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PRELIMINARY REPORT OF GEOTECHNICAL EXPLORATION AND REVIEW

Track and Station Construction

Southwest Light Rail Transit Project, PEC East
Minneapolis, Minnesota

Report No. 01-05697.01

Date:

August 19, 2014

Prepared for:

Kimley-Horn and Associates, Inc.
Southwest Project Office
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August 19, 2014

Kimley-Horn and Associates, Inc.
Southwest Project Office
6465 Wayzata Boulevard, Suite 500
St. Louis Park, MN 55426

Attn: Mark C. Bishop, PE

RE: Geotechnical Exploration and Review
Track and Station Construction
Southwest Light Rail Transit Project, PEC East
Minneapolis, Minnesota
Report No. 01-05697.01

Dear Mr. Bishop:

American Engineering Testing, Inc. (AET) is pleased to present the results of the subsurface exploration and testing program performed to date and our associated geotechnical engineering review for the above grade track and station construction planned for the Southwest LRT in the PEC East segment from Hopkins to Minneapolis, Minnesota. This report supersedes the June 23, 2014 report submitted under AET No. 01-05697.01.

In addition to the pdf electronic copy, we are submitting four copies of the report to you.

Sincerely,
American Engineering Testing, Inc.

Jeffery K. Voyen, PE
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I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under Minnesota Statute Section 326.02 to 326.15

Name: Jeffery K. Voyen

Date: 8/19/14 **License #: 15928**

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1.0 INTRODUCTION

A new light rail transit (LRT) system is proposed for construction from Minneapolis to Eden Prairie, Minnesota. This report concerns the above grade track and station components of the PEC East segment of the project, from Minneapolis to Hopkins. Separate reports are associated with tunnel and bridge components of the project and are therefore not included herein. To assist planning and design of this portion of the project, you have authorized American Engineering Testing, Inc. (AET) to conduct a subsurface exploration/testing program at the site and to perform geotechnical engineering review. This report presents the results of these services and our associated engineering recommendations.

2.0 SCOPE OF SERVICES

AET's services for the SWLRT PEC East project are being performed per our on-going Master Agreement for Continuing Professional Services (January 1, 2011) and our Individual Project Orders. The scope relative to the track and station portion of the project which is contained in this report consists of the following:

- Drill and sample 156 standard penetration test (SPT) borings.
- Conduct four piezocone penetration test (CPT_u) soundings.
- Install and monitor the water level in a piezometer near the Beltline Station (1066 PS). Piezometer installation and monitoring was also conducted north of the Cedar Lake to Lake of the Isles channel; the specific results related to those piezometers can be found in the Kenilworth shallow tunnel report (AET 01-05697.02).
- Perform soil laboratory index testing.
- Conduct engineering analysis based on the gained data, and prepare this geotechnical engineering report.

Numerous SPT borings, CPT_u soundings, and piezometers have been completed for other components of the project, but not presented herein, if not relevant to the specific subjects of this report.

These services were intended for geotechnical purposes. The scope was not intended to explore for the presence or extent of environmental contamination.

3.0 PROJECT INFORMATION

The information stated in this section represents our understanding of the proposed construction. This information is an integral part of our engineering review. It is important that we be contacted if there are changes from that described so that we can evaluate whether modifications to our recommendations are appropriate.

The PEC East portion of the Southwest LRT project extends along current HCRRA property from about 1,000 feet west of 8th Avenue S in Hopkins to the existing bridge at the Target Field Station in Minneapolis. The LRT system is planned to traverse through the Kenilworth Corridor between Cedar Lake and Lake of the Isles. The segment to the south of the channel connecting the lakes is planned to be placed in a shallow tunnel, that portion of the project being the subject of a separate report. Previous plans considered a shallow tunnel to the north of the channel; although the track is now planned to be constructed "near-grade" such that it is now a topic of this report. Planned bridges, which are also the subject of separate reports, include:

- LRT over Excelsior Boulevard
- LRT, Freight, and Trail over Minnehaha Creek
- LRT, Freight, and Trail over Louisiana Avenue
- LRT, Freight, and Trail over TH 100

- LRT, Freight, and Trail over Channel in Kenilworth Corridor
- Glenwood Avenue, with LRT rising within Retained Walls to cross Glenwood at deck grade
- LRT Flyover BNSF, north of Glenwood Avenue
- LRT over 5th Avenue N and N 7th Street

Trail/pedestrian access bridges and underpasses beyond that presented above are also planned as shown below. Again, these components are topics of separate reports.

- Pedestrian bridge to east of Beltline Station
- Pedestrian bridge to west of Penn Station
- Penn Station pedestrian bridge to Penn Ave "kiss and ride" lot, including a retaining wall at Penn Avenue. A vertical circulation structure will also be a part of this bridge, and structural support of that structure is included in this separate bridge/wall report.

An underpass which includes structural retaining walls will be located off the east end of the Louisiana Avenue Station, extending northerly below the crossing freight rail tracks to serve the Cedar Lake Trail. The crossing tracks include a line veering to the south leading up to the Southerly Connector freight rail bridges. Vertical circulation towers also are planned at the West Lake and Van White Stations, serving the adjacent existing bridges. Recommendations for foundation support of the above described structures are included herein.

The Luce Line Trail pedestrian bridge will also be constructed in the vicinity of the Van White Station, although borings were not specifically placed for this bridge as of this report. As such, a separate foundation report (FADR) has not been developed for this particular bridge. Preliminary foundation recommendations are anticipated to be similar to the Van White vertical circulation structure, which are included in this report.

3.1 Mainline Track

Outside of the above bridge/tunnel areas, the PEC East portion of the project will incorporate the following track systems:

- **Ballast-** raised rails supported on concrete ties which are supported on a minimum of 1-foot of ballast. The top of rail to bottom of tie is about 1.25 feet. The rail is raised about 7-inches above the tie. Ballast material surrounds the ties, although may not necessarily be flush with the top of the tie. This is the predominant track system that will be used.
- **Ballast, Special Trackwork-** the same ballast system as described above, except that maximum allowable differential settlement requirements are tighter. This system is used in the locations of Cross-Overs (currently located within approximate Stations 2656+50 to 2662+50, 2720+50 to 2724+00, and 2882+00 to 2885+50).
- **Direct Fixation (DF)-** raised rails bolted into C.I.P. concrete plinths which are anchored directly into 12-inch thick structural slabs. This track system is limited to station and vertical circulation areas.
- **Embedded-** rails contained within an 8-foot wide reinforced concrete slab supported over 8-inches of stabilized subbase (i.e., Class 5 aggregate base). The top of rail to bottom of slab dimension will be 18 inches. This system is used for track bordering roadway pavements, which is basically limited to road crossings on this project

Maximum allowable differential settlement tolerances established for the <u>ballast</u> track system (excluding Special Trackwork areas) is 1 inch over 31 feet longitudinal and ½ inch over 31 feet lateral (cross level variation). These tolerances relate to "geotechnical" settlement following construction, and not construction tolerance or maintenance tolerance.

Maximum allowable differential settlement tolerances established for Special Trackwork areas, Direct Fixation, and Embedded track systems is ½ inch over 31 feet longitudinal and lateral (cross level variation). These tolerances relate to "geotechnical" settlement following construction, and not construction tolerance or maintenance tolerance.

As the above settlement tolerances relate to vertical post-construction movement, we also assume the above tolerances can be applied to differential frost-heave movements. For evaluation of subgrade frost control needs (to meet differential tolerable movement criteria), maximum frost depth is assumed to be up to 66 inches (5½ feet). This is based on our review of MnDOT frost depth records, considering latitude and soil types.

Concerning allowable soil bearing pressures required for track support, it is assumed that the bearing pressure on the subgrade will not exceed 18 psi (2592 psf). AREMA stipulates that subgrade bearing pressures should not exceed 20 psi (2880 psf). Structural slab (DF and embedded) tracks are expected to have reduced bearing pressures of no more than 2400 psf based on better distribution of loads through the concrete slab.

3.2 Stations

A total of ten stations are planned within PEC East; those being the Downtown Hopkins, Blake, Louisiana, Wooddale, Beltline, West Lake, 21st Street, Penn, Van White, and Royalston Stations. The stations have been designed as "on-grade" supported structures, which will rest on a uniform thickness layer of Select Granular Material intended to control differential frost movements. This bottom of the Select Granular Material will be consistent with the bottom used below the adjacent mainline DF track, and is expected to result in minimal movement between the platform and track. Station and vertical circulation structures at Louisiana and West Lake, and the vertical circulation structures at Penn and Van White will require deep foundation support. These areas will require increased depth of Select Granular Material placement to eliminate upward frost forces on the structural slab.

The station platform top elevation will be 1'-2" above the top of rail elevation. The structures will be a monolithically cast 6-inch structural slab supported on grade beam edges and grade beam "cross-webs." The platform will then receive a 4-inch topping slab. The depth of the grade beam adjacent to the embedded track slab will have a bottom matching the bottom of the track slab.

All structures placed upon the platforms will be structurally supported by the platform (i.e., there will not be deeper spread foundations). Although the above grade features result in variable localized loads upon the platform, the platform will be structurally tied together such that it will distribute the loads over the supporting subgrade. Unit bearing pressures are anticipated to be lower than 2400 psf.

3.3 Vertical Circulation Structures at West Lake, Penn, and Van White Stations

Enclosed stairway/elevator/walkway structures are planned as extensions to the West Lake, Penn, and Van White Stations. The structures will allow pedestrian access to the higher in-place bridge decks or new overpass. These structures are planned as follows:

- both north and south sides of the Lake Street bridge
- west side of the Penn Station, with a bridge overpass to the Penn Kiss and Ride lot
- east side of the Van White bridge

Most of the enclosed space area will have an on-grade, lobby slab level in the vicinity of track grade. The elevator pit slabs are anticipated to be about 4-feet deeper than the lobby slabs. We assume the interior lobby areas of the structures are not planned to be heated during the winter, such that floor subgrade soils may freeze. However, we assume the elevator pits are expected to maintain temperatures above freezing.

The structures will have a steel frame and a curtain wall skin. Maximum loads are not available; although we assume the highest loads will be associated with shear wall legs and be no more than 250 kips.

3.4 Underpass at Louisiana Station

The Louisiana Station is planned to be located on the south side of the existing rail/trail embankment. The LRT tracks will lower to the station with a new fill embankment which will be structurally retained on the south side. A new freight rail line will also veer to the south to the Southerly Connector bridges in the area of the station and to the east, and will remain at the higher grade. The Cedar Lake Trail is located on the north side, and will veer to the north and lower to grade similar to the station. To connect with the station, a new underpass will be constructed below the freight rail tracks which will be associated with retaining walls/wing walls.

3.5 OCS Poles

Poles for the Overhead Contact System (OCS) will be constructed along the track system. The poles are expected to have low axial loads, but will be associated with significant lateral/overturning loads. We understand a typical OCS pole may be supported on a 30-inch diameter pier, having a depth of 12 feet. However, for poles on curves or dead-end structures, or where soils may limit the design, this may increase to a 42-inch diameter and/or a 20 foot depth. Our scope does not include design of the OCS pole foundations. Future geotechnical final design services should include estimated soil parameters for the pole designer's use in the foundation design.

4.0 SUBSURFACE EXPLORATION AND TESTING

4.1 Field Exploration Program

The subsurface exploration program which was conducted pertinent to this report consisted of 156 standard penetration test (SPT) borings and four piezocone penetration test (CPT_u) soundings. The test locations appear graphically on the figures in Appendix A. Piezometers were installed north of the channel in the Kenilworth Corridor, due to the possibility of a shallow tunnel is this area. The ground-water level results and review associated with this piezometer monitoring appears in detail in AET Report No. 01-05697.02, so is not repeated in this report. A piezometer was also installed in the Beltline area to provide more accurate measurements of the hydrostatic ground-water level in that area.

4.1.1 Standard Penetration Test Borings

The standard penetration test (SPT) methods used are described in Appendix A. The logs of the SPT borings appear in Appendix B. The logs contain information concerning soil layering, soil classification, geologic description, and moisture condition. Relative density or consistency is also noted for the natural soils, which is based on the standard penetration resistance (N-value). The test locations were measured by AET using GPS (submeter accuracy, but not surveyor accuracy). The boring surface elevations were measured by AET using an engineer's level and rod. These were based on various benchmarks provided to us by the project surveyor (MFRA).

4.1.2 Piezocone Penetration Test Soundings

The CPT_u test method is described in Appendix A. The logs of the CPT_u soundings appear in Appendix B. The piezocone penetration test (CPT_u) logs are computer-generated plots which include data on tip resistance, sleeve friction, friction ratio, pore pressure, and soil behavior (interpreted estimate of soil classification based on tip resistance and friction ratio). The test locations were measured by AET using GPS (submeter accuracy, but not surveyor accuracy). The sounding surface elevations were measured by AET using an engineer's level and rod. These were based on various benchmarks provided to us by the project surveyor (MFRA).

4.1.3 Piezometer Installation and Monitoring

To assist evaluation of the hydrostatic ground-water level in the Beltline area (for consideration of a potential underpass), a piezometer (1066 PS) was installed at Boring 1066 SS. The log of the piezometer installation appears in Appendix A, following the log for Boring 1066 SS. The piezometer location and the top of riser elevation (used as the reference for the water level elevation measurements) were measured by AET. Water level measurements taken appear on the log.

4.2 Laboratory Testing of Soils

During laboratory classification logging, water content tests were conducted on cohesive soil samples. In addition, the following tests were performed relative to the borings contained herein:

- 17 sieve analysis tests
- 14 unconfined compression tests with density
- 4 consolidation tests with density and specific gravity
- 16 Atterberg Limits tests
- 29 organic content tests

The test results appear on the individual boring logs, opposite the samples upon which they were performed and/or on the data sheets following the logs.

5.0 SITE CONDITIONS

5.1 General Review of Geologic Conditions Present

The upper natural soils along much of the alignment consist of sand and gravel glacial ice outwash (alluvial) soils, mostly classified as sands to silty sands with varying gravel content. There are areas of significant organic swamp deposits above the natural sands which are usually buried below fill, most notably in the Louisiana, Beltline, West Lake, 21st Street, Penn, and Van White areas where significant structures are planned. The geology changes in the north end of the project. The segment from the Target Field Station to about 1000 feet west of I-94 lies within the Bassett Creek geologic valley. The deposits in this valley consist of deep-water lake sediment deposited in an ice-block melt-out lake. The soils are primarily soft lean to fat clays which occasional beds of silt or fine grained sand. Although portions of the clays have been found to be firm to stiff due to apparent past overburden (generally in the Glenwood to Royalston area), the lower area to the southwest of this have clays which are soft to very soft. Layers of lean clay to fat clay continue to be present to the west of the deep Bassett Creek valley (to about Penn Avenue), although the thickness is not as great and tends to be variable and interbedded with other deposits. Portions of these alluvial clays are also overlain by organic swamp deposits.

At intermittent locations along the PEC East Corridor, deeper portions of the profile include glacial till deposits, mainly consisting of silty sands, clayey sands, and sandy lean clays. Occasionally, the till rises to near the surface, although will rarely be exposed in anticipated excavation bottoms.

As the corridor traverses though railroad and urban areas, fill is usually present above the natural soils, often having a significant thickness. The fill is generally comprised of soil types similar to surrounding native soils, although tend to be a mixture of varying soil types. Debris, organic soils, roots and ashes/cinders can also be present. Compaction tends to be variable, depending on the function of the past development.

5.2 Ground Water

Ground-water levels were measured during the drilling and sampling procedures, although except for the measurements in piezometers, they were short term measurements and many may not have fully stabilized. Generally, water levels measured in sands (or after penetrating into sands) will provide a more reliable indication of the true ground-water level. Often, we allowed a time period on the order of ten minutes to allow for some stabilization to occur. We refer you to Table A in Appendix A for data on water levels elevations found. The water levels shown at those locations having an adjacent piezometer represent the average water level of the measurements to date. Ground-water levels will fluctuate on an annual and seasonal basis.

6.0 RECOMMENDATIONS

6.1 Definitions

Italicized words used in this report have a specific definition or are a publication or software title. The specific definitions are presented below or in an ASTM Standard.

Top of subgrade: For tracks, *top of subgrade* is the grade which contacts the bottom of the subballast (aggregate base) layer. If the concrete slab or ballast/tie zone between and to the sides of the track area is thinner than beneath the track, the bottom of the thicker track slab or tie/ballast zone still defines the top of subgrade for that entire cross-section.

Suitable Grading Material is an environmentally acceptable mineral soil, which can be from the project site, excluding the following soils:

- soils with Unified Classifications of ML, MH, CL, CH,
- soils which have an organic content of 3% or more
- soils which include debris and/or boulders

The soil must also be capable of attaining the specified compaction level at its current water content or at a water content that can be reasonably scarified, blended, and moisture conditioned to a uniform water content to meet the specified compaction level.

Select Granular Material is defined as soils which meet the requirements of MnDOT Standard Specification 3149.2B2.

Test roll is a means of evaluating the near-surface stability of subgrade soils (usually non-granular). Suitability is determined by the depth of rutting or deflection caused by passage of heavy rubber-tired construction equipment, such as a loaded dump truck, over the test area. Yielding of less than 1 inch is normally considered acceptable, although engineering judgment may be applied depending on equipment used, soil conditions present, depth below final grade, and/or performance expectations.

Unstable soils are soils which do not pass a test roll. Unstable soils typically have water content exceeding the standard optimum water content defined in ASTM:D698 (Standard Proctor test).

Organic soils are soils which have sufficient organic content such that engineering properties are affected (3% or more).

Compaction subcut is the construction of a uniform thickness subcut below a designated grade to provide uniformity and compaction within the subcut zone. The subcut bottom must be exposed to allow bottom shaping and compaction (and further correction if deemed necessary). Replacement fill can be the materials subcut provided they satisfy the Suitable Grading Material requirement. These reused soils should be blended to a uniform condition and re-compacted in lifts per the Specified Density Method described in MnDOT Specification 2105.3F1.

6.2 Mainline Track and Station Grading

Subgrade preparation needs for the various track systems and stations are presented in this section. Please refer to Table A in Appendix A for detailed subcut/*Select Granular Material* placement or *compaction subcut* needs based on each boring. Recognize that the boring is not necessarily located in the track/station location, and field evaluation based on the recommendations presented below certainly overrides the Table A designations.

Also, recognize that in some areas, the existing boring was placed on or adjacent to a raised trail embankment and lower ditches may also be present in the track/station footprint. All track and station areas should be stripped of topsoil and pavements prior to subgrade preparation, and there may be a need for additional controlled fill placement beyond that portrayed by Table A. Controlled fill placement should be performed as recommended later in this section.

6.2.1 Stabilized Subbase and Underlying Drainage Layer for Tracks

The subballast layer to be placed directly below ballast layer or concrete slab is planned to be an 8-inch thick stabilizing subbase layer. Class 5 per MnDOT Specification 3138 is a common aggregate base material in the area and can be used as the stabilizing subbase layer. However, Class 6 or Class 7 (recycled base) could also be used.

The aggregate base thickness should be uniform across the cross-section of the tracks, with only slight (2%) tapers if needed for drainage sloping. If tapers occur in the longitudinal direction, they should be tapered at no steeper than 20H:1V.

Where a sand drainage/frost control layer is required directly beneath the aggregate base layer, the layer should consist of *Select Granular Material*. Note that the aggregate base/*Select Granular Material* soils are not totally non-frost susceptible, but the combination of material thickness and uniformity, and the associated drainage improvement, is considered practical and suitable for the desired movement control (the movement criteria is to limit differential settlement).

6.2.2 Ballast Track Subgrade Needs

For mainline ballast tracks (excluding Special Trackwork), a sand drainage layer below *top of subgrade* is not necessarily required. The exception is when soil not meeting the requirements of *Suitable Grading Material* is present within 18 inches of top of subgrade. In this case, the substandard soils should be subcut and replaced with *Select Granular Material*. The full 18-inch subcut is not required if *Suitable Grading Materials* are reached before this depth.

In some areas, the soils present will meet the requirements of *Select Granular Material*. In these areas, no special subcuts are required. However, the exposed soils should be surface compacted and evaluated as described later.

In areas found to meet the requirements of *Suitable Grading Material*, but not that of *Select Granular Material* (i.e., silty sands, clayey sands), the soils can be relied upon for subgrade support, although should undergo a minimum 12-inch deep *Compaction Subcut* to improve compaction and frost uniformity.

6.2.3 Remaining Track System and Station Subgrade Needs

Direct Fixation track, Embedded track, and Special Trackwork in ballast track areas (Cross-Over areas), will require *Select Granular Material* to be in-place beneath *top of subgrade* due to the tighter differential movement tolerance requirements. Where *Select Granular Material* is not confirmed to already be in-place within 2 feet of *top of subgrade*, a 2-foot thickness of *Select Granular Material* should be placed.

In those areas where driven piles are used to support structural track, station platforms, unheated vertical circulation structures, or other exterior slab areas where the structure is not enclosed and heated during the winter, we recommend increasing the *Select Granular Material* thickness to minimize upward frost forces on the slab (assuming the slab is placed on-grade and not formed and raised above grade by at least 2 inches). The combined concrete slab, subballast (if placed), and *Select Granular Material* thickness should be 5.5 feet thick. In this case, the preparation of the excavation bottom discussed in the next section would not be necessary.

6.2.4 Track Excavation/Bottom Preparation Needs

It will be important to have a stable foundation base upon which to place the *Select Granular Material* layer, or the subballast if the *Select Granular Material* is not required. The soils exposed at the bottom of the excavation will need to meet allowable bearing criteria described in Section 3.1. Bearing requirements of the in-place soils can then be reduced with increasing depth, per the direction of a geotechnical field engineer. Soils lacking cohesion (i.e., granular soils) can be surface compacted for bearing improvement. If encountered, overly compressible soils (likely to be *organic soils*) may need to be removed, even if near-surface soil stability/bearing needs are met. After compaction as recommended, the in-place soils encountered at the boring locations are judged to be suitable without additional subcutting beyond the planned subcuts for *Select Granular Material* placement. However, there may be areas away from the test locations where the in-place soils will need to be corrected to provide a firm foundation base meeting the bearing/settlement criteria.

Excavations and subsequent engineered fill placement should maintain minimum lateral oversizing of the excavation bottom. This lateral excavation oversizing should be a minimum of ½H:1V. The exception would be if organic soils are encountered during the excavation (significantly organic soils were not encountered at the boring locations). If excavation sides expose organic soils, the lateral excavation bottom oversize requirement should be increased to at least 1:1.

Looser granular soils should be surface compacted when exposed in the excavation bottom. This would apply to the natural sands to silty sands (typically coarse alluvium) having N-values of 8 bpf or lower. In those areas of granular fill (fill with predominantly a sand to silty sand component), we recommend surface compaction be applied regardless of N-value due to the increased potential for soil variability. Surface compaction should involve at least six passes of a vibratory roller compactor (3 foot minimum drum diameter, minimum static weight of 6 tons). The deflections under the compaction process should be observed for the purpose of evaluating whether *unstable soils* may still exist within the subgrade. The instability would likely be caused by wet, clayey zones or inclusions within the fill. If unstable zones are detected, they should be subcut and replaced with more favorable granular soils.

For evaluation of clayey soils, a *test roll* process should be performed, and *unstable soils* found should be corrected with subcutting and replacement; or with scarification, drying, and recompaction to proper stability.

6.2.5 Station Excavation/Bottom Preparation Needs

This section relates to those stations which are not structurally supported on pile foundation systems. The station platforms are immediately adjacent to the Direct Fixation track. The bearing, settlement, and frost control needs of the stations should be satisfied by meeting the needs of the track. Accordingly, the recommendations for the stations are consistent with that recommended for the tracks. The track subcut required adjacent to stations is about 5 feet below top of rail. The bottom elevation of the subcut should remain relatively constant across the cross-section of the track and station platform, maintaining no steeper than a 2% slope to promote drainage. Bottom preparation requirements should be consistent with that recommended for the track subgrade.

One exception relates to the placement of the aggregate base layer. The station edge grade beam abutting the track slab is planned to have a common bottom elevation. We recommend the aggregate base layer planned beneath the track slab also be extended laterally to support the abutting grade beam. However, the aggregate base layer can terminate at a lateral distance of 12 inches beyond the grade beam. All fill materials placed beyond (and above) this aggregate termination can then be *Select Granular Material.*

To ensure that the top edge of the platform maintains proper interface with the top of rail, we recommend installing dowels to connect the track slab with the platform foundation.

6.2.6 Fill Placement and Compaction

Suitable Grading Material fill can be used to re-attain bottom of *Select Granular Material* layer grade, or *top of subgrade* in ballasted areas where *Select Granular Material* is not required.

The fill should be compacted in thin lifts, such that the entire lift achieves a minimum compaction level of 100% of the *standard maximum dry unit weight* per ASTM:D698 (Standard Proctor test). The minimum compaction level can be reduced to 95% for fill placed deeper than 3 feet below *top of subgrade*. The fill lift thicknesses should be no greater than 12 inches for granular soils and no greater than 8 inches for more clayey/silty soils. The lifts should be thinner than the above if needed to achieve the minimum specified compaction level with the type of compaction equipment being used.

6.2.7 Select Granular Frost Tapering

Select Granular Material has relatively low frost heave potential. Soils with higher silt and clay content have a higher frost heave potential, and it will be undesirable to have an abrupt change in *Select Granular Material* thickness for frost heave uniformity purposes. To reduce the abruptness of this frost differential, tapering of the *Select Granular Material* is needed within the frost zone (within 5.5 feet of the surface) as described below where the thickness is required to be changed.

Where the *Select Granular Material* thickness changes in a longitudinal (profile) direction, we recommend the thickness have a taper of no steeper than 20H:1V in track areas and no steeper than 10H:1V in roadway areas.

The *Select Granular Material* cross-sectional thickness beneath the mainline tracks and stations should remain constant, with the bottom allowing for the 2% drainage slope. We recommend tapers from station platforms to roadway sections, or from the mainline track to roadway sections be maintained at tapered slopes no steeper than 4H:1V. Tapers which extend into sidewalk or non-driving surface features should be maintained at no steeper than 1.5H:1V. Back-slopes into green areas should satisfy OSHA back-sloping requirements.

6.2.8 Subsurface Drainage

To prevent buildup of water, the *Select Granular Material* layer should be provided with a positive means of subsurface drainage. Properly engineered drainage lines are recommended at the bottom of *Select Granular Material* layers. Placement of drainage lines should be outside of the track footprint, at the points to which the *Select Granular Material* 2% bottom sloping is directed. We recommend perforated drain pipe used be placed within a Coarse Filter Aggregate material (MnDOT Specification 3149.2H). Geotextile separation fabric should envelope the Coarse Filter Aggregate, separating it from the *Select Granular Material*.

It may be possible to eliminate drainage systems if it can be confirmed that sands to sands with silt will be fully present within 5-feet of top of subgrade (soils will adequately infiltrate in this case). There may be other possibilities to eliminate drainage systems, although this would need to be evaluated on a case by case method, possibly with the need for in-place soil confirmation during construction.

6.3 Foundation Support of Vertical Circulation Structures and Adjacent Stations

6.3.1 Foundation Type

In both the West Lake and Van White areas, the soil profile consists of uncontrolled fill over organic swamp deposits and/or soft clays. Foundation support over these soils is expected to undergo settlements beyond tolerable levels. Therefore, we recommend they be placed on driven pile foundation systems.

The Penn Station vertical circulation structure will also need to be supported on driven piling, although recommendations relative to that structure are included in the report pertaining to the overpass bridge and retaining wall at the Penn Kiss and Ride lot to the north of the station. We refer you to that report for specific details. As organic swamp deposits were not found below the Penn Station, it is our opinion the station platform does not require deep foundation support.

The Van White Memorial Boulevard bridge is supported on 16-inch diameter CIP steel pipe piles, having a wall thickness of 0.250 inches. Considering that bedrock or highly resistant material is not at a reasonably shallow depth, the same pile type was analyzed for the Van White vertical circulation structure. A Factored Pile Bearing Resistance value (φR_n) of up to 135 tons can be used with this pile type. The station platform also includes a substantial thickness of underlying organic soil. However, no grade raise to a slight cut is planned in the station area and the organic soils are not highly organic (organic contents in the range of 4.6% to 6.5%). Minor secondary consolidation of the deposit may occur with time, but we estimate this consolidation to be on the order of ½ inch over an extended time period (that time period being the amount of time the existing fill has already been in-place, which is presumed to be longer than the design life). Therefore, it is our opinion that station platform and track system do not need to be supported on piles.

No borings have been specifically placed for the Luce Line Trail pedestrian bridge to be constructed on the west side of the Van White Station, although there is a good probability that similar soft clays are present in this area. Therefore, for preliminary planning, we recommend assuming a bridge foundation system similar to that recommended for the Van White vertical circulation structure.

In addition to the vertical circulation structure in the West Lake area, the station platform will have some areas of minor grade raise and the buried swamp is more highly organic (peat). With this added fill/station load, the area is expected to undergo potential settlement on the order of 2 to 3 inches. Therefore, unless the station area is unloaded with lightweight fill, the station platform, track system, and continued structure attached to the vertical circulation structure should also be supported on piles. If pile supported, the station structure depth would essentially eliminate the "grade raise" zone (hence wouldn't induce settlement which could lead to downdrag loads on the piles). The use of lightweight fill could be explored during advanced design, as this may be needed to transition the pile supported track to the non-pile supported track on the southwest side. A grade raise is also planned for W 31st Street (Chowen Avenue) near the West Lake Station. Lightweight fill may again be considered if settlements are deemed excessive. The use of lightweight fill is discussed in a later section. If the lightweight fill is not incorporated, pile support of the track system may need to extend further southwest of the station, again recommended to be reviewed further during advanced design.

Although piles may approach bedrock in the West Lake station area, the bedrock lowers to the north where the vertical circulation structure will be located and overburden soils are considered sufficient for attaining resistance by means of combined tip resistance and skin friction using conventional 12-inch diameter CIP steel pipe pile. Per normal MnDOT limits, this pile can be designed for a Factored Pile Bearing Resistance value (φR_n) of up to 100 tons, assuming a pile wall thickness of 0.250 inches.

6.3.2 Pile Foundation Analysis Methods

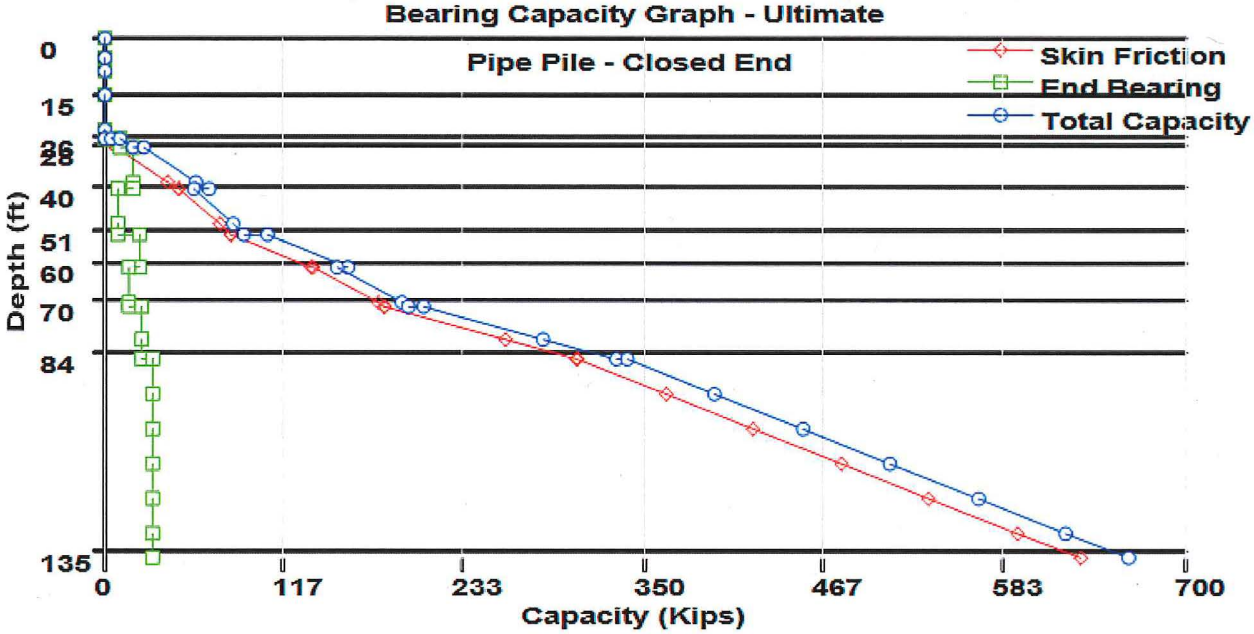
Pile bearing resistance versus pile length was analyzed using *DRIVEN* software (FHWA). This program uses the Nordlund method for granular soils and the Tomlinson method for cohesive soils. The granular soil internal friction angle used was based on its relationship to standard penetration test values as presented by Peck, Hanson, and Thorburn (1974), with the N-values being corrected for the influence of the effective overburden pressure. For cohesive soils, we estimated undrained shear strength based on correlations with the SPT data. The "ultimate capacity" determined from this *DRIVEN* analysis is considered the Nominal Resistance of Single Pile in Axial Compression (R_n) using LRFD terminology.

The nominal resistance (ultimate capacity) needed to be demonstrated in the field depends on the Resistance Factor allowed by the "Condition/Resistance Determination Method" used. A Resistance Factor (φ) of 0.65 can be used when dynamic analysis (High Strain Dynamic Pile Testing) is employed and a Resistance Factor (φ) of 0.50 should be used when field evaluation of steel pipe pile is based on the MPF12 driving formula (MnDOT's new formula). We recommend using dynamic analysis for pile evaluation on these structures.

6.3.3 Analysis Results – Van White Vertical Circulation

A 16-inch CIP steel pipe pile was analyzed for the Van White structure. Assuming a φR_n = 135 tons and a Resistance Factor of 0.65, a nominal resistance of 415 kips would then need to be demonstrated. Figure 6.3.3 presents the analysis results based on the boring drilled in the area for the original bridge construction, attached to this report as Boring 40 SB. In the analysis, we assumed no skin friction contribution from the zone of organic soils and the overlying soils, and assumed a foundation elevation of 812 feet (about 5 feet below surrounding grade). The analysis shows a pile length of 90 feet (tip elevation 722 feet) to achieve the 415 kip nominal resistance.

Figure 6.3.3 – *DRIVEN* Results, Boring 40 SB



6.3.4 Analysis Results – West Lake Vertical Circulation and Station

A 12-inch CIP steel pipe pile was analyzed for the West Lake structures. Assuming a $\varphi R_n = 100$ tons and a Resistance Factor of 0.65, a nominal resistance of 308 kips would then need to be demonstrated. Figures 6.3.4a through 6.3.4c presents the analysis results based on Borings 1228 SS, 1229 SV, and 1230 SV. In the analysis, we assumed no skin friction contribution from the zone of organic soils and the overlying soils in the station area. In the vertical circulation structure locations, we assumed skin friction contribution from both the fill and soft clays, as highly organic soils were not found to be present. Foundation elevations were assumed to be 5 feet below surrounding grade (that grade assumed to be 875 feet at Boring 1228 SS, 873 feet at Boring 1229 SV, and 870.5 at Boring 1230 SV).

Figure 6.3.4a – *DRIVEN* Results, Boring 1228 SS

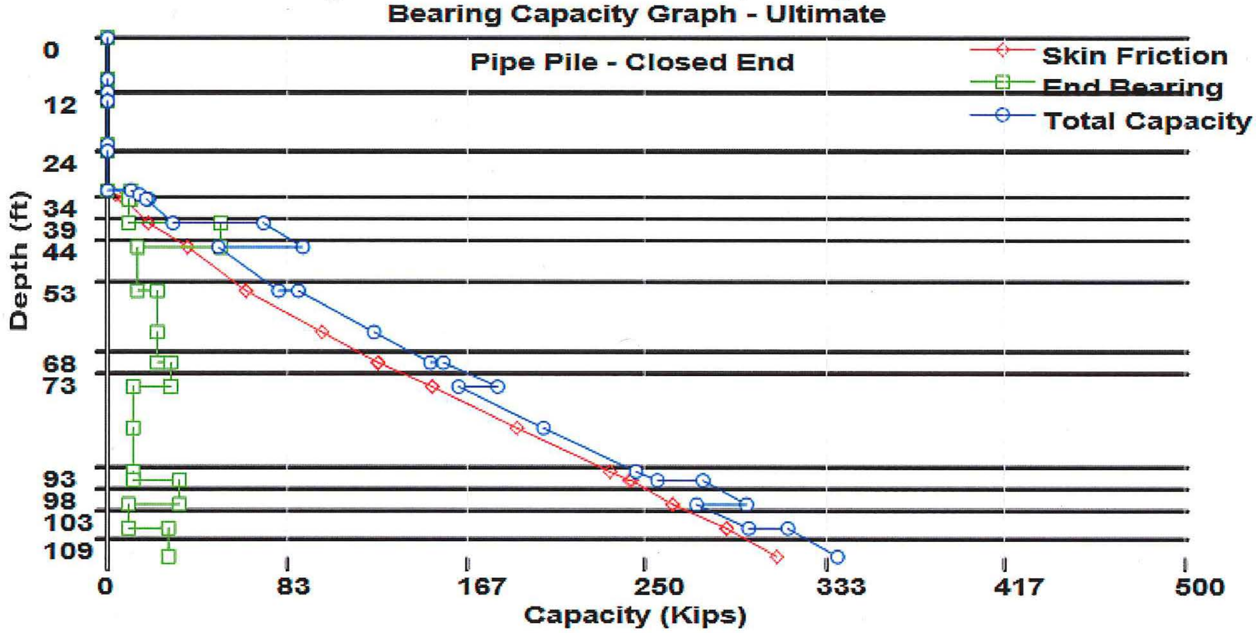


Figure 6.3.4b – *DRIVEN* Results, Boring 1229 SV

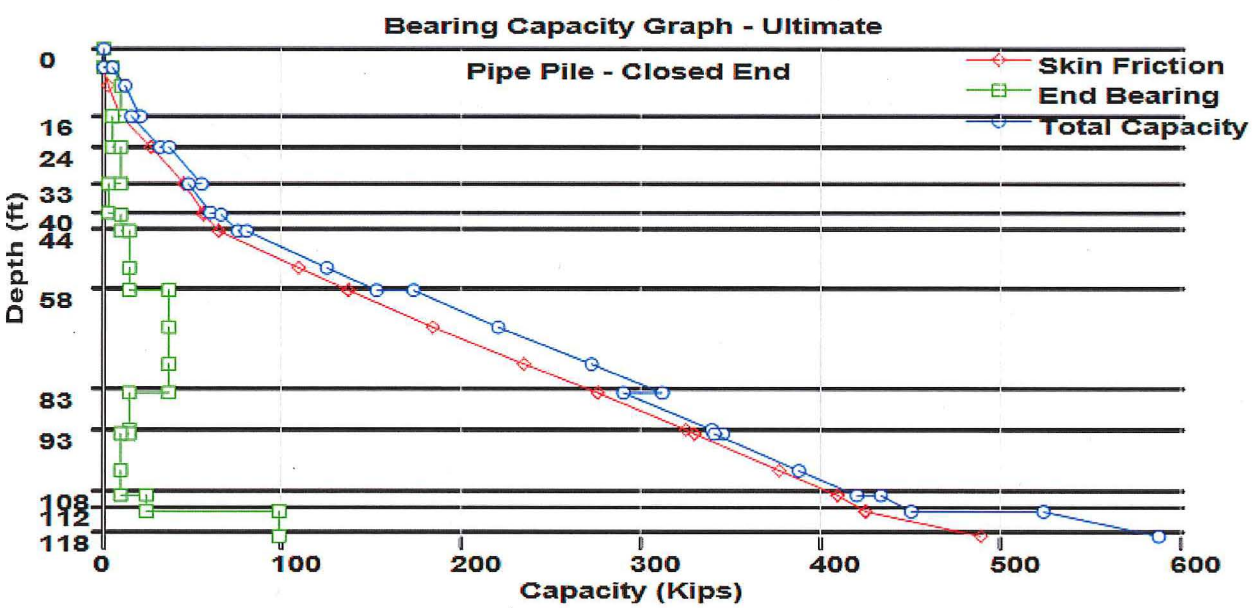
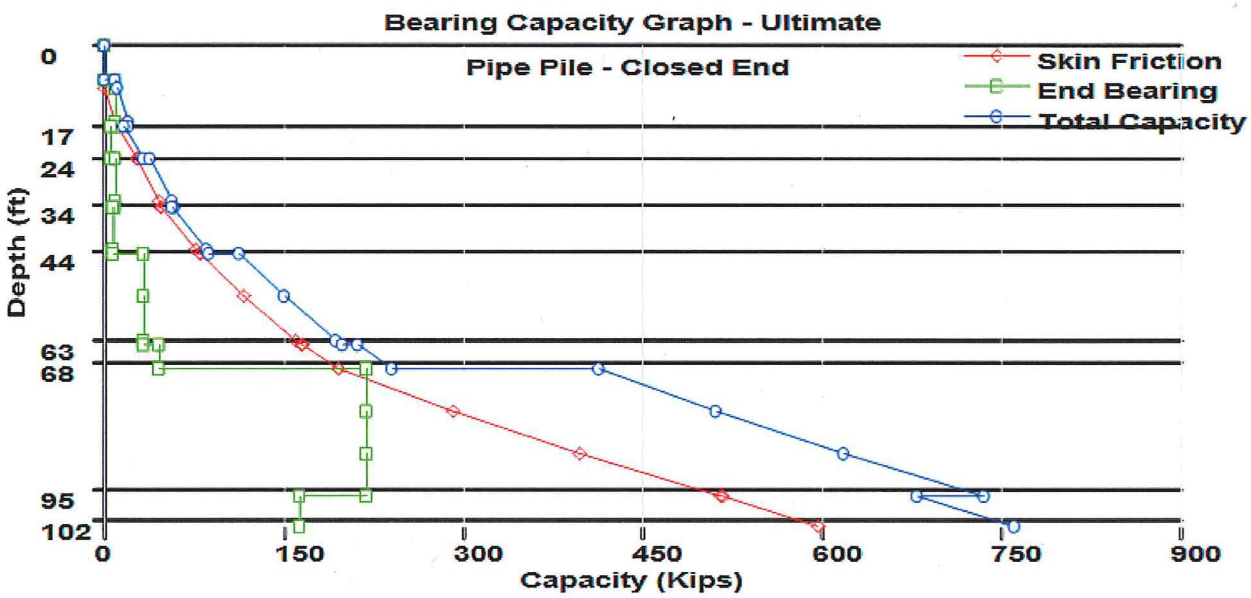


Figure 6.3.4c – *DRIVEN* Results, Boring 1230 SV



The lengths predicted by the preceding computer analyses in order to attain a nominal resistance of 308 kips are shown in Table 6.3.4. This assumes a design φR_n = 100 tons and the use of dynamic analysis for the field evaluation method (allowing φ = 0.65).

Table 6.3.4 – Estimated Pile Lengths from DRIVEN Analyses

Location	Boring Used	Assumed Bottom of Foundation Elevation, ft	Estimated Pile Tip Elevation, ft	Estimated Pile Length, ft
Station Area	1228 SS	870	772	98
Vert Circ, S side	1229 SV	868	790	78
Vert Circ, N side	1230 SV	865.5	805	61

6.3.5 Foundation Recommendations

The foundations for the Van White vertical circulation structure can be supported with 16-inch diameter CIP steel pipe piles. These piles can be designed based on a Factored Pile Bearing Resistance (φR_n) value of up to 135 tons.

The foundations for the West Lake vertical circulation structure and attached station/track structure system can be supported with 12-inch diameter CIP steel pipe piles. These piles can be designed based on a Factored Pile Bearing Resistance (φR_n) value of up to 100 tons.

The pipe piles should have a minimum yield strength (f_y) of 45 ksi and a minimum wall thickness of 0.250 inches. The pipe should be driven with a flat plate welded to the pile tip (closed end). The plate should have a minimum thickness of 0.75 inches and a diameter no greater than the pile diameter. The pipe piles should be inspected and concrete filled in accordance with MnDOT Specification 2452.D6. The minimum compressive strength of the concrete should be 3000 psi at 28-days.

The nominal resistance of the piles should be evaluated using high strain dynamic (PDA) testing, which will allow the Resistance Factor of 0.65. The dynamic testing should meet the minimum requirements listed in Section 10.5.5 of the *AASHTO LRFD Bridge Design Specifications, 2012*. This approach includes Quality Control of non-tested pile by calibrated wave equation analyses.

We refer you to previous Sections 6.3 and 6.4 for the pile lengths predicted to achieve the stated nominal resistance values. The pile lengths shown are based on the varying analysis methods discussed with assumed soil parameters, and the soil layer variations make accurate pile length predictions difficult. It is common for actual pile resistance to differ from the theoretical resistance. The actual pile lengths must be confirmed at the time of driving, and lengths may be more or less than that shown.

During field evaluation of the pile driving, it will be desired to neglect the skin friction contribution through organic swamp zones and the zone of fill above the swamp (Van White and West Lake station areas). During test pile driving, the skin friction through those zones should be determined such that it is known how much of the driving resistance should be discounted.

If piles do not achieve the required resistance at desired depths, pile driving can be stopped and time can be given to allow pile "set-up" to occur. The increase in resistance can then be rechecked with a re-strike on the following day.

It is our opinion that down drag (DD) load does not need to be considered in the pile design, as differential settlement to the track will need to be mitigated in some form (lightweight fill or continued pile support of the track to non-compressible soil areas).

A reduction factor for group effects does not need to be applied provided the pile arrangement maintains a center-to-center spacing of 3 times the diameter.

Foundations should have five or more piles for redundancy purposes. With five or more piles, a reduction factor for a lack of redundancy does not need to be applied.

Boulders or rock slabs may potentially be present within the profile. If pile penetration appears to be obstructed at abnormally variable depths (due to apparent boulders/slabs), additional pile and foundation review may be needed.

6.4 Foundation Support of Louisiana Station, Underpass, and Walls

For pertinent background information relating to this section, we refer you to the Foundation Analysis and Design Reports (FADR) prepared for the bridges over Louisiana Avenue to the west (AET No. 01-05697.07) and the South Connector freight rail bridges to the east (AET No. 01-05697.09). These structures are underlain by buried organic swamp deposits, and will need to be supported by piles driven to or near bedrock; most likely H-piles. Buried swamp deposits continue to be present beneath the Station, underpass, and associated retaining wall areas.

It is our opinion that the foundation system for the Louisiana Station, the underpass, and the associated retaining walls should be consistent with that recommended for the bridges. This includes the track system, as discussed in the referenced FADRs. We recommend the system be H-piles, as discussed in the FADR for the Louisiana Avenue bridges. The H-piles are expected to meet the required nominal resistance with "refusal" on the bedrock, which is expected to be both sandstone of the St. Peter Formation and limestone of the Platteville Formation. The anticipated pile tip elevations for the borings in the area are shown in Table 6.4.

Table 6.4 – Estimated Pile Lengths

Structure	Boring No.	Estimated Tip Elevation, ft
Freight over Louisiana	1012 SB	809
LRT over Louisiana	1213 SB	810
Approach Retaining Wall	1214 SW	824½
Approach Retaining Wall	1215 SW	822
Station Platform	1220 SS	822½
Underpass, south end	1221 SU	826
Underpass, north end	1222 SU	822½

Please refer to the FADR for the Louisiana Avenue bridges for more specific details regarding the pile foundation system, retaining wall backfilling needs, and track support.

6.5 21st Street Station Lightweight Fill Option

A buried swamp deposit is also present in the 21st Street Station area and in the track area to the south. This station does not include a vertical circulation structure, such that new loads imposed are somewhat lighter; primarily related to the station platform, items supported on the platform, and any new fill increase loads. The area will have several feet of fill added to attain rail grades, with the platform being higher than this. We estimate settlements on the order of 3 inches to 5 inches could occur as a result of these added loads.

It would be possible to avoid pile foundation support of the station and affected track by replacing heavier underlying mineral fill with lightweight fill such that the deeper compressible soils do not experience a load increase. The lightweight fill could be expanded polystyrene (geofoam) rigid blocks or foamed concrete. Considering that geofoam only weighs about 1 to 2 pcf, versus a density on the order of 25 pcf for foamed concrete, geofoam is most often used. The volume of geofoam would only need to be slightly greater than the volume of the new material (soil, concrete, equipment, etc.) placed above the existing grade. The geofoam should be buried beneath about 4 feet of total overburden material (i.e., ballast, subballast, and Select Granular Material). The geofoam should also be directly covered by an EPS Geomembrane, which is a flexible, impervious membrane intended to prevent damage from petroleum-based materials. The segment needing this treatment is from Station 2816+50 to 2824+00. The specific thickness and cross-sectional extent should be further refined during advanced design if this approach is to be used.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Excavation Backsloping

Where excavation faces are not retained, the excavations should maintain maximum allowable slopes in accordance with *OSHA Regulations (Standards 29 CFR), Part 1926, Subpart P, "Excavations"* (can be found on www.osha.gov). Even with the required OSHA sloping, water seepage or surface runoff can potentially induce side-slope erosion or running which could require slope maintenance. The responsibility for excavation face maintenance in accordance with OSHA requirements should lie with the contractor, and we recommend the construction documents be prepared as such.

7.2 Observation and Testing

The recommendations in this report are based on the subsurface conditions found at the boring/CPT locations. Since the soil conditions can be expected to vary away from the soil boring locations, we recommend on-site observation by an AET geotechnical engineer or technician during construction to evaluate these potential changes.

Sieve analysis tests should be performed on engineered fill in order to document that materials used meet the intended gradation specifications.

Soil density and Proctor testing should be performed on new fill placed in order to document that project specifications for compaction have been satisfied. If on-site soils are to be re-used, we recommend the fill placement and compaction be monitored on a full-time basis.

8.0 LIMITATIONS

Within the limitations of scope, budget, and schedule, our services have been conducted according to generally accepted geotechnical engineering practices at this time and location. Other than this, no warranty, either express or implied, is intended.

Important information regarding risk management and proper use of this report is given in Appendix C entitled "Geotechnical Report Limitations and Guidelines for Use."

Appendix A

Geotechnical Field Exploration and Testing
Boring Log Notes
Unified Soil Classification System
AASHTO Soil Classification System
Figures 1 to 18– Boring Locations
Table A – Select Granular and Compaction Subcut Needs

Appendix A
Geotechnical Field Exploration and Testing
Report No. 01-05697.01

A.1 FIELD EXPLORATION

The subsurface conditions for the project components contained herein were explored by drilling and sampling 156 standard penetration test (SPT) borings and conducting four piezocone penetration test (CPT_u) soundings. The test locations appear on Figures 1 to 18 contained in this appendix.

A.2 SOIL BORING SAMPLING METHODS

A.2.1 Split-Spoon Samples (SS) - Calibrated to N_{60} Values

Standard penetration (split-spoon) samples were collected in general accordance with ASTM:D1586 with one primary modification. The ASTM test method consists of driving a 2-inch O.D. split-barrel sampler into the in-situ soil with a 140-pound hammer dropped from a height of 30 inches. The sampler is driven a total of 18 inches into the soil. After an initial set of 6 inches, the number of hammer blows to drive the sampler the final 12 inches is known as the standard penetration resistance or N-value. Our method uses a modified hammer weight, which is determined by measuring the system energy using a Pile Driving Analyzer (PDA) and an instrumented rod.

In the past, standard penetration N-value tests were performed using a rope and cathead for the lift and drop system. The energy transferred to the split-spoon sampler was typically limited to about 60% of its potential energy due to the friction inherent in this system. This converted energy then provides what is known as an N_{60} blow count.

Most newer drill rigs incorporate an automatic hammer lift and drop system, which has higher energy efficiency and subsequently results in lower N-values than the traditional N_{60} values. By using the PDA energy measurement equipment, we are able to determine actual energy generated by the drop hammer. With the various hammer systems available, we have found highly variable energies ranging from 55% to over 100%. Therefore, the intent of AET's hammer calibrations is to vary the hammer weight such that hammer energies lie within about 60% to 65% of the theoretical energy of a 140-pound weight falling 30 inches. The current ASTM procedure acknowledges the wide variation in N-values, stating that N-values of 100% or more have been observed. Although we have not yet determined the statistical measurement uncertainty of our calibrated method to date, we can state that the accuracy deviation of the N-values using this method is significantly better than the standard ASTM Method.

A.2.2 Disturbed Samples (DS)/Spin-up Samples (SU)

Sample types described as "DS" or "SU" on the boring logs are disturbed samples, which are taken from the flights of the auger. Because the auger disturbs the samples, possible soil layering and contact depths should be considered approximate.

A.2.3 Sampling Limitations

Unless actually observed in a sample, contacts between soil layers are estimated based on the spacing of samples and the action of drilling tools. Cobbles, boulders, and other large objects generally cannot be recovered from test borings, and they may be present in the ground even if they are not noted on the boring logs.

A.3 SOIL BORING CLASSIFICATION METHODS

Soil descriptions shown on the boring logs are based on the Unified Soil Classification (USC) system. The USC system is described in ASTM:D2487 and D2488. Where laboratory classification tests (sieve analysis or Atterberg Limits) have been performed, accurate classifications per ASTM:D2487 are possible. Otherwise, soil descriptions shown on the boring logs are visual-manual judgments. Charts are attached which provide information on the USC system, the descriptive terminology, and the symbols used on the boring logs.

Visual-manual judgment of the AASHTO Soil Group is also noted as a part of the soil description. A chart presenting details of the AASHTO Soil Classification System is also attached.

The boring logs include descriptions of apparent geology. The geologic depositional origin of each soil layer is interpreted primarily by observation of the soil samples, which can be limited. Observations of the surrounding topography, vegetation, and development can sometimes aid this judgment.

A.4 SOIL BORING WATER LEVEL MEASUREMENTS

The ground water level measurements are shown at the bottom of the boring logs. The following information appears under "Water Level Measurements" on the logs:

- Date and Time of measurement
- Sampled Depth: lowest depth of soil sampling at the time of measurement
- Casing Depth: depth to bottom of casing or hollow-stem auger at time of measurement
- Cave-in Depth: depth at which measuring tape stops in the borehole
- Water Level: depth in the borehole where free water is encountered
- Drilling Fluid Level: same as Water Level, except that the liquid in the borehole is drilling fluid

The true location of the water table at the boring locations may be different than the water levels measured in the boreholes. This is possible because there are several factors that can affect the water level measurements in the borehole. Some of these factors include: permeability of each soil layer in profile, presence of perched water, amount of time between water level readings, presence of drilling fluid, weather conditions, and use of borehole casing.

A.5 LABORATORY TEST METHODS

A.5.1 Water Content Tests

Conducted in general accordance with ASTM:D2216.

A.5.2 Sieve Analysis of Soils (thru #200 Sieve)

Conducted in general conformance with ASTM:D6913, Method A.

A.5.3 Organic Content Tests

Conducted in general conformance with ASTM:D2216, Method B.

A.5.4 Atterberg Limits Test

Conducted in general conformance with ASTM:D4318.

A.5.5 Unconfined Compressive Strength of Cohesive Soil

Conducted in general accordance with ASTM:D2166. Dry density is also determined during this test (sample is trimmed to known diameter and height).

A.5.6 One-Dimensional Consolidation Testing

Conducted in general conformance with ASTM:D2435.

A.6 TEST STANDARD LIMITATIONS

Field and laboratory testing is done in general conformance with the described procedures. Compliance with any other standards referenced within the specified standard is neither inferred nor implied.

A.7 SAMPLE STORAGE

Unless notified to do otherwise, we routinely retain representative samples of the soils recovered from the borings for a period of 30 days.

A.8 PIEZOCONE PENETRATION TEST (CPT_u) METHODS

The test method is described in ASTM: D5778. This cone test method determines the resistance to penetration of a conical pointed penetrometer and the frictional resistance of a cylindrical sleeve located behind the conical point as the cone is advanced through subsurface soils at a slow and steady rate. The piezocone adds the measurement of pore pressure development behind the tip. The equipment provides a detailed record of cone resistance which is useful for evaluation of site stratigraphy, homogeneity and depth to firm layers, voids or cavities, and other discontinuities. In addition, the cone resistance and friction data can be used to estimate soil classification, and correlations with engineering properties of soils. The pore pressure readings also provide information on soil type and water table depth. Pore pressure dissipation, after a push, can also be monitored for correlation to soil consolidation and permeability. Therefore, the test provides a rapid means for determining subsurface conditions, and can be used for estimating engineering properties of soils for structures, and the behavior of soils under static and dynamic loads.

During the testing, a penetrometer tip with a conical point having a 60° apex angle and a cone base area of 10 cm^2 or 15cm^2 is advanced through the soil at a constant rate of 2 cm/sec. The friction sleeve is present on the penetrometer immediately behind the cone tip. The forces exerted on the conical point (cone) and the friction sleeve required to penetrate the soil are measured by electrical methods, at every 2 cm of penetration. The cone resistance (q_t) is calculated by dividing the measured total cone force by the cone base area. The friction sleeve resistance (f_s) is obtained by dividing the measured force exerted on the sleeve by its surface area. Pore pressure is measured directly behind the cone (U_2 position).

A.9 SEISMIC PIEZOCONE METHODS

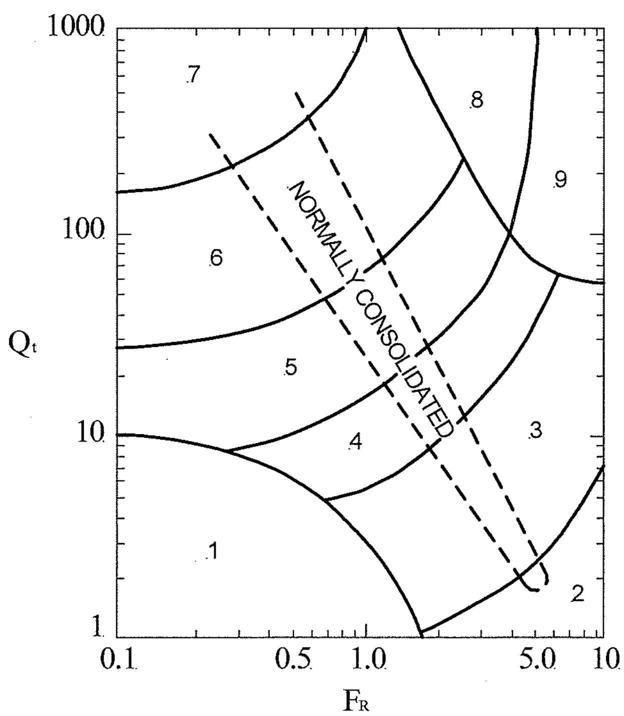
A seismic cone is similar to the standard piezocone (described above) with geophones added to the cone. During a normal piezocone sounding (where tip resistance, sleeve friction and pore pressure are measured), the sounding is paused at various depths where shear (S) wave velocities are measured. The S wave source is a wooded beam pressed against the ground surface. The S waves are generated by striking the beam with a hammer with an electronic trigger. The measured S wave velocities can be used to evaluate the stress-strain modulus of the various soil layers.

A.9 CPT_u SOIL BEHAVIOR TYPE

Soil Classification methods for the Cone Penetration Test is based on correlation charts developed from observations of CPT data and conventional borings. Please note that these classification charts are meant to provide a, guide to Soil Behavior Type and should not be used to infer a soil classification based on grain size distribution.

The following chart is used to provide a Soil Behavior Type of the CPT Data.

Figure 1: Robertson CPT 1990 (Soil Behavior Type based on Friction Ratio)



The numbers corresponding to different regions on the Charts represent the following soil behavior types:

1. Sensitive, Fine Grained
2. Organic Soils - Peats
3. Clays - Clay to Silty Clay
4. Silt Mixtures - Clayey Silt to Silty Clay
5. Sand Mixtures - Silty Sand to Sandy Silt
6. Sands - Clean Sand to Silty Sand
7. Gravelly Sand to Sand
8. Very Stiff Sand to Clayey Sand
9. Very Stiff, Fine Grained

$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \quad F_R = \frac{f_s}{q_t - \sigma_{vo}} \times 100\%$$

where . . .
Q_T...........normalized cone resistance
F_R...........normalized friction ratio

Note that engineering judgment and comparison with conventional borings is especially important in the proper interpretation of CPT data in certain geo-materials.

BORING LOG NOTES

DRILLING AND SAMPLING SYMBOLS

Symbol	Definition
AR:	Sample of material obtained from cuttings blown out the top of the borehole during air rotary procedure.
B, H, N:	Size of flush-joint casing
CAS:	Pipe casing, number indicates nominal diameter in inches
COT:	Clean-out tube
DC:	Drive casing; number indicates diameter in inches
DM:	Drilling mud or bentonite slurry
DR:	Driller (initials)
DS:	Disturbed sample from auger flights
DP:	Direct push drilling; a 2.125 inch OD outer casing with an inner 1½ inch ID plastic tube is driven continuously into the ground.
FA:	Flight auger; number indicates outside diameter in inches
HA:	Hand auger; number indicates outside diameter
HSA:	Hollow stem auger; number indicates inside diameter in inches
LG:	Field logger (initials)
MC:	Column used to describe moisture condition of samples and for the ground water level symbols
N (BPF):	Standard penetration resistance (N-value) in blows per foot (see notes)
NQ:	NQ wireline core barrel
PQ:	PQ wireline core barrel
RDA:	Rotary drilling with compressed air and roller or drag bit.
RDF:	Rotary drilling with drilling fluid and roller or drag bit
REC:	In split-spoon (see notes), direct push and thin-walled tube sampling, the recovered length (in inches) of sample. In rock coring, the length of core recovered (expressed as percent of the total core run). Zero indicates no sample recovered.
SS:	Standard split-spoon sampler (steel; 1.5" is inside diameter; 2" outside diameter); unless indicated otherwise
SU	Spin-up sample from hollow stem auger
TW:	Thin-walled tube; number indicates inside diameter in inches
WASH:	Sample of material obtained by screening returning rotary drilling fluid or by which has collected inside the borehole after "falling" through drilling fluid
WH:	Sampler advanced by static weight of drill rod and hammer
WR:	Sampler advanced by static weight of drill rod
94mm:	94 millimeter wireline core barrel
▼:	Water level directly measured in boring
▽:	Estimated water level based solely on sample appearance

TEST SYMBOLS

Symbol	Definition
CONS:	One-dimensional consolidation test
DEN:	Dry density, pcf
DST:	Direct shear test
E:	Pressuremeter Modulus, tsf
HYD:	Hydrometer analysis
LL:	Liquid Limit, %
LP:	Pressuremeter Limit Pressure, tsf
OC:	Organic Content, %
PERM:	Coefficient of permeability (K) test; F - Field; L - Laboratory
PL:	Plastic Limit, %
q_p:	Pocket Penetrometer strength, tsf (approximate)
q_c:	Static cone bearing pressure, tsf
q_u:	Unconfined compressive strength, psf
R:	Electrical Resistivity, ohm-cms
RQD:	Rock Quality Designation of Rock Core, in percent (aggregate length of core pieces 4" or more in length as a percent of total core run)
SA:	Sieve analysis
TRX:	Triaxial compression test
VSR:	Vane shear strength, remolded (field), psf
VSU:	Vane shear strength, undisturbed (field), psf
WC:	Water content, as percent of dry weight
%-200:	Percent of material finer than #200 sieve

STANDARD PENETRATION TEST NOTES (Calibrated Hammer Weight)

The standard penetration test consists of driving a split-spoon sampler with a drop hammer (calibrated weight varies to provide N_{60} values) and counting the number of blows applied in each of three 6" increments of penetration. If the sampler is driven less than 18" (usually in highly resistant material), permitted in ASTM: D1586, the blows for each complete 6" increment and for each partial increment is on the boring log. For partial increments, the number of blows is shown to the nearest 0.1' below the slash.

The length of sample recovered, as shown on the "REC" column, may be greater than the distance indicated in the N column. The disparity is because the N-value is recorded below the initial 6" set (unless partial penetration defined in ASTM: D1586 is encountered) whereas the length of sample recovered is for the entire sampler drive (which may even extend more than 18").

UNIFIED SOIL CLASSIFICATION SYSTEM
ASTM Designations: D 2487, D2488

AMERICAN ENGINEERING TESTING, INC.

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification: Group Symbol	Soil Classification: Group Name[B]
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]	GW	Well graded gravel[F]
			$Cu<4$ and/or $1>Cc>3$[E]	GP	Poorly graded gravel[F]
		Gravels with Fines more than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel[F,G,H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]	SW	Well-graded sand[I]
			$Cu<6$ and/or $1>Cc>3$[E]	SP	Poorly-graded sand[I]
		Sands with Fines more than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G,H,I]
			Fines classify as CL or CH	SC	Clayey sand[G,H,I]
Fine-Grained Soils 50% or more passes the No. 200 sieve (see Plasticity Chart below)	Silts and Clays Liquid limit less than 50	inorganic	PI>7 and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
			PI<4 or plots below "A" line[J]	ML	Silt[K,L,M]
		organic	Liquid limit–oven dried / Liquid limit – not dried <0.75	OL	Organic clay[K,L,M,N] Organic silt[K,L,M,O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	Fat clay[K,L,M]
			PI plots below "A" line	MH	Elastic silt[K,L,M]
		organic	Liquid limit–oven dried / Liquid limit – not dried <0.75	OH	Organic clay[K,L,M,P] Organic silt[K,L,M,Q]
Highly organic soil			Primarily organic matter, dark in color, and organic in odor	PT	Peat[R]

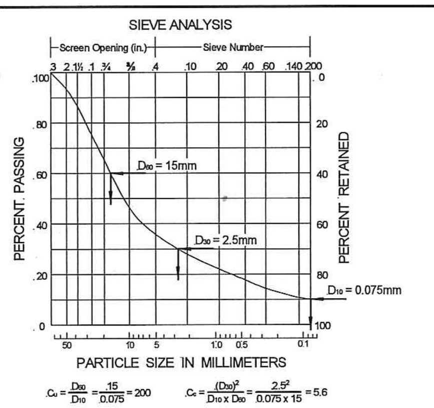
Notes

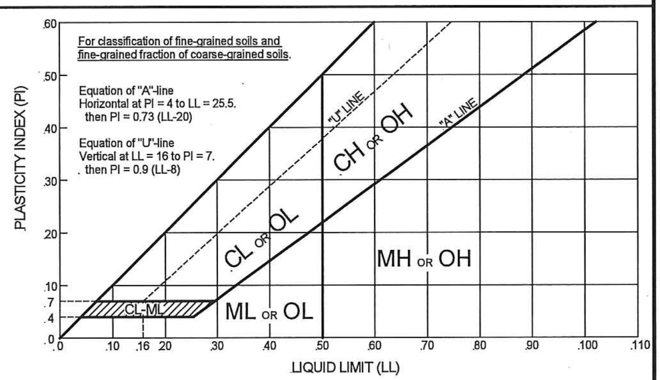
[A]Based on the material passing the 3-in (75-mm) sieve.
[B]If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C]Gravels with 5 to 12% fines require dual symbols:
- GW-GM well-graded gravel with silt
- GW-GC well-graded gravel with clay
- GP-GM poorly graded gravel with silt
- GP-GC poorly graded gravel with clay

[D]Sands with 5 to 12% fines require dual symbols:
- SW-SM well-graded sand with silt
- SW-SC well-graded sand with clay
- SP-SM poorly graded sand with silt
- SP-SC poorly graded sand with clay

[E]$Cu = D_{60} / D_{10}$, $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[F]If soil contains ≥15% sand, add "with sand" to group name.
[G]If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H]If fines are organic, add "with organic fines" to group name.
[I]If soil contains ≥15% gravel, add "with gravel" to group name.
[J]If Atterberg limits plot is hatched area, soils is a CL-ML silty clay.
[K]If soil contains 15 to 29% plus No. 200 add "with sand" or "with gravel", whichever is predominant.
[L]If soil contains ≥30% plus No. 200, predominantly sand, add "sandy" to group name.
[M]If soil contains ≥30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N]PI≥4 and plots on or above "A" line.
[O]PI<4 or plots below "A" line.
[P]PI plots on or above "A" line.
[Q]PI plots below "A" line.
[R]Fiber Content description shown below.





Plasticity Chart

ADDITIONAL TERMINOLOGY NOTES USED BY AET FOR SOIL IDENTIFICATION AND DESCRIPTION

Grain Size

Term	Particle Size
Boulders	Over 12"
Cobbles	3" to 12"
Gravel	#4 sieve to 3"
Sand	#200 to #4 sieve
Fines (silt & clay)	Pass #200 sieve

Gravel Percentages

Term	Percent
A Little Gravel	3% - 14%
With Gravel	15% - 29%
Gravelly	30% - 50%

Consistency of Plastic Soils

Term	N-Value, BPF
Very Soft	less than 2
Soft	2 - 4
Firm	5 - 8
Stiff	9 - 15
Very Stiff	16 - 30
Hard	Greater than 30

Relative Density of Non-Plastic Soils

Term	N-Value, BPF
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	Greater than 50

Moisture/Frost Condition
(MC Column)

D (Dry):	Absence of moisture, dusty, dry to touch.
M (Moist):	Damp, although free water not visible. Soil may still have a high water content (over "optimum").
W (Wet/ Waterbearing):	Free water visible intended to describe non-plastic soils. Waterbearing usually relates to sands and sand with silt.
F (Frozen):	Soil frozen

Layering Notes

Laminations:	Layers less than ½" thick of differing material or color.
Lenses:	Pockets or layers greater than ½" thick of differing material or color.

Peat Description

Term	Fiber Content (Visual Estimate)
Fibric Peat:	Greater than 67%
Hemic Peat:	33 – 67%
Sapric Peat:	Less than 33%

Organic Description (if no lab tests)

Soils are described as *organic*, if soil is not peat and is judged to have sufficient organic fines content to influence the Liquid Limit properties. *Slightly organic* used for borderline cases.

Root Inclusions

With roots: Judged to have sufficient quantity of roots to influence the soil properties.

Trace roots: Small roots present, but not judged to be in sufficient quantity to significantly affect soil properties.

AASHTO SOIL CLASSIFICATION SYSTEM

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS

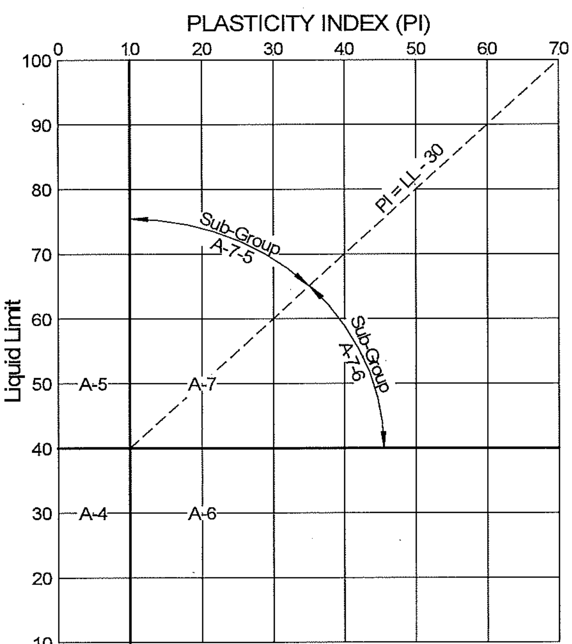
Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35% or less passing No. 200 sieve)							Silt-Clay Materials (More than 35% passing No. 200 sieve)			
Group Classification	A-1			A-2							A-7
	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5 A-7-6
Sieve Analysis, Percent passing:											
No. 10 (2.00 mm)	50 max.	. . . .	. . . .	. . . .	. . . .	. . . .	. . . .	. . . .	. . . .	. . . .	. . . .
No. 40 (0.425 mm)	30 max.	50 max.	51 min.	. . . .	. . . .	. . . .	. . . .	. . . .	. . . .	. . . .	. . . .
No. 200 (0.075 mm)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of Fraction Passing No. 40 (0.425 mm)											
Liquid limit	. . . .		. . . .	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.
Usual Types of Significant Constituent Materials	Stone Fragments, Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Ratings as Subgrade	Excellent to Good							Fair to Poor			

The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30.

Group A-8 soils are organic clays or peat with organic content >5%.



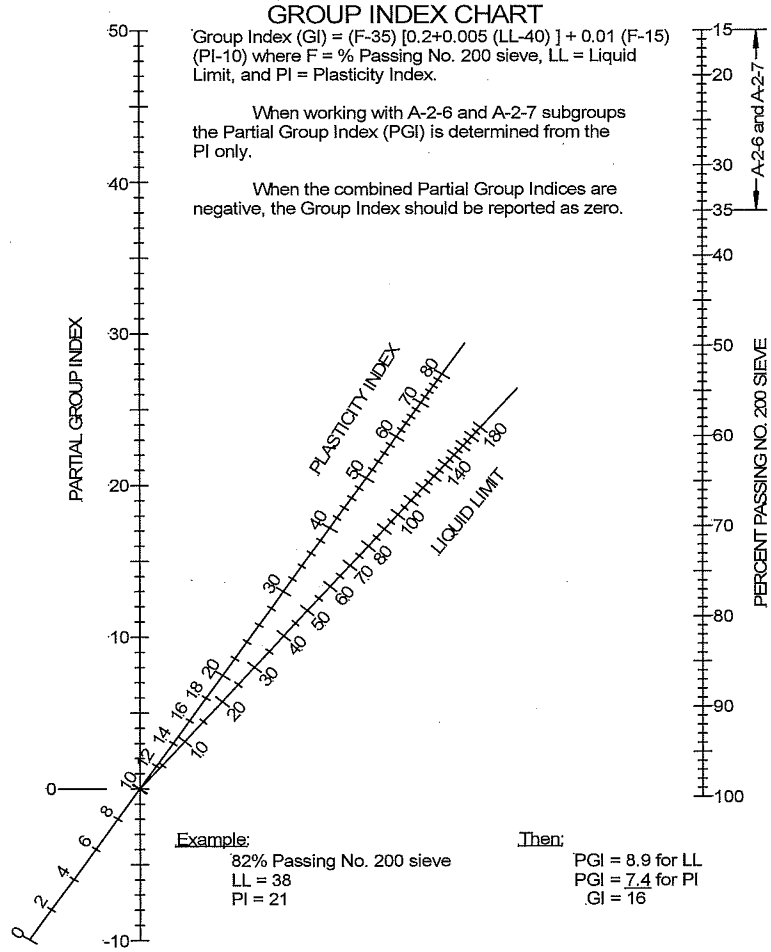
Liquid Limit and Plasticity Index Ranges for the A-4, A-5, A-6 and A-7 Subgroups

GROUP INDEX CHART

Group Index (GI) = (F-35) [0.2+0.005 (LL-40)] + 0.01 (F-15) (PI-10) where F = % Passing No. 200 sieve, LL = Liquid Limit, and PI = Plasticity Index.

When working with A-2-6 and A-2-7 subgroups the Partial Group Index (PGI) is determined from the PI only.

When the combined Partial Group Indices are negative, the Group Index should be reported as zero.



Example:
82% Passing No. 200 sieve
LL = 38
PI = 21

Then:
PGI = 8.9 for LL
PGI = 7.4 for PI
GI = 16

Definitions of Gravel, Sand and Silt-Clay

The terms "gravel", "coarse sand", "fine sand" and "silt-clay", as determinable from the minimum test data required in this classification arrangement and as used in subsequent word descriptions are defined as follows:

GRAVEL - Material passing sieve with 3-in. square openings and retained on the No. 10 sieve.

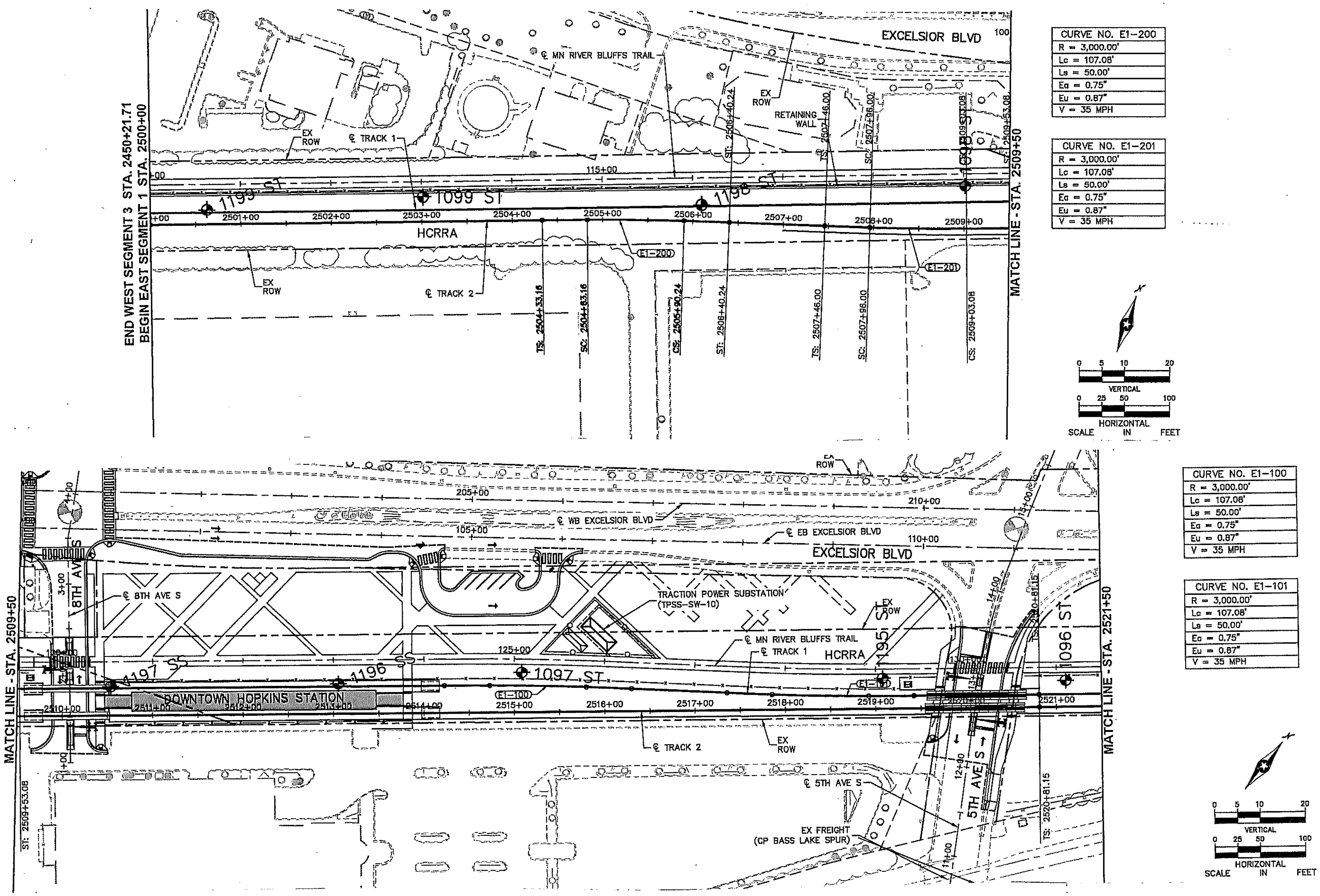
COARSE SAND - Material passing the No. 10 sieve and retained on the No. 40 sieve.

FINE SAND - Material passing the No. 40 sieve and retained on the No. 200 sieve.

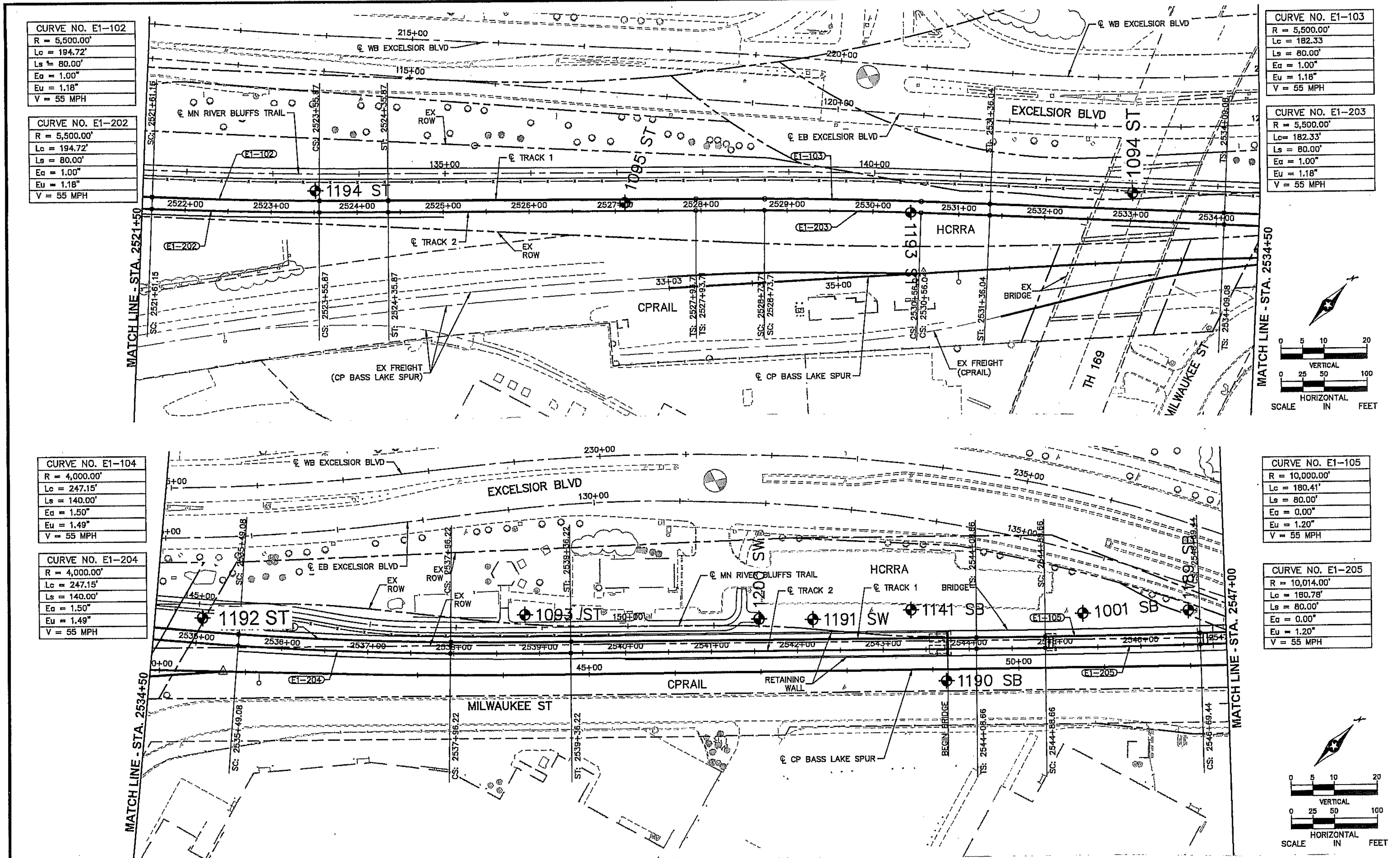
COMBINED SILT AND CLAY - Material passing the No. 200 sieve

BOULDERS (retained on 3-in. sieve) should be excluded from the portion of the sample to which the classificaiton is applied, but the percentage of such material, if any, in the sample should be recorded.

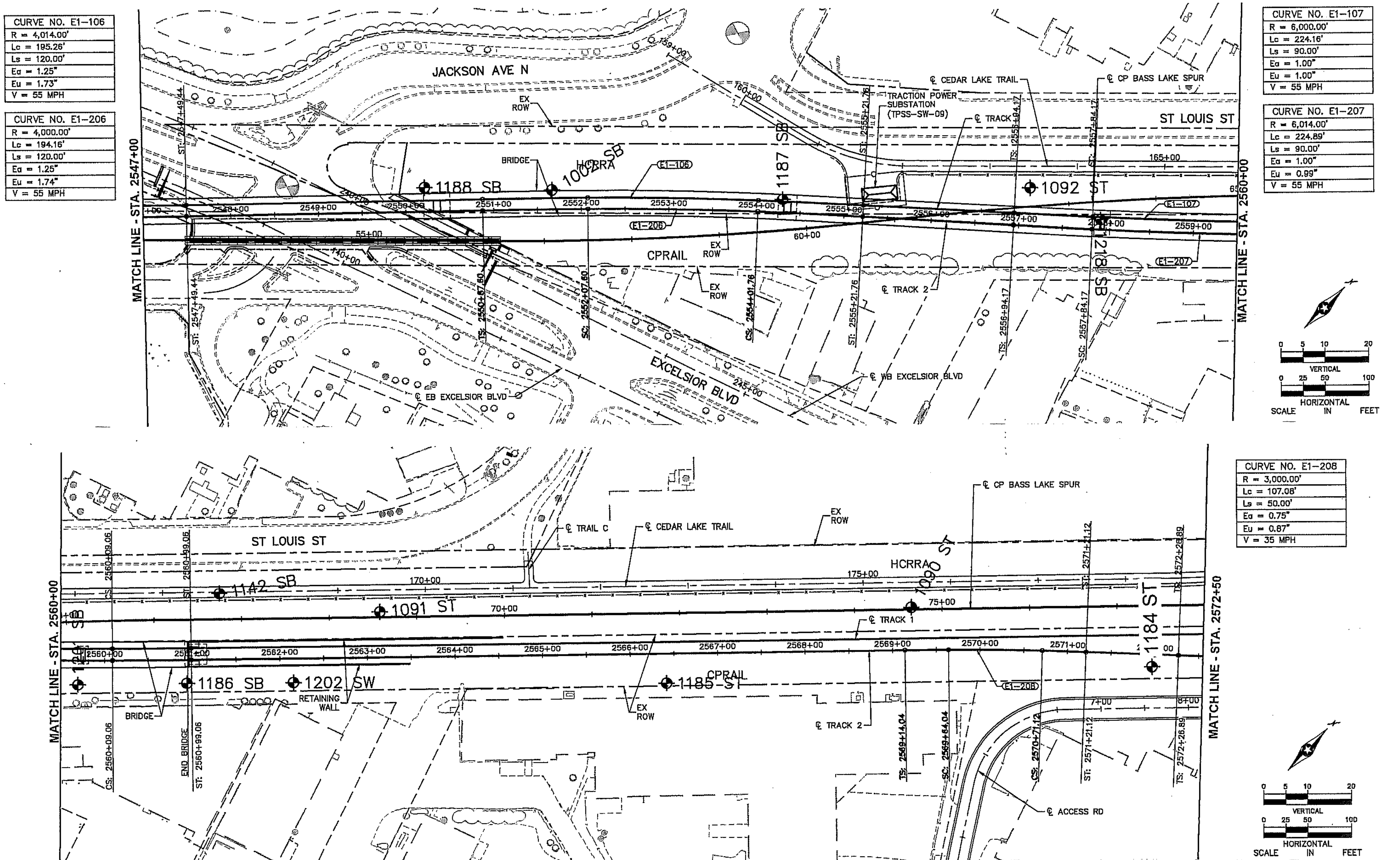
The term "silty" is applied to fine material having plasticity index of 10 or less and the term "clayey" is applied to fine material having plasticity index of 11 or greater.



AMERICAN ENGINEERING TESTING, INC.			
PROJECT Southwest LRT, PEC East			**AET NO.** 01-05697.01
SUBJECT Boring Locations, Station 2500+00 to 2521+50			**DATE** June 22, 2014
SCALE 1" = 160'±	**PREPARED BY** KHA/JV	**CHECKED BY** JV	**FIGURE 1**

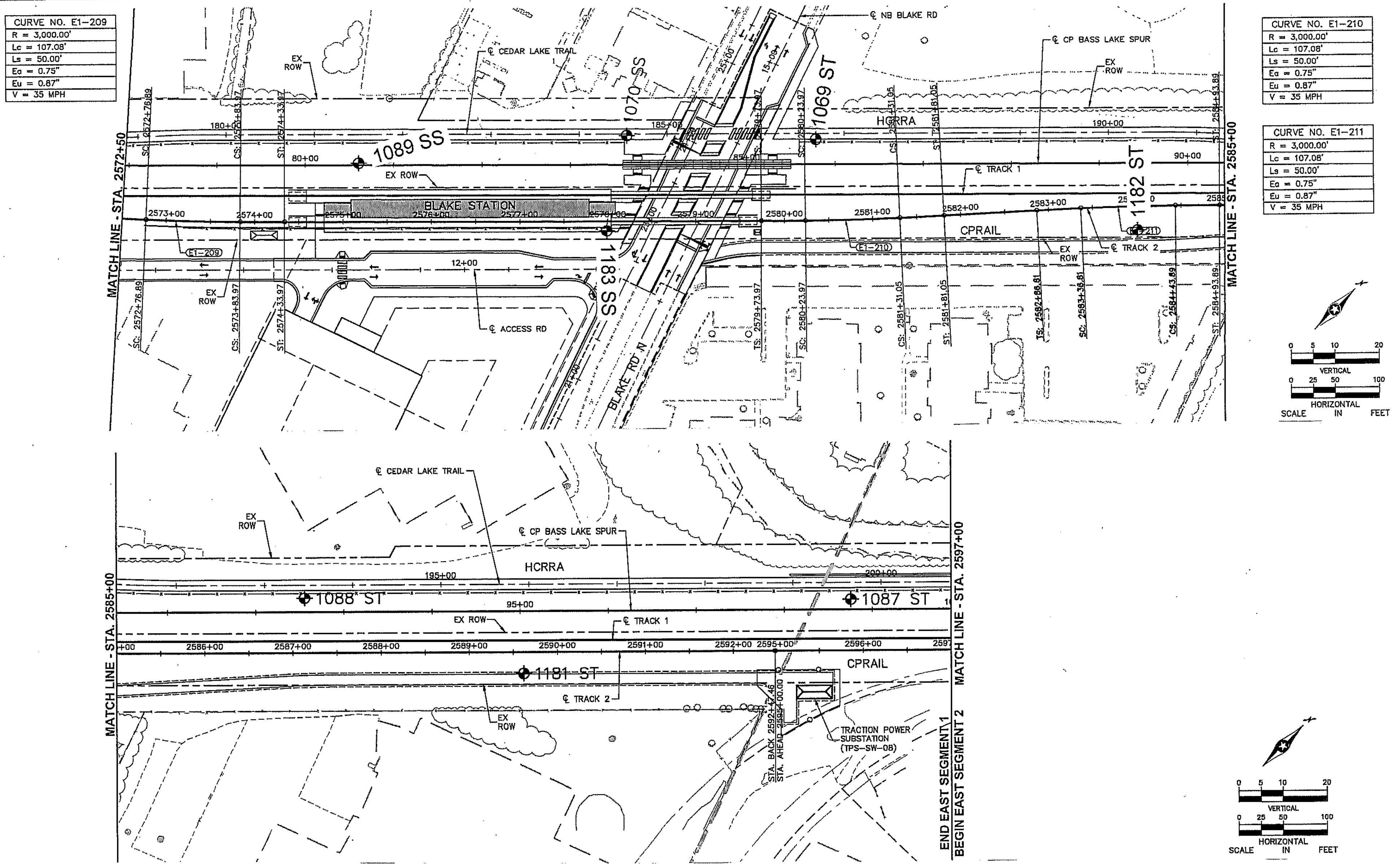


AMERICAN ENGINEERING TESTING, INC.	**PROJECT** Southwest LRT, PEC East	**AET NO.** 01-05697.01
	SUBJECT Boring Locations, Station 2521+50 to 2547+00	**DATE** June 22, 2014
	SCALE 1" = 160'± / **PREPARED BY** KHA/JV / **CHECKED BY** JV	**FIGURE 2**

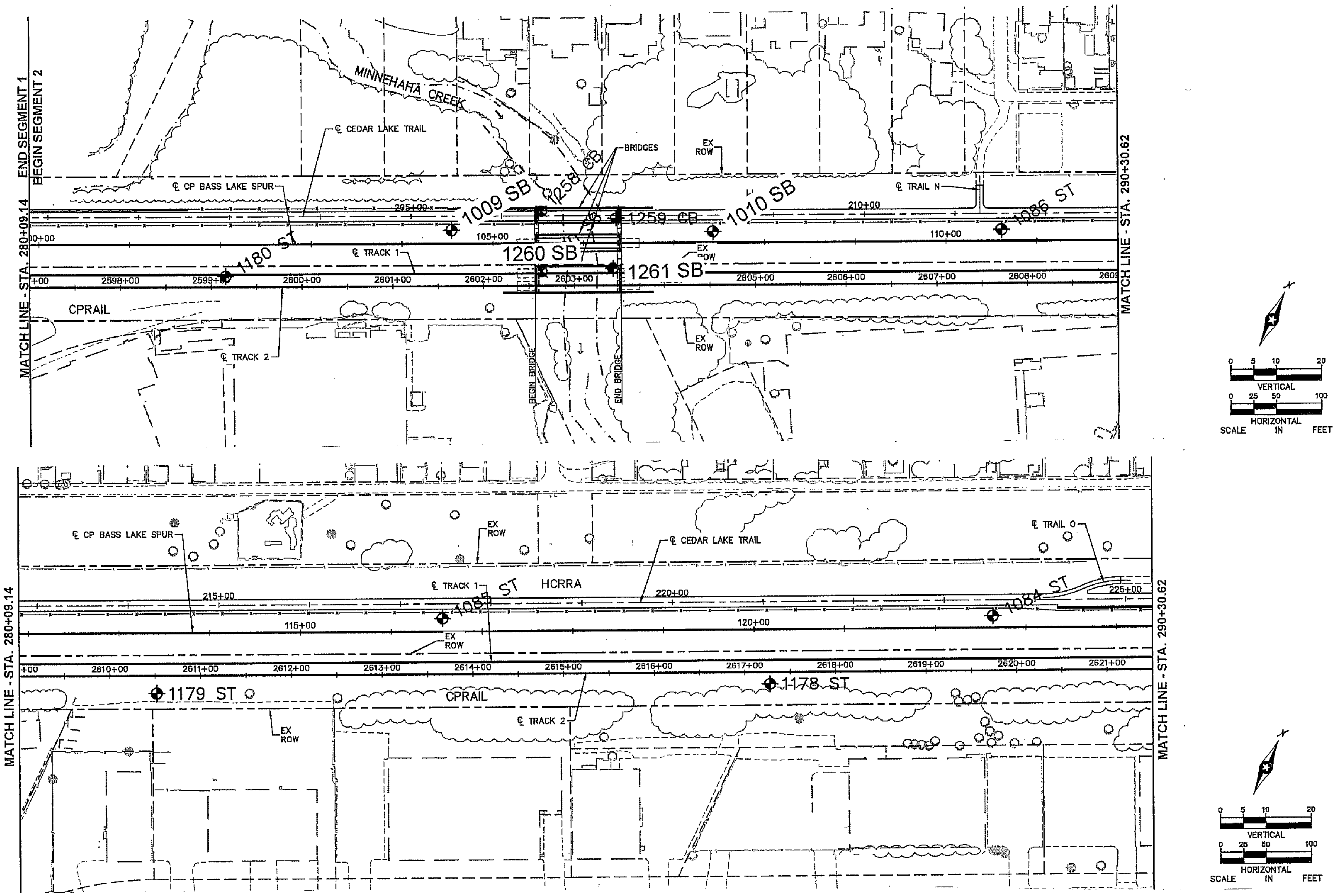


AMERICAN ENGINEERING TESTING, INC.

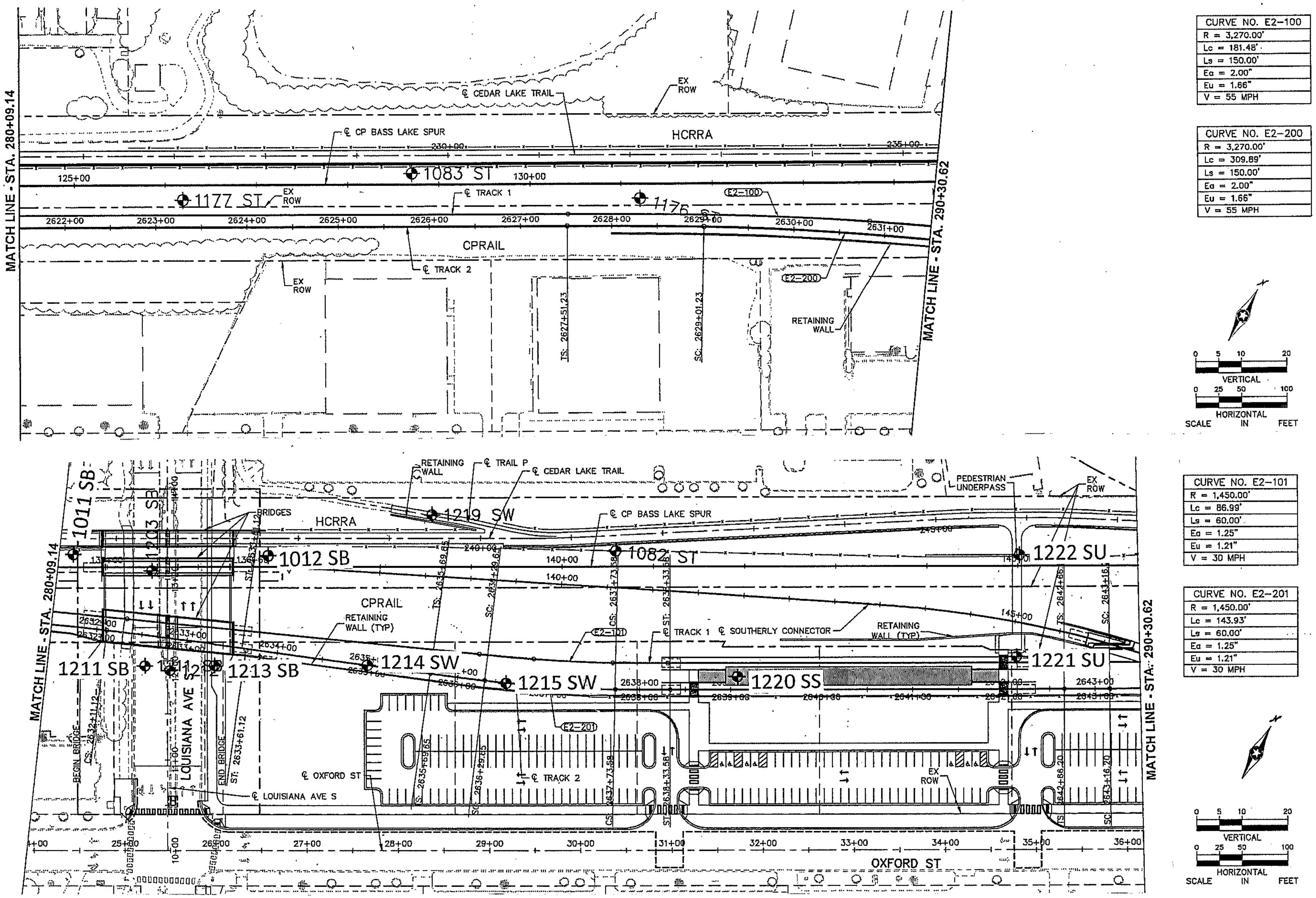
PROJECT Southwest LRT, PEC East		**AET NO.** 01-05697.01
SUBJECT Boring Locations, Station 2547+00 to 2572+50		**DATE** June 22, 2014
SCALE 1" = 160'±	**PREPARED BY** KHA/JV	**CHECKED BY** JV
		FIGURE 3



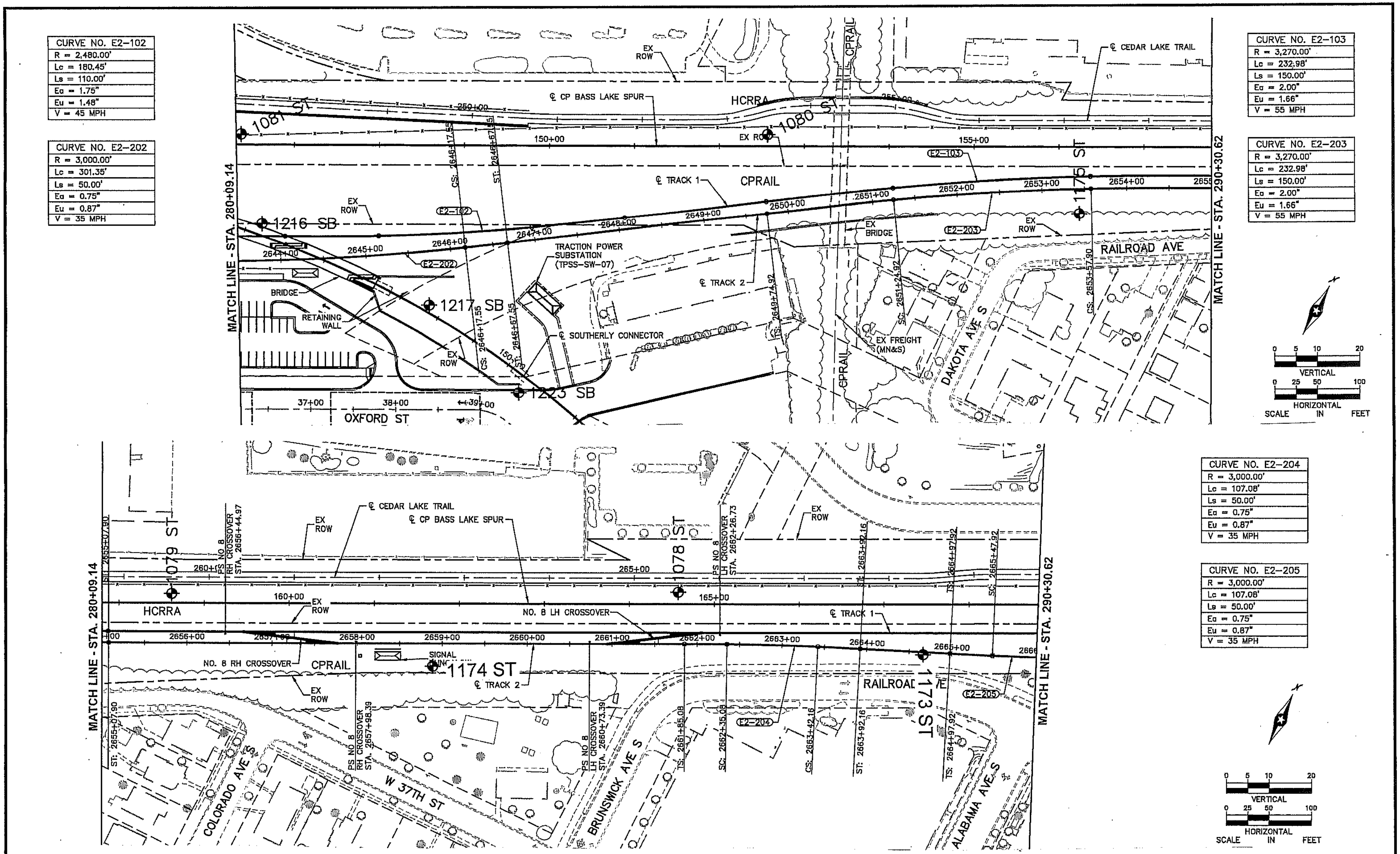
AMERICAN ENGINEERING TESTING, INC.	**PROJECT** Southwest LRT, PEC East		**AET NO.** 01-05697.01
	SUBJECT Boring Locations, Station 2572+50 to 2597+00		**DATE** June 22, 2014
	SCALE 1" = 160'±	**PREPARED BY** KHA/JV / **CHECKED BY** JV	**FIGURE 4**



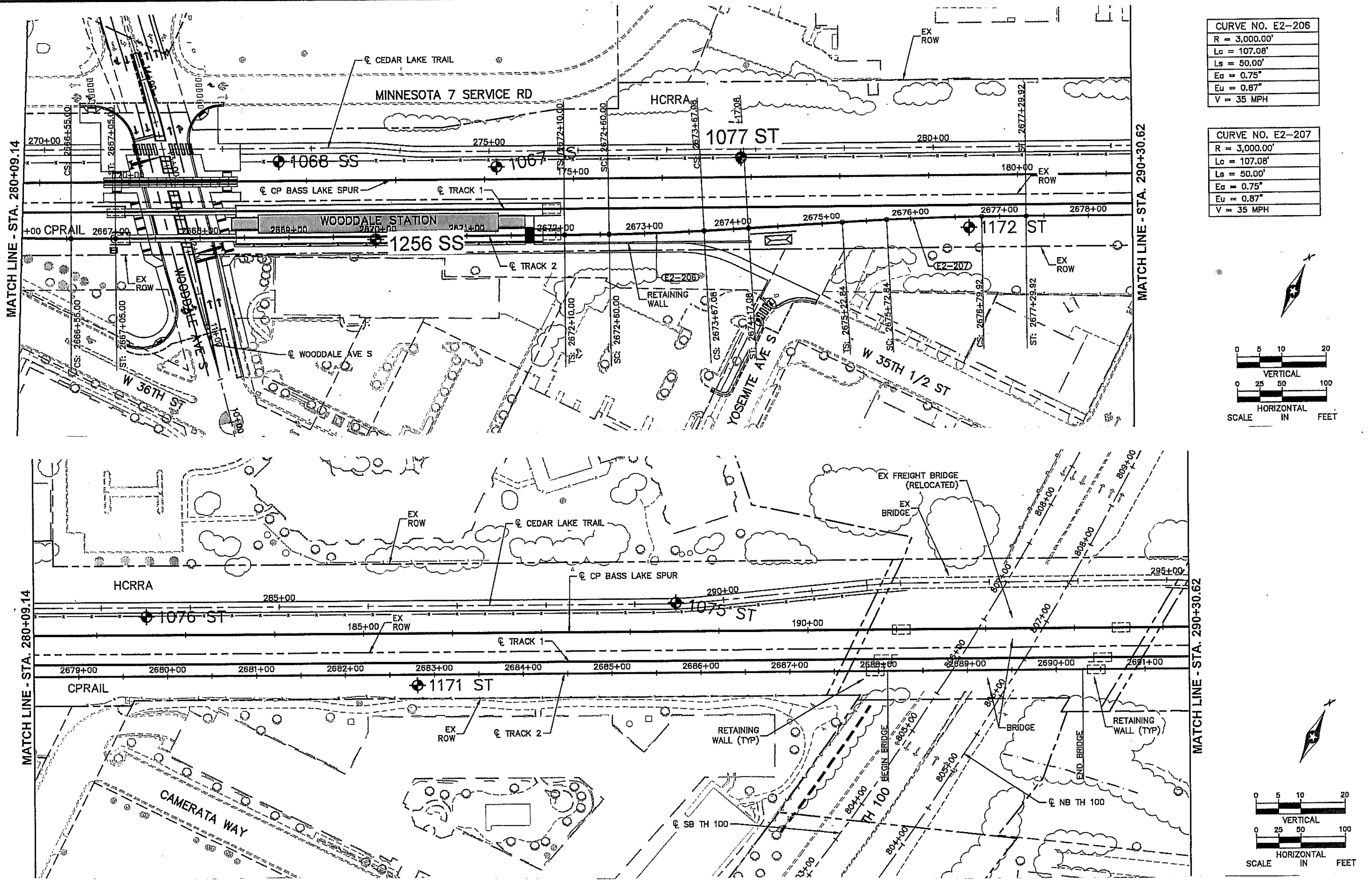
AMERICAN ENGINEERING TESTING, INC.	**PROJECT** Southwest LRT, PEC East	**AET NO.** 01-05697.01
	SUBJECT Boring Locations, Station 2597+00 to 2621+50	**DATE** June 22, 2014
	SCALE 1" = 160'± / **PREPARED BY** KHA/JV / **CHECKED BY** JV	**FIGURE 5**



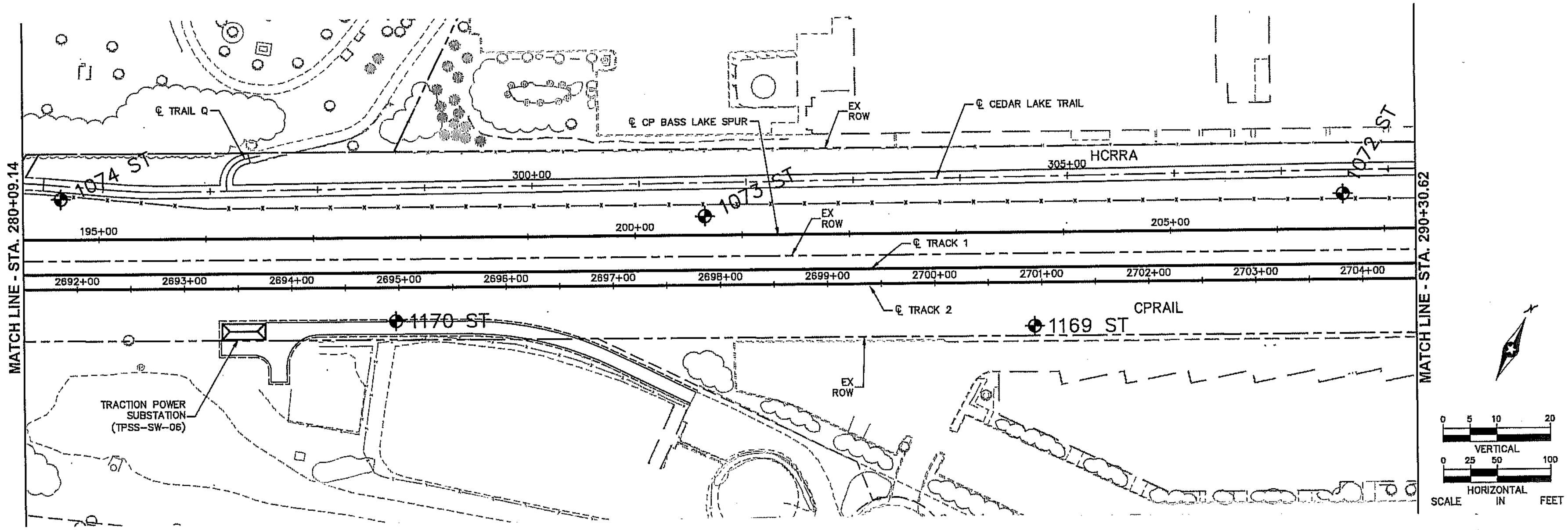
AMERICAN ENGINEERING TESTING, INC.	**PROJECT** Southwest LRT, PEC East		**AET NO.** 01-05697.01
	SUBJECT Boring Locations, Station 2651+50 to 2643+50		**DATE** August 18, 2014
	SCALE 1" = 160'	**PREPARED BY** KHA/JV	**CHECKED BY** JV	**FIGURE 6**



AMERICAN ENGINEERING TESTING, INC.	**PROJECT** Southwest LRT, PEC East		**AET NO.** 01-05697.01
	SUBJECT Boring Locations, Station 2643+50 to 2666+00		**DATE** June 22, 2014
	SCALE 1" = 160'±	**PREPARED BY** KHA/JV	**CHECKED BY** JV	**FIGURE 7**

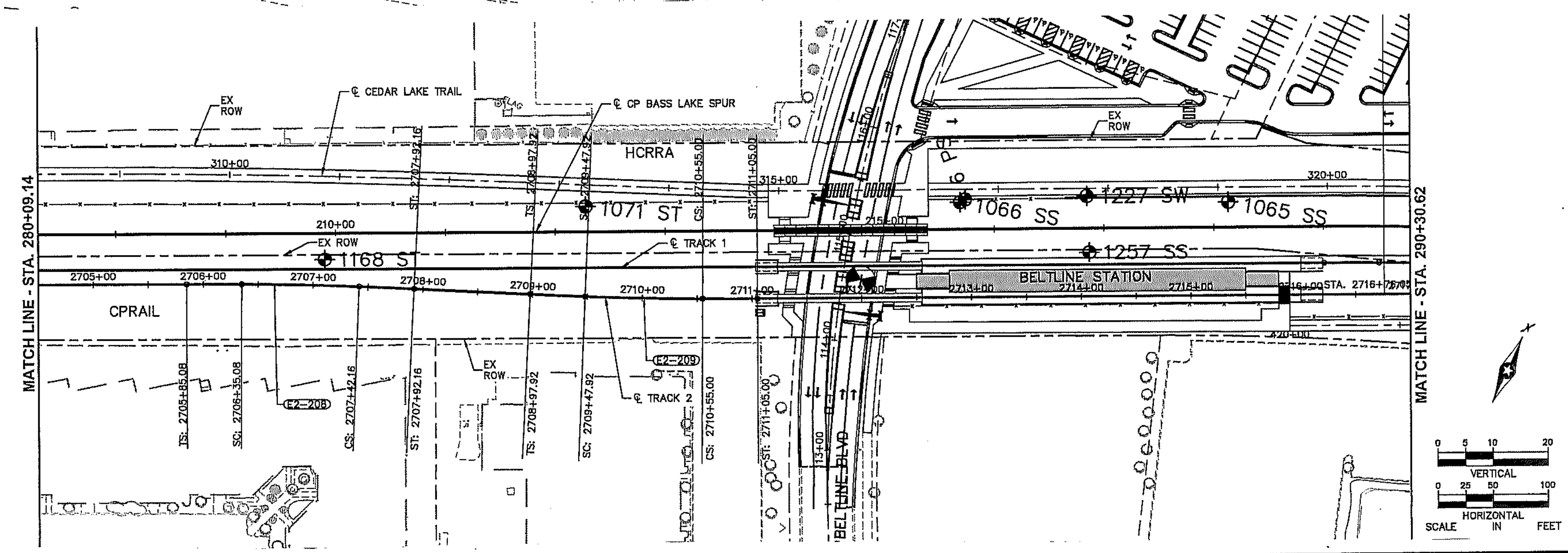


AMERICAN ENGINEERING TESTING, INC.	PROJECT Southwest LRT, PEC East		AET NO. 01-05697.01
	SUBJECT Boring Locations, Station 2666+00 to 2691+50		DATE June 22, 2014
	SCALE 1" = 160'±	PREPARED BY KHA/JV	CHECKED BY JV
			FIGURE 8



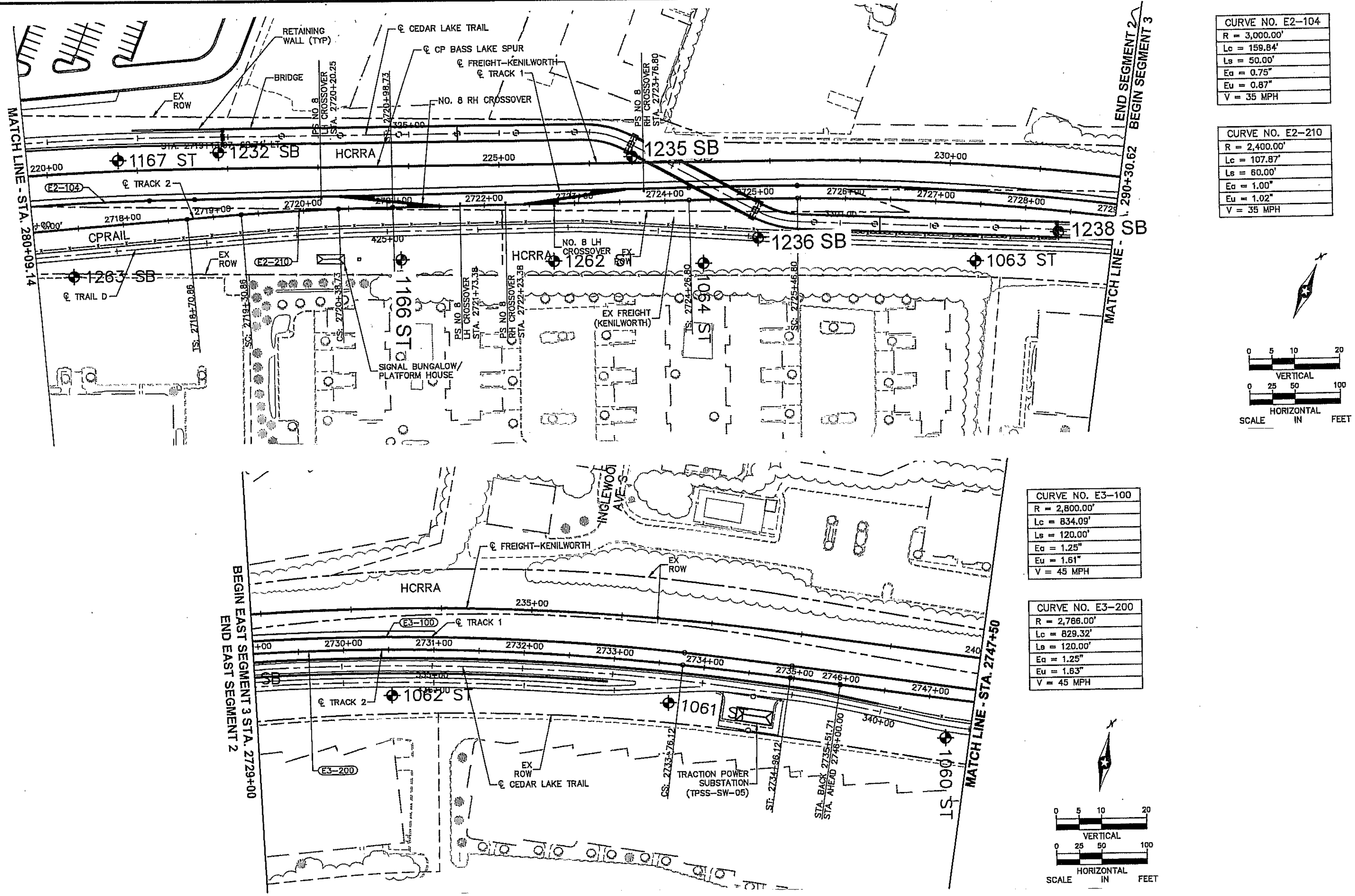
CURVE NO. E2-208
R = 3,000.00'
Lc = 107.08'
Ls = 50.00'
Ea = 0.75"
Eu = 0.87"
V = 35 MPH

CURVE NO. E2-209
R = 3,000.00'
Lc = 107.08'
Ls = 50.00'
Ea = 0.75"
Eu = 0.87"
V = 35 MPH

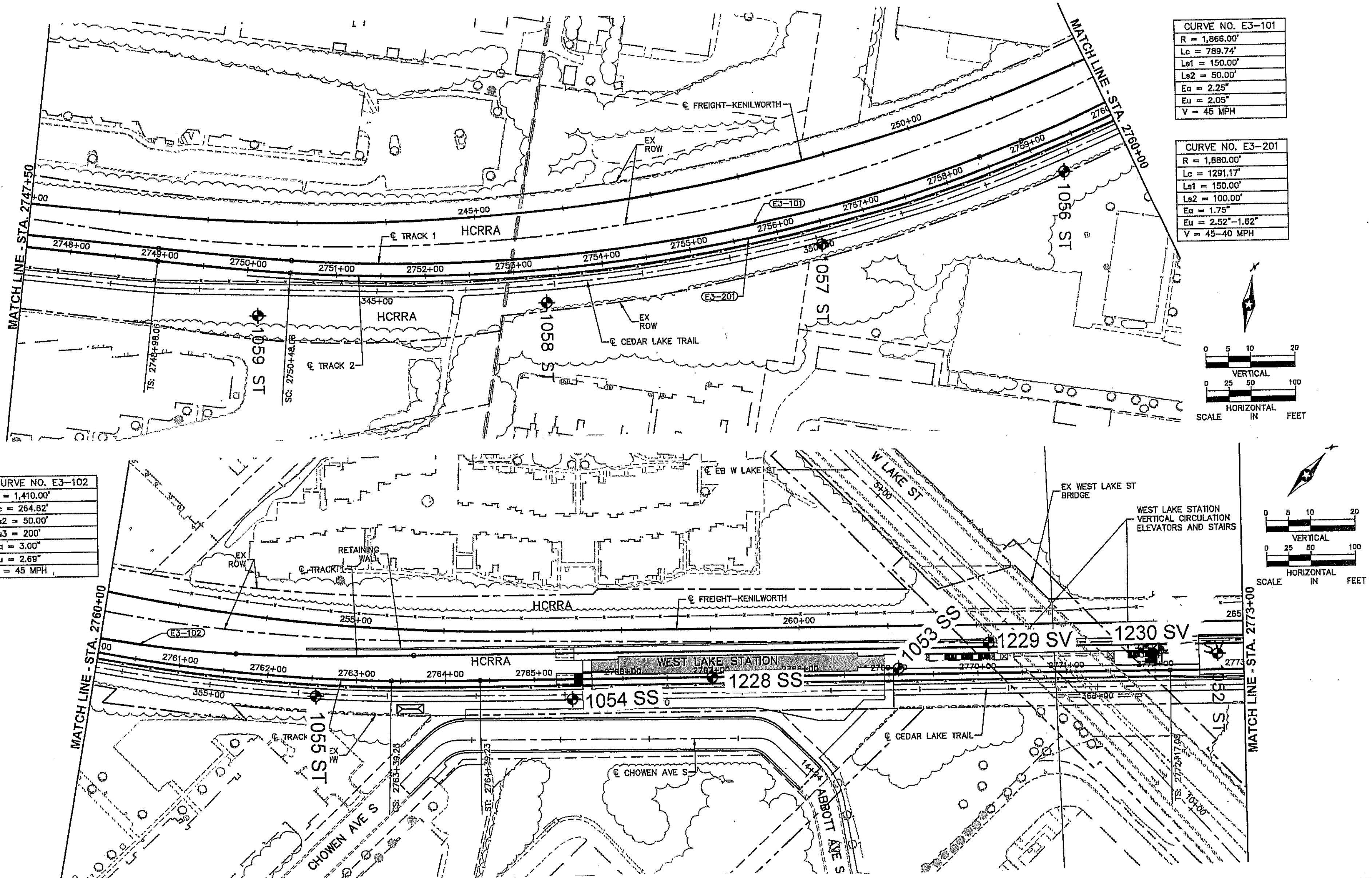


AMERICAN ENGINEERING TESTING, INC.

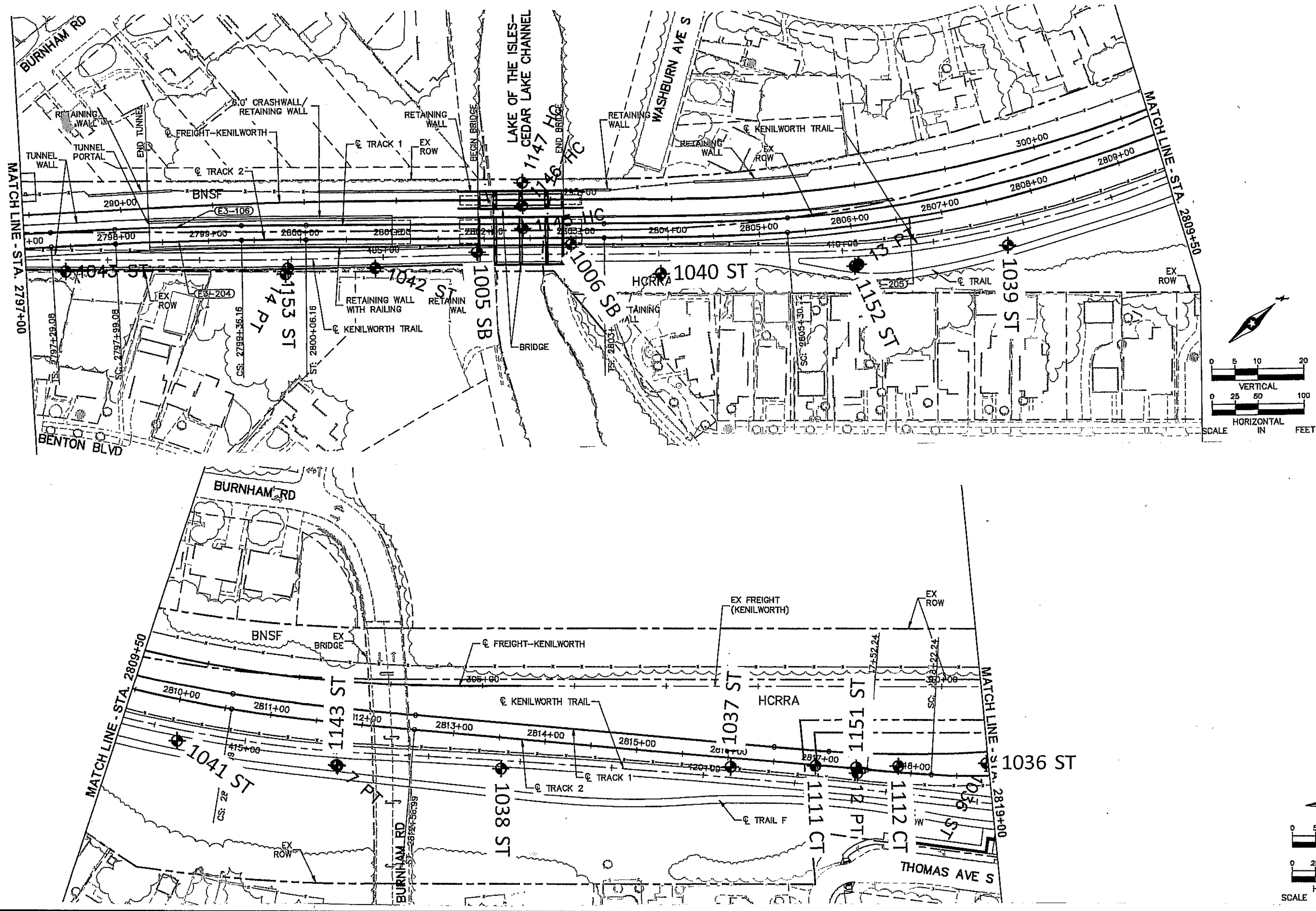
PROJECT Southwest LRT, PEC East			**AET NO.** 01-05697.01
SUBJECT Boring Locations, Station 2691+50 to 2717+00			**DATE** June 22, 2014
SCALE 1" = 160'±	**PREPARED BY** KHA/JV	**CHECKED BY** JV	**FIGURE 9**



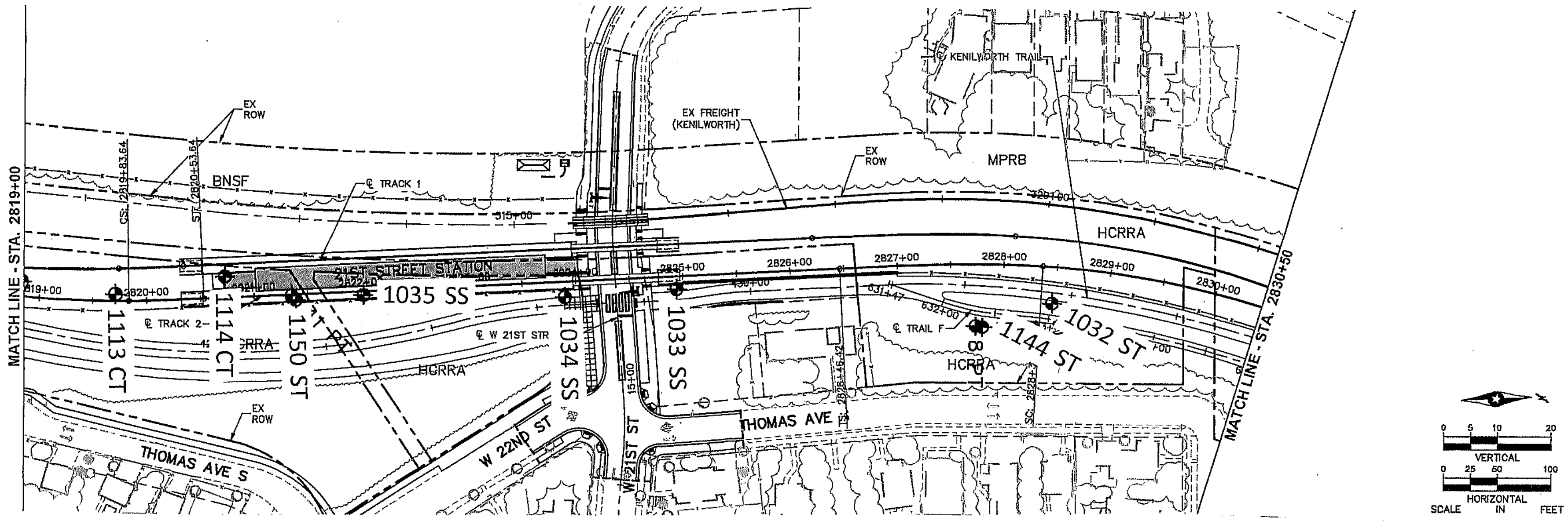
AMERICAN ENGINEERING TESTING, INC.	**PROJECT** Southwest LRT, PEC East	**AET NO.** 01-05697.01
	SUBJECT Boring Locations, Station 2717+00 to 2747+50	**DATE** June 22, 2014
	SCALE 1" = 160'± / **PREPARED BY** KHA/JV / **CHECKED BY** JV	**FIGURE 10**

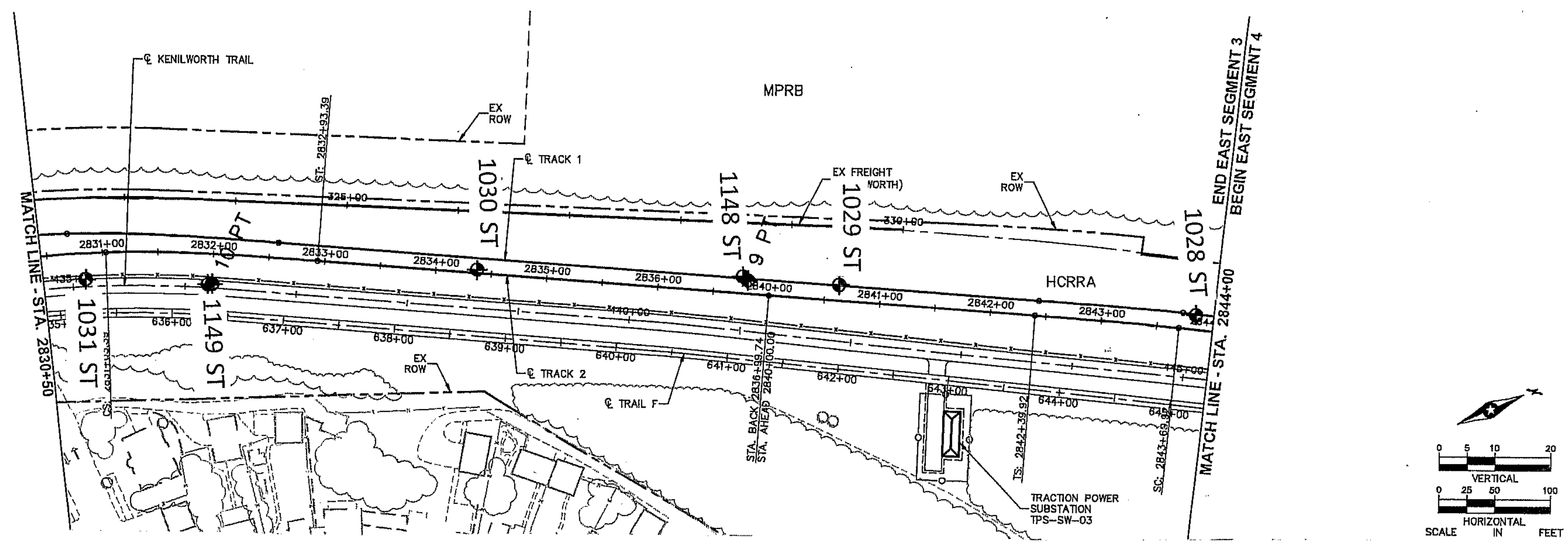


AMERICAN ENGINEERING TESTING, INC.	**PROJECT** Southwest LRT, PEC East		**AET NO.** 01-05697.01
	SUBJECT Boring Locations, Station 2747+50 to 2773+00		**DATE** June 22, 2014
	SCALE 1" = 160'±	**PREPARED BY** KHA/JV	**CHECKED BY** JV	**FIGURE 11**

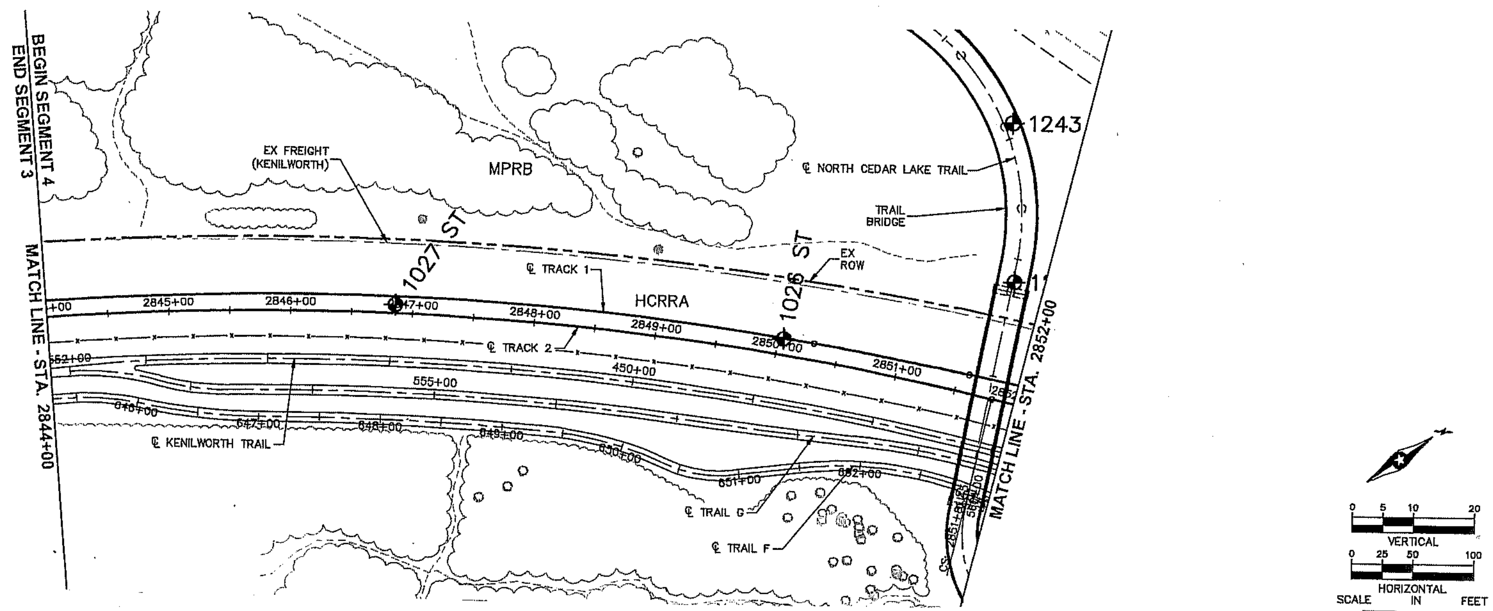


AMERICAN ENGINEERING TESTING, INC.	PROJECT Southwest LRT, PEC East		AET NO. 01-05697.01
	SUBJECT Boring Locations, Station 2797+00 to 2819+00		DATE August 18, 2014
	SCALE 1" = 160'	PREPARED BY KHA/JV	CHECKED BY JV
			FIGURE 12



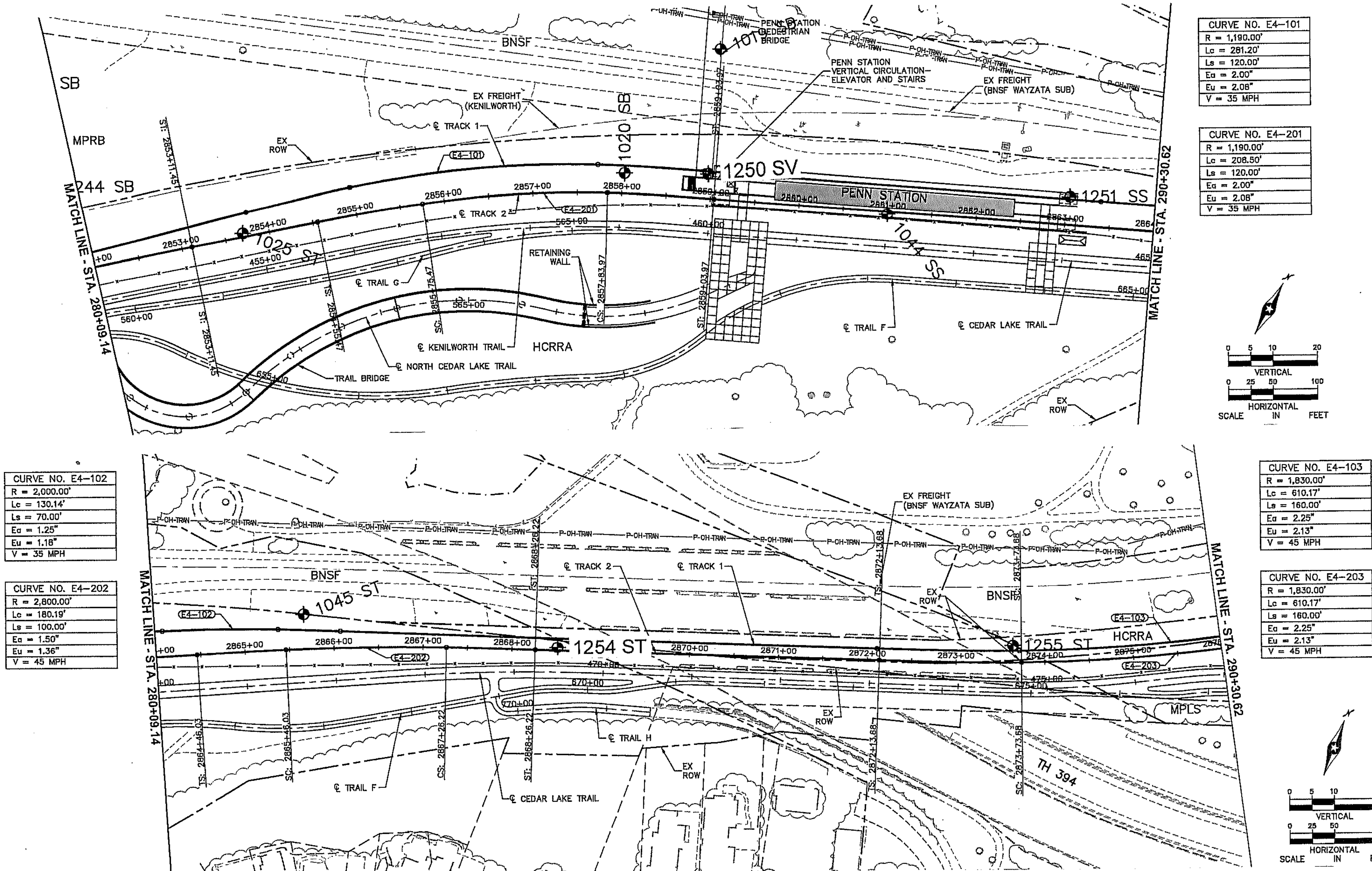


AMERICAN ENGINEERING TESTING, INC.	**PROJECT** Southwest LRT, PEC East		**AET NO.** 01-05697.01
	SUBJECT Boring Locations, Station 2819+00 to 2844+00		**DATE** August 18, 2014
	SCALE 1" = 160'	**PREPARED BY** KHA/JV — **CHECKED BY** JV	**FIGURE 13**

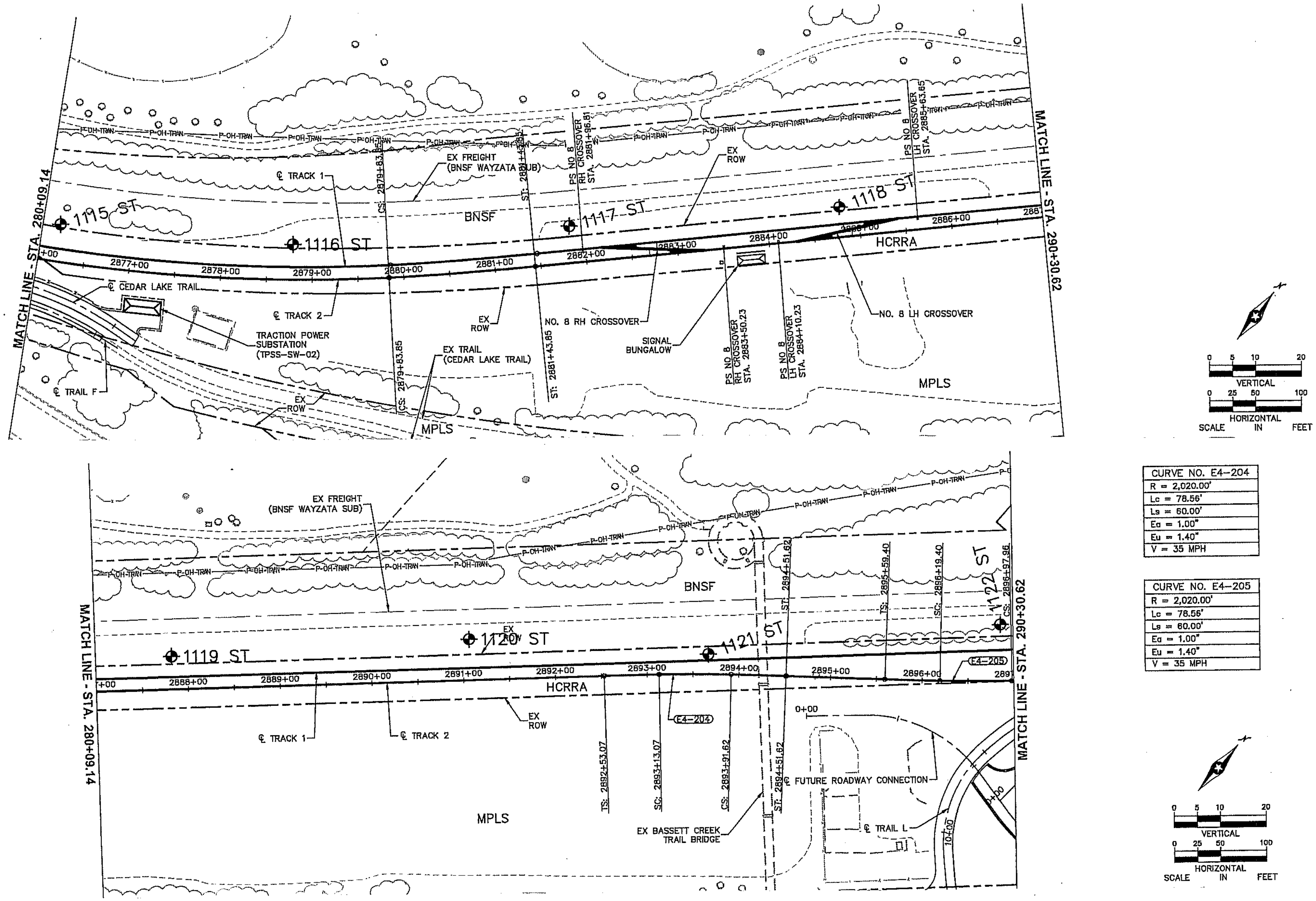


AMERICAN ENGINEERING TESTING, INC.	PROJECT Southwest LRT, PEC East		AET NO. 01-05697.01
	SUBJECT Boring Locations, Station 2844+00 to 2852+00		DATE August 18, 2014
	SCALE 1" = 160'	PREPARED BY KHA/JV	CHECKED BY JV

FIGURE 14

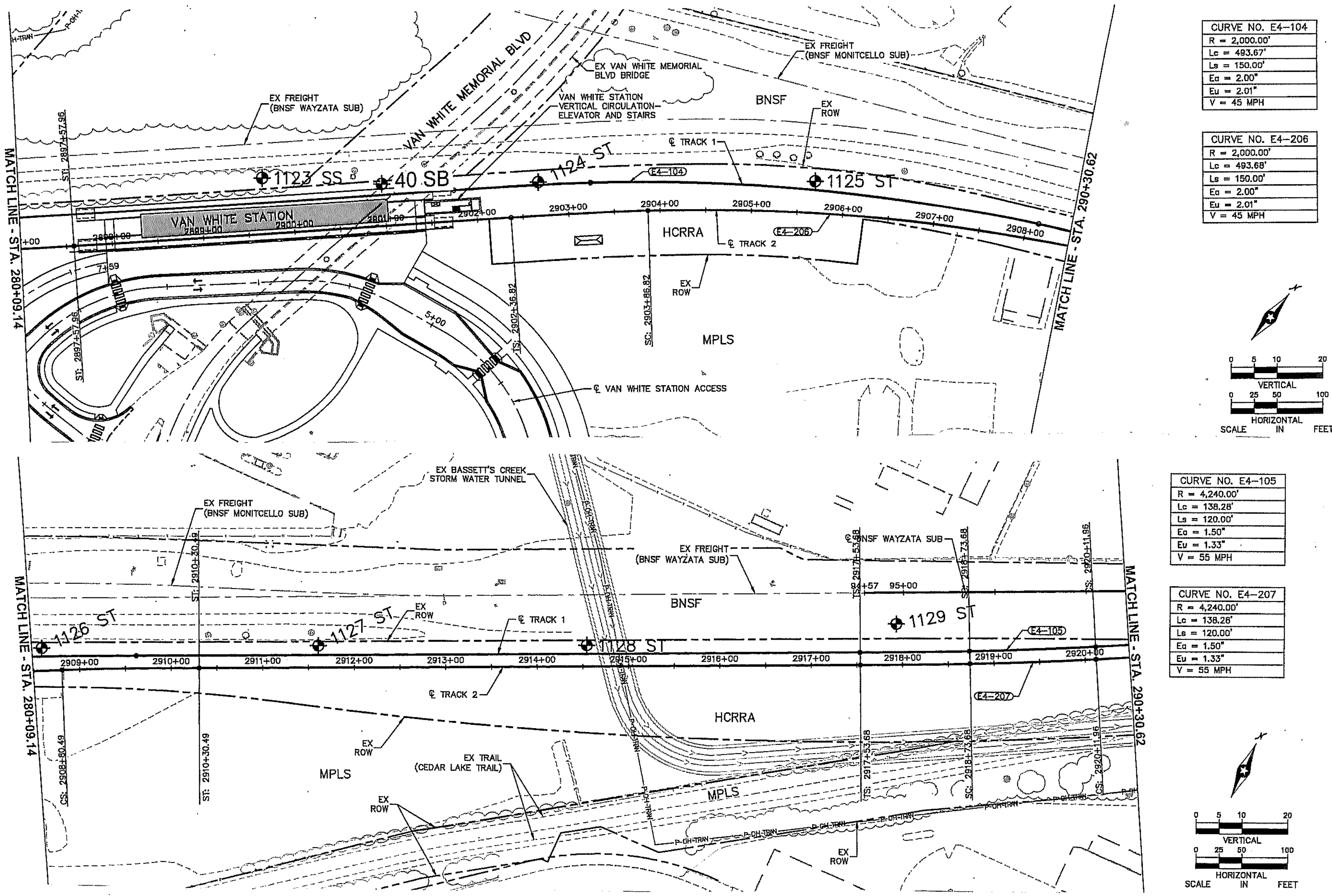


AMERICAN ENGINEERING TESTING, INC.	**PROJECT** Southwest LRT, PEC East		**AET NO.** 01-05697.01
	SUBJECT Boring Locations, Station 2852+00 to 2876+00		**DATE** June 22, 2014
	SCALE 1" = 160'±	**PREPARED BY** KHA/JV	**CHECKED BY** JV	**FIGURE 15**



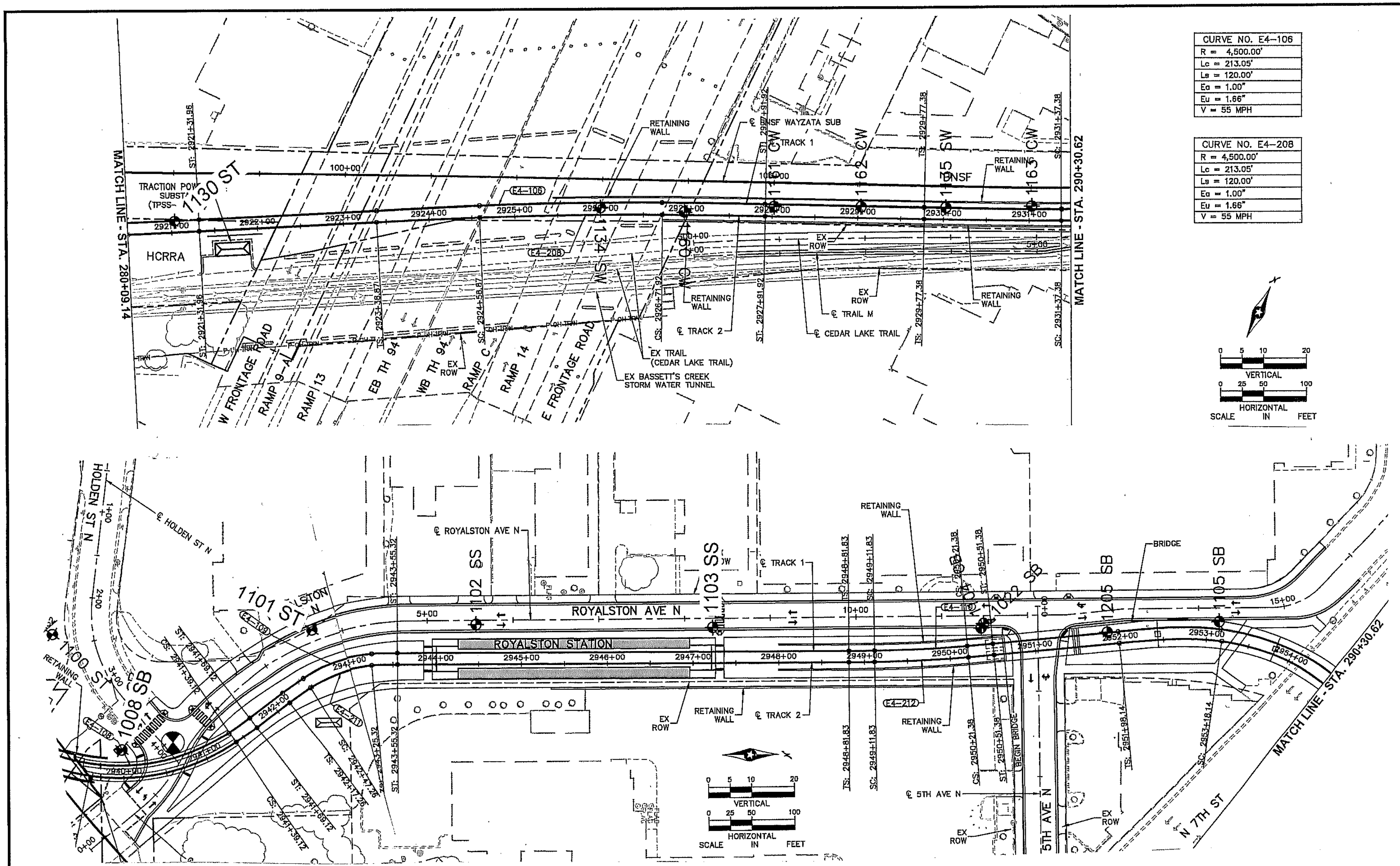
AMERICAN ENGINEERING TESTING, INC.

PROJECT Southwest LRT, PEC East		**AET NO.** 01-05697.01
SUBJECT Boring Locations, Station 2876+00 to 2897+00		**DATE** June 22, 2014
SCALE 1" = 160'±	**PREPARED BY** KHA/JV	**CHECKED BY** JV
		FIGURE 16



AMERICAN ENGINEERING TESTING, INC.

PROJECT Southwest LRT, PEC East		**AET NO.** 01-05697.01
SUBJECT Boring Locations, Station 2897+00 to 2920+50		**DATE** June 22, 2014
SCALE 1" = 160'±	**PREPARED BY** KHA/JV / **CHECKED BY** JV	**FIGURE 17**



AMERICAN ENGINEERING TESTING, INC.	**PROJECT** Southwest LRT, PEC East		**AET NO.** 01-05697.01
	SUBJECT Boring Locations, Station 2920+50 to 2931+50, 2939+50 to 2954+50		**DATE** June 22, 2014
	SCALE 1" = 160'±	**PREPARED BY** KHA/JV	**CHECKED BY** JV
			FIGURE 18

Table A - Select Granular and Compaction Subcut Needs
Southwest LRT, PEC East, Hopkins to Minneapolis
AET No. 01-05697

GWL - Ground-Water Lavel
TOR - Top of Rail
TOS - Top of Subgrade
CS - Compaction Subcut
**below existing ground

Boring No.	Boring Suffix	Coordinates		Stationing	Ground Elevation	GWL Elevation	TOR Elevation	TOS Elevation	TOS Depth**	Select Required	CS Required
		North	East								
1199	ST	146897	492214	2500+63	916.1	<898.1	913.6	910.7	-5.4	2	
1099	ST	146989	492435	2503+02	917.5	<896.5	915.4	912.5	-5.0		no
1198	ST	147082	492729	2506+09	920.7	<909.2	920.2	917.3	-3.5		no
1098	ST	147197	492998	2509+02	922.5		923.4	920.5	-2.0		yes
1197	SS	147234	493146	2510+54	924.1	<903.1	923.7	920.8	-3.3	2	
1196	SS	147321	493382	2513+06	924.2	<907.7	924.2	921.3	-2.9	1.1	
1097	ST	147401	493569	2515+09	923.3	905.0	924.6	921.7	-1.6		yes
1195	ST	147527	493948	2519+08	924.0	<907.5	922.2	919.3	-4.7		no
1096	ST	147592	494140	2521+11	923.2	907.8	920.9	918.0	-5.2		yes
1194	ST	147674	494365	2523+51	922.5	<911.0	921.4	918.5	-4.0		yes
1095	ST	147799	494703	2527+11	923.0	905.8	922.3	919.4	-3.6		yes
1193	ST	147915	495015	2530+44	922.1	903.7	923.0	920.1	-2.0		no
1094	ST	148035	495245	2533+01	921.9	903.8	923.4	920.5	-1.4		yes
1192	ST	148101	495438	2535+05	922.3	902.8	923.3	920.4	-1.9		yes
1093	ST	148263	495776	2538+83	921.8	906.0	922.6	919.7	-2.1		yes
1200	SW	148374	496025	2541+56	921.9	906.9	926.04	923.1	1.2		no
1092	ST	149051	497427	2557+11	925.1	906.3	962.1	959.2	34.1		n/a
1091	ST	149310	497970	2563+15	925.4	<908.4	934.8	931.9	6.5		no
1185	ST	149372	498302	2566+41	925.5	<904.5	926.44	923.5	-2.0		no
1090	ST	149569	498519	2569+22	924.3	901.8	924.9	922.0	-2.3		yes
1184	ST	149621	498798	2571+97	922.4	902.4	923.59	920.7	-1.7		yes
1089	SS	149826	499056	2575+18	921.6	<904.6	922.1	919.2	-2.4		no
1183	SS	149878	499343	2577+99	919.5	<898.5	920.74	917.8	-1.7	2	
1070	SS	149985	499317	2578+22	920.0	896.7	920.7	917.8	-2.2	2	
1069	ST	150073	499512	2580+37	918.0	896.8	919.6	916.7	-1.3		yes
1182	ST	150137	499886	2584+00	916.1	898.1	917.89	915.0	-1.1		yes
1088	ST	150355	500128	2587+13	916.2	897.8	916.4	913.5	-2.7		yes
1181	ST	150383	500396	2589+62	913	897.4	915.23	912.3	-0.7		yes
1087	ST	150619	500689	2595+96	913.7	894.7	913.7	910.8	-2.9		no
1180	ST	150719	501007	2599+16	910.7	894.1	912.66	909.8	-0.9		yes
1009	SB	150872	501210	2601+65	912.8	889.5	912.3	909.4	-3.4		yes
1260	SB	150873	501319	2602+64	910.1	893.2	912.22	909.3	-0.8		yes
1261	SB	150910	501389	2603+43	910.1	893.1	912.11	909.2	-0.9		yes
1010	SB	150994	501471	2604+53	912.9	888.7	912.0	909.1	-3.8		yes
1086	ST	151132	501758	2607+72	914.2	<892.7	911.6	908.7	-5.5		no
1179	ST	151181	502045	2610+52	910	<889.0	911.23	908.3	-1.7		yes
1085	ST	151391	502294	2613+67	913.1	<891.6	910.8	907.9	-5.2		no
1178	ST	151479	502652	2617+28	907.1	887.0	910.37	907.5	0.4		yes
1084	ST	151652	502842	2619+74	911.5	<890.0	910.1	907.2	-4.3		no
1177	ST	151779	503176	2623+30	905.9	885.6	909.92	907.0	1.1		yes
1083	ST	151913	503390	2625+81	910.1	<890.6	909.8	906.9	-3.2		yes
1176	ST	151996	503628	2628+31	904.9	886.6	909.62	906.7	1.8		no
1011	SB	152171	503923	2631+65	909.0	884.5	911.8	908.9	-0.1		yes
1211	SB	152093	504048	2632+58	888.1	883.6	912.10	909.2	21.1	*	
1213	SB	152126	504117	2633+34	888.8	-	911.26	908.4	19.6	*	
1012	SB	152262	504117	2633+75	909.6	884.3	910.4	907.5	-2.1		no

GWL - Ground-Water Lavel
TOR - Top of Rail
TOS - Top of Subgrade
CS - Compaction Subcut
**below existing ground

Boring No.	Boring Suffix	Coordinates		Stationing	Ground Elevation	GWL Elevation	TOR Elevation	TOS Elevation	TOS Depth**	Select Required	CS Required
		North	East								
1214	SW	152175	504279	2635+00	889.9	883.7	906.34	903.4	13.5	*	
1219	SW	152380	504260	2635+47	902.8	882.7	904.18	901.3	-1.5		n/a
1215	SW	152250	504420	2636+53	890.6	882.7	898.86	896.0	5.4	*	
1082	ST	152431	504459	2637+71	911.0	<889.5	893.2	890.3	-20.7	*	
1220	SS	152357	504654	2639+36	891.7	883.1	892.4	889.5	-2.2	*	
1221	SU	152515	504907	2642+14	893.7	881.9	893.90	891.0	-2.7	*	
1222	SU	152618	504861	2642+17	911.3	887.8	893.91	891.0	-20.3	*	
1081	ST	152677	504983	2643+56	911.7	<890.2	894.6	891.7	-20.0		n/a
1080	ST	152946	505541	2649+86	913.5	<892.0	905.5	902.6	-10.9		no
1175	ST	153022	505912	2653+43	912.3	<891.3	914.16	911.3	-1.0		no
1079	ST	153205	506088	2655+82	915.2	<893.7	916.0	913.1	-2.1		no
1174	ST	153260	506402	2658+89	915	<894.0	916.60	913.7	-1.3	2	
1078	ST	153465	506624	2661+78	918.1	<896.6	917.2	914.3	-3.8	2	
1173	ST	153523	506916	2664+66	916.6	<895.6	917.64	914.7	-1.9	1.5	
1068	SS	153779	507262	2668+89	917.2	<885.7	917.2	914.3	-2.9		no
1256	SS	153746	508398	2669+98	914.9	<888.9	916.83	913.9	-1.0	1	
1067	SS	153878	507486	2671+34	915.6	<884.1	916.6	913.7	-1.9		no
1077	ST	154004	507731	2674+14	913.2	<891.7	916.1	913.2	0.0		yes
1172	ST	154041	507997	2676+65	911.5	890.5	915.09	912.2	0.7		yes
1076	ST	154248	508243	2679+77	911.8	<890.3	913.7	910.8	-1.0		yes
1171	ST	154307	508552	2682+81	909	<888.0	912.48	909.6	0.6	1.5	
1075	ST	154516	508775	2685+72	909.4	<887.9	911.2	908.3	-1.1	1.5	
1074	ST	154785	509326	2691+86	904.1	<882.6	906.9	904.0	-0.1		no
1170	ST	154815	509657	2694+97	900.9	<879.9	903.74	900.8	-0.1		yes
1073	ST	155027	509876	2697+86	898.6	<877.1	900.8	897.9	-0.7		yes
1169	ST	155064	510199	2700+93	896.6	<877.6	897.78	894.9	-1.7		yes
1072	ST	155298	510407	2703+82	894.2	875.0	894.8	891.9	-2.3		no
1168	ST	155389	510728	2707+10	890.9	874.2	892.30	889.4	-1.5		yes
1071	ST	155534	510922	2709+45	889.8	873.7	890.6	887.7	-2.1		yes
1066	SS	155683	511229	2712+90	887.9	-	887.9	885.0	-2.9	2	
1066	PS	155687	511232	2712+95	887.8	873.8	888.0	885.1	-2.7		n/a
1227	SW	155737	511331	2714+06	886.9	879.5	887.2	884.3	-2.6		yes
1257	SS	155692	511355	2714+07	886.2	876.0	887.16	884.3	-1.9	2	
1065	SS	155787	511450	2715+34	886.9	873.3	886.2	883.3	-3.6		no
1263	SB	155750	511696	2717+40	884.9	872.8	885.27	882.4	-2.5		yes
1167	ST	155888	511697	2718+01	886.4	873.9	885.41	882.5	-3.9		yes
1232	SB	155935	511798	2719+12	885.6	869.4	885.90	883.0	-2.6		yes
1166	ST	155893	512029	2721+07	884.3	871.6	886.18	883.3	-1.0	2	
1262	ST	155950	512188	2722+76	884.7	873.0	887.36	884.5	-0.2		no
1235	SB	156089	512229	2723+65	886.5	870.0	887.73	884.8	-1.7		no
1064	SR	156005	512344	2724+41	884.9	874.9	888.0	885.1	0.2		yes
1236	SB	156052	512391	2725+02	886.1	869.6	888.29	885.4	-0.7		yes
1063	ST	156111	512626	2727+48	887.6	872.4	889.3	886.4	-1.3		no
1238	SB	156173	512701	2728+39	888.4	872.1	889.23	886.3	-2.1		yes
1062	ST	156191	512914	2730+54	887.8	871.2	888.3	885.4	-2.4		no
1061	ST	156234	513217	2733+65	886.4	872.0	886.7	883.8	-2.6		no

Table A - Select Granular and Compaction Subcut Needs
Southwest LRT, PEC East, Hopkins to Minneapolis
AET No. 01-05697

GWL - Ground-Water Lavel
TOR - Top of Rail
TOS - Top of Subgrade
CS - Compaction Subcut
**below existing ground

Boring No.	Boring Suffix	Coordinates		Stationing	Ground Elevation	GWL Elevation	TOR Elevation	TOS Elevation	TOS Depth**	Select Required	CS Required
		North	East								
1060	ST	156247	513526	2747+24	883.1	874.1	885.1	882.2	-0.9		yes
1059	ST	156251	513819	2750+15	881.1	870.8	883.6	880.7	-0.4		no
1058	ST	156311	514139	2753+34	880.6	875.5	882.1	879.2	-1.4		no
1057	ST	156419	514437	2756+46	879.4	874.2	880.5	877.6	-1.8		no
1056	ST	156538	514696	2759+26	877.9	868.1	879.1	876.2	-1.7		yes
1055	ST	156748	514956	2762+55	876.4	868.2	877.4	874.5	-1.9		yes
1054	SS	156948	515165	2765+43	875.7	863.4	876.1	873.2	-2.5	*	
1228	SS	157076	515260	2767+01	874.5	861.1	875.25	872.4	-2.1	*	
1053	SS	157230	515403	2769+11	875.0	861.4	874.2	871.3	-3.7	*	
1229	SV	157322	515455	2770+13	872.8	856.2	873.69	870.8	-2.0	*	
1230	SV	157440	515584	2771+88	873	856.8	871.53	868.6	-4.4	*	
1005	SB	159918	517230	2801+92	869.1	848.0	869.9	867.0	-2.1		yes
1006	SB	160002	517289	2802+95	868.4	847.4	872.0	869.1	0.7	0.5	
1040	ST	160056	517377	2803+92	867.9	849.6	872.7	869.8	1.9		no
1152	ST	160223	517507	2805+99	867.9	852.5	871.1	868.2	0.3	2	
1039	ST	160365	517597	2807+62	867	851.9	869.2	866.3	-0.7	1.3	
1041	ST	160602	517682	2810+05	865.9	850.5	866.4	863.5	-2.4		yes
1143	ST	160773	517726	2811+77	866.2	851.3	864.8	861.9	-4.3	2.2	
1038	ST	160950	517746	2813+55	865.4	850.6	865.1	862.2	-3.2		no
1037	ST	161199	517768	2816+04	864.9	850.1	865.6	862.7	-2.2		no
1151	ST	161335	517782	2817+40	865.3	851.0	865.9	863.0	-2.3	**	
1036	ST	161477	517791	2818+83	863.8	857.7	866.1	863.2	-0.6	**	
1150	ST	161728	517802	2821+35	864.8	852.1	866.7	863.8	-1.0	**	
1035	SS	161795	517801	2822+02	865.3	852.1	866.8	863.9	-1.4	**	
1034	SS	161983	517802	2823+90	867.2	849.8	867.2	864.3	-2.9	**	
1033	SS	162087	517794	2824+94	867.8	849.6	867.4	864.5	-3.3		yes
1144	ST	162372	517829	2827+80	868.9	850.7	868.0	865.1	-3.8		no
1032	ST	162437	517806	2828+47	868.7	858.1	868.1	865.2	-3.5		yes
1031	ST	162662	517844	2830+84	868.1	849.0	868.3	865.4	-2.7	1.3	
1149	ST	162761	517891	2831+96	867.4	850.3	867.9	865.0	-2.4		yes
1030	ST	162989	517973	2834+37	866.5	-	866.4	863.5	-3.0		yes
1148	ST	163206	518072	2836+75	864.6	849.8	865.0	862.1	-2.5		yes
1029	ST	163283	518113	2840+63	864.2	<851.2	864.5	861.6	-2.6		yes
1028	ST	163567	518264	2843+84	861.4	849.2	862.5	859.6	-1.8	2.7	
1027	ST	163818	518433	2846+85	859.6	848.6	860.7	857.8	-1.8		yes
1026	ST	164067	518634	2850+04	857.7	846.7	858.8	855.9	-1.9	2.1	
1025	ST	164308	518911	2853+70	854.7	849.7	856.6	853.7	-1.0		yes
1020	SB	164578	519252	2858+02	854.9	-	854.1	851.2	-3.8		yes
1250	SV	164619	519336	2858+95	853.3	847.7	853.46	850.6	-2.7	*	
1044	SS	164681	519533	2861+00	851.0	841.2	852.3	849.4	-1.6	2	
1251	SS	164787	519705	2863+01	850.5	839.0	851.03	848.1	-2.4	2	
1045	ST	164915	519941	2865+67	847.2	838.4	849.4	846.5	-0.7		yes
1254	ST	164981	520225	2868+55	846.4	837.6	847.45	844.6	-1.8		yes
1255	ST	165163	520702	2873+66	841.2	828.1	842.23	839.3	-1.9		yes
1115	ST	165292	520917	2876+19	836.9	823.8	839.7	836.8	-0.1		yes
1116	ST	165430	521130	2879+79	834.9	823.9	837.0	834.1	-0.8		yes

Table A - Select Granular and Compaction Subcut Needs
Southwest LRT, PEC East, Hopkins to Minneapolis
AET No. 01-05697

GWL - Ground-Water Lavel
TOR - Top of Rail
TOS - Top of Subgrade
CS - Compaction Subcut
**below existing ground

Boring No.	Boring Suffix	Coordinates		Stationing	Ground Elevation	GWL Elevation	TOR Elevation	TOS Elevation	TOS Depth**	Select Required	CS Required
		North	East								
1117	ST	165630	521356	2881+85	831.7	812.8	833.8	830.9	-0.8	2	
1118	ST	165827	521576	2884+80	829.3	812.7	830.8	827.9	-1.4	2	
1119	ST	166037	521794	2887+83	825.7	812.2	827.6	824.7	-1.0		yes
1120	ST	166265	522025	2891+07	823.8	808.6	823.8	820.9	-2.9		no
1121	ST	166425	522232	2893+67	822.4	805.5	821.7	818.8	-3.6		no
1122	ST	166659	522450	2896+86	819.3	805.1	818.6	815.7	-3.6	1.5	
1123	SS	166850	522656	2899+68	818.1	814.1	818.0	815.1	-3.0	2	
40	SB	166928	522761	2900+98	817.6	-	817.69	814.8	-2.8	*	
1124	ST	167038	522892	2902+69	817.8	798.8	817.8	814.9	-2.9		yes
1125	ST	167231	523126	2905+68	818.0	811.8	820.0	817.1	-0.9		yes
1126	ST	167378	523383	2908+60	818.9	800.9	822.9	820.0	1.1		yes
1127	ST	167506	523657	2911+61	821.7	<805.2	825.4	822.5	0.8		yes
1128	ST	167628	523924	2914+54	822.1	<810.1	826.4	823.5	1.4		yes
1129	ST	167789	524223	2917+93	822.6	<808.1	825.3	822.4	-0.2		yes
1130	ST	167899	524513	2921+04	822.4	815.1	824.0	821.1	-1.3		yes
1100	ST	168772	525926	2938+42	842.9	<823.4	851.4	848.5	5.6		n/a
1008	SB	168850	526064	2939+98	849.8	<825.8	850.9	848.0	-1.8		yes
1101	ST	169074	525926	2942+75	844.8	<828.8	844.6	841.7	-3.1		no
1102	SS	169267	525922	2944+47	844.5	<823.5	843.0	840.1	-4.4		no
1103	SS	169543	525927	2947+23	842.6	<823.1	841.4	838.5	-4.1		no

*Pile supported area. Slab, subballast, and Select Granular Material thickness should total 5.5 feet.
**Geofoam lightweight fill area.