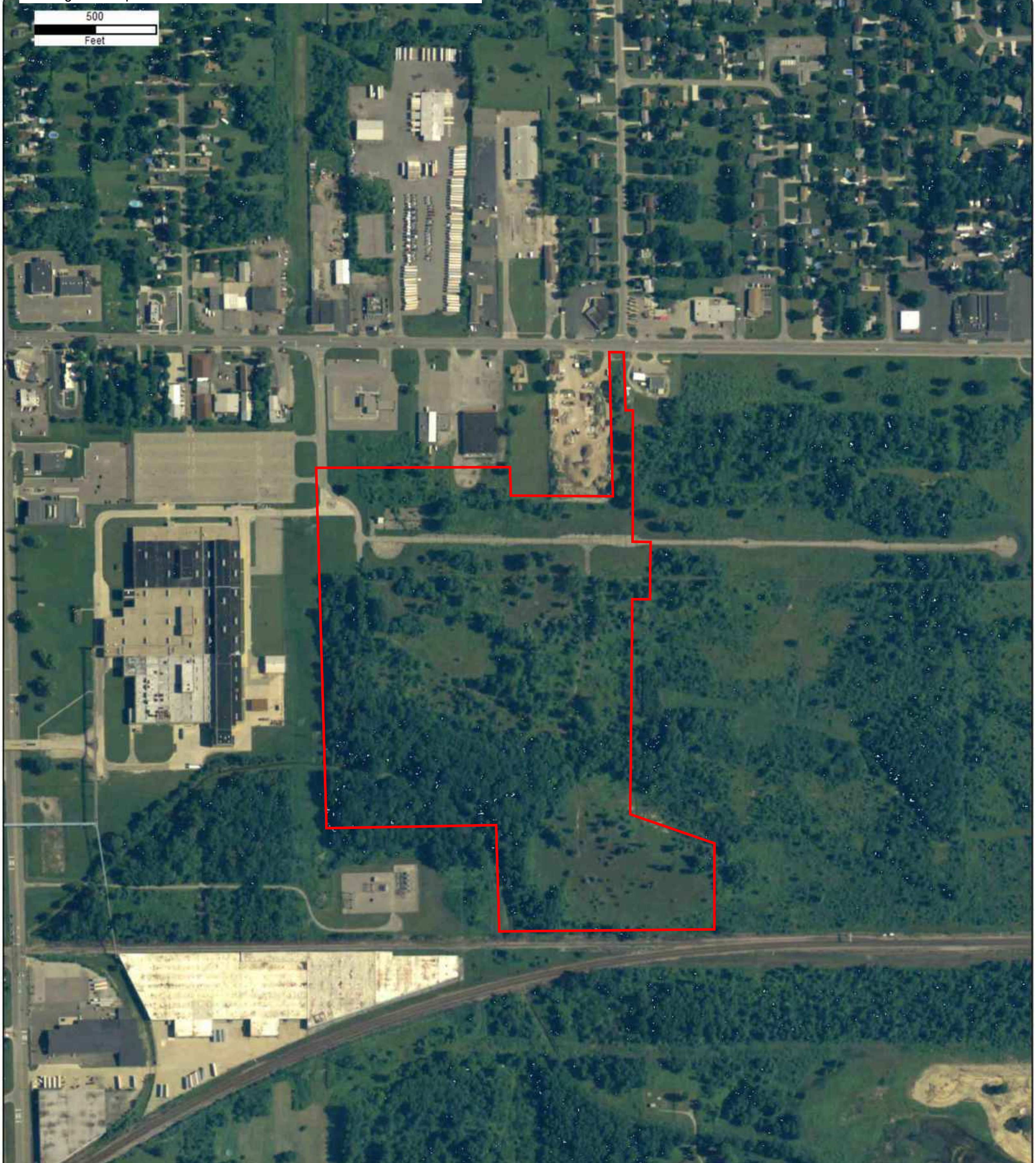

Year: 2005
Source: USDA
Scale: 1" = 500'
Comment:

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Order No: 25060600551

ERIS

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Year: 1999
Source: USGS
Scale: 1" = 500'
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Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

Year: 1993
Source: USGS
Scale: 1" = 500'
Comment: Best Copy Available

Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

ERIS

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

Year: 1988
Source: USGS
Scale: 1" = 500'
Comment:

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ERIS

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Year: 1982
Source: USGS
Scale: 1" = 500'
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Approx Center: -83.62750729,43.02913904

Order No: 25060600551

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Year: 1975
Source: USGS
Scale: 1" = 500'
Comment:

Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

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Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

Year: 1973

Source: USGS

Scale: 1" = 500'

Comment: Best Copy Available

Address: Davidson Road, Burton, MI

Approx Center: -83.62750729,43.02913904

Order No: 25060600551

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Year: 1964
Source: ASCS
Scale: 1" = 500'
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Year: 1957
Source: ASCS
Scale: 1" = 500'
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Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

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Year: 1950
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Scale: 1" = 500'
Comment:

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Approx Center: -83.62750729,43.02913904

Order No: 25060600551

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

Year: 1941
Source: ASCS
Scale: 1" = 500'
Comment:

Address: Davidson Road, Burton, MI
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Order No: 25060600551

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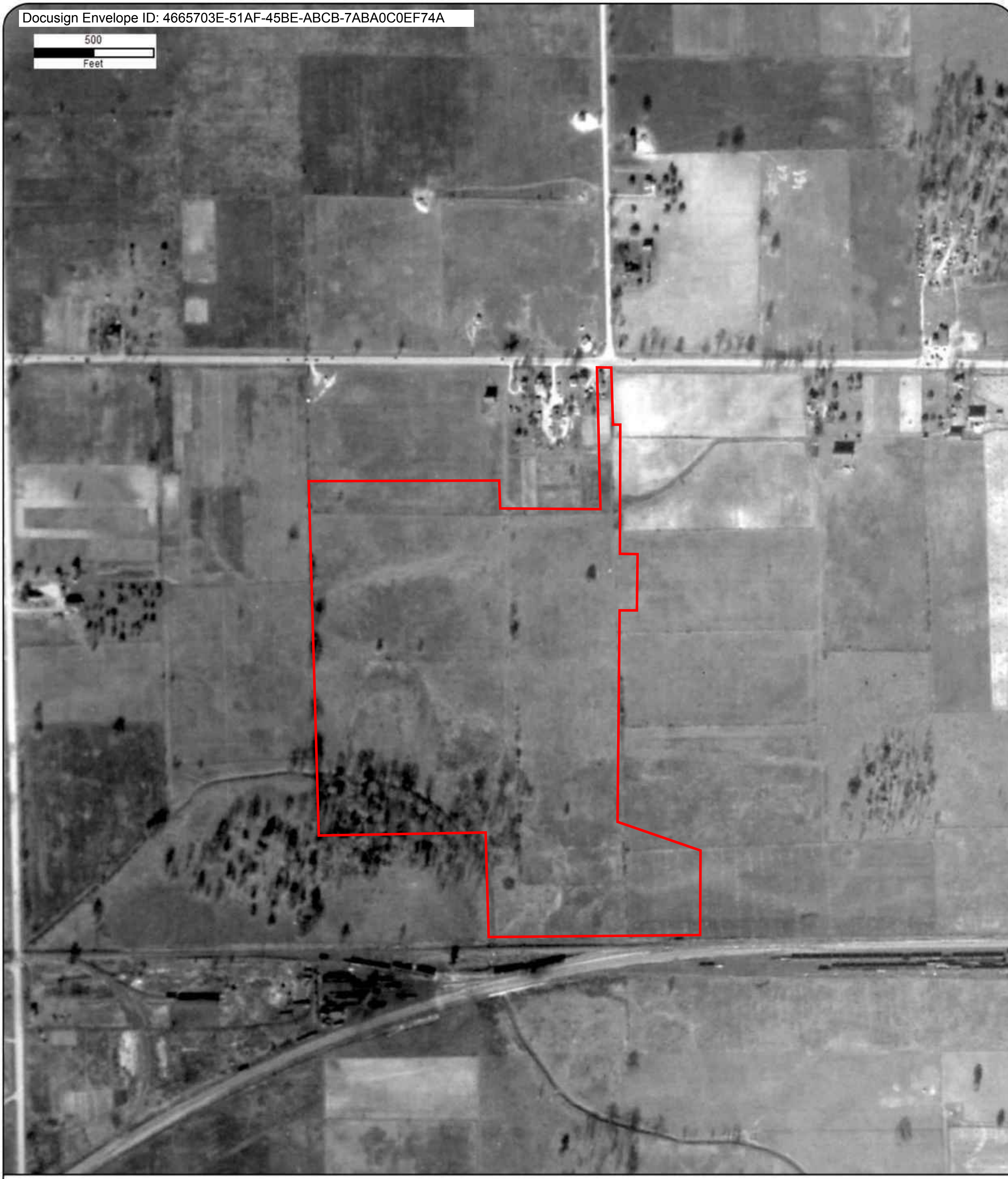

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Source: ASCS
Scale: 1" = 500'
Comment:

Address: Davidson Road, Burton, MI
Approx Center: -83.62750729,43.02913904

Order No: 25060600551

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Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

ERIS

TOPOGRAPHIC MAPS

Project Property:	Davidson Road Industrial Land
	Davidson Road Burton MI 48509
Project No:	100047.00.002
Requested By:	SME-USA
Order No:	25060600551
Date Completed:	June 08, 2025

Environmental Risk Information Services
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We have searched USGS collections of current topographic maps and historical topographic maps for the project property. Below is a list of maps found for the project property and adjacent area. Maps are from 7.5 and 15 minute topographic map series, if available.

Year	Map Series
2019	7.5
2017	7.5
2014	7.5
1975	7.5
1969	7.5
1943	15
1922	15
1920	15

Topographic Map Symbology for the maps may be available in the following documents:

Pre-1947

- Page 223 of 1918 Topographic Instructions
- Page 130 of 1928 Topographic Instructions

1947-2009

- Topographic Map Symbols

2009-present

- US Topo Map Symbols

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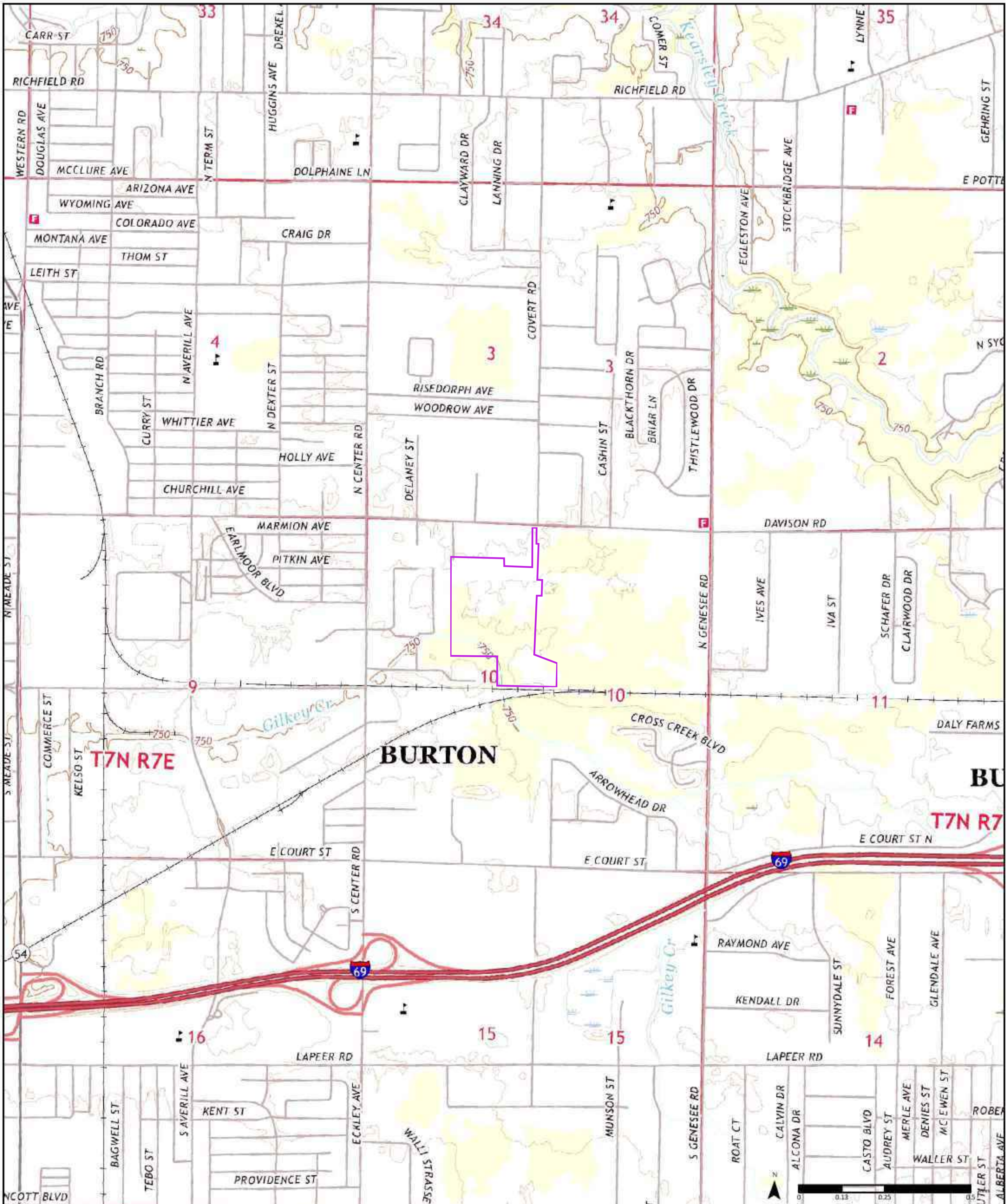

2019

Order No. 25060600551

Available Quadrangle(s): Flint North, MI
Davison, MI

Source: USGS 7.5 Minute Topographic Map

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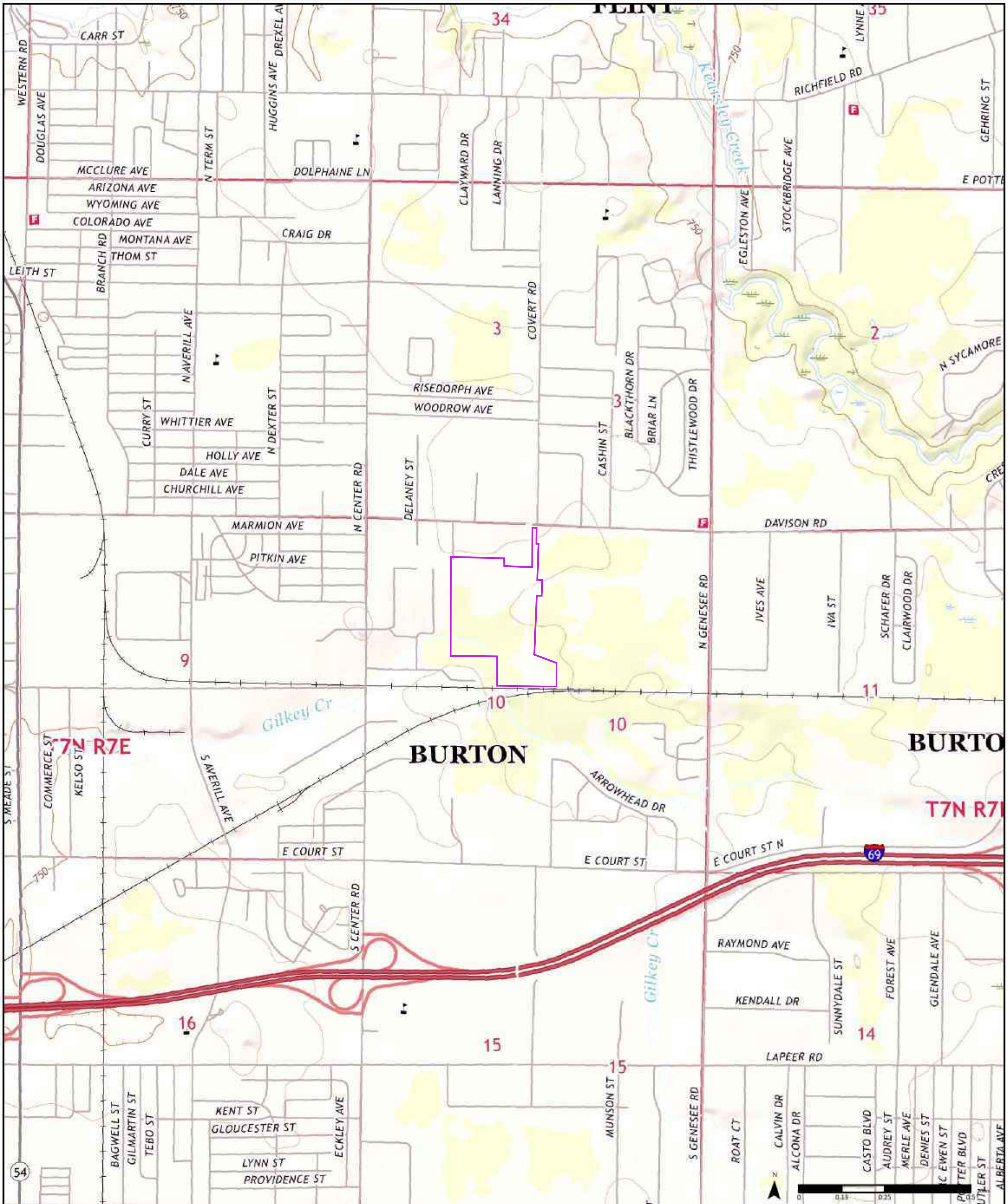

2017

Order No. 25060600551

Available Quadrangle(s): Flint North, MI
Davison, MI

Source: USGS 7.5 Minute Topographic Map

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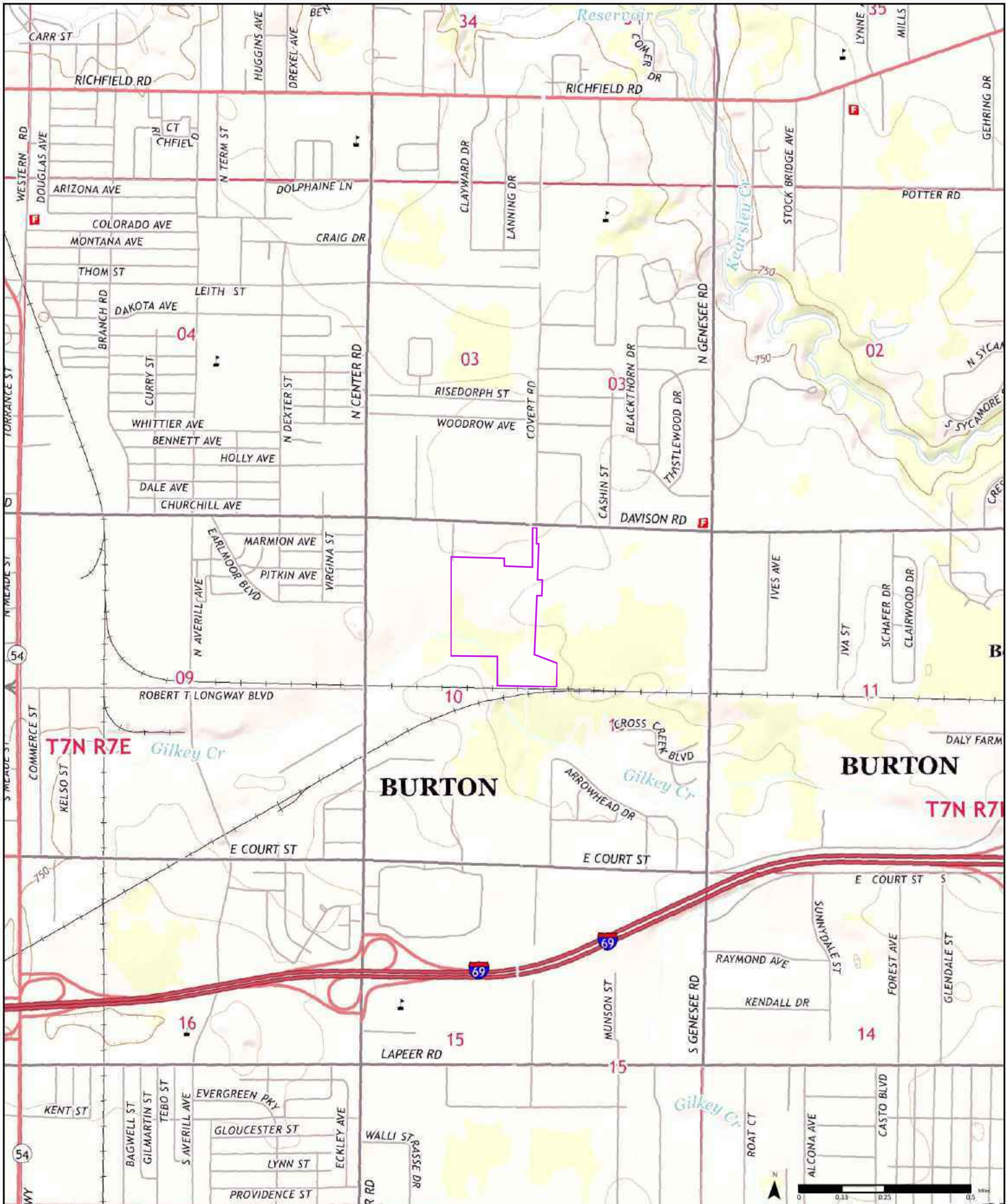

2014

Order No. 25060600551

Available Quadrangle(s): Flint North, MI
Davison, MI

ERIS

Source: USGS 7.5 Minute Topographic Map

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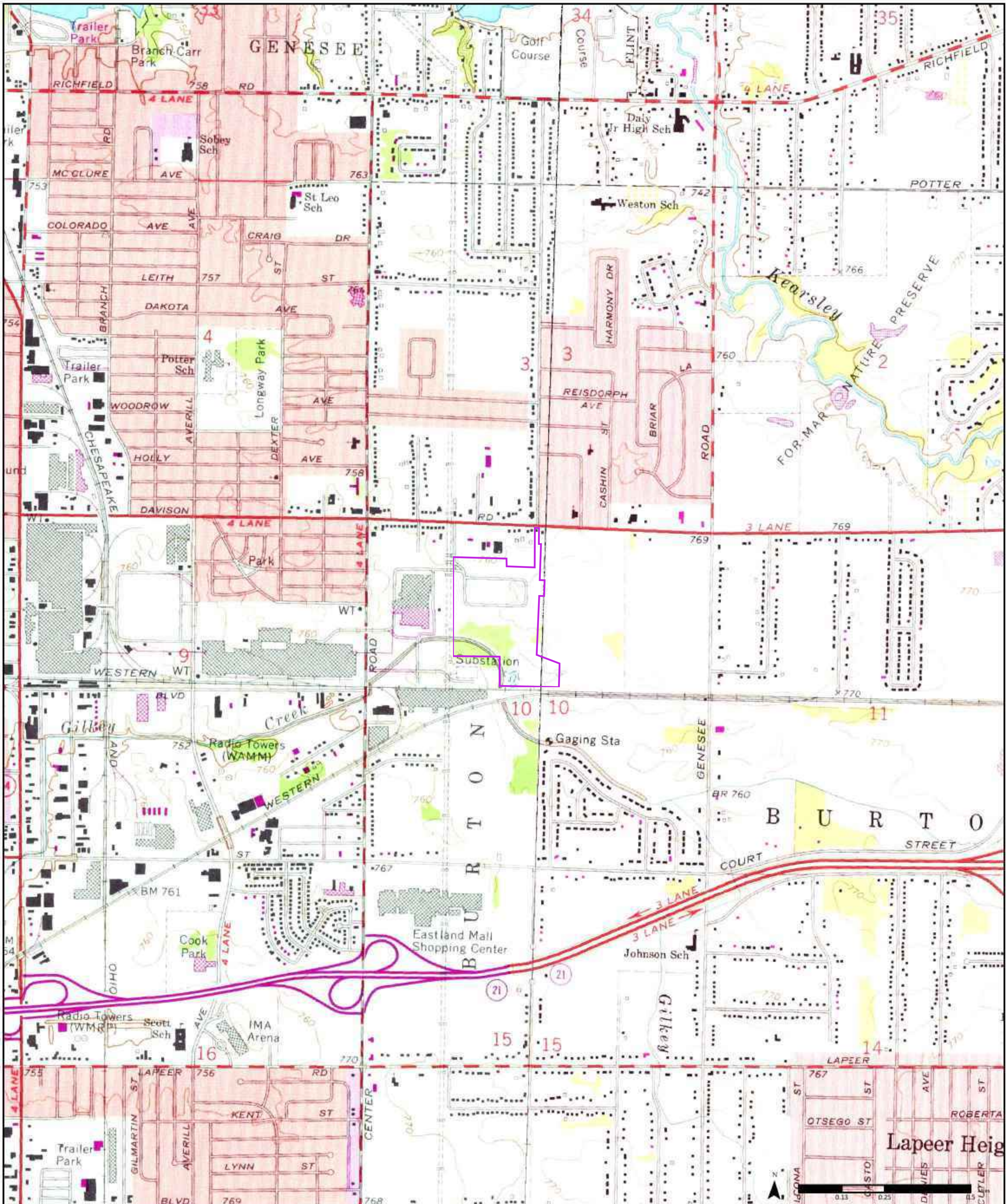


1975

(1-1975)
Aerial Photo Year: 1975
Photo Revision Year: 1975

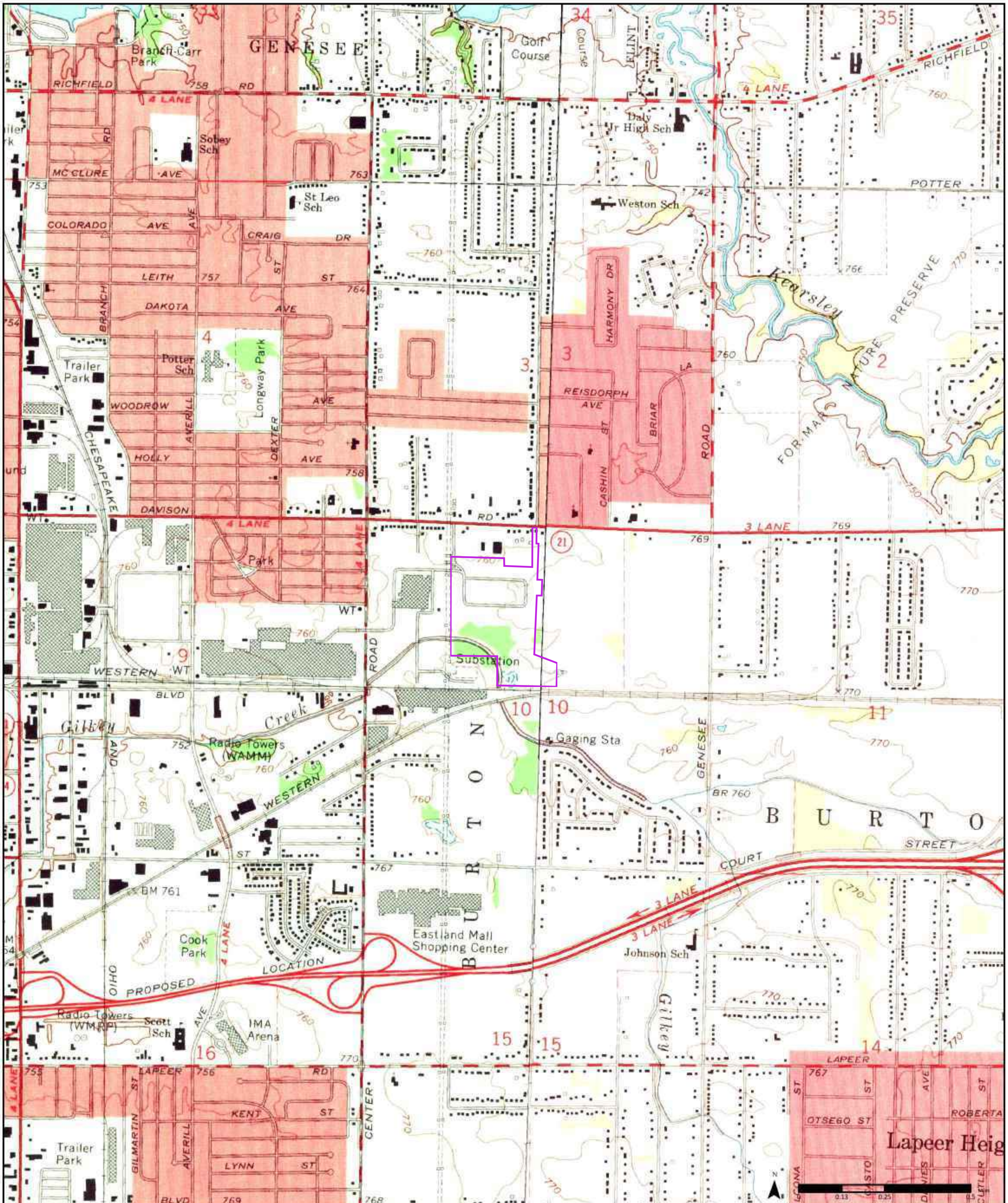
(2-1975)
Aerial Photo Year: 1975
Photo Revision Year: 1975

Order No. 25060600551

Available Quadrangle(s): Flint North, MI(1-1975)
Davison, MI(2-1975)

ERIS

Source: USGS 7.5 Minute Topographic Map



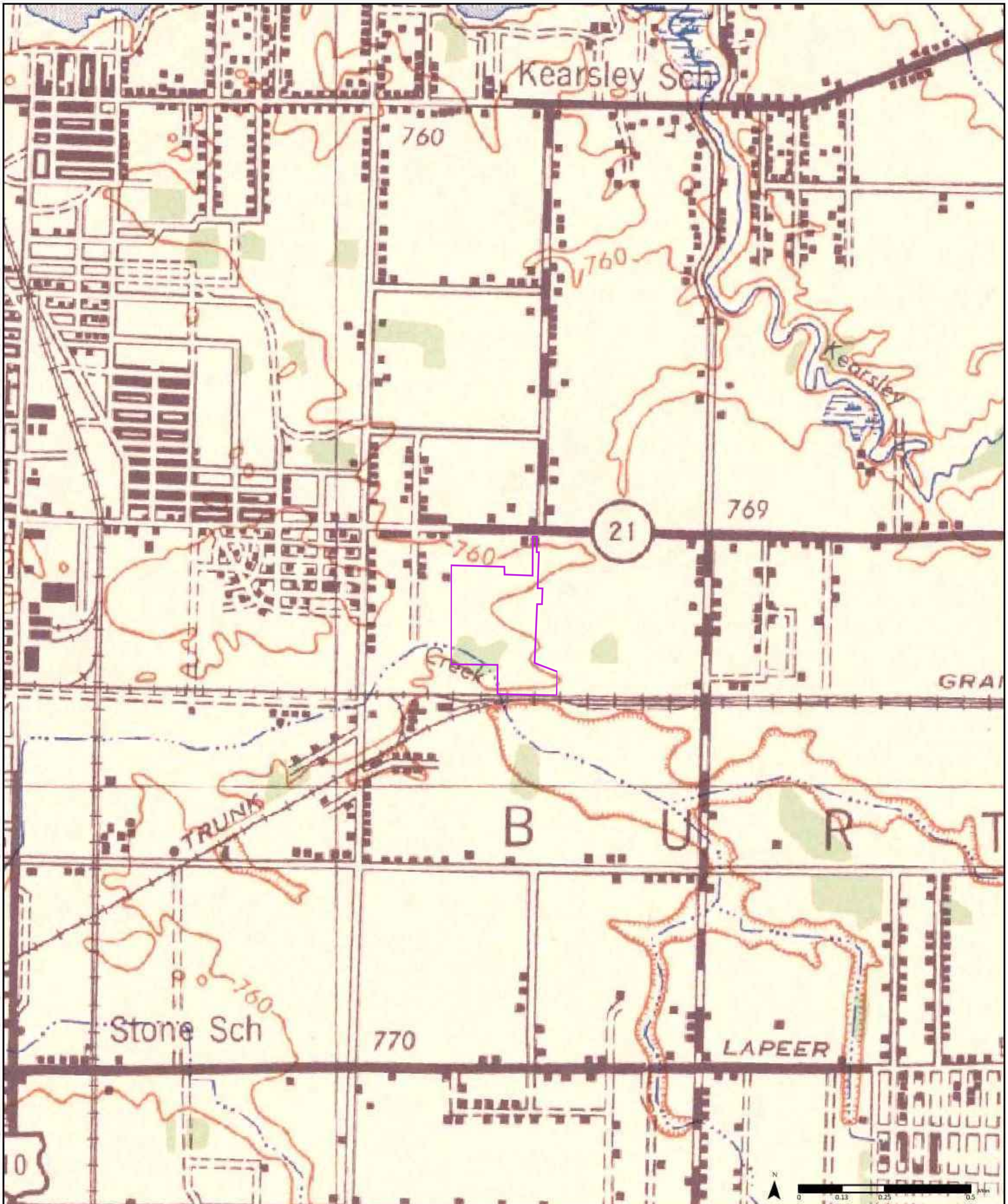
1969 [(1-1969)] Aerial Photo Year: 1966 [(2-1969)] Aerial Photo Year: 1967

Order No. 25060600551

Available Quadrangle(s): Flint North, MI(1-1969)
Davison, MI(2-1969)

Source: USGS 7.5 Minute Topographic Map

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A



Order No. 25060600551

1943 (1-1943)
Aerial Photo Year: 1941

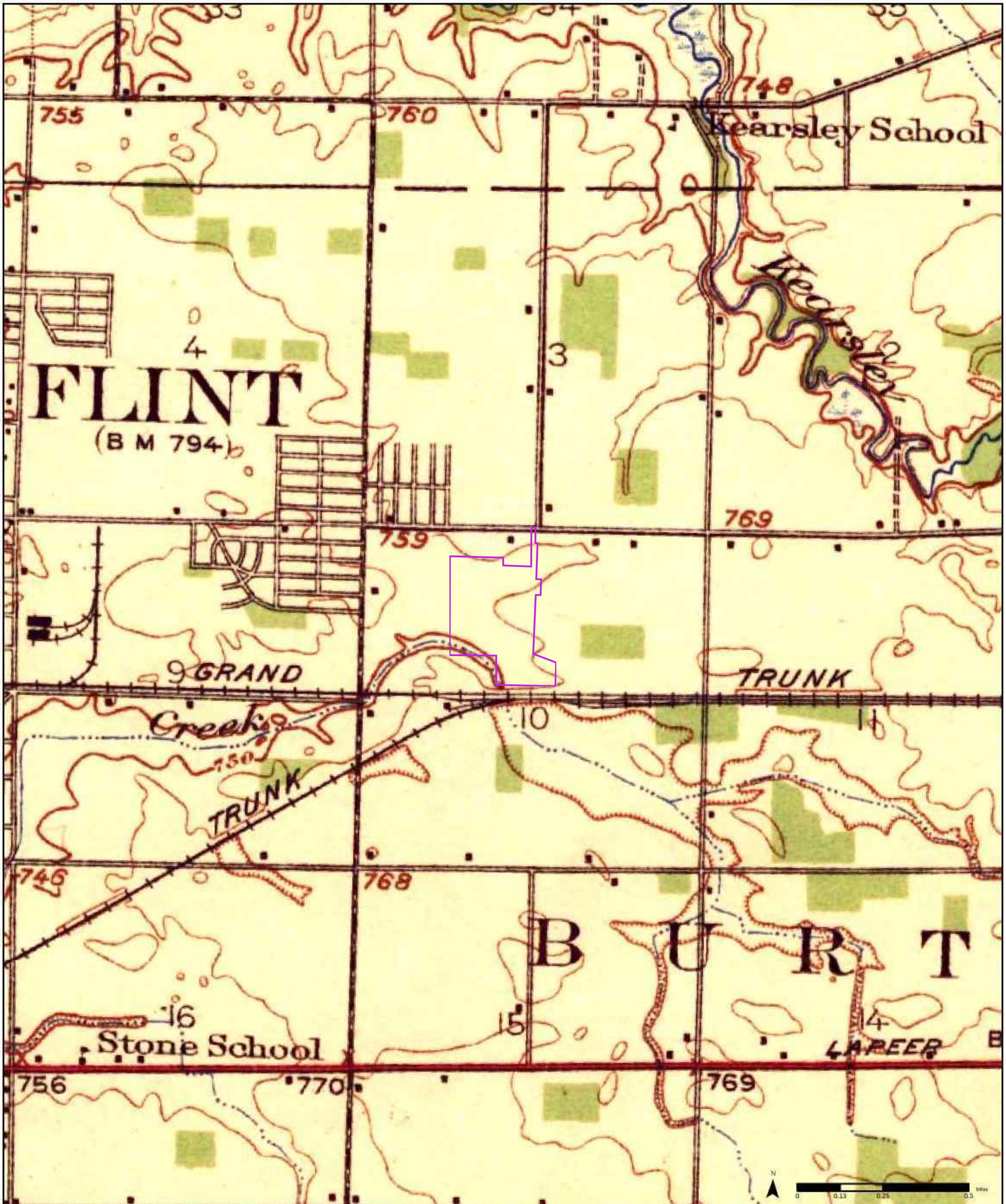
Flint

Available Quadrangle(s): Flint, MI(1-1943)

ERIS

Source: USGS 15 Minute Topographic Map

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

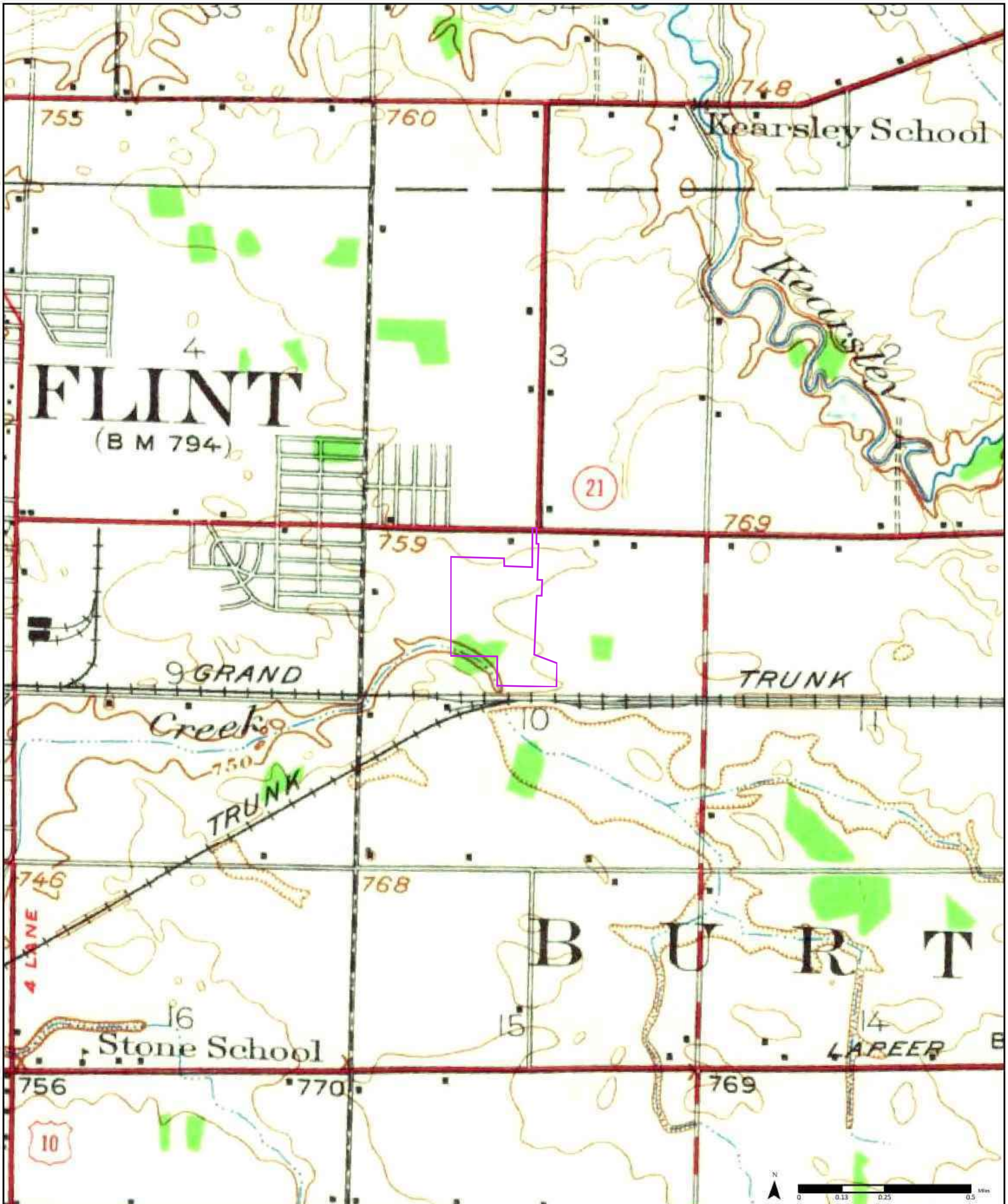


1922

Order No. 25060600551

Available Quadrangle(s): Flint, MI

Source: USGS 15 Minute Topographic Map



1920

Order No. 25060600551

Flint

Available Quadrangle(s): Flint, MI

Source: USGS 15 Minute Topographic Map

Docusign Envelope ID: 4665703E-51AF-45BE-ABCB-7ABA0C0EF74A

APPENDIX B

FIELD TESTING PROCEDURES
LABORATORY TESTING PROCEDURES
LIMITATIONS PERTAINING TO SUBSURFACE CONDITIONS

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FIELD TESTING PROCEDURES

STANDARD PENETRATION TESTS

The Standard Penetration Tests (SPT) generally follow the American Standard Test Method (ASTM) D-1586 "Standard Test Method for SPT and Split-Barrel Sampling of Soils". It is typically performed using a 2-inch O.D. split-spoon sampler, which is driven to obtain samples at selected intervals. The number of blows of a 140-pound hammer dropping 30 inches is recorded for each of three or four, 6-inch penetration intervals for an 18 or 24-inch drive at each sample location. The sum of blow counts for the second and third 6-inch penetration intervals equals the raw (uncorrected) N-value for a given sample interval (i.e. 5-4-2, N = 6). We periodically calibrate SME's drill rig auto-hammers. The hammer efficiency determined from this calibration is used to calculate the corrected N-values (N_{60}), as reported on the logs. When sampling in rock or hard soil, where a penetration of 6 inches or less was obtained for 50 hammer blows, the actual blow count and depth of penetration in inches for that interval is recorded (i.e. 50/2"). When sampling in very loose or very soft soil, where a penetration of more than 6 inches is obtained for a single hammer blow, the actual depth of penetration for that hammer blow is recorded (i.e. 1-0-0). Where the sampling equipment advanced under its own weight, "WOH" (weight of hammer) and the corresponding penetration depth are shown on the boring logs.

INFILTRATION TESTS

In-situ infiltration tests generally follow the double-ring infiltrometer field test procedures outlined in Appendix E in the Low Impact Development (LID) Manual for Michigan (dated 2008) prepared by the Southeast Michigan Council of Governments (SEMCOG). This procedure is also referenced in the States of Indiana, Ohio, and Kentucky applicable local manuals. The double-ring infiltrometer field test set-up consists of performing a boring or test pit to the test depth, installing an outer 6-inch-diameter standpipe and an inner 4-inch-diameter standpipe, and then driving the standpipes a suitable distance, per the referenced manual, below the bottom of the test depth. Soil is pre-soaked with approximately 12 inches of water for approximately one hour. The water drop rate per the last 30 minutes of the soaking period determines the subsequent interval for infiltration readings (i.e., 10- or 30-minute intervals). After completing the soaking period, standpipes are filled with water to a height of approximately 12 inches above the test depth, and water level changes in the standpipes are measured with a water level measuring tape with markings every 0.01 feet and recorded after the time intervals. This procedure is repeated until a minimum of four consecutive height changes within 1/4-inch of one another are measured. The height drop that occurred during the final time interval or the average stabilized rate is used to calculate the infiltration rate. After completion of the double-ring infiltrometer field test, the standpipes are removed, and the test hole is backfilled.

DYNAMIC CONE PENETROMETER (DCP)

USACE TESTS

Dynamic Cone Penetrometer (DCP) testing designed by the U.S. Army Corps of Engineers (USACE) is conducted to estimate the California Bearing Ratio (CBR) of the subgrade materials and existing pavement sub-layers. The USACE DCP consists of a 5/8-inch-diameter steel rod with a steel cone attached to one end driven by a sliding dual mass hammer. The rate of penetration per blow is measured at selected penetration, or hammer drop, intervals. CBR is an index commonly used in pavement design that provides an indication of subgrade support characteristics. The Corps of Engineers developed relationships to estimate the CBR value from the results of the USACE DCP test.

SME DCP TESTS

SME Dynamic Cone Penetrometer (DCP) test consists of dropping a 10-pound slide hammer that falls 24 inches and drives a rod with a 1-1/8-inch conical tip into the subgrade. The number of hammer drops required to drive the cone penetrometer are recorded for each six-inch increment and are used to estimate the relative density of the granular soils. The DCP blow counts were used to estimate Standard Penetration Test resistances (N-values) commonly used in geotechnical evaluations, based on empirical correlations developed by SME.

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PRESSUREMETER TESTING

Pressuremeter testing in the field models the static loading characteristics of the soil and the resulting analyses based on pressuremeter test results are considered to be a more accurate indicator of the ultimate foundation bearing pressure, and associated settlement, than analyses using empirical correlations based on dynamic test methods, such as the Standard Penetration Test (SPT) and/or dynamic cone penetrometer (DCP) tests. Results of the SPT and/or DCP were recorded, at the pressuremeter test locations and depths to provide additional information on the relative density of the in-place subgrade, and to correlate the pressuremeter test results with data obtained at other site locations. The pressuremeter test depths were selected to provide representative information corresponding to the bearing soils anticipated within the stress influence zone of the proposed footings at the design bearing levels.

In the pressuremeter test, a radial expandable cylindrical probe is inserted into a prepared borehole at the selected testing depth. After obtaining N-values from driving a standard 2-inch O.D. diameter split-spoon sampler, borehole preparation consisted of then driving a 3-inch O.D. split-spoon sampler (or using a roller bit with wash rotary methods) to develop the appropriately sized borehole diameter for the pressuremeter probe. The cylindrical probe was inserted into the borehole to the sampling depth and then expanded against the sides of the borehole by pressurizing fluid within the system using a hydraulic screw-jack console positioned at the ground surface.

Simultaneous measurements of pressure and volume change within the probe were observed at the pressuremeter console and recorded. The pressure was incrementally increased until the maximum probe volume was reached, or until significant creep deformation (soil failure) was observed.

MUCK PROBE

The muck probe consists of a smooth rod about 1/2-inch in diameter manually pushed into the subgrade until encountering significant resistance (determined “by feel”), presumably indicating the bottom of organic soil stratum.

VANE SHEAR TESTING

In-situ vane shear testing generally follows the American Standard Test Method (ASTM) D-2573 “Standard Test Method for Field Vane Shear Test in Cohesive Soil”. Per the ASTM, the field vane shear test consists of placing a four-bladed vane (sized based on the expected cohesive soil strength) in the undisturbed soil and rotating it from the surface at a constant rate to determine the torque required to shear a cylindrical surface with the vane. This torque, or moment, is then converted to the unit shearing resistance of the failure surface by limit equilibrium analysis. Friction of the vane rod and instrument is either minimized during readings by an open hole, casing, or accounted for and subtracted from the total torque to determine the torque applied to the vane. After initially shearing the soil to determine the peak “undisturbed” ultimate shear strength, the test can be repeated to determine the remolded “residual” ultimate shear strength. The ratio of the peak shear strength divided by the remolded shear strength equals the degree of sensitivity.

DEGREE OF SENSITIVITY

DEGREE OF SENSITIVITY	DESCRIPTION
2	Insensitive
4	Moderately Sensitive
8	Extra Sensitive

ELECTRICAL RESISTIVITY TESTING

Field or laboratory resistivity testing generally follows the American Standard Test Method (ASTM) G-57 “Standard Test Method for Measurement of Soil Resistivity Using the Four-Electrode Method”.

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FIELD TESTING

Per the ASTM, the field Wenner four-electrode method requires four metal electrode probes placed with equal separation at various distances in a straight line. The probes are inserted in the surface of the soil to a depth not exceeding 5 percent of the minimum separation of the electrodes (or 12 inches maximum, whichever is less). The electrode

separation is selected with consideration of the soil strata location and depth of interest. A voltage is impressed between the outer electrodes and the voltage drop between the inner electrodes is measured. The resulting resistivity measurement represents the average resistivity of a hemisphere of soil of a radius equal to the electrode separation.

LABORATORY TESTING

Soil is tamped into a soil box to resemble the compaction where the soil sample was taken until the soil is level with the top of the box. Two brass pins are inserted at the premanufactured distances into the soil sample with two endpins connected to the box. The four test leads are connected to the soil box. A voltage is impressed between the two endpins to measure the resistance, and soil resistivity is calculated based on the product-specified conversion.

CONE PENETRATION TESTING

Cone Penetration Tests (CPT) measures the soil resistance to the penetration of a standard 10 square centimeter (cm) projected area. The cone is hydraulically pushed into the soil at approximately a 2 cm per second rate. Soil resistance is recorded in kilograms per square cm at 20 cm depth intervals. Soil friction values are measured by a friction sleeve at each test interval.

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LABORATORY TESTING PROCEDURES

VISUAL ENGINEERING CLASSIFICATION

Visual classification was performed on recovered samples. The appended General Notes and Unified Soil Classification System (USCS) sheets include a brief summary of the general method used visually classify the soil and assign an appropriate USCS group symbol. The estimated group symbol, according to the USCS, is shown in parentheses following the textural description of the various strata on the boring logs appended to this report. The soil descriptions developed from visual classifications are sometimes modified to reflect the results of laboratory testing.

MOISTURE CONTENT

Moisture content tests were performed by weighing samples from the field at their in-situ moisture condition. These samples were then dried at a constant temperature (approximately 110° C) overnight in an oven. After drying, the samples were weighed to determine the dry weight of the sample and the weight of the water that was expelled during drying. The moisture content of the specimen is expressed as a percent and is the weight of the water compared to the dry weight of the specimen.

HAND PENETROMETER TESTS

In the hand penetrometer test, the unconfined compressive strength of a cohesive soil sample is estimated by measuring the resistance of the sample to the penetration of a small calibrated, spring-loaded cylinder. The maximum capacity of the penetrometer is 4.5 tons per square-foot (tsf). Theoretically, the undrained shear strength of the cohesive sample is one-half the unconfined compressive strength. The undrained shear strength (based on the hand penetrometer test) presented on the boring logs is reported in units of kips per square-foot (ksf).

TORVANE SHEAR TESTS

In the Torvane test, the shear strength of a low strength, cohesive soil sample is estimated by measuring the resistance of the sample to a torque applied through vanes inserted into the sample. The undrained shear strength of the samples is measured from the maximum torque required to shear the sample and is reported in units of kips per square-foot (ksf).

LOSS-ON-IGNITION (ORGANIC CONTENT) TESTS

Loss-on-ignition (LOI) tests are conducted by first weighing the sample and then heating the sample to dry the moisture from the sample (in the same manner as determining the moisture content of the soil). The sample is then re-weighed to determine the dry weight and then heated for four hours in a muffle furnace at a high temperature (approximately 440° C). After cooling, the sample is re-weighed to calculate the amount of ash remaining, which in turn is used to determine the amount of organic matter burned from the original dry sample. The organic matter content of the specimen is expressed as a percent compared to the dry weight of the sample.

ATTERBERG LIMITS TESTS

Atterberg limits tests consist of two components. The plastic limit of a cohesive sample is determined by rolling the sample into a thread and the plastic limit is the moisture content where a 1/8-inch thread begins to crumble. The liquid limit is determined by placing a 1/2-inch-thick soil pat into the liquid limits cup and using a grooving tool to divide the soil pat in half. The cup is then tapped on the base of the liquid limits device using a crank handle. The number of drops of the cup to close the gap formed by the grooving tool 1/2 inch is recorded along with the corresponding moisture content of the sample. This procedure is repeated several times at different moisture contents and a graph of moisture content, and the corresponding number of blows is plotted. The liquid limit is defined as the moisture content at a nominal 25 drops of the cup. From this test, the plasticity index can be determined by subtracting the plastic limit from the liquid limit.

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GRAIN SIZE DISTRIBUTION ANALYSIS

COARSE-GRAINED (GRANULAR) SAMPLES WITH LOW FINES CONTENT

Grain size distribution tests performed on granular samples involves oven-drying a representative sample of soil and washing out the fines (passing the No. 200 sieve) with tap water. The sample retained on the No. 200 sieve is then oven-dried, cooled and sieved on a series of stacked sieves beginning with the largest sieve on top and progressing to the smallest on the bottom. The portions of the sample retained on each sieve are then weighed and used to develop the grain size distribution curve in the report for each sample tested.

FINE-GRAINED (SILT OR CLAY) SAMPLES OR COARSE-GRAINED SAMPLES WITH HIGH FINES CONTENT

Particle size distribution tests performed on fine-grained or coarse-grained samples with a high fines content involves oven-drying a representative sample and mixing the sample with a liquid deflocculant to disperse the soil particles. The slurry is placed in a graduated cylinder and shaken to suspend the soil particles in the slurry. The graduated cylinder is then placed on a tabletop; a calibrated hydrometer is floated in the slurry to determine its density. The hydrometer measurements are made at selected time intervals as the soil in the cylinder settles and slurry density decreases. When the hydrometer measurements are completed, the slurry is poured onto a No. 200 sieve and the fines are washed out with tap water. The sample retained on the No. 200 sieve is then oven-dried, cooled and sieved on a series of stacked sieves beginning with the largest sieve on top and progressing to the smallest on the bottom. The portions of the sample retained on each sieve are then weighed and used with the hydrometer data to develop the grain size distribution curve in the report for each sample tested.

WET/DRY DENSITY TESTS

Wet/dry density tests involve extracting a representative soil sample from either a Shelby tube or sample liner, trimming the ends perpendicular to the length of the sample and measuring the length and diameter. The sample is then weighed, oven-dried and weighed again after drying. The wet density is equal to the wet weight of the sample (prior to drying) divided by the volume, while the dry density is the dry weight of the sample divided by the volume.

UNCONFINED COMPRESSIVE STRENGTH TESTS

In addition to the hand penetrometer and Torvane tests, unconfined compression tests were performed to better estimate the undrained shear strength of selected cohesive samples recovered from either Shelby tubes or liners taken in conjunction with the Standard Penetration Test. In the unconfined compression test, the unconfined compressive strength of a soil sample is determined by axially loading the soil sample at a slow, constant rate of strain. The unconfined compressive strength is the maximum compressive stress in the soil sample, up to 15 percent strain. Theoretically, the undrained shear strength of the cohesive sample is one-half the unconfined compressive strength. The undrained shear strength presented on the boring logs is reported in units of kips per square-foot (ksf).

CORROSION TESTS

The soil corrosion tests may include measuring the electrical resistivity, pH and concentrations of soluble chlorides and sulfates. Soil samples tested are generally taken from a composite of two or more selected soil samples with generally similar visual characteristics. The electrical resistivity of the selected soil samples was performed on natural-state and saturated samples using a Miller multi-combination meter with a soil box configured in a four-pin arrangement. pH tests are typically conducted using litmus test paper. The soil samples for the soluble sulfates and chlorides were prepared as a water-soil solution, typically at a water-to-soil ratio of 20:1, and tested in general accordance with local laboratory methods for measuring sulfate and chloride concentrations.

MOISTURE-DRY DENSITY RELATIONSHIPS (COMPACTION) TESTS

Moisture-dry density tests involve the preparation of a bulk soil sample by compacting the sample at a given energy into a calibrated mold with a known volume of 0.0333 cubic feet at various moisture contents. A graph of the moisture content vs. dry density is developed, which results in an inverted U-shaped curve. The maximum dry density is the peak of the curve, and the corresponding moisture content is the optimum moisture. Two methods can be performed, namely:

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STANDARD PROCTOR METHOD

This method involves a standard energy of 12,400 ft-lbs. per cubic foot of soil volume to compact the sample. The sample is compacted in three layers of equal thickness using a 5.5-pound hammer dropped 12 inches using 25 blows per layer.

MODIFIED PROCTOR METHOD

This method involves a modified energy of 56,000 ft-lbs. per cubic foot of soil volume to compact the sample. The sample is compacted in five layers of equal thickness using a 10-pound hammer dropped 18 inches using 25 blows per layer.

SPECIFIC GRAVITY TESTS

This test involves the determination of the ratio of the weight of a known volume of soil particles in air to weight of the same volume of water in air. The test is performed by oven drying a soil sample and placing the sample with water into a calibrated pycnometer, boiling the soil/water mixture, filling the pycnometer with distilled water to its calibration mark, weighing the pycnometer and soil/water mixture and measuring the temperature of the mixture. The specific gravity is equal to the weight of the dry soil particles multiplied by the specific gravity of distilled water at the temperature measured for the soil/water mixture divided by the sum of the weight of the dry soil particles plus the weight of the pycnometer, soil/water mixture plus the weight of the pycnometer plus water from the calibration curve developed for the pycnometer.

DIRECT SHEAR TESTS

A bulk sample is compacted in a direct shear mold at a specified density and moisture content. Shear tests are then performed using the direct shear procedure. The direct shear test is performed at several overburden pressures or normal stresses that represent approximate potential stresses in the proposed construction. Values of both peak friction angle and residual friction angle are determined from the tests for each overburden pressure.

CONSOLIDATION TESTS

Consolidation tests are used to evaluate the magnitude and rate of consolidation of soil when it is restrained laterally and drained on the top and bottom while subjected to vertical load applied in controlled increments. The range of test loads applied is generally selected to represent the anticipated vertical stress conditions resulting from existing conditions and the proposed construction. Plots of the percent strain vs. log pressure are constructed from the data to assess consolidation characteristics, while the rate of consolidation is evaluated from plots of deformation vs. time for each vertical load increment.

PERMEABILITY TESTS

The permeability of either relatively undisturbed or compacted soils can be determined by various laboratory test equipment including a triaxial cell, permeameter mold or from a liner sample. The type of permeability equipment used and test performed will be based on the soil type being evaluated.

CLAY, SILT AND OTHER LOW PERMEABLE SOIL SAMPLES

For samples with relatively low permeability characteristics, an undisturbed or compacted soil sample is placed in a triaxial cell. Prior to performing the permeability test, the sample must be fully saturated by forcing water into the sample using a backpressure (water under pressure from an air supply) which is slightly less than the cell pressure. Once the sample is saturated, water is forced through the top of the sample with pressure from an air supply (which is slightly less than the cell pressure) and water forced out of the bottom of the sample is measured in a burette. The volume of water displaced from the sample is recorded with time and from that information, the coefficient of permeability is calculated. This method is a constant head permeability test.

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SAND SAMPLES

Due to the nature of relatively clean granular soils, the use of a triaxial cell is generally not practical and the permeability of these types of soils is typically determined from either a liner sample (either recovered directly from a split-spoon in the field or a sample compacted in the liner) or a bulk sample compacted in a 6-inch diameter permeameter mold. A falling head permeability test can be performed on most granular samples by filling a standpipe with water and measuring the head drop with time. For highly permeable soils, the rate of drop in a falling head test may be too rapid to obtain reliable volume and time measurements. Thus, a constant head test will be required where a constant head of water is maintained, and the volume of water discharged from the sample is measured with time.

TRIAXIAL TESTS

Triaxial tests were conducted on samples trimmed from Shelby tubes or liners. There are several types of triaxial tests which can be performed, and each are described below:

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST METHOD

The strength and stress-strain relationships of a cylindrical soil sample are determined for a sample subjected to a selected confining fluid pressure in a triaxial chamber. No drainage of the sample is permitted during the test, and the sample is sheared in compression at a constant rate of axial deformation. The peak stress measured for the sample is recorded, up to a maximum 15 percent strain. At least three triaxial tests are performed at various confining fluid pressures to model in-situ stress conditions for loading.

CONSOLIDATED-DRAINED TRIAXIAL TEST METHOD

The strength and stress-strain relationships of a cylindrical soil sample are determined for a sample subjected to a selected confining fluid pressure in a triaxial chamber. The sample is isotropically consolidated prior to applying axial loads and sheared in compression at a slow constant rate of axial deformation while allowing the sample to drain. The peak stress measured for the sample is recorded, up to a maximum 15 percent strain. At least three triaxial tests are performed at various confining fluid pressures to model in-situ stress conditions for loading.

CONSOLIDATED-UNDRAINED TRIAXIAL TEST METHOD

The strength and stress-strain relationships of a cylindrical soil sample are determined for a sample subjected to a selected confining fluid pressure in a triaxial chamber. The sample is isotropically consolidated prior to applying axial loads and sheared undrained in compression at a constant rate of axial deformation. Pore water pressure measurements can also be measured during the shearing of the sample. The peak stress measured for the sample is recorded, up to a maximum 15 percent strain. At least three triaxial tests are performed at various confining fluid pressures to model in-situ stress conditions for loading.

DENSITY TESTS ON ROCK CORES

Density tests involve trimming the ends of an intact rock core sample perpendicular to the length of the sample and measuring the length and diameter. The sample is then weighed, and the weight is divided by the volume to calculate the density.

UNCONFINED COMPRESSIVE STRENGTH TESTS ON ROCK CORES

Unconfined compression tests were performed to estimate the compressive strength of selected rock core samples. Representative rock cores were selected and cut perpendicular to the length of the sample on both ends to a specified length with a wet saw. In the unconfined compression test, the unconfined compressive strength of a rock core sample is determined by axially loading the rock core sample at a slow, constant rate of strain. The unconfined compressive strength is the maximum compressive stress in the rock core sample, or the load applied when a predetermined amount of strain is achieved.

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LIMITATIONS PERTAINING TO SUBSURFACE CONDITIONS

EXISTING FILL

It is sometimes difficult to distinguish between existing fill present at a site and natural soils based on samples and cuttings from small-diameter boreholes, especially if portions of the fill do not contain man-made materials, debris, topsoil, or organic layers, and when the fill appears similar in composition to the local natural soils. Therefore, consider the delineation of fill described on the logs, if encountered, to be approximate.

The composition of existing site fill, if encountered, may change abruptly over short distances and will vary from what is reported on the logs. The descriptions of debris within fill may not accurately indicate the quantity, composition, or size of the debris, and may not fully include the types of debris existing within the site fills. Perform test pits if existing fill is encountered to further evaluate the condition of the fill, particularly if the fill will be utilized for support of overlying structures and/or other improvements.

GROUNDWATER

Hydrostatic groundwater levels, and perched groundwater conditions, will fluctuate throughout the year, based on variations in precipitation, evaporation, run-off, and other factors. The groundwater information reported on the logs represent conditions at the time the readings were taken and may vary from the groundwater conditions encountered at other times.

SUBSURFACE PROFILE

The profile described in this report and included on the logs is a generalized description of the conditions observed. The stratification depths described in this report and shown on the logs indicate a zone of transition from one soil type to another. They are not meant to delineate exact depths of change between soil types. Soil conditions may vary between or away from the exploration locations. Refer to the logs for the soil descriptions, rock descriptions (when applicable), and results of the field and laboratory tests at the specific exploration locations.

If only borings with hollow-stem or solid-stem augers are performed, consider thickness measurements of surficial materials reported on the logs (e.g., gravel, asphalt, concrete, aggregate base) to be approximate since mixing of the surface materials with the underlying subgrade can occur while advancing the augers, and it is difficult to measure the thickness of surface materials in small-diameter boreholes. Perform additional evaluations for more accurate measurements of surface materials, such as test pits for topsoil and gravel thicknesses, coring for pavement thicknesses, and hand sampling for aggregate thickness.

RADON

The need for radon control systems for this project was not evaluated as part of our current scope of services. Contact the local building authority to verify whether radon control systems are necessary to meet the applicable local building codes or other requirements. If radon control methods are required, incorporate the recommendations regarding the below-slab leveling course materials and vapor retarders presented in this report with the specific materials and measures necessary to meet the applicable radon control methods. Contact SME for further recommendations.

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APPENDIX C

IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL ENGINEERING REPORT
GENERAL COMMENTS

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Important Information about This Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

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This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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GENERAL COMMENTS

BASIS OF GEOTECHNICAL REPORT

This report has been prepared in accordance with generally accepted geotechnical engineering practices to assist in the design and/or evaluation of this project. If the project plans, design criteria, and/or other project information referenced in this report and utilized by SME to prepare our recommendations are changed, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions and recommendations of this report are modified or approved in writing.

The discussions and recommendations contained in this report are based on the available project information, described in this report, and the geotechnical data obtained from the field exploration at the locations indicated in the report. Variations in soil and groundwater conditions commonly occur between or away from sampling locations. The nature and extent of the variations may not become evident until the time of construction. If significant variations are observed during construction, SME must be contacted to reevaluate the recommendations of this report.

In the process of obtaining and testing samples and preparing this report, procedures are followed that represent reasonable and accepted practice in the field of geotechnical engineering. Specifically, field logs are prepared during the field exploration that describe field occurrences, sampling locations, and other information. Samples obtained in the field are frequently subjected to additional testing and reclassification in the laboratory and differences may exist between the field logs and the report logs.

The engineer preparing the report reviews the field logs, laboratory classifications, and test data and then prepares the report logs. Our recommendations are based on the contents of the report logs and the information contained therein.

REVIEW OF DESIGN DETAILS, PLANS, AND SPECIFICATIONS

Retain SME to review the design details, project plans, and specifications to verify those documents are consistent with the recommendations contained in this report.

REVIEW OF REPORT INFORMATION WITH PROJECT TEAM

Implementation of our recommendations may affect the design, construction, and performance of the proposed improvements, along with the potential inherent risks involved with the proposed construction. The client and key members of the design team, including SME, should discuss the issues covered in this report so the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk, and expectations for performance and maintenance.

FIELD VERIFICATION OF GEOTECHNICAL CONDITIONS

SME needs to be retained to continue our services through construction so we may observe and evaluate the actual subsurface conditions relative to the recommendations made in this report, and so we can verify the recommendations of this report are properly implemented during construction. This may avoid misinterpretation of our recommendations by other parties and will allow us to review and modify our recommendations if variations in the site subsurface conditions are encountered.

PROJECT INFORMATION FOR CONTRACTOR

This report and any future addenda or other reports regarding this site needs to be made available to prospective contractors prior to submitting their proposals for their information only and to supply them with facts relative to the subsurface evaluation and laboratory test results. If the selected contractor encounters subsurface conditions during construction, which differ from those presented in this report, the contractor needs to promptly describe the nature and extent of the differing conditions in writing and SME needs to be notified so we can verify those conditions. The construction contract needs to include provisions for dealing with differing conditions, and contingency funds for potential problems during earthwork and foundation construction. We would be pleased to assist with the development of contract provisions based on our experience.

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The contractor needs to be prepared to handle environmental conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers. Any Environmental Assessment reports prepared for this site need to be made available for review by bidders and the successful contractor.

THIRD PARTY RELIANCE/REUSE OF THIS REPORT

This report has been prepared solely for the use of our Client for the project specifically described in this report. This report cannot be relied upon by other parties not involved in the project, unless specifically allowed by SME in writing. SME also is not responsible for the interpretation by other parties of the geotechnical data and the recommendations provided herein.

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