Geotechnical Report Addendum

To: Mr. Adrian Reid, P.E. – AZTEC Engineering Group, Inc.

From: Mr. Kellen P. Heavin, P.E. – Earth Exploration, Inc. (EEI)

CC: Mr. Curtis R. Bradburn, P.E. – EEI

Date: September 5, 2017

Re: Updated Retaining Wall Design Parameters
17th Street Improvements
Bloomington, Indiana
EEI Project No. 1-17-052

The purpose of this memorandum is to provide updated retaining wall parameters in response to new information regarding the type of walls planned for the referenced project. Based on correspondence with Mr. Mario Colecchia with AZTEC , we understand that the proposed retaining walls are planned to consist of reinforced masonry block without geotextile reinforcement. Table 1, summarizing our understanding of the location, condition, and approximate exposed height of these walls is provided below.

Table 1: Summary of Retaining Wall Information

Start	End	Offset (ft), Line	Condition	Approximate Max Exposed Height (ft)
18+75	20+75	30 Lt, "PR-B"	Cut	9
18+90	21+20	20 Rt, "PR-B"	Cut	9
20+85	22+75	30 Lt, "PR-B"	Cut	9
23+90	25+05	35 Lt, "PR-B"	Cut	5
24+75	26+15	20 Rt, "PR-B"	Fill	5
25+80	26+85	32 Lt, "PR-B"	Cut	5
27+00	27+75	35 Lt, "PR-B"	Cut	7

Discussion and Considerations

Based on the subsurface conditions encountered at the exploratory locations, it is our opinion that the retaining walls can be supported on conventional spread foundations, as planned. Foundations established a minimum of 3 ft below the existing ground surface are anticipated to be established on medium stiff or better cohesive soil and/or limestone. For design of the foundations, we recommend that they be proportioned using an allowable bearing capacity of 3,000 lb/sq ft (psf). This lower bearing pressure will reduce the potential for differential settlement where portions of a wall are supported at the soil/rock interface. We recommend that the cohesive subgrades be prepared as discussed in our geotechnical report. Although not observed at the boring locations, if soft cohesive soils are encountered at the foundation subgrade, we recommend that they be removed and replaced with compacted granular fill.

Lateral Earth Pressures

Based on our understanding of the wall construction, the walls will deform somewhat, creating an active earth pressure condition. This condition assumes that relatively free-draining granular soils are used as wall backfill and that the backfill extends horizontally from the wall a distance equal to at least 24 in. from the base to the top of the wall. For design of the wall, we recommend a moist unit weight of 120 pcf and an angle of internal friction of 30 degrees. Also, an active earth pressure coefficient (K_a) of 0.33 may be utilized.

Though groundwater was not observed during our subsurface evaluation, it is our experience that water can become trapped at the soil/rock interface. Where excavation to construct the walls exposes or extends into the underlying rock, the potential for groundwater seepage to enter the granular backfill exists. We recommend that adequate drainage be provided behind the walls to control seepage from groundwater and surface water sources. This is commonly accommodated by the use of a perforated drain pipe located at the back of the drainage fill. In addition, we recommend that weep holes be provided through the wall located near the toe, and the spacing of these weep holes should not exceed 20 ft.

In addition to the lateral earth pressures, surcharges from temporary loads during construction and loads associated with adjacent foundations should be taken into account in the wall design. We recommend that backfill placed immediately adjacent to the walls be compacted to 95 percent of the standard Proctor dry density. Compaction of backfill within 3 ft of the walls should be performed with a hand-guided compactor to avoid over-stressing the walls. The friction acting along the base of the footings founded on suitable foundation soils may be computed using an ultimate adhesion equal to 1,750 psf where founded on soil, and an ultimate coefficient of friction of 0.7.

GEOTECHNICAL EVALUATION

17^{TH} STREET IMPROVEMENTS
BLOOMINGTON, INDIANA

Prepared for

AZTEC ENGINEERING GROUP, INC.
1145 N. SUNRISE GREETINGS COURT
BLOOMINGTON, INDIANA 47404

By

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August 30, 2017

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August 30, 2017

Mr. Adrian Reid, P.E.
AZTEC Engineering Group, Inc.
1145 N. Sunrise Greetings Ct.
Bloomington, IN 47404

Re: Geotechnical Evaluation
17th Street Improvements
Bloomington, Indiana
EEI Project No. 1-17-052

Dear Adrian:

We are pleased to submit our geotechnical evaluation for the above-referenced project. This report presents the results of our subsurface exploration and laboratory testing and provides geotechnical recommendations for design and construction of the proposed improvements. The work for this project was authorized via acceptance of Earth Exploration, Inc. (EEI) Proposal No. P1-16-657.

The opinions and recommendations submitted in this report are based, in part, on our interpretation of the subsurface information revealed at the exploratory locations as indicated on an attached plan. Understandably, this report does not reflect variations in subsurface conditions between or beyond these locations. Therefore, variations in these conditions can be expected, and fluctuation of the groundwater levels will occur with time. Other important limitations of this report are discussed in Appendix A.

PROJECT DESCRIPTION

We understand that representatives of the City of Bloomington are planning to make improvements to 17th Street (Line "PR-B") using local funds only. The project start (Station 13+66.58) is about 310 ft east of Crescent Road and the project end (about Station 31+00) is about 640 ft west of the roundabout at Monroe Street for a total length of approximately 1,730 ft. Based on plans provided by AZTEC Engineering Group, Inc. (AZTEC), the improvements are anticipated to include reconstruction of the pavement, retaining walls, drainage improvements, and a pedestrian path. The typical section includes two travel lanes with curb and gutter, as well as a 10-ft wide multi-use path to the north of the roadway.

Along with the curb and gutter, drainage improvements are planned to include new storm sewers with inverts established about 4 ft below the existing grade. Pipe sizes were not known at the time of this report. Grading information shown on the plans indicates that earth cuts and fills are generally on the order of about 5 ft or less with an exception from Station 18+00 to Station 22+00 where a cut of up to 8 ft is planned for improvements to a vertical curve. East of Station 18+50, the roadway is topographically lower than the surrounding grade, and retaining walls are planned to assist with the cuts necessary for the vertical curve improvements and the construction of the path. Based on our conversations, we understand that the walls are planned to be constructed using a segmental block system. Our knowledge of the retaining walls is based on the cross sections and is summarized in the table below.

Table 1: Summary of Retaining Wall Information

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Based on information provided on the plans, proposed sideslopes are anticipated to be 3 Horizontal (H): 1 Vertical (V) or flatter.

We understand that the INDOT Standard Specifications (ISS) will be utilized for construction. At this time, other information such as anticipated construction schedule is not known. In the event that the nature, design or location of the proposed construction changes, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed, and the conclusions are modified or confirmed in writing.

FIELD EXPLORATION AND LABORATORY TESTING

Subsurface conditions for the proposed improvements were explored by performing five soil borings for the roadway improvements (designated RB-1 through RB-5) and three soil borings for the retaining walls (designated RW-1 through RW-3). Auger refusal on bedrock was encountered at Borings RB-3, RB-4, RW-2, and RW-3 prior to achieving the planned termination depth. In addition, soundings to refusal on bedrock were performed at eight locations between the soil borings. The number, location, and depths (where refusal was not encountered) of the borings were determined by EEI. The exploratory locations are shown on Drawing No. 1-17-052.B1 in Appendix C. The borings were located in the field by EEI personnel referencing identifiable features shown on the plans. Ground surface elevations at the exploratory locations were estimated to the nearest 1 ft based on topographic information provided on the plans. The exploratory locations and elevations should be considered accurate only to the degree implied by the methods used.

Exploratory field activities were performed by EEI on July 18 and 20, 2017. Exploratory activities at the soil boring locations were performed using hollow stem augers to advance the boreholes. The soundings were performed by direct pushing or hammering a solid rod to refusal. Representative samples of the soil conditions using Standard Penetration Test (SPT) procedures (AASHTO T 206) were obtained at predetermined intervals. After obtaining groundwater observations, each borehole was backfilled with auger cuttings and a bentonite chip plug was placed near the ground surface. At the borings performed in the roadway, the surface was restored with a pavement patch. Additional details of the drilling and sampling procedures are provided in Appendix B.

Following the field activities, the soil and rock samples were visually classified by an EEI engineering technician and later reviewed by an EEI geotechnical engineer. After visually classifying the soils, representative samples were selected and submitted for index property testing. These tests included: natural moisture content (AASHTO T 265); Atterberg limits (AASHTO T 89 and T 90); soil pH; and hand penetrometer readings. The results of these tests are provided on the boring logs and/or respective laboratory reports in Appendix C. For your information, soil descriptions on the boring logs are in general accordance with the AASHTO system and the INDOT Standard Specifications (ISS[1]) (textural classification, e.g., clay, A-7-6). The boring logs represent our interpretation of the individual samples and field logs and results of the laboratory tests. The stratification lines on the boring logs represent the approximate boundary between soil types; although, the transition may actually be gradual.

SITE CONDITIONS

Surface Conditions

The ground surface within the project limits is relatively flat near the western and eastern project extents and gently to moderately sloping near the center. Grades along Line "PR-B" range from about El. 875½ at the project start and about El. 793 at the end. In addition, a creek is present to the south of 17th Street, with the closest proximity to the roadway being about 40 ft near Station 26+00.

At the boring locations performed in the roadways (e.g., Borings RB-1 through RB-5, and RW-3), the surface conditions consisted of asphaltic concrete pavement (HMA) with a thickness in the range of 6 to 16 in. At Boring RB-1, the HMA was underlain by 6 in. of a crushed stone subbase. Also, an exception to these conditions was observed at Boring RW-3 where the HMA was found to be about 2½ in. thick. This boring was performed in a shoulder area. The surface conditions at the remainder of the soil boring locations consisted of about 4 to 6 in. of topsoil.

Subsurface Conditions

The subsurface profile generally consisted of cohesive soils underlain by rock. In general, the cohesive soils were described as A-6 (clay loam and silty loam) from below the surface conditions to depths of about 3½ to 6 ft below the existing grade. At these depths, A-7-6 clay was typically observed to the depth at which rock was encountered or the maximum depth explored. However, the A-7-6 soils were observed at a shallower depth at Borings RB-2, RW-1, and RW-2. The underlying rock consisted of siltstone and limestone. Table 1 provides a summary of the depth to rock and auger refusal at the boring and sounding locations.

Table 2: Summary of Depths to Rock and Refusal at Exploratory Locations

Boring Location	Ground Surface El.	Depth to Rock (ft)	El. of Rock	Depth of Refusal (ft)	El. of Refusal
RB-1	868	NO	NO	NO	NO
RB-2	857	NO	NO	NO	NO
RB-3	814	3.5	810.5	9	805
RB-4	800	7	793	7	793
RB-5	826	11.5	814.5	NO	NO
RW-1	853	NO	NO	NO	NO
RW-2	847	7.5	839.5	9	838
RW-3	858	10.5	847.5	10.5	847.5

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[1] References the Indiana Department of Transportation (INDOT) Standard Specifications.

Table 2: Summary of Depths to Rock and Refusal at Exploratory Locations (continued)

Boring Location	Ground Surface El.	Depth to Rock (ft)	El. of Rock	Depth of Refusal (ft)	El. of Refusal
S-1	857	5	852	5	852
S-2	858	12.5	845.5	12.5	845.5
S-3	857	12.5	844.5	12.5	844.5
S-4	833	10	823	10	823
S-5	844	12.5	831.5	12.5	831.5
S-6	812	2	810	2	810
S-7	810	5	805	5	805
S-8	808	5	803	5	803

NO: Not observed

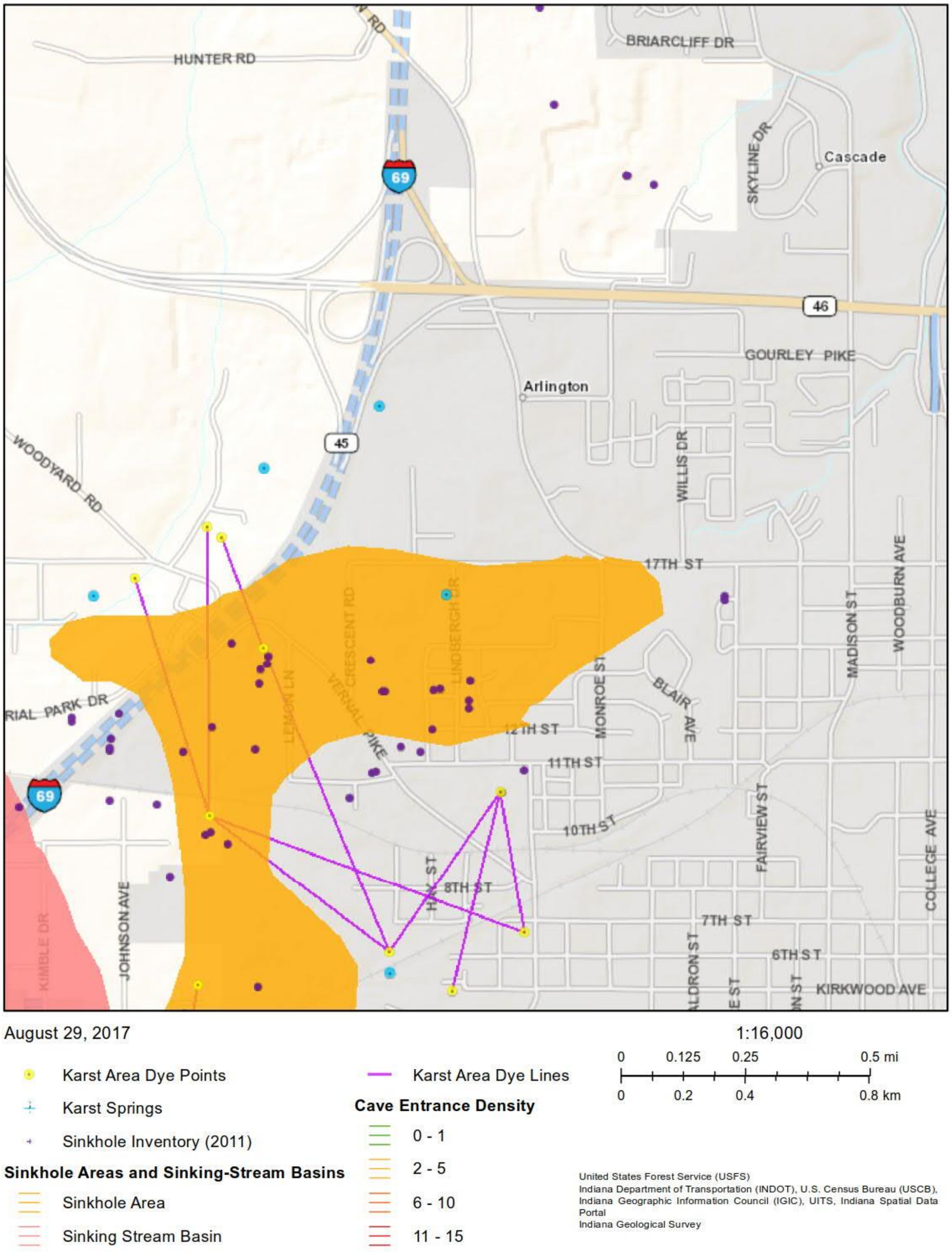
It should be noted that a siltstone boulder or floater was observed at Boring RB-3 from about 3½ to 6 ft below the existing grade. Underlying this rock, A-7-6 clay was observed to a depth of about 8½ ft, where siltstone with limestone seams was observed.

The consistency of the cohesive soils was typically stiff to very stiff based on hand penetrometer readings in the range of 1½ to 4 tons/sq. ft (tsf) and moisture contents were generally in the range of 21 to 31 percent for the A-6 soils and 30 to 48 percent for the A-7-6 soils. A layer of medium stiff A-6 clay loam was observed at Boring RB-5 near a depth of 2½ ft. Atterberg limit determinations performed on the A-6 soils indicated plasticity indices (PI) of 14 and 19 with liquid limits (LL) of 31 and 38 percent indicating medium plasticity. An Atterberg limit determination performed on the A-7-6 clay indicated a PI of 62 and a LL of 83 percent indicating very high plasticity.

As mentioned, the cohesive soils were typically underlain by siltstone and limestone. The rock was typically soft at the surface, based on the ability to scratch recovered samples with a metallic object, but quickly became hard based on observations and auger refusal.

It should be noted that the project lies in a general area of karst topography resulting from the solutioning of the underlying limestone bedrock. A search on the Indiana Map GIS system indicated that there are mapped karst features near the site. Figure 1 on the following page provides a map of these features and was generated using IndianaMap referencing information from the Indiana Geological Survey. However, an in-depth evaluation of the presence of karst features such as sinkholes, caverns, or springs was not included in our scope. It is also important to recognize in areas of karst that the bedrock surface can vary significantly and abruptly over very short distances (e.g., pinnacles and crevices).

Figure 1: Map of Karstic Features



United States Forest Service (USFS)
Indiana Department of Transportation (INDOT), U.S. Census Bureau (USCB), Indiana Geographic Information Council (IGIC), UITS, Indiana Spatial Data Portal
Indiana Geological Survey

Groundwater Conditions

Groundwater level observations were made during and shortly after completion of the sampling activities and are noted at the bottom of the boring logs. Groundwater was not observed at the soil boring locations within the timeframe of the exploratory activities. Based on our experience, the "piezometric" groundwater level is possible near the soil/rock interface but is likely deeper than the maximum depth explored. As additional input, review of the *Soil Survey of Monroe County* suggests that the water level remains below a depth of 6 ft throughout the year. It should be recognized that groundwater levels either piezometric or perched can fluctuate due to changes in precipitation, infiltration, surface run-off, and other hydrogeological factors.

DISCUSSION AND RECOMMENDATIONS

General

The subsurface conditions observed at the exploratory locations consisted of cohesive soils exhibiting medium to very high plasticity at a shallow depth. Based upon our understanding of the improvements and information obtained from the exploratory locations, it is our opinion that the subsurface conditions are generally conducive for the support of the roadway improvements, sewers, and modular block walls with reinforcement. The most critical aspect of this project, from a geotechnical perspective, will be preparation of the subgrades for support of these elements. Given the presence of high plasticity clay, improvement of the roadway and wall subgrades will be required. Additional discussion and recommendations regarding these issues are provided in the following paragraphs.

Earthwork

Subgrade Preparation

In areas to receive new pavement components and embankment fill, we recommend that topsoil, wet or soft near-surface soils, and existing pavement components be removed from within the construction limits. We recommend that root masses and soils containing organics be removed (grubbed) and the area regraded to avoid leaving depressions and areas that may collect water. In addition, we recommend that existing underground utilities in conflict with the proposed construction be appropriately relocated. Where utilities are relocated, we recommend that the resulting excavations be backfilled with B borrow in accordance with Section 203.09 of the ISS.

Once the subgrade is exposed, we recommend that the cohesive soils be proofrolled in accordance with the ISS. The purpose of proofrolling is to provide a first-order evaluation of how the subgrade is anticipated to react to construction traffic and gain an additional understanding of the conditions for support of the planned improvements. We recommend that the proofrolling be observed by an EEI geotechnical engineer or engineering technician. Based on observations at our test borings, we anticipate that yielding subgrade conditions will be exposed during the proofroll observations where the A-7-6 clay is present (i.e., Borings RB-2, RW-1, and RW-2) and possibly in other areas. We anticipate that improvement of the subgrade in at-grade sections and in areas where minimal earthwork (i.e., less than 1 ft of fill) is planned to establish the planned profile grade could be accomplished within the range of the subgrade treatment for the pavement (i.e., Type 1B). For areas with limited access (i.e., fill areas and retaining wall foundations), a proofroll may not be practical and evaluation of these areas may require a dynamic cone penetrometer (DCP) or probe rod. We recommend that undistributed quantities of undercutting (maximum depth of 1½ ft) and replacement with compacted crushed stone (INDOT No. 53) be included in the contract quantities for the purpose of addressing poor subgrades that would not be addressed via the subgrade treatment for pavement. A quantity of these items equal to 20 percent of the area below embankment fill should be included as a contingency.

In areas of planned embankment fill, we recommend that soft/yielding or otherwise unstable soils encountered during the proof-rolling operations which will not readily compact be aerated (if feasible) to reduce the moisture content and be recompacted per the ISS. However, based on the plasticity of the shallow soils, we anticipate that other means of stabilization of the foundation soils such as chemical drying (ISS 217) or the undercut discussed above may be required. The final decision regarding stabilization should be made at the time of construction, based on the observed actual conditions after removal of the surficial elements.

Fill Placement and Compaction

Based on the anticipated earthwork requirements both cut and fill will be required to establish proposed grades. Based on INDOT criteria, the A-7-6 soil is not suitable for use as fill. The A-6 soils are anticipated to be suitable for reuse as fill, as needed, provided they satisfy the recommendations for use as fill. The soils observed in the anticipated cut areas generally exhibited moisture contents in the range of 15 to 30 percent which is above the anticipated optimum moisture content for this soil type. Prior to use, they will require moisture conditioning in order to obtain adequate compaction. Moisture conditioning is typically accomplished by continuously discing the soils to reduce the moisture content and breakdown soil clods. However, this method requires favorable weather conditions (i.e., dry warm weather) and space to spread and work the soil. If the project timeline will not permit the use of discing or if the moisture contents during construction exceed those observed in our laboratory evaluation, chemical drying of the soils (ISS 217) may be utilized to dry the soils, or preconditioned imported soil may be necessary. For chemical drying, we recommend 5 percent (by mass [tons]) of product be considered for estimating purposes. In areas where smaller equipment will be necessary for compaction (i.e., due to space constraints), we recommend granular soil for fill.

The maximum anticipated earth fill placement height on the project will be about 5 ft. Based on the information obtained at the boring locations, the soil/rock interface is anticipated to be near the embankment foundation elevation from Station 26+50 to 27+50. Therefore, we recommend that a quantity of rock excavation (discussed later in this report) be included at this location. Standard embankment construction practices outlined in the ISS and as discussed above should provide an adequate subgrade for embankment construction.

Based on a review of the plans, sideslopes as steep as 3H:1V are anticipated. Global instability of these slopes is not of concern; however the performance of these slopes will be directly dependent on the subgrade preparation and quality of compaction achieved in the embankments, as previously discussed. Benches should be cut into any existing slopes steeper than 6H:1V before fill placement so as to key the new fill into the slope. Due to the relatively short embankment heights, 6-ft wide benches (i.e., minimum) are recommended. Scarifying of the slope will also aid in keying the new fill into the slope. Additionally, finished slopes steeper than 3H:1V can create maintenance issues as they are not accessible with conventional mowing equipment and tend to slough (surficial). To minimize sloughing and erosion, it is important to provide adequate compaction and erosion and sloughing protection at the face of the embankment.

Pavement Design Considerations

Based on our observations at the exploratory locations, the pavement subgrade is anticipated to consist of cohesive soils having medium to very high plasticity. Based on the results of the resilient modulus testing, we recommend that the information in Table 3 be considered for pavement design.

Table 3: Pavement Design Parameters

M_r for Improved Subgrade	7,500 psi
M_r for Natural Subgrade	4,000 psi
Subgrade Material	Clay (A-7-6)
Depth to Water	> 6 ft
Subgrade Treatment	Type IB

* From field observations and the Soil Survey of Monroe County, Indiana

The use of Type IB subgrade treatment, utilizing a slurry if necessary due to the nearby residences, will improve the subgrade's ability to support the proposed pavements. This subgrade treatment is recommended anticipating a full closure, allowing contractors to utilize equipment to chemically treat large areas. If alternate MOT plans are utilized to maintain traffic during construction, the subgrade treatment will be applied in phases, and a consistent use of Type IB subgrade treatment may not be feasible. Type IC subgrade treatment may be utilized in isolated areas where a Type IB treatment is not practical. However, difficulty achieving compaction in implementing a Type IC subgrade treatment should be anticipated at locations where the A-7-6 Clay is present. As a result, additional subgrade stabilization in addition to the subgrade treatment may be necessary depending on the site conditions at the time of construction. If a phased MOT will be considered, we recommend including additional quantities for undercut and No. 53 crushed stone in conjunction with a Type IB geogrid to address these areas.

It is very important to provide positive drainage during construction before the subgrade treatment is performed in order to reduce the risk of any wet soil conditions. We recommend that the new storm sewers, discussed in the next section of this report, be constructed early in the project to help improve site drainage and reduce the risk of ponding water on proposed pavement subgrades. In addition, the subgrade should be graded at the end of each day to facilitate positive drainage. Water infiltration into cohesive subgrade soils will reduce the life of a pavement section. Since these soils have a low permeability, any water which may infiltrate the subgrade would affect the long-term performance of the pavement. To reduce the impact of moisture on the pavement performance, we recommend that the pavement surface and the subgrade be sloped to drain towards the proposed sewers. The long-term performance of pavement is a function of routine maintenance (e.g., crack sealing) which will be the responsibility of the owner to perform.

Modular Block Wall Considerations

Based on the soil boring information and the proposed construction, the conditions are generally conducive for support of the proposed modular block walls. We recommend that the foundation soil below the walls be evaluated using a dynamic cone penetrometer (DCP) and improved as necessary in accordance with ISS 731.07. Based on the observed conditions at the test boring locations, rock is anticipated to be at or near the leveling pad elevation for the wall on the south side of 17th Street near Station 19+50, and for the wall to the north of the path from about Station 21+00 to about Station 22+00. At these locations, rock excavation for the leveling pad and the reinforced soil zone should be anticipated. At other locations, stiff to very stiff cohesive soils are expected to be present and undercutting is not anticipated to be necessary provided care is taken to protect the foundation soils from exposure to moisture and repeated construction traffic. The cohesive-type soils anticipated at the base of the walls are moisture-sensitive and will soften if exposed to water. Consequently, the quality of the foundation soil and the need for undercutting will be directly contingent on the workmanship of the contractor. If undercutting is required, we recommend the undercut areas be replaced with INDOT No. 53 crushed stone compacted to 100 percent of the maximum dry density in accordance with AASHTO T 99. Any undercutting performed will be required below the influence of the modular block wall fill (i.e., with a 1H:1V line of influence beyond the front face and rear of the reinforced area).

In evaluating the design for modular block retaining walls, the external and internal stability should be analyzed. For external stability, the following four standard modes of failure are typically addressed: 1) sliding [minimum resistance factor ≥ 1.0]; 2) eccentricity [$e \leq L/4$]; 3) bearing capacity ($q \leq øq_n$); and 4) global stability (resistance factor of 0.65 to 0.75). To evaluate the internal stability, three standard modes of failure are typically addressed. These include: 1) pullout of the soil reinforcement; 2) tensile overstress of the soil reinforcement and wall connection; and 3) corrosion (steel) and or creep (for high-density polypropylene products) of the soil reinforcement. We understand that the wall manufacturer will evaluate the internal stability of the wall system, including that of any gravity-type wall elements.

Analyses of a cut condition and a fill condition were performed considering the respective maximum retained heights, foundation soils consisting of stiff cohesive soil, and with traffic surcharge loading, where appropriate. The analysis for over-turning and sliding considered the maximum height of the retaining walls and a minimum width of 0.7 times the height or 6 ft (whichever is greater). Our analysis indicated adequate resistance to sliding, eccentricity, and bearing, provided the foundation is adequately prepared and improved, where necessary, as described. Provided the subgrade and foundation is prepared as discussed (including the undercutting), the global stability indicated the proposed geometry exhibited an acceptable factors of safety. A factored bearing resistance of 5,000 psf for all of the walls is acceptable. The same resistance is recommended for walls founded on rock and soil to reduce the risk of a bearing failure in the soil. Due to the relatively shallow bedrock in conjunction with the stiff to very stiff (overconsolidated) soils, consolidation settlement of the foundation soil is not of concern.

Due to the cut condition and the location of the creek, we recommend that the structure backfill be INDOT No. 8 crushed stone and that adequate drainage be provided (with cleanouts) in the structure backfill and at the base of the modular block walls in accordance with INDOT Design Memorandum No. 17-03. Also, in accordance with Chapter 11 of AASHTO, we recommend a 4-ft wide horizontal bench be constructed at the base of modular block walls constructed on a slope. The recommended embedment depth of the leveling course is also based on the slope. Refer to Chapter 11 for additional commentary on geometric requirements. For the fill wall, we recommend scour and erosion protection be provided via appropriately sized riprap.

East of Station 23+50, the walls planned on the north side of 17th Street and the path are in residential yards, many of which feature mature trees. It is important to remember that construction of a modular block wall in a cut condition requires excavation for the reinforced zone and excavation behind this zone to create a safe slope during construction. The wall near Station 27+00 is planned to be about 8 ft tall and the house on this property is located about 20 ft behind the wall. Considering the estimate of reinforcement length and a typical 1H:1V cutback slope during construction, the construction geometry is anticipated to come close to the houses. We recommend that you evaluate the walls and construction excavation geometry relative to the location of existing mature trees and houses. We recommend that EEI be retained to evaluate any changes in the design. An alternate wall type (e.g., cast-in-place concrete, gravity block) and/or detailed evaluation of cut slopes may be necessary.

Storm Sewer Considerations

We understand that invert subgrades for the sewers are planned to be established within about 4 ft of the existing ground surface. Based on our observations at the exploratory locations, relatively stiff cohesive soils are anticipated to be encountered at the subgrade. However, as evidenced by the observations at Boring RB-3 and the variation in the depth to rock in Table 2, rock excavation will likely be necessary in isolated locations during the construction of the sewer. In our opinion, these conditions are generally adequate for support of the pipes (i.e., the net load on the supporting conditions is anticipated to be nominal [possibly less than the overburden]). The condition of the subgrade will be, in part, a function of the care and workmanship of the contractor in protecting the subgrade from water. The cohesive soils observed at the test boring locations are moisture-sensitive and will soften when exposed to water. If soft soils are encountered at the base of the trench excavations or the condition of the subgrade deteriorates in the presence of moisture, it is our opinion they should be removed and replaced with compacted structure backfill material to achieve a stable base. Although not anticipated, if the use of structure backfill is not feasible due to the depth of unstable materials, the use of geogrid and/or compacted crushed aggregate may be required to stabilize the trench. In this case, a minimum of 2 ft of the soft soils should be removed prior to stabilization.

In our opinion, a minimum 6-in. thick bedding layer consisting of structure backfill material should be provided for pipe support. This includes areas where rock is present at the invert. Since the pipes are anticipated to be located beneath or adjacent to the proposed roadways, the trenches should be backfilled to grade with structure backfill material. In our opinion, the structure backfill material should be compacted to 95 percent of maximum dry density obtained in accordance with AASHTO T 99 and INDOT Specifications. Hand or remote guided vibratory compactors are recommended for compacting the bedding material and material on either side of the pipe. The first several lifts of backfill over the pipe should also be compacted with small vibratory compactors to assure proper compaction is achieved and to prevent damage to the pipe from heavier, high-energy compactors.

Excavations and Dewatering

We recommend that excavated soil not be stockpiled immediately adjacent to the top of the excavation nor should equipment be allowed to operate too closely to excavations. Furthermore, all excavations should conform to Occupational Safety and Health Administration (OSHA) requirements (i.e., 29 CFR Part 1926). Excavation safety is solely the responsibility of the contractor.

As mentioned previously, auger refusal on siltstone and limestone bedrock was encountered at several boring locations along the proposed roadway and modular block walls. At locations where the hollow-stem augers were able to penetrate the rock, the rock at this location may be rippable and/or could possibly be broken with a hydraulic hammer or with conventional earthwork equipment. However due to variations in the rock that were observed at the test boring locations and that are inherent with the geology of Bloomington, some areas may not be rippable with conventional earthwork equipment. The actual method of rock removal to be used cannot be speculated with certainty. However where hard rock is encountered, from experience, other methods have included hydraulic hammers and heavier mechanical equipment. We recommend that the quantities for rock excavation be based on the top of rock and not on the depth of auger refusal. Rock excavation is anticipated along portions of the walls west of Station 22+00 and, possibly, near the toe of the embankment fill planned from Station 26+50 to 27+50.

For shallow excavations in the observed cohesive soils, dewatering is anticipated to consist of traditional pumps and filtered sumps possibly in combination with collection trenches provided the level of the creek does not rise.

CONCLUDING REMARKS

In closing, EEI's professional services were performed, our findings obtained, and our preliminary recommendations prepared in accordance with generally and currently accepted geotechnical engineering practices. This warranty is in lieu of all other warranties either expressed or implied.

We appreciate the opportunity to provide our services to you on this project. Please contact our office if you have any questions or need further assistance with the project.

Sincerely,

EARTH EXPLORATION, INC.

Kellen P. Heavin, P.E.
Senior Geotechnical Engineer

KELLEN PATRICK HEAVIN
PE11011658

Curtis R. Bradburn, P.E.
Senior Geotechnical Engineer

Attachments –
APPENDIX A - Important Information about This Geotechnical Engineering Report
APPENDIX B - Field Methods for Exploring and Sampling Soils and Rock
APPENDIX C - Exploratory Location Plan (Drawing No. 1-17-052.B1)
Log of Test Boring - General Notes
Log of Test Boring (8)
Summary of Soundings
Unconfined Compression Test (2)

APPENDIX A

IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer about Change

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

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

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

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

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

This Report May Not Be Reliable

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX B

FIELD METHODS FOR EXPLORING AND SAMPLING SOILS AND ROCK

FIELD METHODS FOR EXPLORING AND SAMPLING SOILS AND ROCK

A. Boring Procedures Between Samples

The boring is extended downward, between samples, by a hollow stem auger (AASHTO* Designation T251), continuous flight auger, driven and washed-out casing, or rotary boring with drilling mud or water.

B. Standard Penetration Test and Split-Barrel Sampling of Soils

(AASHTO* Designation: T206)

This method consists of driving a 2-in. outside diameter split-barrel sampler using a 140-lb weight falling freely through a distance of 30 in. The sampler is first seated 6 in. into the material to be sampled and then driven 12 in. The number of blows required to drive the sampler the final 12 in. is recorded on the Log of Test Boring and known as the Standard Penetration Resistance or N-value. Recovered samples are first classified as to texture by the field personnel. Later in the laboratory, the field classification is reviewed by a geotechnical engineer who observes each sample.

C. Thin-walled Tube Sampling of Soils

(AASHTO* Designation: T207)

This method consists of hydraulically pushing a 2-in. or 3-in. outside diameter thin wall tube into the soil, usually cohesive types. Relatively undisturbed samples are recovered.

D. Soil Investigation and Sampling by Auger Borings

(AASHTO* Designation: T203)

This method consists of augering a hole and removing representative soil samples from the auger flight or bucket at 5-ft intervals or with each change in the substrata. Relatively disturbed samples are obtained and its use is therefore limited to situations where it is satisfactory to determine approximate subsurface profile.

E. Diamond Core Drilling for Site Investigation

(AASHTO* Designation: T225)

This method consists of advancing a hole in rock or other hard strata by rotating downward a single tube or double tube core barrel equipped with a cutting bit. Diamond, tungsten carbide, or other cutting agents may be used for the bit. Wash water is used to remove the cuttings. Normally, a 3-in. outside diameter by 2-in. inside diameter coring bit is used unless otherwise noted. The rock or hard material recovered within the core barrel is examined in the field and laboratory. Cores are stored in partitioned boxes and the length of recovered material is expressed as a percentage of the actual distance penetrated.

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* American Association of State Highway and Transportation Officials, Washington D.C.

APPENDIX C

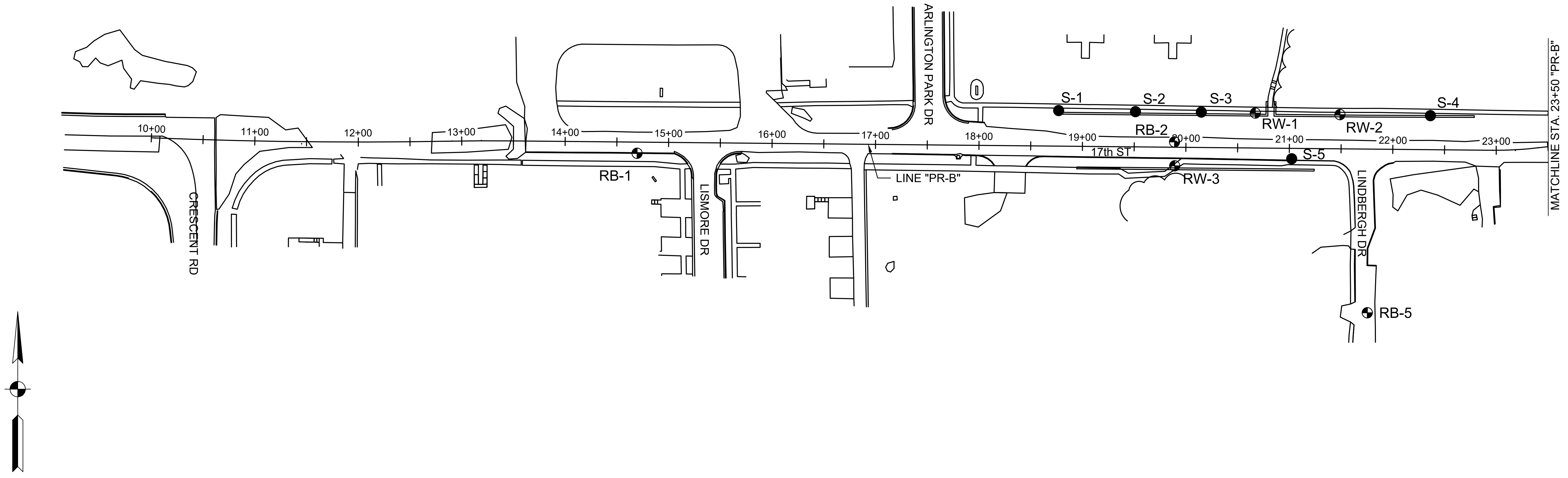
EXPLORATORY LOCATION PLAN (Drawing No. 1-17-052.B1)

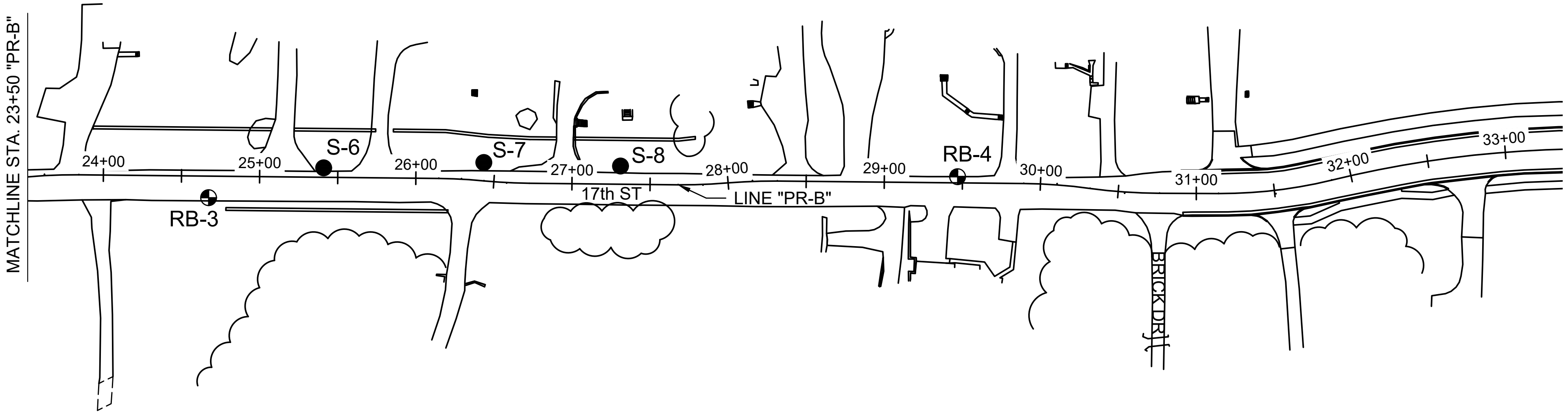
LOG OF TEST BORING - GENERAL NOTES

LOG OF TEST BORING (8)

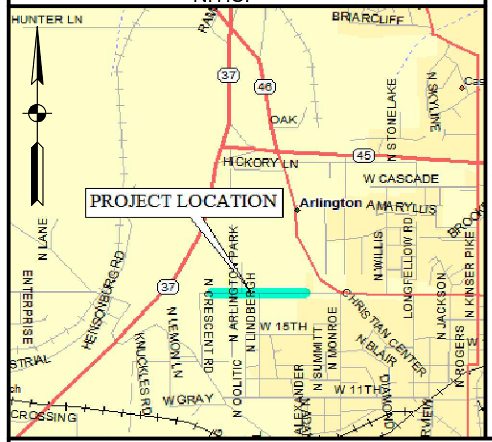
SUMMARY OF SOUNDINGS

UNCONFINED COMPRESSION TEST (2)





VICINITY MAP
N.T.S.



LEGEND

RB-1	⊕	Test Boring Location and Designation
S-1	●	Sounding Location and Designation

NOTES

1. Base map developed from an electronic file provided by Aztec Engineering Group, Inc. on July 27, 2017.
2. Vicinity map generated using commercially-available software by DeLorme (Street Atlas USA ver. 8.0).
3. Borings and soundings were located in the field by Earth Exploration, Inc. on June 20, 2017.
4. Ground surface elevations at the test boring locations were interpolated to the nearest 1 ft based on topographic information provided on the previously mentioned plan.
5. Exploratory locations are approximate.

EXPLORATORY LOCATION PLAN

PROJECT: 17th Street Improvements
LOCATION: Bloomington, Indiana
CLIENT: Aztec Engineering Group, Inc.
EEI PROJECT NO.: 1-17-052
SCALE: 1" = 100'

PROJECT ENG: KPH
APPROVED BY: MSW
DRAWN BY: JBF
DATE: 8/20/17
DRAWING NO.: 1-17-052.B1

EARTH EXPLORATION INC
A Terracon COMPANY

LOG OF TEST BORING – GENERAL NOTES

DESCRIPTIVE CLASSIFICATION

GRAIN SIZE TERMINOLOGY

Soil Fraction	Particle Size	US Standard Sieve Size
Boulders	Larger than 75 mm	Larger than 3”
Gravel	4.76 mm to 75 mm	#10 to 75 mm
Sand: Coarse	2.00 to 4.76 mm	#40 to #10
Fine	0.075 to 0.42 mm	#200 to #40
Silt	0.002 to 0.075 mm	Smaller than #200
Clay	Smaller than 0.002 mm	Smaller than #200

GENERAL TERMINOLOGY

Physical Characteristics
- Color, moisture, grain shape fineness, etc.

Major Constituents
- Clay silt, sand, gravel

Structure
- Laminated, varved, fibrous, stratified, cemented, fissured, etc.

Geologic Origin
- Glacial, alluvial, eolian, residual, etc.

PERCENTAGE MODIFIERS

Term	Defining Range by % of Weight
Trace	1 – 10%
Little	11 – 20%
Some	21 – 35%
And	36 – 50%

ORGANIC CONTENT BY COMBUSTION METHOD

Soil Description	LOI
w/ organic matter	4 – 15 %
Organic Soil (A-8)	16 – 30%
Peat (A-8)	More than 30%

RELATIVE DENSITY

Term	“N” Value
Very loose	0 – 5
Loose	6 – 10
Medium dense	11 – 30
Dense	31 – 50
Very Dense	51+

CONSISTENCY

Term	“N Value”
Very soft	0 - 3
Soft	4 - 5
Medium	6 - 10
Stiff	11 - 15
Very Stiff	16 - 30
Hard	31+

PLASTICITY

Term	Plastic Index
None to slight	0 – 4
Slight	5 – 7
Medium	8 – 22
High/Very High	Over 22

The penetration resistance, N, is the summation of the number of blows required to effect two successive 6-in. penetrations of the 2-in. split-barrel sampler. The sampler is driven with a 140-lb weight falling 30 in. and is seated to a depth of 6 in. before commencing the standard penetration test.

SYMBOLS

DRILLING AND SAMPLING

AS – Auger Sample
BS – Bag Sample
C – Casing Size 2½”, NW, 4”, HW
COA – Clean-Out Auger
CS – Continuous Sampling
CW – Clear Water
DC – Driven Casing
DM – Drilling Mud
FA – Flight Auger
FT – Fish Tail
HA – Hand Auger
HSA – Hollow Stem Auger
NR – No Recovery
PMT – Borehole Pressuremeter Test
PT – 3” O.D. Piston Tube Sample
PTS – Peat Sample
RB – Rock Bit
RC – Rock Coring
REC – Recovery
RQD – Rock Quality Designation
RS – Rock Sounding
S – Soil Sounding
SS – 2” O.D. Split-Barrel Sample
2ST – 2” O.D. Thin-Walled Tube Sample
3ST – 3” O.D. Thin-Walled Tube Sample
VS – Vane Shear Test
WPT – Water Pressure Test

LABORATORY TESTS

q_p – Penetrometer Reading, tsf
q_u – Unconfined Strength, tsf
W – Moisture Content, %
LL – Liquid Limit, %
PL – Plastic Limit, %
PI – Plasticity Index
SL – Shrinkage Limit, %
LOI – Loss on Ignition, %
γ_d – Dry Unit Weight, pcf
pH – Measure of Soil Alkalinity/Acidity

WATER LEVEL MEASUREMENT

BF – Backfilled upon Completion
NW – No Water Encountered

Note: Water level measurements shown on the boring logs represent conditions at the time indicated and may not reflect static levels, especially in cohesive soils.

LOG OF TEST BORING

BORING NO.: **RB-1**
SHEET 1 OF 1
NORTHING : 1432123
EASTING : 3102499
DATUM : USC & GS
DATE STARTED : 07-20-17
DATE COMPLETED : 07-20-17

CLIENT : AZTEC Engineering Group, Inc.
DES NO. : ---
STRUCTURE # : ---
PROJECT TYPE : Roadway Improvements
LOCATION : 17th Street
COUNTY : Monroe
PROJECT NO.: 1-17-052

ELEVATION : 868.0
STATION : 14+70
OFFSET : 7.0 ft Right
LINE : "PR-B"
DEPTH : 12.5 ft

BORING METHOD : Hollow Stem Auger
RIG TYPE : Geoprobe 7822
CASING DIA. : 3¼
CORE SIZE : ---

HAMMER : Auto
DRILLER/INSP : Z.M.
TEMPERATURE : 85 °F
WEATHER : Overcast

GROUNDWATER: ∇ Encountered at NW ▼ At completion NW ▨ Caved in at 8.5 ft

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION		SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS			REMARKS
											LL	PL	PI	
		Asphaltic Concrete	0.5											
		Granular Subbase, (crushed stone)	1.0											
	2.5	**Clay Loam,** stiff, moist, brown, with rock fragments near 5 ft, A-6		SS 1	5-5-12	78	21.8		1.5		31	17	14	1.7, pH = 7.4
865.0	5.0		6.0	SS 2	5-5-6	67	24.0		2.0					
	7.5	**Clay,** very stiff, moist, brown, with rock fragments near 6 ft, A-7-6		SS 3	3-3-4	89	34.2		2.75					
860.0	10.0			SS 4	3-5-7	100	26.5		3.75					
	12.5		12.5	SS 5	3-4-6	100	27.1		2.25					
855.0		Bottom of Boring at 12.5 ft												
	15.0													
850.0	17.5													
	20.0													
845.0	22.5													
	25.0													
840.0	27.5													
	30.0													

LOG OF TEST BORING

BORING NO.: **RB-2**
SHEET 1 OF 1
NORTHING : 1432134
EASTING : 3103019
DATUM : USC & GS
DATE STARTED : 07-18-17
DATE COMPLETED : 07-18-17

CLIENT : AZTEC Engineering Group, Inc.
DES NO. : ---
STRUCTURE # : ---
PROJECT TYPE : Roadway Improvements
LOCATION : 17th Street
COUNTY : Monroe
PROJECT NO.: 1-17-052

ELEVATION : 857.0
STATION : 19+88
OFFSET : 6.0 ft Left
LINE : "PR-B"
DEPTH : 20.0 ft

BORING METHOD : Hollow Stem Auger
RIG TYPE : CME 55 Track
CASING DIA. : 3¼
CORE SIZE : ---

HAMMER : Auto
DRILLER/INSP : J.S.
TEMPERATURE : 80 °F
WEATHER : Partialy Cloudy

GROUNDWATER: ▽ Encountered at NW ▼ At completion NW ☒ Caved in at 16.0 ft

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS LL	PL	PI	REMARKS
		Asphaltic Concrete 1.0											
855.0	2.5	**Clay,** stiff to very stiff, moist, brown, with silt seams near 2 ft, 10 ft, and 15 ft, A-7-6	SS 1	3-2-3	89	35.1		1.5					
	5.0		SS 2	2-3-4	100	32.4		1.75					
850.0	7.5		SS 3	2-4-5	100	34.8		2.5		83	21	62	6.7, pH = 7.5
	10.0		SS 4	2-2-5	100	31.6		2.0					
845.0	12.5		SS 5	2-3-4	100	42.2		1.75					
	15.0		SS 6	2-3-4	100	36.4		1.75					
840.0	17.5	19.0	SS 7	1-2-4	100	47.8		1.5					
	20.0	**Silty Clay,** stiff, moist, brown 20.0	SS 8	3-5-8	100	16.4		2.0					
835.0	22.5	Bottom of Boring at 20.0 ft											
	25.0												
830.0	27.5												
	30.0												

LOG OF TEST BORING

BORING NO.: **RB-3**
SHEET 1 OF 1
NORTHING : 1432116
EASTING : 3103497
DATUM : USC & GS
DATE STARTED : 07-18-17
DATE COMPLETED : 07-18-17

CLIENT : AZTEC Engineering Group, Inc.
DES NO. : ---
STRUCTURE # : ---
PROJECT TYPE : Roadway Improvements
LOCATION : 17th Street
COUNTY : Monroe
PROJECT NO.: 1-17-052

ELEVATION : 814.0
STATION : 24+68
OFFSET : 14.0 ft Right
LINE : "PR-B"
DEPTH : 9.0 ft

BORING METHOD : Hollow Stem Auger
RIG TYPE : CME 55 Track
CASING DIA. : 3¼
CORE SIZE : ---

HAMMER : Auto
DRILLER/INSP : J.S.
TEMPERATURE : 80 °F
WEATHER : Partialy Cloudy

GROUNDWATER: ▽ Encountered at NW ▼ At completion NW ☒ Caved in at 7.5 ft

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS LL	PL	PI	REMARKS
		Asphaltic Concrete 1.3											
	2.5	**Silty Loam,** stiff, moist, brown 3.5	SS 1	3-2-5	67	29.5		1.0					
810.0	5.0	**Weathered Siltstone**, soft, brown, with limestone seam near 5 ft 6.0	SS 2	2-4-23	33								
	☒ 7.5	**Clay,** stiff, black, with limestone fragments near 8.5 ft, A-7-6 8.5	SS 3	3-3-3	33	32.4		1.75					
805.0		**Weathered Siltstone**, with limestone seams 9.0	SS 4	50/2	83								
	10.0	Bottom of Boring at 9.0 ft											
		Auger refusal at 9 ft											
	12.5												
800.0	15.0												
	17.5												
795.0	20.0												
	22.5												
790.0	25.0												
	27.5												
785.0	30.0												

LOG OF TEST BORING

BORING NO.: **RB-4**
SHEET 1 OF 1
NORTHING : 1432129
EASTING : 3103976
DATUM : USC & GS
DATE STARTED : 07-18-17
DATE COMPLETED : 07-18-17

CLIENT : AZTEC Engineering Group, Inc.
DES NO. : ---
STRUCTURE # : ---
PROJECT TYPE : Roadway Improvements
LOCATION : 17th Street
COUNTY : Monroe
PROJECT NO.: 1-17-052

ELEVATION : 800.0
STATION : 29+56
OFFSET : 0.0 ft
LINE : "PR-B"
DEPTH : 7.0 ft

BORING METHOD : Hollow Stem Auger
RIG TYPE : CME 55 Track
CASING DIA. : 3¼
CORE SIZE : ---

HAMMER : Auto
DRILLER/INSP : J.S.
TEMPERATURE : 87 °F
WEATHER : Partialy Cloudy

GROUNDWATER: ▽ Encountered at NW ▼ At completion NW ☒ Caved in at 5.0 ft

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS LL	PL	PI	REMARKS
		Asphaltic Concrete 1.1											
	2.5	**Clay,** stiff, moist, brown, A-6	SS 1	3-4-5	67	29.8		1.5		38	19	19	1.7, pH = 4.6
795.0 ☒	5.0		SS 2	2-3-4	89	30.6		1.0					
		6.0 **Clay,** very stiff, moist, brown, with limestone fragments near 6.5 ft, A-7-6 7.0	SS 3	5-50/3	83	32.1		2.25					
	7.5	Bottom of Boring at 7.0 ft Auger refusal at 7 ft											
790.0	10.0												
	12.5												
785.0	15.0												
	17.5												
780.0	20.0												
	22.5												
775.0	25.0												
	27.5												
770.0	30.0												

LOG OF TEST BORING

BORING NO.: **RB-5**
SHEET 1 OF 1
NORTHING : 1431968
EASTING : 3103205
DATUM : USC & GS
DATE STARTED : 07-18-17
DATE COMPLETED : 07-18-17

CLIENT : AZTEC Engineering Group, Inc.
DES NO. : ---
STRUCTURE # : ---
PROJECT TYPE : Roadway Improvements
LOCATION : 17th Street
COUNTY : Monroe
PROJECT NO.: 1-17-052

ELEVATION : 826.0
STATION : 21+73
OFFSET : 160.0 ft Right
LINE : "PR-B"
DEPTH : 11.8 ft

BORING METHOD : Hollow Stem Auger
RIG TYPE : CME 55 Track
CASING DIA. : 3¼
CORE SIZE : ---

HAMMER : Auto
DRILLER/INSP : J.S.
TEMPERATURE : 90 °F
WEATHER : Partialy Cloudy

GROUNDWATER: ∇ Encountered at NW ▼ At completion NW ☒ Caved in at 13.0 ft

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS LL	PL	PI	REMARKS
		Asphaltic Concrete 0.8											
825.0	2.5	**Clay Loam,** medium stiff to stiff, moist, brown, A-6	SS 1	2-2-3	100	25.8		0.75					
	5.0	6.0	SS 2	2-4-7	89	15.7		1.5					
820.0	7.5	**Clay,** stiff, moist, brown, A-7-6	SS 3	2-4-6	100	35.9		2.0					
	10.0	11.5	SS 4	3-3-3	100	29.9		1.75					
815.0	12.5	**Weathered Siltstone**, soft, brown, with limestone fragments near 11.5 ft 11.8	SS 5	44-50/3	63								
		Bottom of Boring at 11.8 ft											
810.0	15.0 17.5												
805.0	20.0 22.5												
800.0	25.0 27.5 30.0												

LOG OF TEST BORING

BORING NO.: **RW-1**
SHEET 1 OF 1
NORTHING : 1432162
EASTING : 3103097
DATUM : USC & GS
DATE STARTED : 07-18-17
DATE COMPLETED : 07-18-17

CLIENT : AZTEC Engineering Group, Inc.
DES NO. : ---
STRUCTURE # : ---
PROJECT TYPE : Roadway Improvements
LOCATION : 17th Street
COUNTY : Monroe
PROJECT NO.: 1-17-052

ELEVATION : 853.0
STATION : 20+66
OFFSET : 33.0 ft Left
LINE : "PR-B"
DEPTH : 15.0 ft

BORING METHOD : Hollow Stem Auger
RIG TYPE : CME 55 Track
CASING DIA. : 3¼
CORE SIZE : ---

HAMMER : Auto
DRILLER/INSP : J.S.
TEMPERATURE : 91 °F
WEATHER : Partialy Cloudy

GROUNDWATER: ▽ Encountered at NW ▼ At completion NW ☒ Caved in at 12.0 ft

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS			REMARKS
										LL	PL	PI	
		Topsoil 0.5											
	2.5		SS 1	3-4-4	89	32.7		4.0					
850.0													
	5.0		SS 2	2-4-4	100	30.4	89.0	2.5	3.14				
	7.5	**Clay,** very stiff, moist, brown, with rock fragments near 2 ft and 10 ft, A-7-6	SS 3	3-4-4	100	28.5		2.5					
845.0													
	10.0		SS 4	4-4-6	100	30.9		2.75					
☒	12.5		SS 5	2-4-6	100	29.5		2.75					
840.0													
	15.0	15.0	SS 6	2-4-5	100	25.8		2.25					
		Bottom of Boring at 15.0 ft											
	17.5												
835.0													
	20.0												
	22.5												
830.0													
	25.0												
	27.5												
825.0													
	30.0												

LOG OF TEST BORING

BORING NO.: **RW-2**
SHEET 1 OF 1
NORTHING : 1432160
EASTING : 3103179
DATUM : USC & GS
DATE STARTED : 07-20-17
DATE COMPLETED : 07-20-17

CLIENT : AZTEC Engineering Group, Inc.
DES NO. : ---
STRUCTURE # : ---
PROJECT TYPE : Roadway Improvements
LOCATION : 17th Street
COUNTY : Monroe
PROJECT NO.: 1-17-052

ELEVATION : 847.0
STATION : 21+48
OFFSET : 32.0 ft Left
LINE : "PR-B"
DEPTH : 9.0 ft

BORING METHOD : Hollow Stem Auger
RIG TYPE : Geoprobe 7822
CASING DIA. : 3¼
CORE SIZE : ---

HAMMER : Auto
DRILLER/INSP : Z.M.
TEMPERATURE : 91 °F
WEATHER : Sunny

GROUNDWATER: ∇ Encountered at NW ▼ At completion NW ☒ Caved in at 5.0 ft

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS			REMARKS
										LL	PL	PI	
		Topsoil 0.3											
845.0	2.5		SS 1	3-3-3	78	31.7		2.75					
	5.0	**Clay,** very stiff, moist, brown, with rock fragments near 7 ft, A-7-6	SS 2	3-2-3	100	34.0	88.1	2.25	2.57				
840.0	7.5	7.5	SS 3	4-7-10	100	35.0		2.25					
		Weathered Limestone, with clay seams 9.0	SS 4	50/5	83								
	10.0	Bottom of Boring at 9.0 ft											
		Auger refusal at 9 ft											
835.0	12.5												
	15.0												
830.0	17.5												
	20.0												
825.0	22.5												
	25.0												
820.0	27.5												
	30.0												

LOG OF TEST BORING

BORING NO.: **RW-3**
SHEET 1 OF 1
NORTHING : 1432111
EASTING : 3103019
DATUM : USC & GS
DATE STARTED : 07-20-17
DATE COMPLETED : 07-20-17

CLIENT : AZTEC Engineering Group, Inc.
DES NO. : ---
STRUCTURE # : ---
PROJECT TYPE : Roadway Improvements
LOCATION : 17th Street
COUNTY : Monroe
PROJECT NO.: 1-17-052

ELEVATION : 858.0
STATION : 19+88
OFFSET : 18.0 ft Right
LINE : "PR-B"
DEPTH : 10.5 ft

BORING METHOD : Hollow Stem Auger
RIG TYPE : Geoprobe 7822
CASING DIA. : 3¼
CORE SIZE : ---

HAMMER : Auto
DRILLER/INSP : Z.M.
TEMPERATURE : 85 °F
WEATHER : Overcast

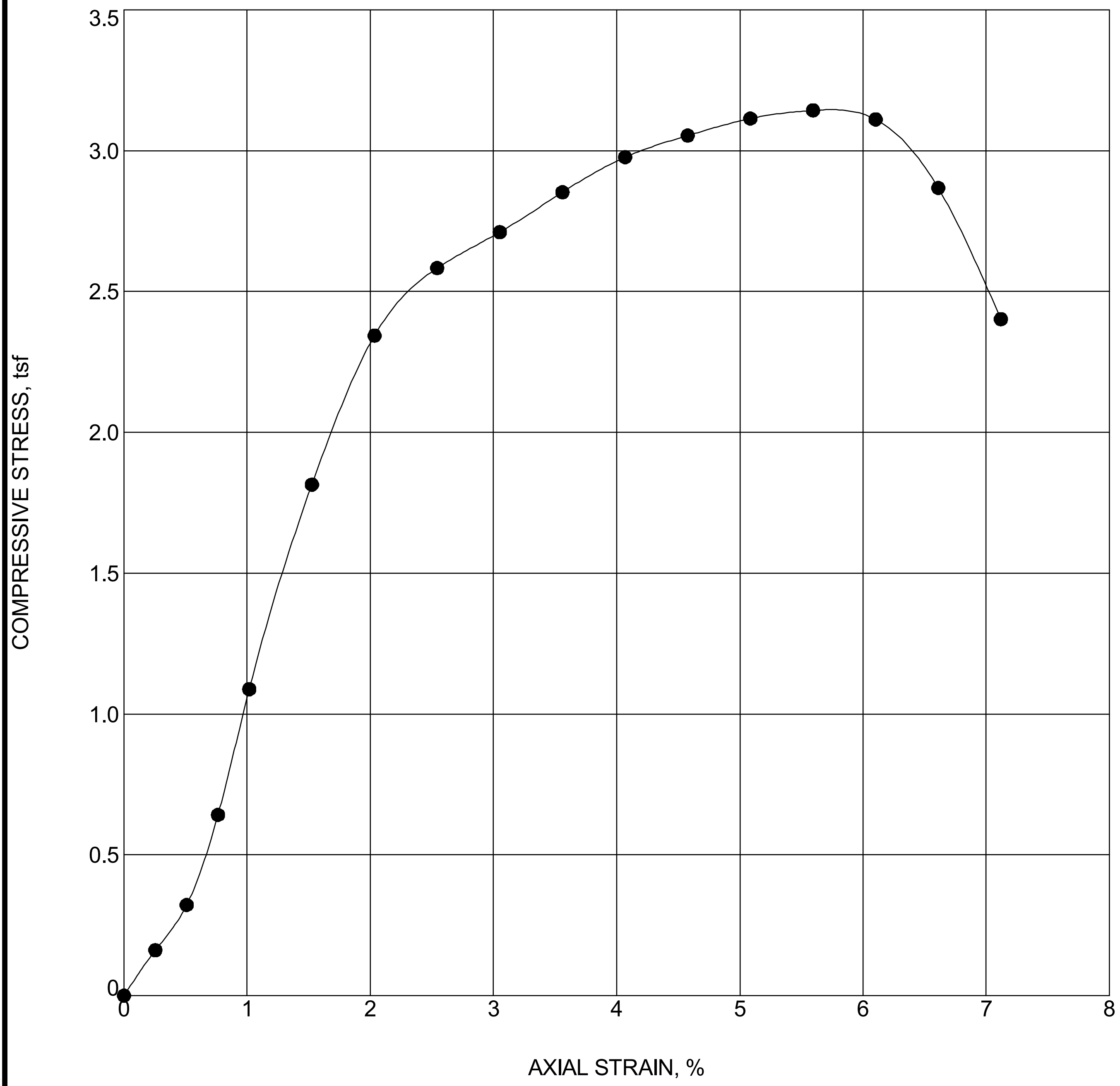
GROUNDWATER: ∇ Encountered at NW ▼ At completion NW

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS LL	PL	PI	REMARKS
		Asphaltic Concrete 0.2											
855.0	2.5	**Silty Loam,** very stiff, moist, brown 3.5	SS 1	4-4-5	89	15.5		2.25					
	5.0	**Clay Loam,** hard, moist, brown, A-6 5.5	SS 2	5-6-6	100	21.1		>4.5					
850.0	7.5	**Clay,** very stiff to hard, moist, brown, A-7-6 10.5	SS 3	4-4-4	100	30.0	91.8	>4.5					
	10.0		SS 4	4-4-5	100	43.6		2.25					
845.0	12.5	Bottom of Boring at 10.5 ft Auger refusal at 10.5 ft											
	15.0												
840.0	17.5												
	20.0												
835.0	22.5												
	25.0												
830.0	27.5												
	30.0												

SUMMARY OF SOUNDINGS

Project: 17th Street Roadway Improvements
Location: Bloomington, Monroe Co., IN
Client: AZTEC Engineering Group, Inc.
EEI Project No.: 1-17-052

Sounding Designation	Depth to Top of Rock (ft)	Depth to Refusal (ft)
S-1	5.0	5.0
S-2	12.5	12.5
S-3	12.5	12.5
S-4	10.0	10.0
S-5	12.5	12.5
S-6	2.0	2.0
S-7	5.0	7.5
S-8	5.0	7.5



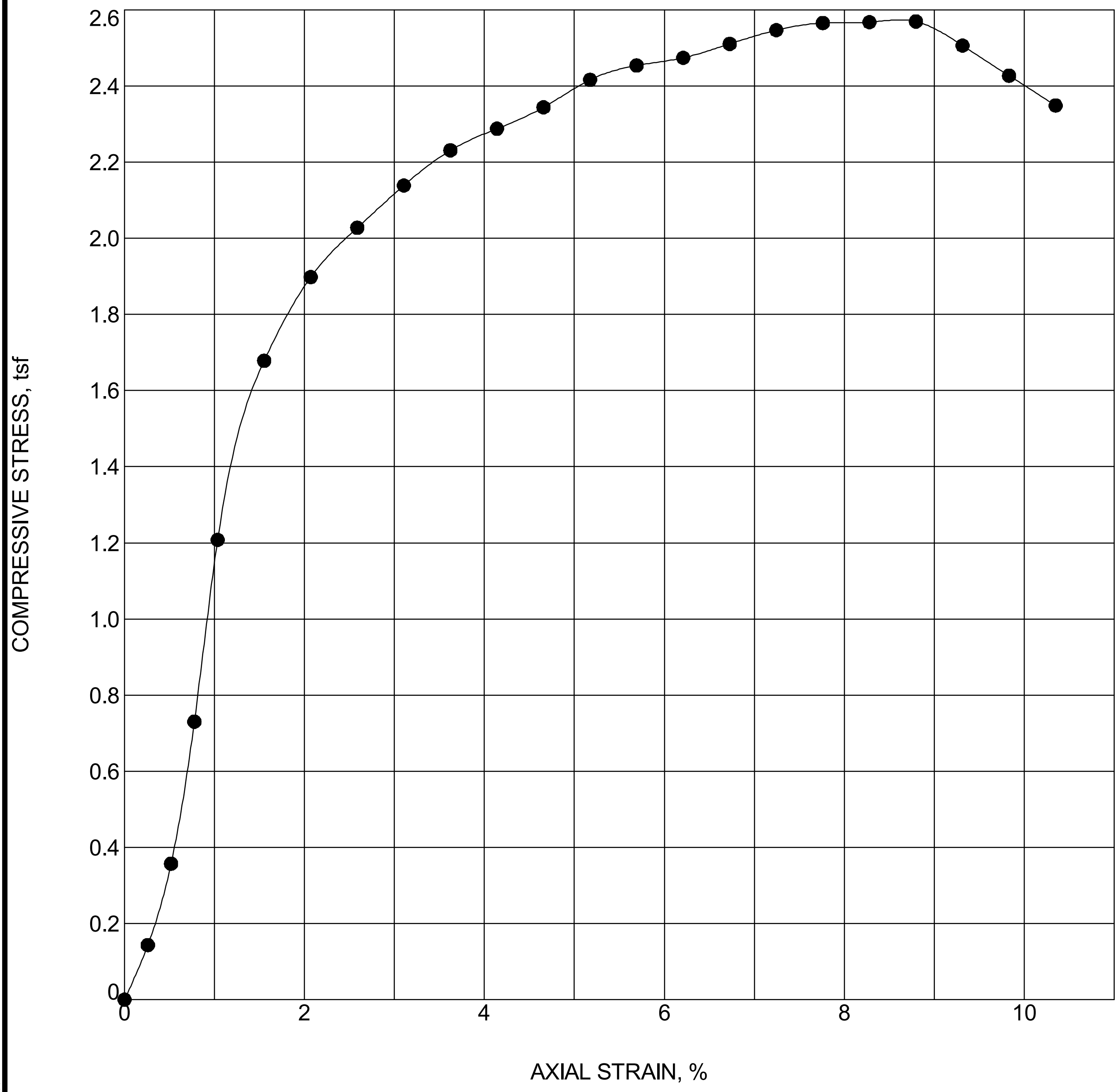
Boring	Sample	Depth	Classification
RW-1	SS-2	3.5 - 5	CLAY

Moisture Content (%)	Moist Density (pcf)	Dry Density (pcf)	Unconfined Strength (tsf)	Strain Rate (%)	Failure Strain (%)
30.4	116.1	89.0	3.14	1.0	5.6
Shear Strength (tsf)	**Saturation (%)**	**Void Ratio**	**Specimen Diameter (mm)**	**Specimen Height (mm)**	**Height/Diameter Ratio**
1.57	91	0.908	33.93	69.93	2.1

Earth Exploration, Inc.
7770 West New York Street
Indianapolis, IN 46214
Telephone: 317-273-1690
Fax: 317-273-2250

UNCONFINED COMPRESSION TEST

DES #: --- Structure #: ---
Project #: 1-17-052
County: Monroe
Location: 17th Street



Boring	Sample	Depth	Classification
RW-2	SS-2	3.5 - 5	CLAY

Moisture Content (%)	Moist Density (pcf)	Dry Density (pcf)	Unconfined Strength (tsf)	Strain Rate (%)	Failure Strain (%)
34.0	118.1	88.1	2.57	1.0	8.8
Shear Strength (tsf)	**Saturation (%)**	**Void Ratio**	**Specimen Diameter (mm)**	**Specimen Height (mm)**	**Height/Diameter Ratio**
1.28	100	0.926	32.2	68.72	2.1

Earth Exploration, Inc.
7770 West New York Street
Indianapolis, IN 46214
Telephone: 317-273-1690
Fax: 317-273-2250

UNCONFINED COMPRESSION TEST

DES #: ---   Structure #: ---
Project #: 1-17-052
County: Monroe
Location: 17th Street

EARTH EXPLORATION INC.
A Terracon COMPANY
7770 West New York Street
Indianapolis, IN 46214
(317) 273 1690
(317) 273 2250 (FAX)

Memorandum for Pavement Design

DATE: September 27, 2017

TO: Mr. Adrian Reid, P.E. – AZTEC Engineering Group, Inc. (AZTEC)

FROM: Kellen P. Heavin, P.E. - Earth Exploration, Inc. (EEI)
CC: Michael S. Wigger, P.E. - EEI

SUBJECT: Pavement Analysis
RE: 17th Street Improvements
Bloomington, Indiana
EEI Project No. 1-17-052

KELLEN PATRICK HEAVIN
PE11011658

September 26, 2017

Project Description/Scope

We understand that representatives of the City of Bloomington are planning to make improvements to 17th Street (Line "PR-B") using local funds only. The project start (Station 13+66.58) is about 310 ft east of Crescent Road, and the project end (about Station 31+00) is about 640 ft west of the roundabout at Monroe Street for a total length of approximately 1,730 ft. Based on plans provided by AZTEC, the improvements are anticipated to include reconstruction of the pavement, retaining walls, drainage improvements, and a pedestrian path. The typical section includes two travel lanes with curb and gutter, as well as a 10-ft wide multi-use path to the north of the roadway. MOT is planned to be accomplished using partial closures and restrictions to local traffic only.

Since the pavement improvements include reconstruction, a detailed review of the history and condition of the existing pavement was not necessary.

Pavement Analysis Parameters

Based on our correspondence and observations, the pavement at the project extents is currently or has recently been reconstructed as part of other improvements along 17th Street. The pavement at the east extent was reconstructed during the construction of a roundabout intersection in 2014. In addition, at the time of this memorandum, construction of a new overpass for 17th Street/Vernal Pike over I-69 as part of the I-69 Section 5 project was near completion. The typical sections for the pavement for both of these projects were provided by AZTEC along with the traffic information, which was obtained from the I-69 technical provisions. The parameters utilized in our analysis are summarized on the following page.

Summary of Pavement Analysis Parameters

Design Data	17th Street	Notes
Functional Classification	Urban Arterial	From the plans
Net length	About 1,690 ft	From the plans
AADT 2018	11,057 (2016)	11,525 calculated from growth factor
Growth Factor (%)	2.07	Linear
% of Trucks	1.0	From the plans
AADTT (2018)	116	Rounded up
Design Speed (mph)	30	From the plans
Resilient Modulus, psi (improved subgrade)	7,500	Type IB
Resilient Modulus, psi (in-situ)	4,000	A-7-6
Depth of Water Table (ft)	> 6	From the geotech report

Rationale & Recommendations

Per your request, the new pavement section considered an analysis of the adjacent sections. We started with an analysis of the thinner section (i.e., the approach for the overpass) which consists of 7-in. of asphaltic concrete (HMA) on 3 in. of compacted aggregate base. The anticipated performance of this section was modeled using AASHTOWare Pavement ME Design, Version 2.3.0 (Revision 65), which is the software required by INDOT. The "7 on 3" section satisfied the performance criteria per Chapter 304. In addition, to match the adjacent pavement, we recommend the use of QC/QA HMA. As such, our recommendations for the pavement includes:

17th Street
165 lb/yd^2 QC/QA HMA, 3, 64, Surface, 9.5 mm, on
275 lb/yd^2 QC/QA HMA, 2, 64, Intermediate, 19.0 mm, on
330 lb/yd^2 QC/QA HMA, 2, 64, Base, 19.0 mm, on
3 in. Compacted Aggregate No. 53, on
Subgrade Treatment Type IB (14 in. of chemical modification)

Pedestrian Path
Refer to standard drawing for paths (Drawing No. E 604-NVUF-01).

Pavement Life and Predicted Distress Modes

Note that the service life is directly dependent on the maintenance activities being performed.
Design Life: 20 yrs
Functional Service Life: no less than 20 yrs
Structural Service Life: no less than 20 yrs

Predicted distress mode at end of functional (service) life: Age-related surficial distresses
Predicted distress mode at end of structural life: Advanced age-related surface distresses, and possibly IRI

As you know, proper drainage characteristics, or lack thereof, has a significant impact on the performance of any pavement. As such, the performance of the recommended pavement section will be, in part, dependent on the drainage conditions in these areas. We understand that storm sewers are planned as part of this project.

Potholes and isolated failures should be patched as soon as possible. It should be noted that wooded areas and residential properties with several trees are located along the project length. It will be the responsibility of the owner to clear debris and leaves from the storm sewer inlets. We anticipate that standard INDOT PM tasks and frequencies (e.g., crack and joint sealing every 3 yrs) should be sufficient for the recommended sections during the anticipated performance period.

Attachments:

- Plan Title Sheet
- Typical Section from 17th St/Vernal Pike Overpass Project
- LTPPBind Report
- Pavement ME Output:
  - 7 on 3

PROJECT	DESIGNATION
17TH ST. RECONSTRUCTION	N/A
CONTRACT	
17TH ST. RECONSTRUCTION	

CITY OF BLOOMINGTON

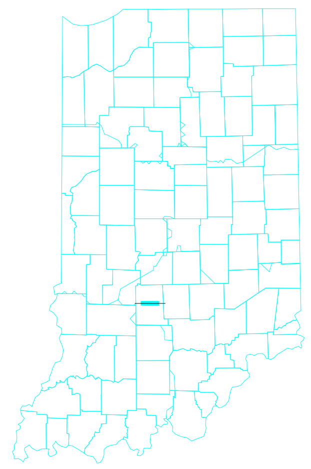
ROAD PLANS
17TH STREET RECONSTRUCTION

PROJECT NO. 17TH ST. RECONSTRUCTION P.E.
R/W
CONST.

Beginning at a point approximately 309.17 feet east and 6.47 feet south of the northwest corner of Section 32, Township 9 North, Range 1 West on the centerline of 17th Street and running along said centerline in an easterly direction 1733.42 feet to a point approximately 623.75 feet west and 6.50 feet south of the northeast corner of the northwest quarter of Section 32, Township 9 North, Range 1 West being a part of Sections 29 & 32 of T-9-N, R-1-W, Bloomington Township, Monroe County, Indiana

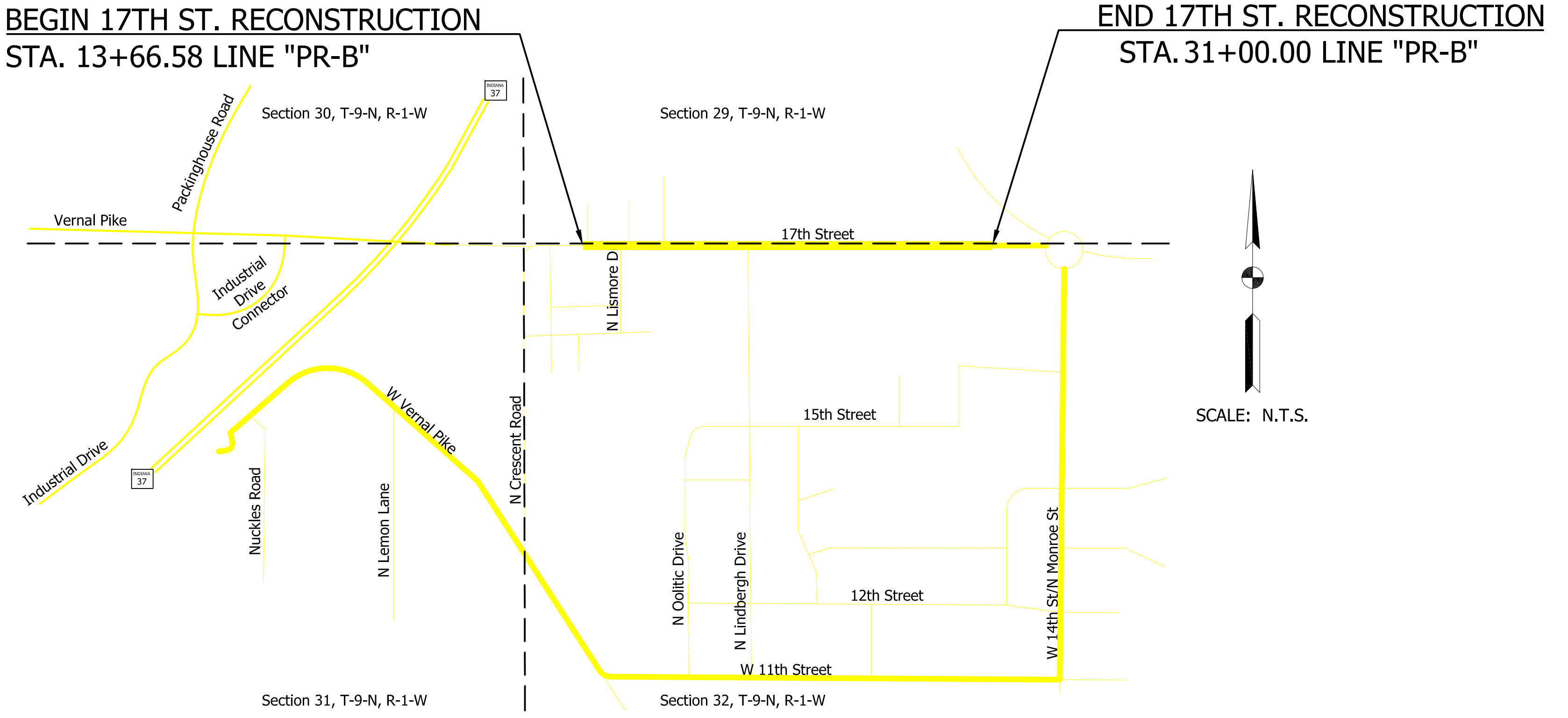
TRAFFIC DATA		17TH STREET
A.A.D.T.	(2016)	11,057 V.P.D.
A.A.D.T.	(2035)	15,410 V.P.D.
D.H.V	(2035)	1,650 V.P.H.
DIRECTIONAL DISTRIBUTION		50 %
TRUCKS		1 % A.A.D.T.
		1 % D.H.V.

DESIGN DATA	
DESIGN SPEED	30 M.P.H.
PROJECT DESIGN CRITERIA	3R (NON-FREEWAY)
FUNCTIONAL CLASSIFICATION	ARTERIAL
RURAL/URBAN	URBAN
TERRAIN	ROLLING
ACCESS CONTROL	NONE


PROJECT LOCATION SHOWN BY ▬

LATITUDE: 39 10' 45" N LONGITUDE: 86 33' 09" W

GROSS LENGTH:	0.33	MI.
NET LENGTH:	0.33	MI.
MAX. GRADE:	10	%



John Hamilton, Mayor

Adam Wason, Director Board of Public Works

Kyla Cox Deckard, President Board of Public Works

Andrew Cibor, Traffic & Transportation Engineer

AZTEC TYPSAGroup

BLOOMINGTON TOWNSHIP
MONROE COUNTY
LOCATION MAP

INDIANA DEPARTMENT OF TRANSPORTATION
STANDARD SPECIFICATIONS DATED 2016
TO BE USED WITH THESE PLANS

PLANS PREPARED BY: PHONE NUMBER

"THIS MEDIA SHOULD NOT BE CONSIDERED A CERTIFIED DOCUMENT."

CERTIFIED BY: SIG_DATE DATE

APPROVED FOR LETTING: INDIANA DEPARTMENT OF TRANSPORTATION DATE

SURVEY BOOK	DESIGNATION
	N/A
	SHEETS
.	1 of $TOTAL_PAGES$
CONTRACT	PROJECT
17TH ST. RECONSTRUCTION	17TH ST. RECONSTRUCTION

Abbreviations

R/W	Right of Way
L.A. R/W	Limited Access Right-of-Way
A.C.L	Access Control Line
C.L.T.F.	Chain Link Type Fence
F.F.T.F.	Farm Field Type Fence
APP. P.L.	Apparent Property Line
APP. EXIST. R/W	Apparent Existing Right-of-Way
B	Beginning L.A. R/W
E	Ending L.A. R/W
N.E.P.L.	No Evidence of Property Line
s.e. / e	Superelevation
P.G.	Profile Grade
N.B.	Northbound
S.B.	Southbound
O.M.L.	Operation & Maintenance Limits
A.O.R.	Axis of Rotation
P.E.S.	Pipe and Section

General Notes

1. All Earth Shoulders, Median Areas, Cut and Fill Slopes shall be Plain or Mulch Seeded except where Sodding is specified.

2. All existing Storm Drainage Pipes, Inlets, and Manholes Shall be removed unless otherwise noted.

3. All Limited Access Right-of-Way (L.A. R/W) is to be fenced with Farm Field Type Fence (F.F.T.F.) or Chain Link Type Fence (C.L.T.F.) and shall be placed in lieu of Right-of-Way Markers, unless otherwise noted.

4. All existing Right-of-Way Fence shall not be disturbed unless otherwise noted.

5. For Utility Contacts, See Utility Contacts List included in Project Details Package.

6. A karst monitor is required during construction.

Drawing Index

Sheet No.	DWG No.	Title
1	RD-TI-04	Title Sheet - Vernal Pike & Roads
2	RD-IX-04	Index of Sheets
3	RD-UP-02	Utility Protection Plan
4	KAR-CD-01	Permanent Karst Treatment Table
5-10	RD-TS-07/12	Typical Cross Sections - Vernal Pike & Roads
11-14	RD-GL-07/12	Geometric Layout - Vernal Pike & Roads
15-26	TR-MT-01/12	Maintenance of Traffic - Vernal Pike & Roads
27-38	RD-PN-08/13 RD-PR-11/16	Plan and Profile Sheets - Vernal Pike & Roads
39-44	RD-SE-11/16	Superelevation Sheet - Vernal Pike & Roads
45-52	RD-STK-13,14	Staking Plan - Vernal Pike & Crescent Rd.
53-56	RD-DT-03/06	Detection Basin. Details
57-66	BR-WD-01/10	Retaining Walls
67-69	BR-GE-01/03	Vernal Pike over Packinghouse Road Structure
70-79	TR-SP-12/18	Proposed Signing & Marking - Vernal Pike & Roads
80	RD-AT-01	Approach Table
81-86	RD-SD- 06,07	Structure Data Table - Vernal Pike
87-89	RD-PM-03/ 06	Pipe Material Table - Vernal Pike
	GEO11	Previous geometric design (For information only)
	PP23/27	Right of Way Plans (For information only)

Traffic & Design Data

TRAFFIC DATA		Line "S-8-A" Vernal Pike	Line "S-8-Pack" Industrial Park Rd.	Line "S-8-Pack" North Packing house Rd.	Line "S-8-Ind" Industrial connector	Line "S-8-Cre" Crescent Rd.
A.A.D.T.	(2010)	7.477	4.440	<400	4.840	N/A
A.A.D.T.	(2035)	15.410	6.840	<400	7.240	N/A
D.H.V.	(2035)	1.650	605	<400	655	N/A
Directional Distribution		53	51	50	51	N/A
Percent Trucks (2035)		1 (A.A.D.T.)	1	N/A	1 (A.A.D.T.)	N/A
		1 (D.H.V.)	1	N/A	1 (D.H.V.)	N/A
DESIGN DATA						
Design Speed		40 M.P.H.	35 M.P.H.	35 M.P.H.	35 M.P.H.	30 M.P.H.
Project Design Criteria		IDM Fig. 55-3F	IDM Fig. 55-3G	IDM Fig. 55-3G	IDM Fig. 55-3G	IDM Fig. 55-3H
Functional Classification		Minor Arterial	Collector	Collector	Collector	Local
Rural/Urban		Urban (Suburban)	Urban (Suburban)	Urban (Suburban)	Urban (Suburban)	Urban (Suburban)
Terrain		Rolling	Rolling	Rolling	Rolling	Rolling
Access Control		N/A	N/A	N/A	N/A	N/A

Revisions

Date	Sheet No.	Description

7/15/2015

Plotted By: user

DATE	REVISION

ISOLUX CORSAN USA
RFC 023 – 07/20/15

JEFFREY LYLE CALLICOTT
REGISTERED NO. 11400561
STATE OF INDIANA
PROFESSIONAL ENGINEER

RECOMMENDED FOR APPROVAL Jeff Callicott 07/15/2015
DESIGN ENGINEER DATE

DESIGNED: APF 07/15
DRAWN: PMG 07/15
CHECKED: JGJ 07/15
CHECKED: APF 07/15

INDIANA
DEPARTMENT OF TRANSPORTATION

INDEX SHEET
VERNAL PIKE & ROADS

SCALE	BRIDGE FILE	
N/A	N/A	
	DESIGNATION	
	1297885	
SURVEY BOOK	DWG NO.	SHEET
	RD-IX-04	2 of 89
CONTRACT	PROJECT	
	1297885	

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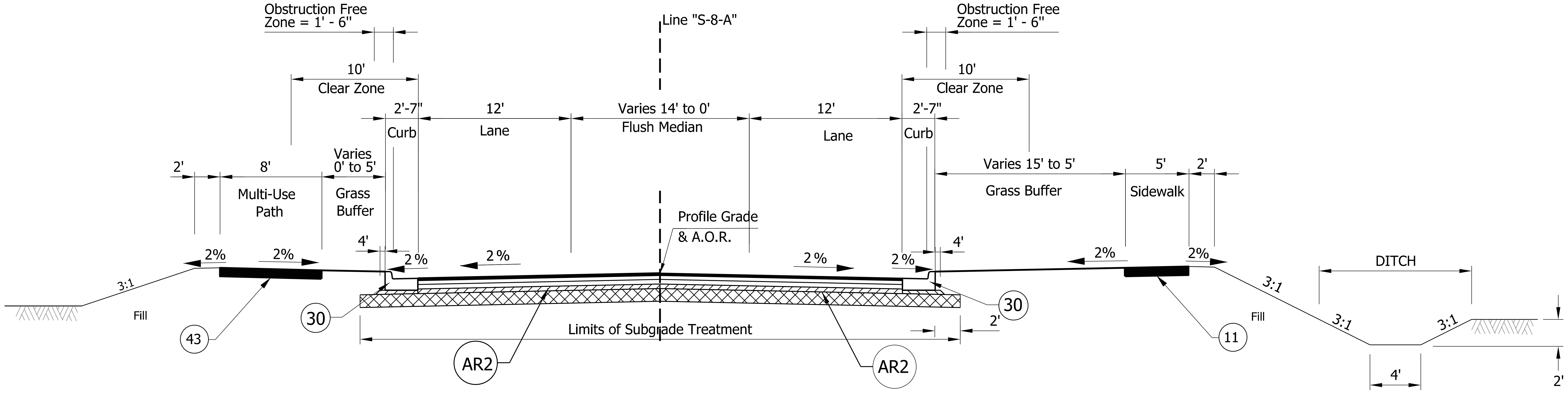
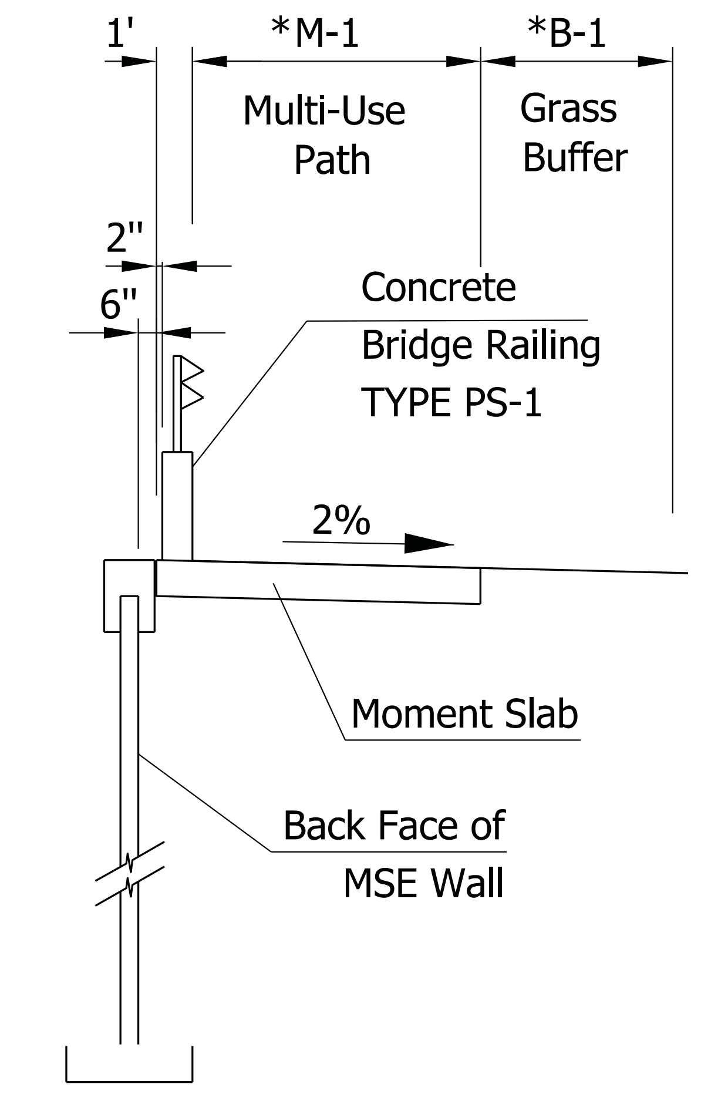


VERNAL PIKE TYPICAL SECTION

STA. 61+36.24 TO STA. 64+45.00



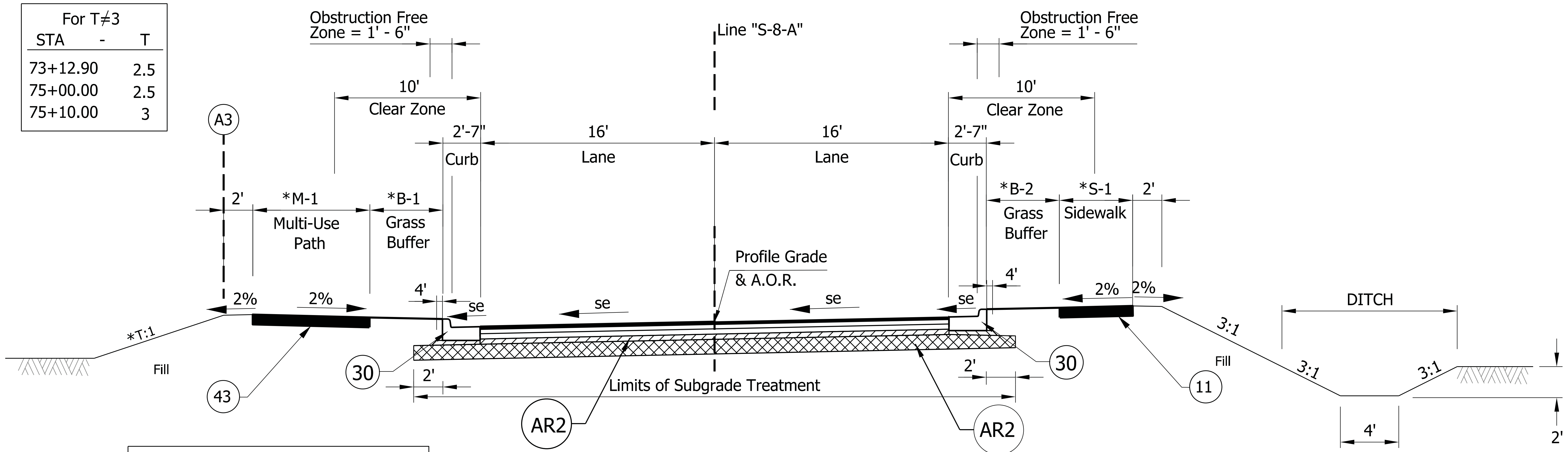
LEFT WALL RAILING

WALL	STA	-	STA
RW465-1	65+81.78	-	68+09.25
RW465-3	69+15.74	-	71+26.38
RW465-5	72+00.56	-	73+12.90

(See Dwg No. BR-WD-01/10
for more Detail)

For T≠3

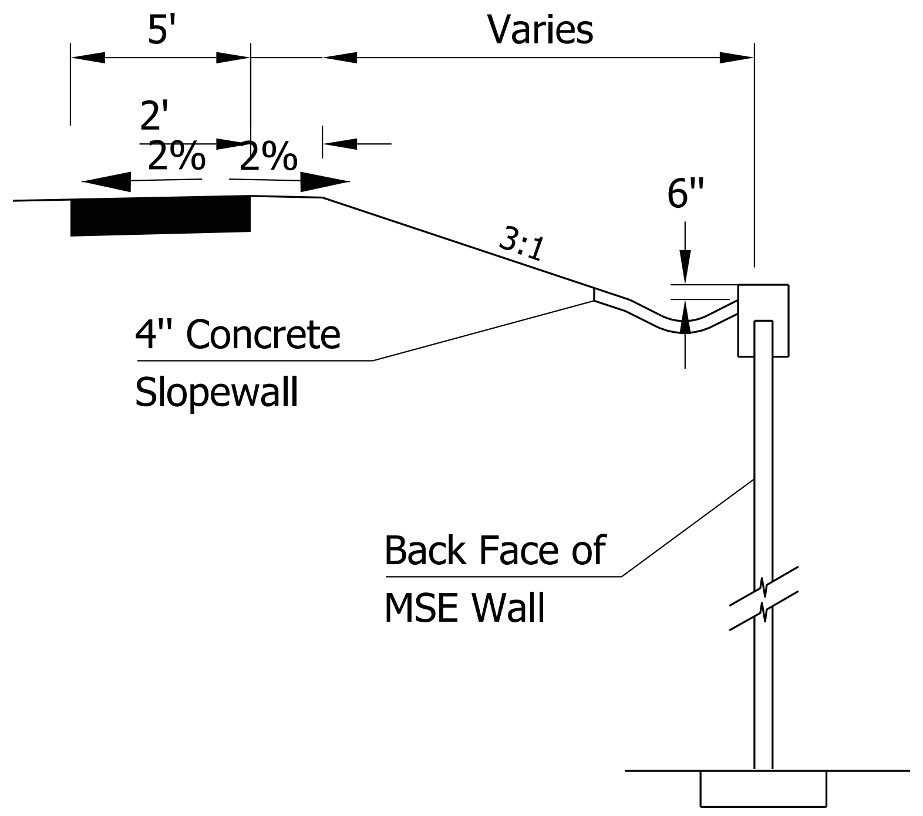
STA	-	T
73+12.90		2.5
75+00.00		2.5
75+10.00		3



STA	*M-1	*B-1	*B-2	*S-1
64+45.00	8'	5'	5'	5'
70+86.50	8'	5'	5'	5'
71+36.50	9'	0'	0'	6'
71+90.50	9'	0'	0'	6'
72+40.50	8'	5'	5'	5'
73+34.15	8'	5'	5'	5'

VERNAL PIKE TYPICAL SECTION

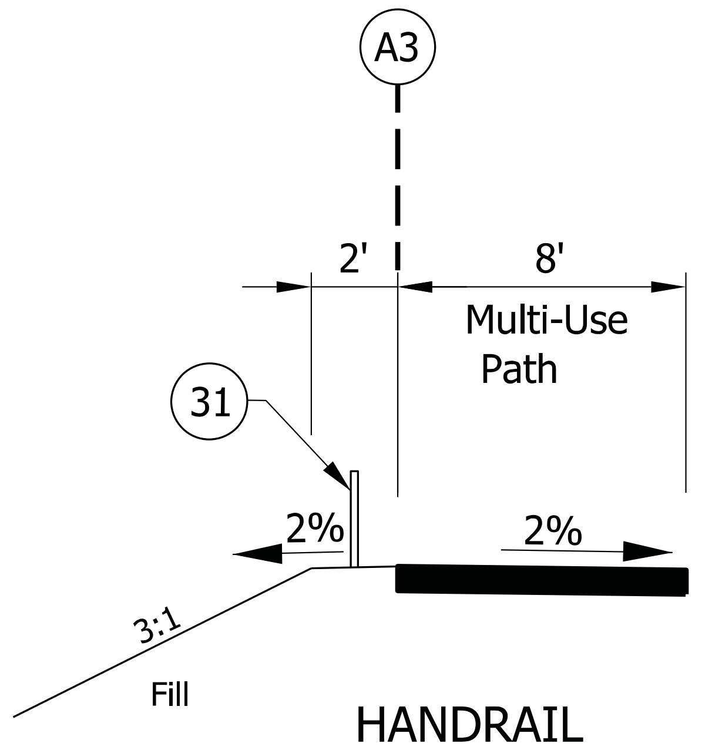
STA. 64+45.00 TO STA. 67+34.08 (se max = 4% Turning Right)
STA. 67+34.08 TO STA. 73+34.15 (se max = 4% Turning Left)



WALL	STA	-	STA
RW465-4	68+45.16	-	71+31.91
RW465-6	71+95.45	-	72+25.53

(See Dwg No. BR-WD-01/10
for more Detail)

RIGHT WALL RAILING



HANDRAIL

STA. 73+12.90 TO STA. 75+10

Legend

AR2	165 lb/sys QC/QA HMA-2, 64, Surface 9.5 mm 275 lb/sys QC/QA HMA-2, 64, Intermediate 19.0 mm 330 lb/sys QC/QA HMA-2, 64, Base 19.0 mm 3" No. 53 Aggregate Base Type 1B or 1C Subgrade Treatment
11	4" Concrete Sidewalk
43	Multi-Use Path, HMA Asphalt 165 Lbs/Sys. HMA Surface, Type A, 9.5mm on 275 Lbs/Sys. HMA Intermediate, Type A, 19mm
30	Combined Concrete Curb and Gutter (Vertical) per Std. Dwg. E605-CCCG-01
31	Handrail, Pedestrian

DATE	REVISION

ISOLUX CORSAN USA
RFC 023 – 07/20/15

JEFFREY LYLE CALLICOTT
REGISTERED NO. 11400561
STATE OF INDIANA
PROFESSIONAL ENGINEER

RECOMMENDED FOR APPROVAL Jeff Callicott 07/15/2015
DESIGN ENGINEER DATE

DESIGNED: APF 07/15 DRAWN: PSL 07/15
CHECKED: JGJ 07/15 CHECKED: APF 07/15

INDIANA
DEPARTMENT OF TRANSPORTATION

TYPICAL SECTIONS
VERNAL PIKE

SCALE: AS SHOWN
BRIDGE FILE
DESIGNATION: 1401617
SURVEY BOOK
CONTRACT
DWG NO.: RD-TS-07
SHEET: 5 of 89
PROJECT: 1297885

7/15/2015
Plotted By: user

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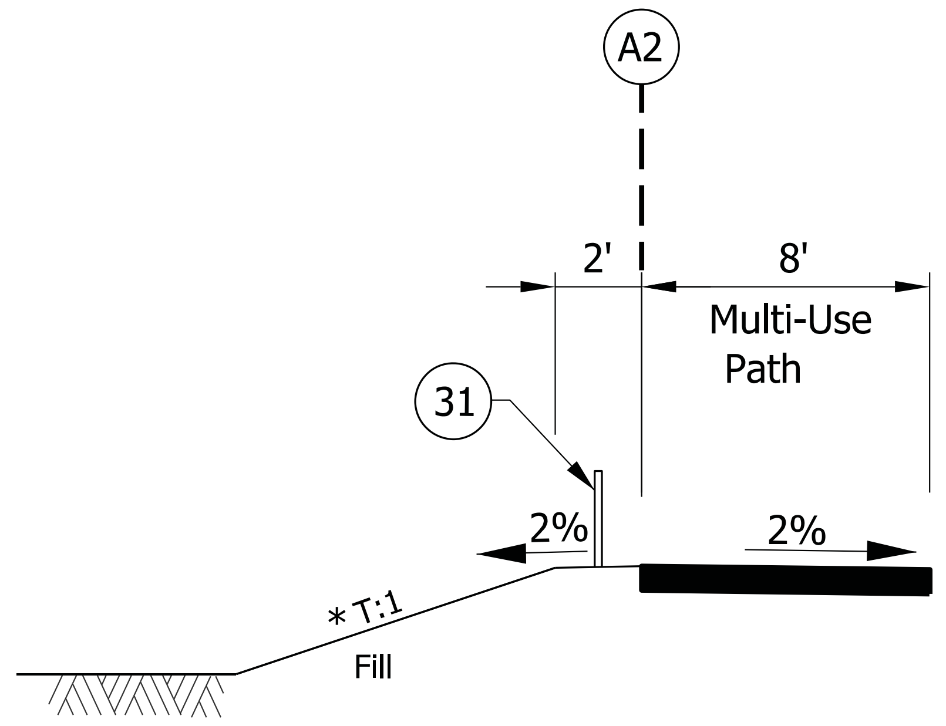
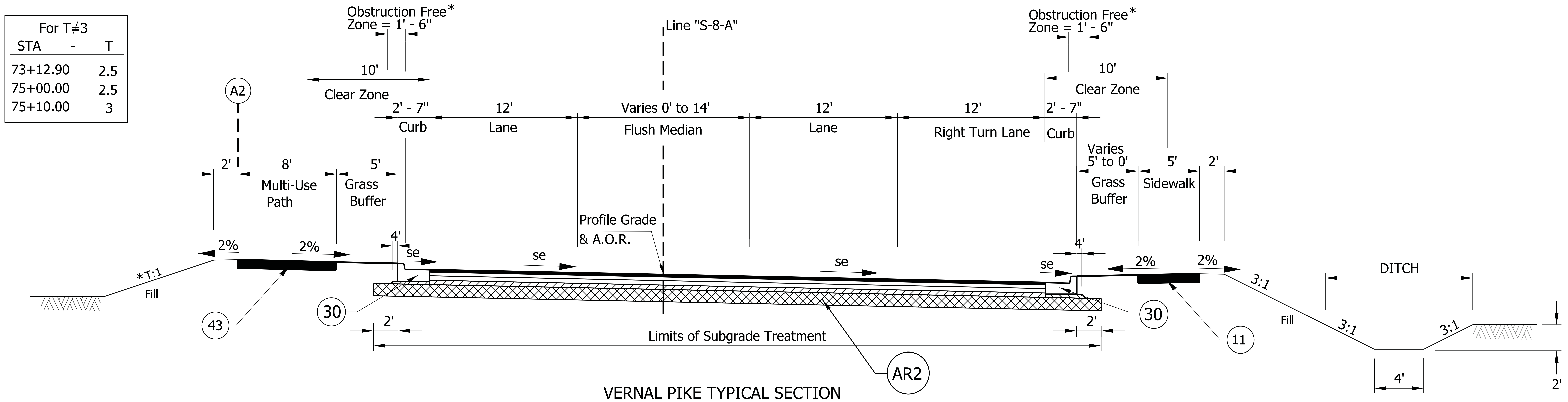


HANDRAIL

STA. 73+12.90 TO STA. 75.10.00
STA. 85+85.10 TO STA. 88+50.00

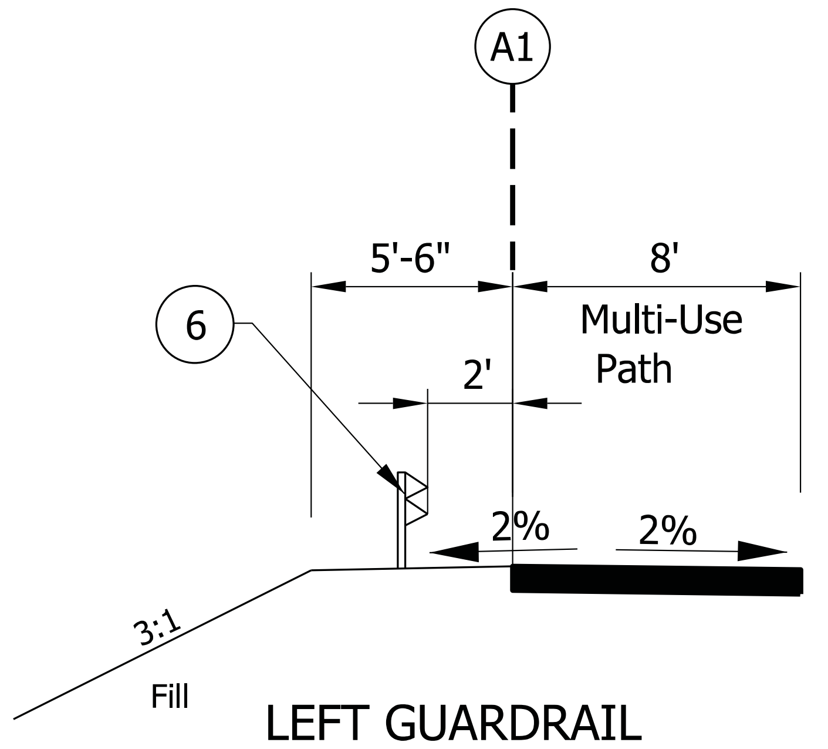
For T≠3	
STA -	T
73+12.90	2.5
75+00.00	2.5
75+10.00	3



VERNAL PIKE TYPICAL SECTION

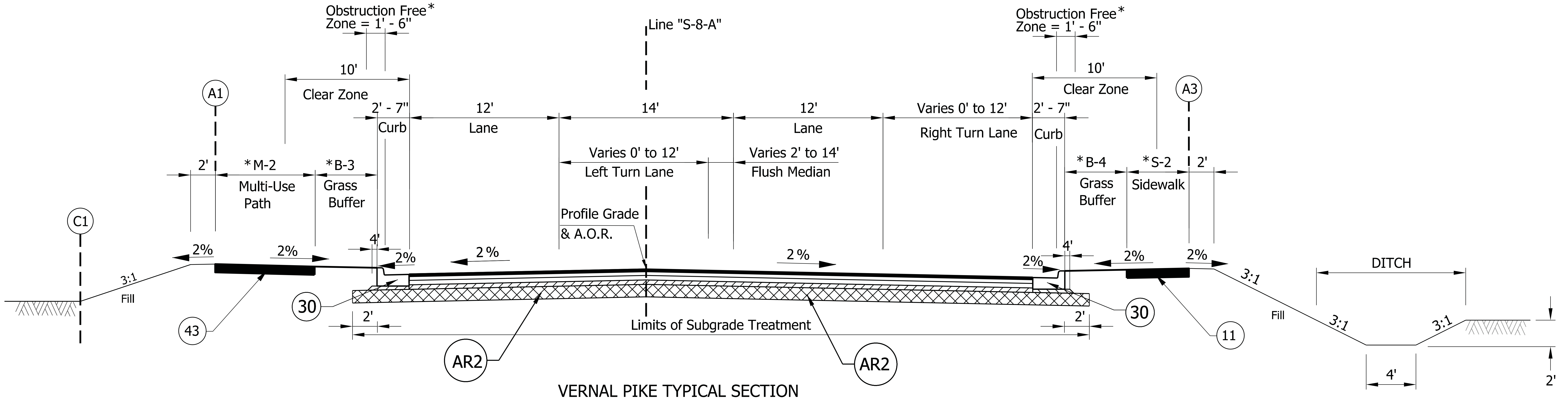
STA. 73+34.15 TO STA. 75+80.00 (se max = 4% Turning Right)

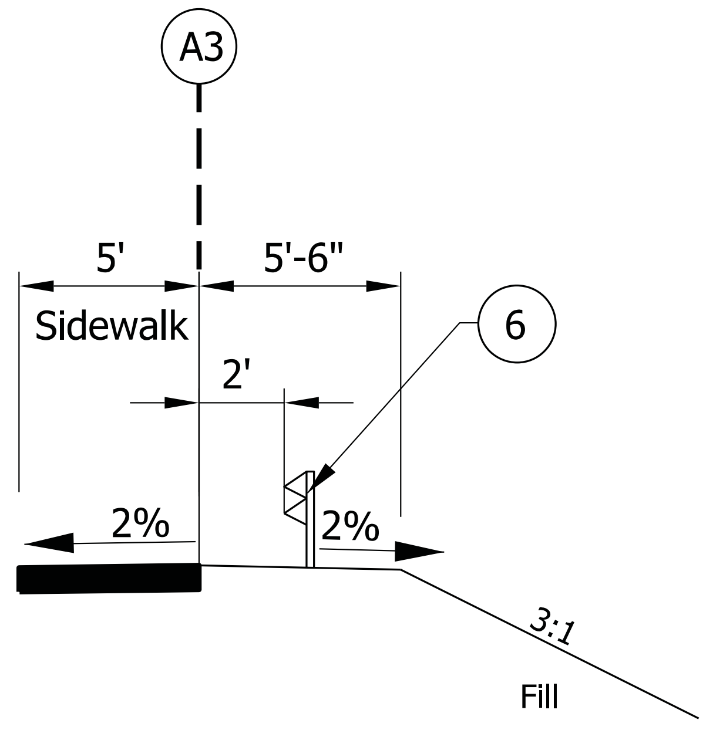
* OFZ is 2' - 6" in the vicinity of Vernal Pike / Industrial Connector



LEFT GUARDRAIL

STA. 83+21.00 TO STA. 83+71.00
STA. 79+82.00 TO STA. 80+32.00





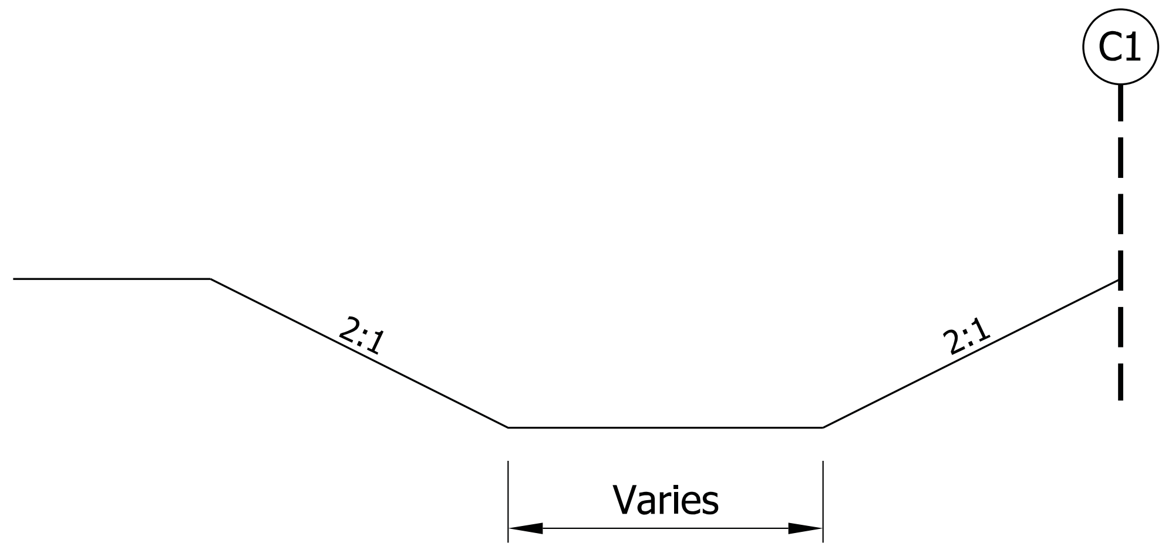
VERNAL PIKE TYPICAL SECTION

STA. 75+80.00 TO STA. 87+93.48
(Pending of ground Treatment solution)

RIGHT GUARDRAIL

STA. 79+30.50 TO STA. 79+80.50
STA. 82+69.50 TO STA. 83+19.50

* OFZ is 2' - 6" in the vicinity of Vernal Pike / Industrial Connector



DETENTION BASIN

STA. 72+60 TO STA. 74+60 - D.B. "D"
STA. 85+88.75 TO STA. 87+11.86 - D.B. 402-A
(See Detail Dwg No. RD-DT-03/06)

Legend

AR2	165 lb/sys QC/QA HMA-2, 64, Surface 9.5 mm 275 lb/sys QC/QA HMA-2, 64, Intermediate 19.0 mm 330 lb/sys QC/QA HMA-2, 64, Base 19.0 mm 3" No. 53 Aggregate Base Type 1B or 1C Subgrade Treatment
11	4" Concrete Sidewalk
43	Multi-Use Path, HMA Asphalt 165 Lbs/Sys. HMA Surface, Type A, 9.5mm on 275 Lbs/Sys. HMA Intermediate, Type A, 19mm
30	Combined Concrete Curb and Gutter (Vertical) per Std. Dwg. E605-CCCG-01
6	Guardrail
31	Handrail, Pedestrian, (See Plan for Locations)

STA	*M-2	*B-3	*B-4	*S-2
75+80.00	8'	5'	5'	5'
79+26.00	8'	5'	5'	5'
79+76.00	9'	0'	0'	6'
83+26.00	9'	0'	0'	6'
83+76.00	8'	5'	5'	5'
85+50.00	8'	5'	5'	5'
86+00.00	8'	5'	0'	6'
87+12.60	8'	5'	0'	6'

7/15/2015

Plotted By: user

DATE	REVISION

ISOLUX CORSAN USA
RFC 023 – 07/20/15

JEFFREY LYLE CALLICOTT
NO. 11400561

RECOMMENDED FOR APPROVAL Jeff Callicott 07/15/2015
DESIGN ENGINEER DATE

DESIGNED: APF 07/15 DRAWN: PSL 07/15
CHECKED: JGJ 07/15 CHECKED: APF 07/15

INDIANA
DEPARTMENT OF TRANSPORTATION

TYPICAL SECTIONS
VERNAL PIKE

SCALE: AS SHOWN
BRIDGE FILE:
DESIGNATION: 1401617
SURVEY BOOK:
DWG NO.: RD-TS-08
SHEET: 6 of 89
CONTRACT:
PROJECT: 1297885

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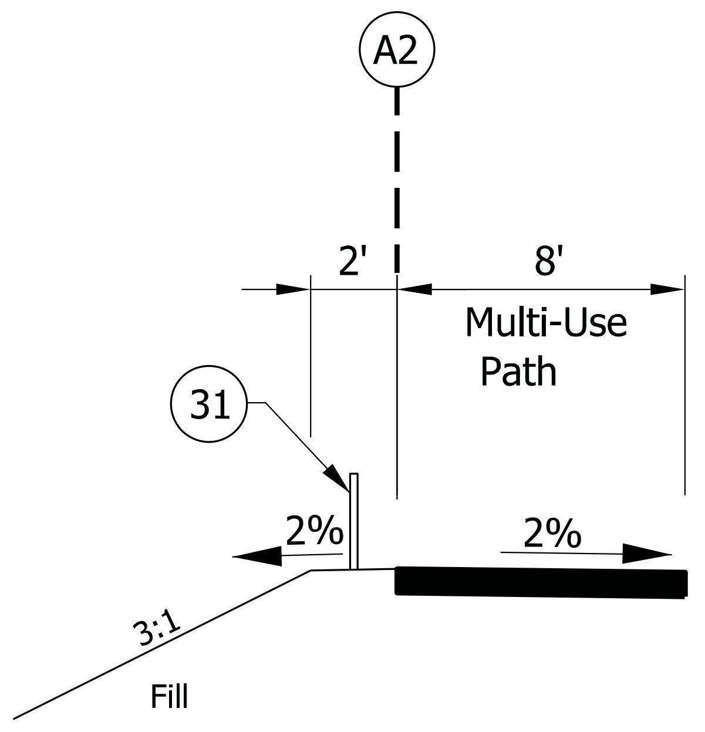
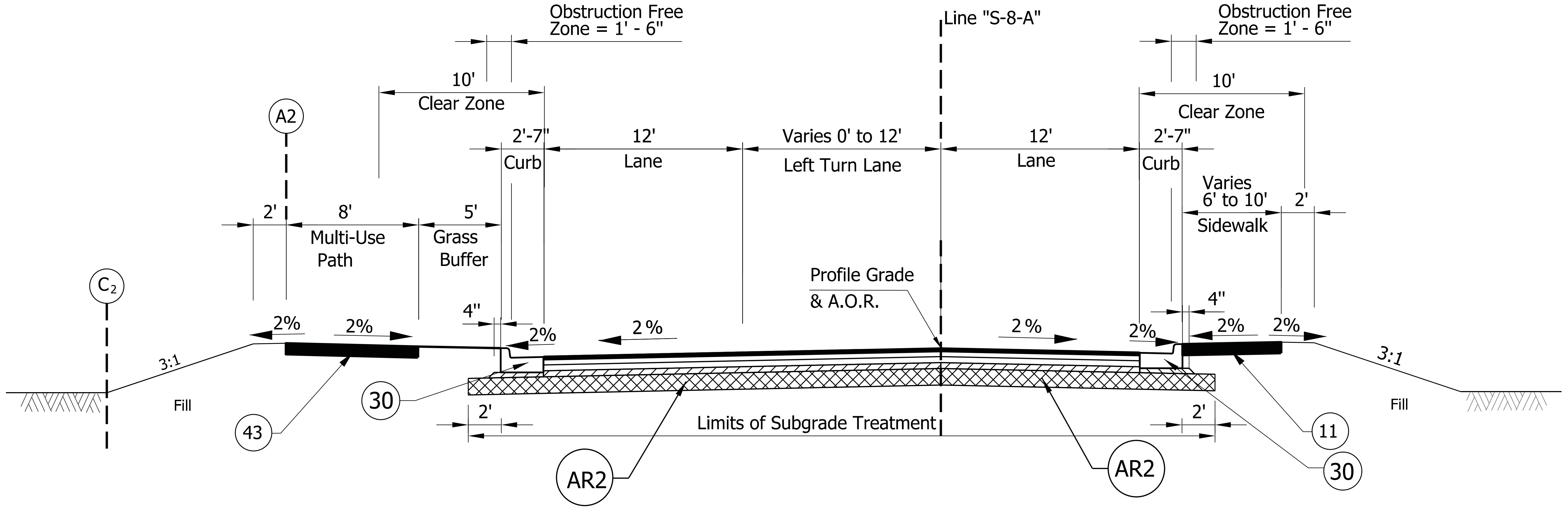
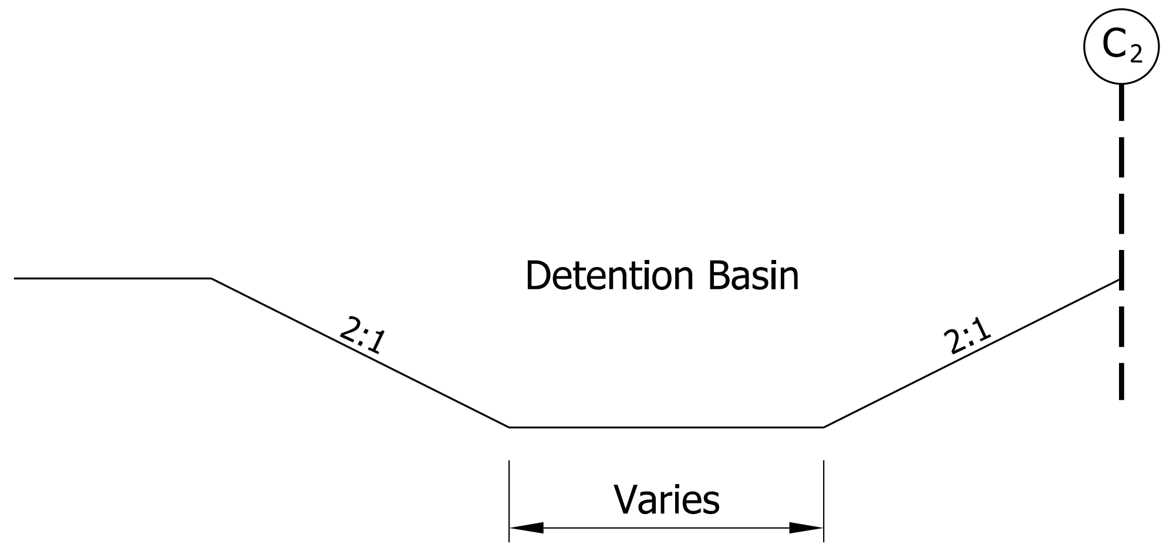


STA. 85+85.00 TO STA. 88+50.00



VERNAL PIKE TYPICAL SECTION

STA. 87+93.48 TO STA. 91+25.00



STA. 87+27.01 TO STA. 88+41.42 - D.B. 402-B
(See Detail Dwg No. RD-DT-03/06)

Legend

AR2	165 lb/sys QC/QA HMA-2, 64, Surface 9.5 mm 275 lb/sys QC/QA HMA-2, 64, Intermediate 19.0 mm 330 lb/sys QC/QA HMA-2, 64, Base 19.0 mm 3" No. 53 Aggregate Base Type 1B or 1C Subgrade Treatment
11	4" Concrete Sidewalk
43	Multi-Use Path, HMA Asphalt 165 Lbs/Sys. HMA Surface, Type A, 9.5mm on 275 Lbs/Sys. HMA Intermediate, Type A, 19mm
30	Combined Concrete Curb and Gutter (Vertical) per Std. Dwg. E605-CCCG-01
6	Guardrail
31	Handrail, Pedestrian (See Plan for Locations)

DATE	REVISION

ISOLUX CORSAN
USA
RFC 023 – 07/20/15

JEFFREY LYLE CALLICOTT
REGISTERED
NO. 11400561
STATE OF INDIANA
PROFESSIONAL ENGINEER

RECOMMENDED FOR APPROVAL Jeff Callicott 07/15/2015
DESIGN ENGINEER DATE

DESIGNED: APF 07/15 DRAWN: PSL 07/15
CHECKED: JGJ 07/15 CHECKED: APF 07/15

INDIANA
DEPARTMENT OF TRANSPORTATION

TYPICAL SECTIONS
VERNAL PIKE

SCALE	BRIDGE FILE
AS SHOWN	
	DESIGNATION
	1401617
SURVEY BOOK	DWG NO. RD-TS-09 / SHEET 7 of 89
CONTRACT	PROJECT
	1297885

7/15/2015

Plotted By: user

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Legend

AR2	165 lb/sy QC/QA HMA-2, 64, Surface 9.5 mm 275 lb/sy QC/QA HMA-2, 64, Intermediate 19.0 mm 330 lb/sy QC/QA HMA-2, 64, Base 19.0 mm 3'' No. 53 Aggregate Base Type 1B or 1C Subgrade Treatment
▨	Pavement Removal
XX	Utility Test Hole Number
6	Guardrail, W-Beam, 6'-3" Spacing
11	4'' Concrete Sidewalk
12	Ramp Type G
13	Ramp Type H
14	Detectable Warning Surface
18	Guardrail Transition, Type TGB
19	Guardrail End Transition, Type OS
43	Multi-Use Path, HMA Asphalt 165 lb/sy HMA, Surface Type A 9.5 mm 275 lb/sy HMA, Intermediate Type A 19.0 mm
30	Combined Concrete Curb and Gutter (Vertical)
31	Handrail, Pedestrian

DATE	REVISION

ISOLUX CORSAN
USA
RFC 023 – 07/20/15

JEFFREY LYLE CALLICOTT
NO. 11400561

RECOMMENDED FOR APPROVAL Jeff Callicott 07/16/2015
DESIGN ENGINEER DATE

DESIGNED: APF 07/15 DRAWN: PMG 07/15
CHECKED: JGJ 07/15 CHECKED: APF 07/15

INDIANA
DEPARTMENT OF TRANSPORTATION

ROADWAY PLAN
STA 77+00 TO STA 91+25 - VERNAL PIKE

SCALE	BRIDGE FILE
1"=50'	
	DESIGNATION
	1297885

SURVEY BOOK	DWG NO.	SHEET
	RD-PN-09	29 of 89
CONTRACT	PROJECT	
	1297885	

```
PG Binder Selection Report Title

LTPPBind V3.0 PG Binder Selection Report (Date: 9/25/2017 )


Parameter                A=4 km    B=26 km   C=32 km   D=36 km   E=43 km
Station ID               IN6580    IN1869    IN0784    IN8036    IN6705
Elevation, m             603       678       771       510       520
Degree-Days >10 C        2946      3185      2936      3091      3120
Low Air Temperature, C   -23.3     -21.6     -21.6     -22.5     -23.9
Low Air Temp. Std Dev    4.4       4.1       3.7       4.3       5


Input Data

Latitude, Degree                   38.91
Yearly Degree-Days>10C             3056
Lowest Yearly Air Temp., Deg. C    -22.6
Low Temp. Std. Dev., Deg. C        4.3
Base HT PG                         58


Traffic Adjustments for HT

Desired Reliability, Percent       98
Traffic Loading, Million ESAL       Up to 3 M. ESAL
Traffic Speed                      Fast
High Temp. Adjustment               0.0


PG Temperature                     HIGH      LOW
PG Temp. at 50%  Reliability        57.0      -15.1
PG Temp. at Desired Reliability     59.2      -22.8
Adjustments for Traffic             0
Adjustments for Depth               0.0       0.0
Adjusted PG Temperature             59.2      -22.8
Selected PG Binder Grade            64        -28
```

7 on 3

File Name: C:\My ME Design\Projects\17th St\7 on 3.dgpx

Design Inputs

Design Life:	50 years	Base construction:	May, 2018	Climate Data Sources (Lat/Lon)	38.043, -87.537
Design Type:	FLEXIBLE	Pavement construction:	June, 2018		
		Traffic opening:	September, 2018		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	9.5mm Surface PG64	1.5
Flexible	19mm Intermediate PG64	2.5
Flexible	19mm Base PG64	3.0
NonStabilized	CA No 53	3.0
Subgrade	Improved subgrade type IB	14.0
Subgrade	Clay (A-7-6)	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	11.6
Air voids (%)	8.0

Traffic

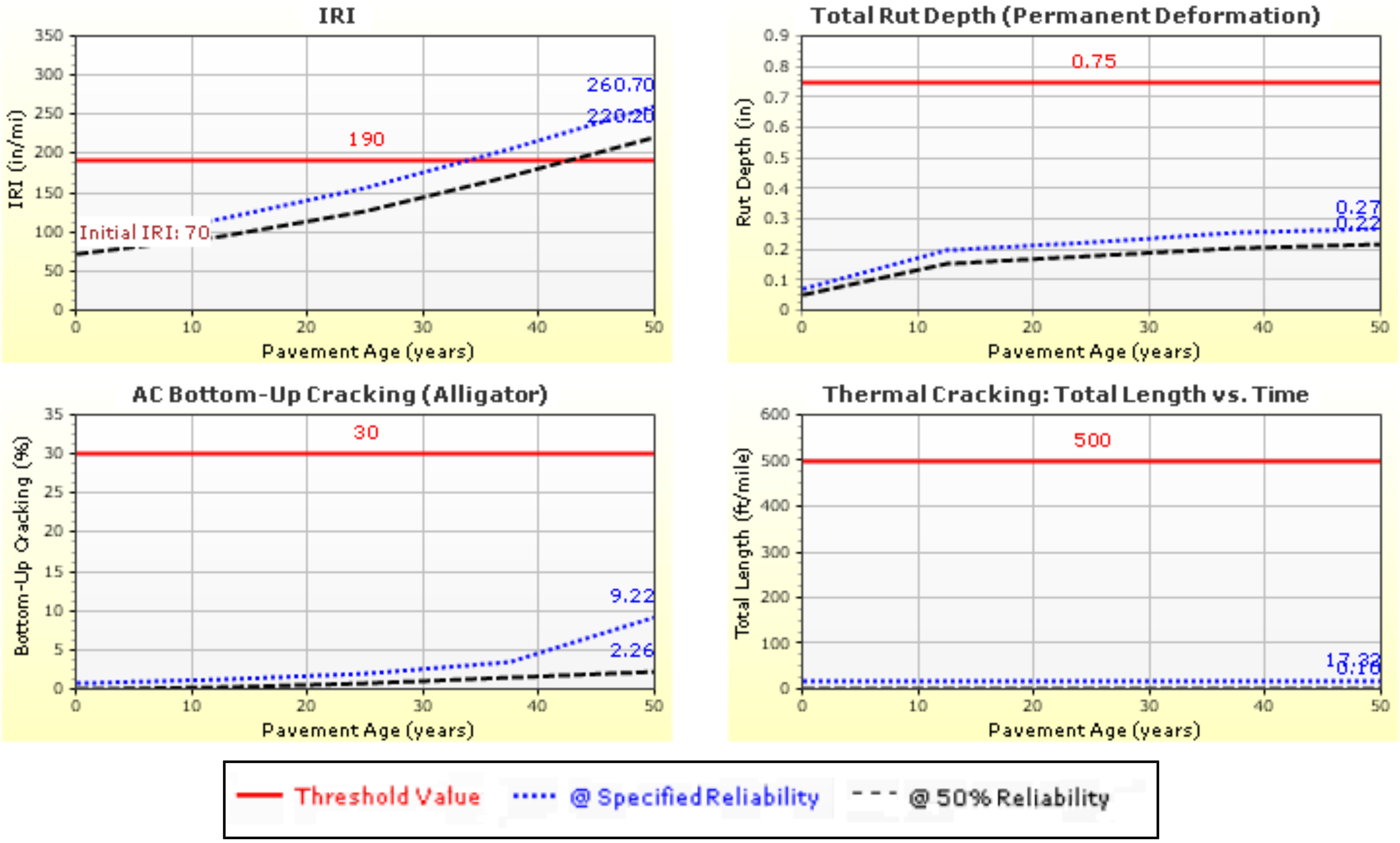
Age (year)	Heavy Trucks (cumulative)
2018 (initial)	116
2043 (25 years)	661,168
2068 (50 years)	1,596,410

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	190.00	260.67	80.00	26.48	Fail
Permanent deformation - total pavement (in)	0.75	0.27	80.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	30.00	9.22	80.00	99.96	Pass
AC thermal cracking (ft/mile)	500.00	17.32	80.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	1369.30	80.00	89.54	Pass
Permanent deformation - AC only (in)	0.40	0.23	80.00	99.98	Pass

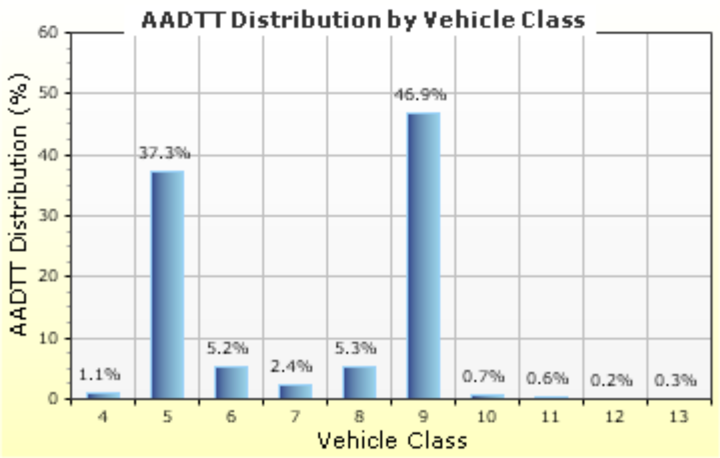
Distress Charts



Traffic Inputs

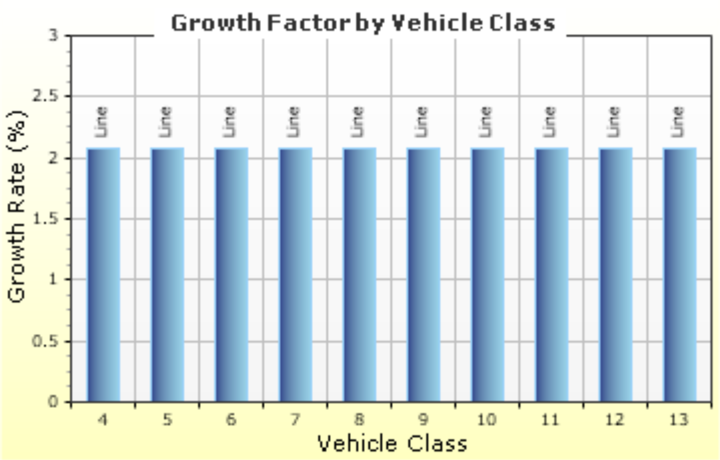
Graphical Representation of Traffic Inputs

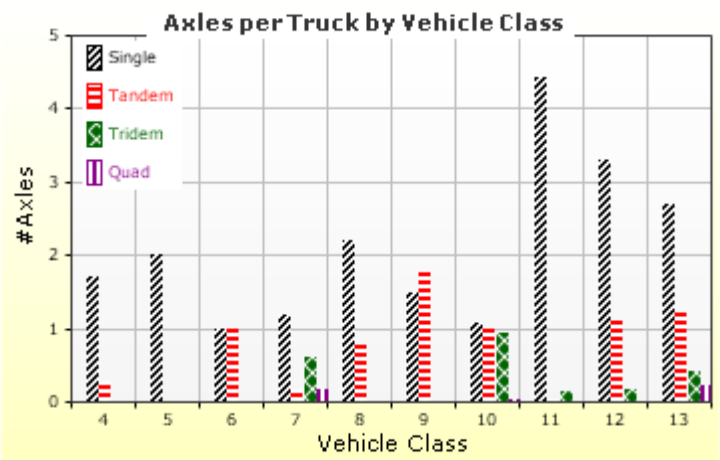
Initial two-way AADTT:	116	Percent of trucks in design direction (%):	50.0
Number of lanes in design direction:	1	Percent of trucks in design lane (%):	100.0
		Operational speed (mph)	30.0

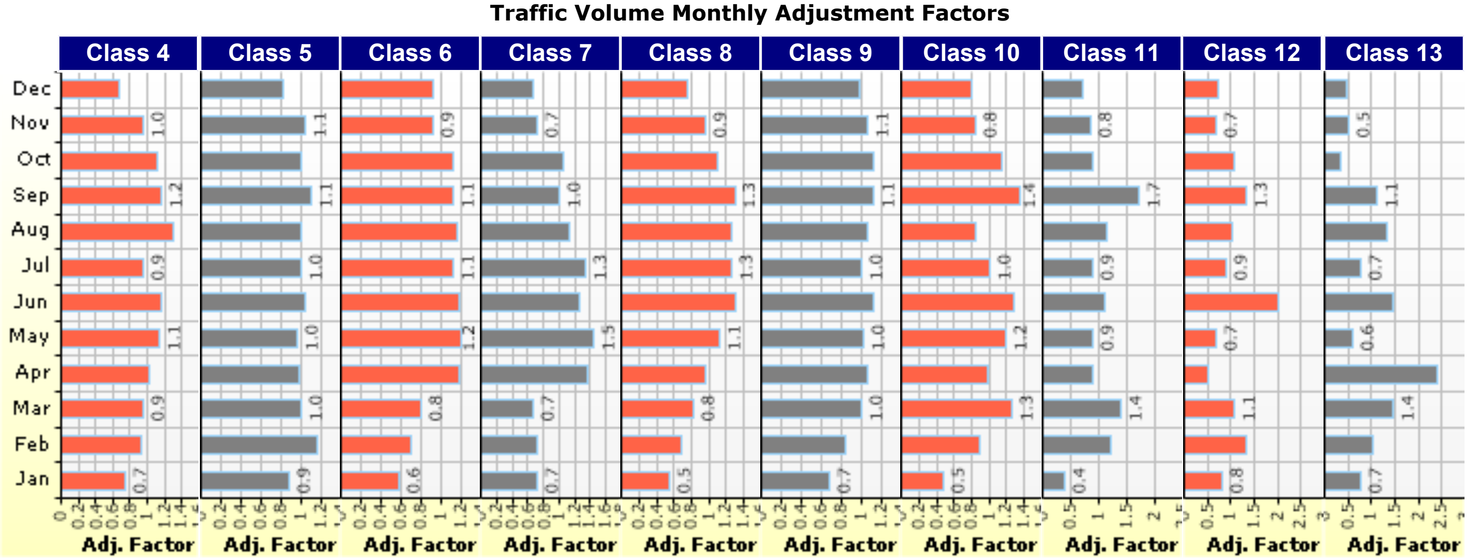


Truck Distribution by Hour

This chart does not apply to the design type







Tabular Representation of Traffic Inputs

Volume Monthly Adjustment Factors

Level 3: Default MAF

Month	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
January	0.7	0.9	0.6	0.7	0.5	0.7	0.5	0.4	0.8	0.7
February	0.9	1.2	0.7	0.7	0.7	0.8	0.9	1.2	1.3	1.0
March	0.9	1.0	0.8	0.7	0.8	1.0	1.3	1.4	1.1	1.4
April	1.0	1.0	1.2	1.4	1.0	1.1	1.0	0.9	0.5	2.4
May	1.1	1.0	1.2	1.5	1.1	1.0	1.2	0.9	0.7	0.6
June	1.2	1.0	1.2	1.3	1.3	1.1	1.3	1.1	2.0	1.4
July	0.9	1.0	1.1	1.3	1.3	1.0	1.0	0.9	0.9	0.7
August	1.3	1.0	1.2	1.1	1.3	1.1	0.8	1.1	1.0	1.3
September	1.2	1.1	1.1	1.0	1.3	1.1	1.4	1.7	1.3	1.1
October	1.1	1.0	1.1	1.1	1.1	1.1	1.2	0.9	1.1	0.3
November	1.0	1.1	0.9	0.7	0.9	1.1	0.8	0.8	0.7	0.5
December	0.7	0.8	0.9	0.7	0.7	1.0	0.8	0.7	0.7	0.5

Distributions by Vehicle Class

Vehicle Class	AADTT Distribution (%) (Level 3)	Growth Factor	
		Rate (%)	Function
Class 4	1.1%	2.07%	Linear
Class 5	37.3%	2.07%	Linear
Class 6	5.2%	2.07%	Linear
Class 7	2.4%	2.07%	Linear
Class 8	5.3%	2.07%	Linear
Class 9	46.9%	2.07%	Linear
Class 10	0.7%	2.07%	Linear
Class 11	0.6%	2.07%	Linear
Class 12	0.2%	2.07%	Linear
Class 13	0.3%	2.07%	Linear

Truck Distribution by Hour does not apply

Axle Configuration

Traffic Wander	
Mean wheel location (in)	18.0
Traffic wander standard deviation (in)	10.0
Design lane width (ft)	12.0

Axle Configuration	
Average axle width (ft)	8.5
Dual tire spacing (in)	12.0
Tire pressure (psi)	120.0

Average Axle Spacing	
Tandem axle spacing (in)	51.6
Tridem axle spacing (in)	49.2
Quad axle spacing (in)	49.2

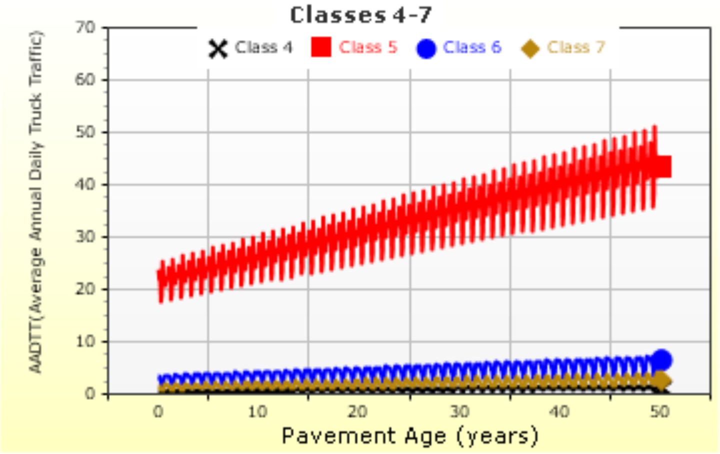
Wheelbase does not apply

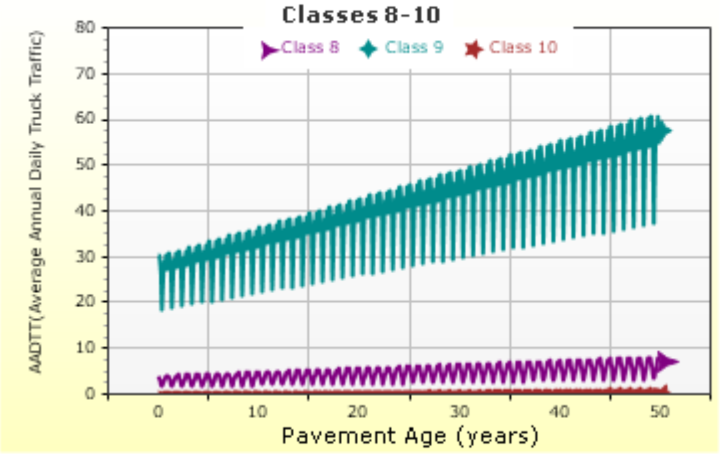
Number of Axles per Truck

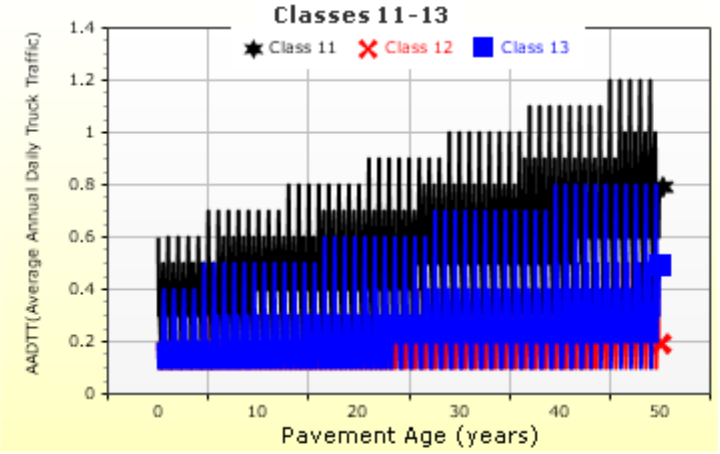
Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.7	0.29	0	0
Class 5	2	0	0	0
Class 6	1	1	0	0
Class 7	1.18	0.18	0.63	0.18
Class 8	2.21	0.78	0	0
Class 9	1.48	1.75	0	0
Class 10	1.08	0.99	0.94	0.03
Class 11	4.43	0.03	0.16	0
Class 12	3.29	1.09	0.17	0
Class 13	2.7	1.22	0.43	0.24

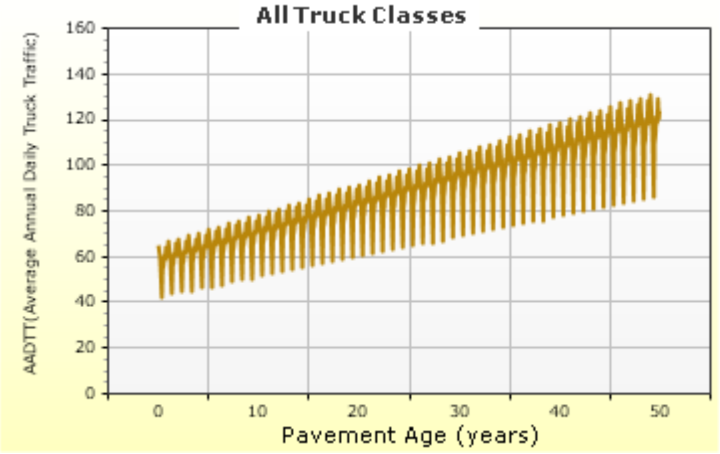
AADTT (Average Annual Daily Truck Traffic) Growth

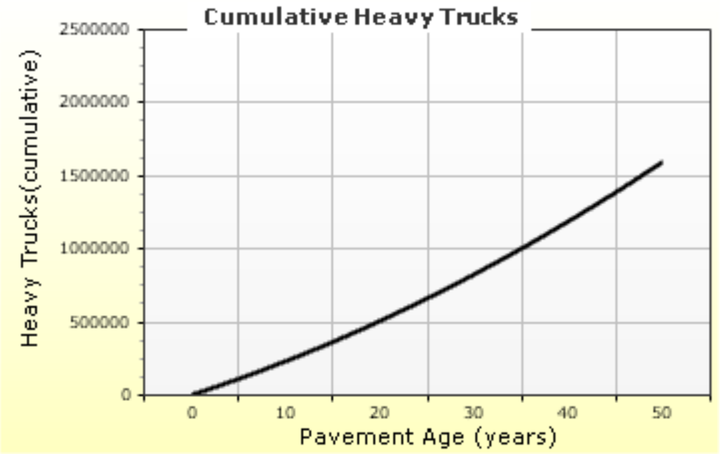
*** Traffic cap is not enforced**







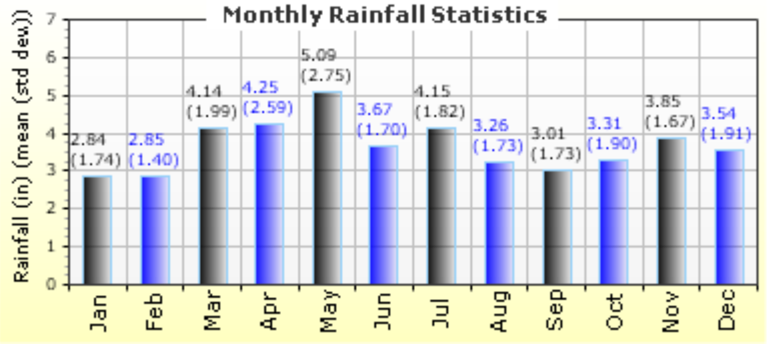




Climate Inputs

Climate Data Sources:

Climate Station Cities:	Location (lat lon elevation(ft))
EVANSVILLE, IN	38.04300 -87.53700 400

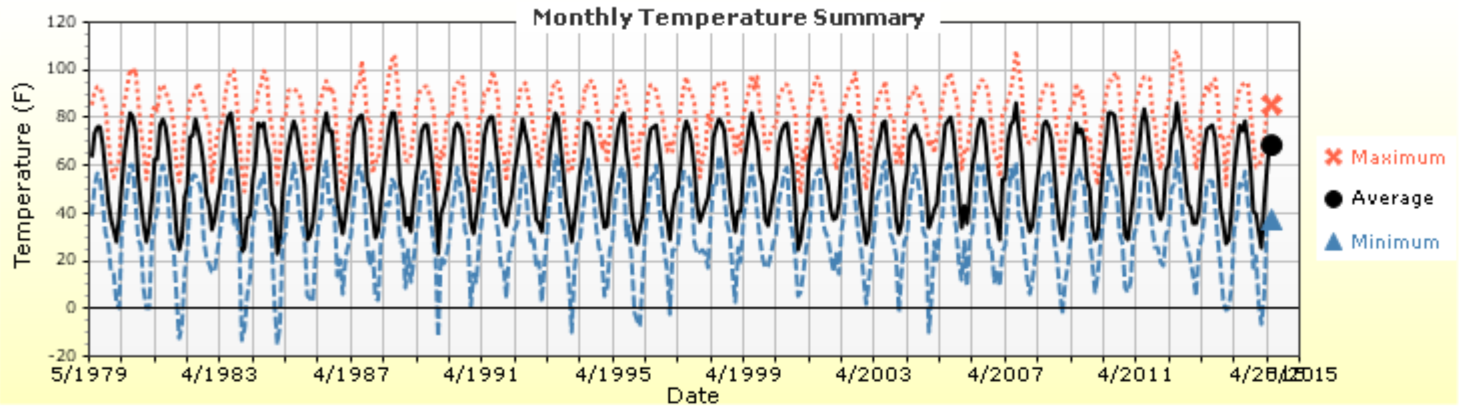


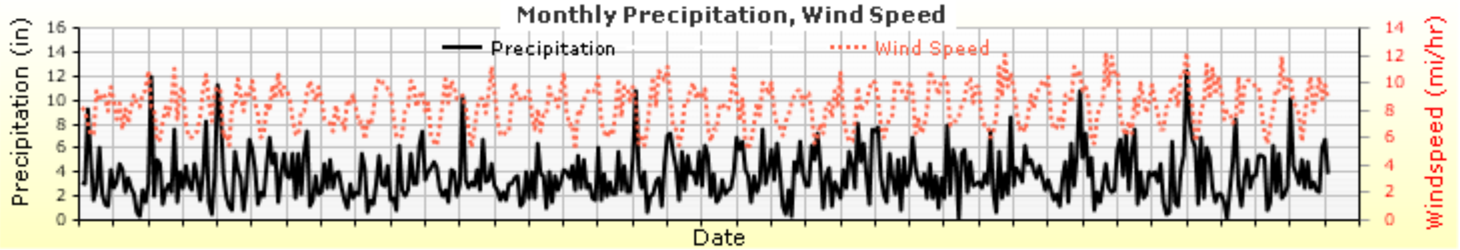
Annual Statistics:

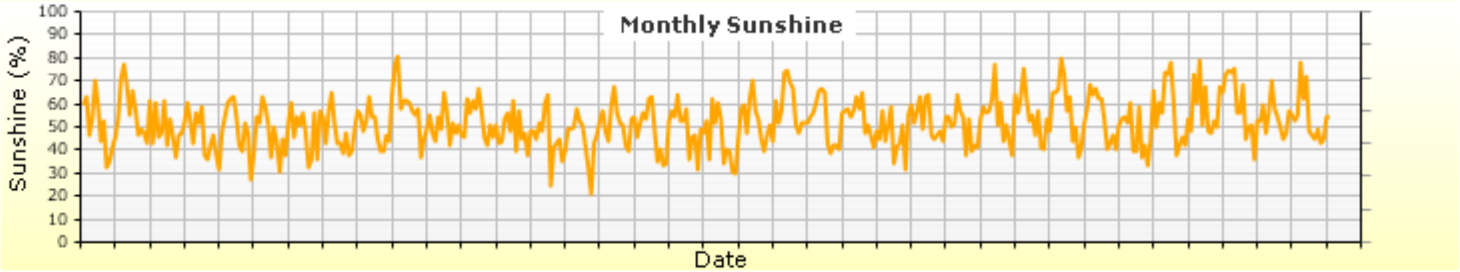
Mean annual air temperature (ºF)	56.72
Mean annual precipitation (in)	43.98
Freezing index (ºF - days)	294.36
Average annual number of freeze/thaw cycles:	56.22

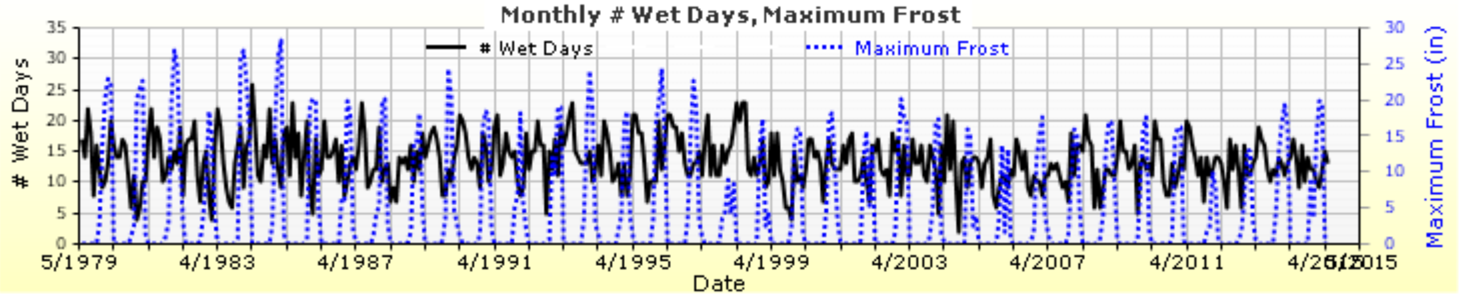
Water table depth (ft)	6.00

Monthly Climate Summary:

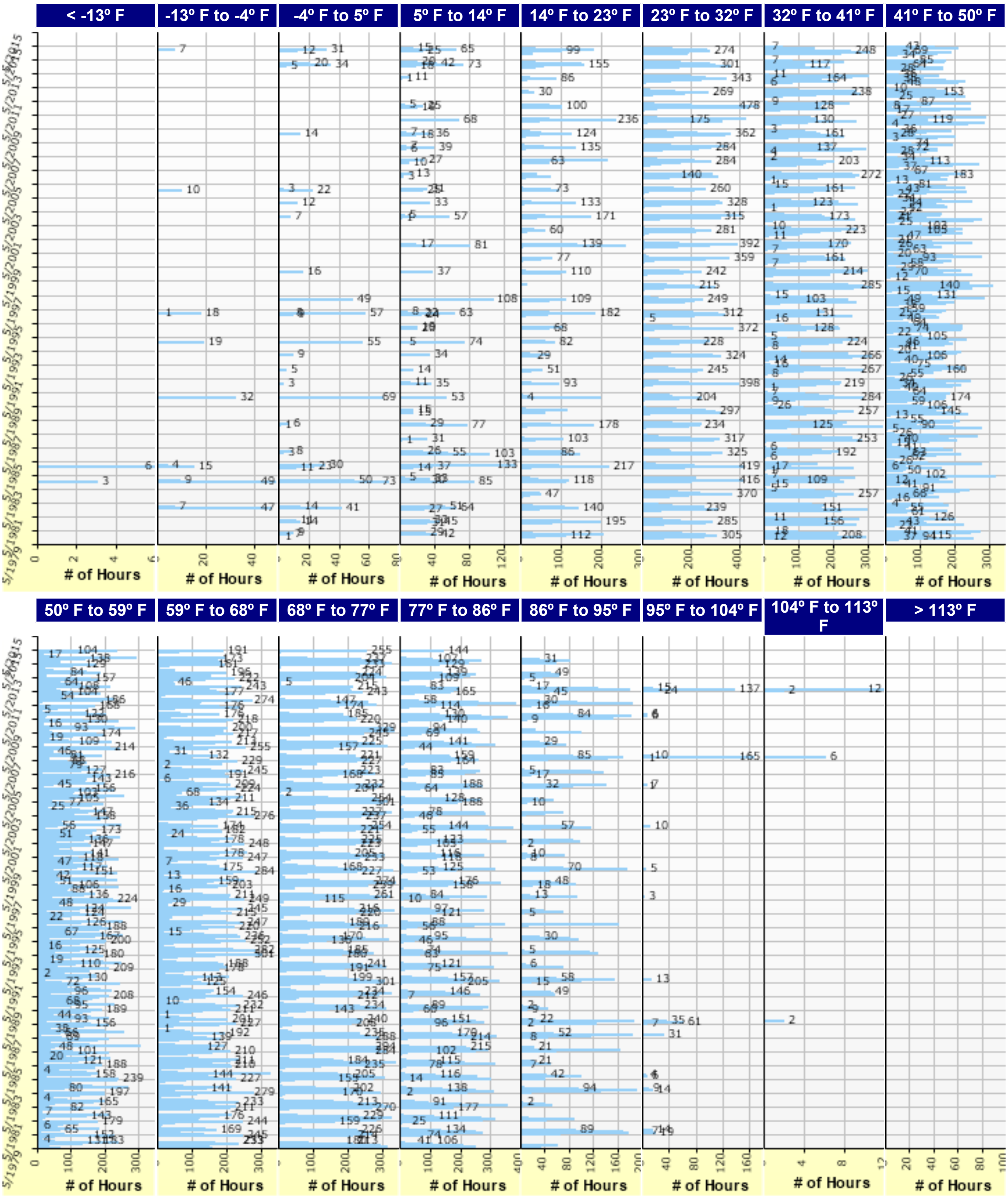








Hourly Air Temperature Distribution by Month:



Design Properties

HMA Design Properties

Use Multilayer Rutting Model	False
Using G* based model (not nationally calibrated)	False
Is NCHRP 1-37A HMA Rutting Model Coefficients	True
Endurance Limit	-
Use Reflective Cracking	True

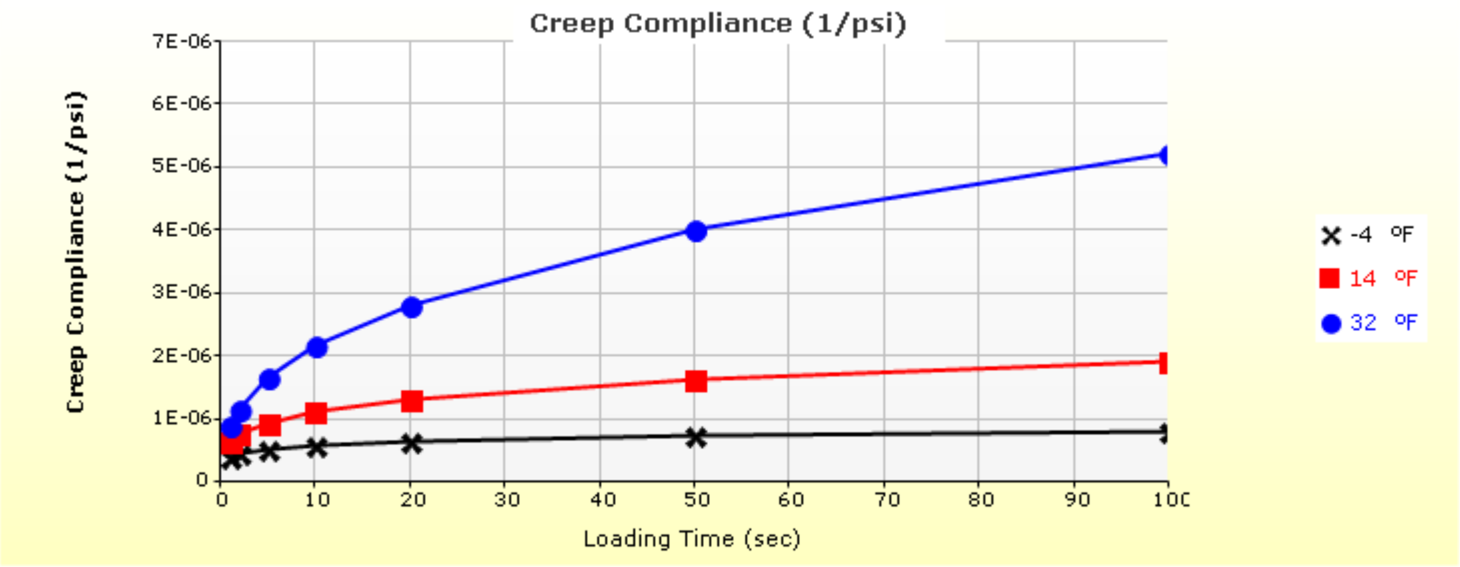
Structure - ICM Properties	
AC surface shortwave absorptivity	0.85

Layer Name	**Layer Type**	**Interface Friction**
Layer 1 Flexible : 9.5mm Surface PG64	Flexible (1)	1.00
Layer 2 Flexible : 19mm Intermediate PG64	Flexible (1)	1.00
Layer 3 Flexible : 19mm Base PG64	Flexible (1)	1.00
Layer 4 Non-stabilized Base : CA No 53	Non-stabilized Base (4)	1.00
Layer 5 Subgrade : Improved subgrade type IB	Subgrade (5)	1.00
Layer 6 Subgrade : Clay (A-7-6)	Subgrade (5)	-

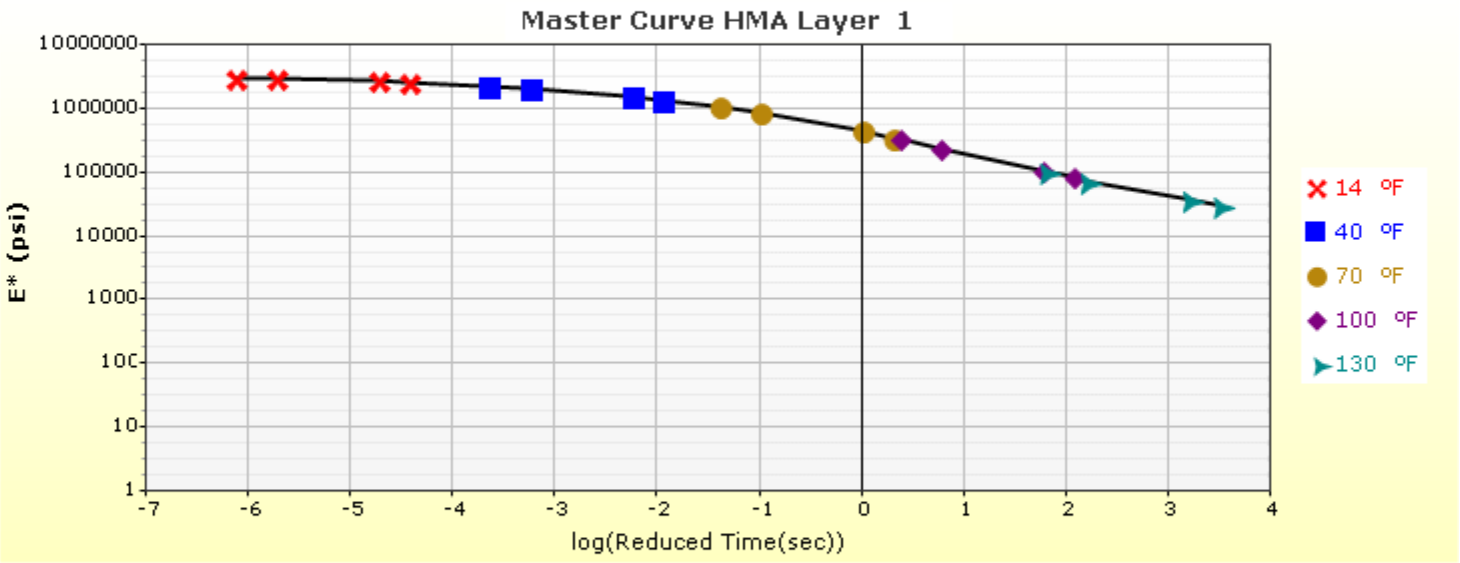
Thermal Cracking (Input Level: 3)

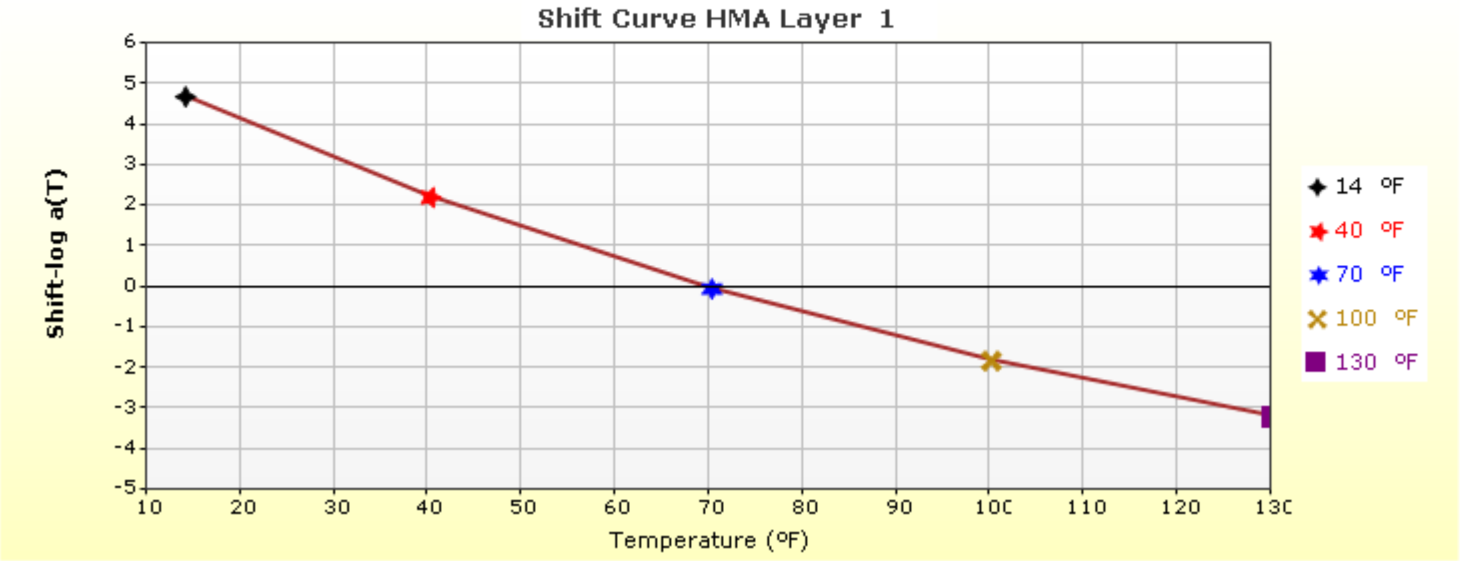
Indirect tensile strength at 14 ºF (psi)	419.80
Thermal Contraction	
Is thermal contraction calculated?	True
Mix coefficient of thermal contraction (in/in/ºF)	-
Aggregate coefficient of thermal contraction (in/in/ºF)	6.1e-006
Voids in Mineral Aggregate (%)	19.6

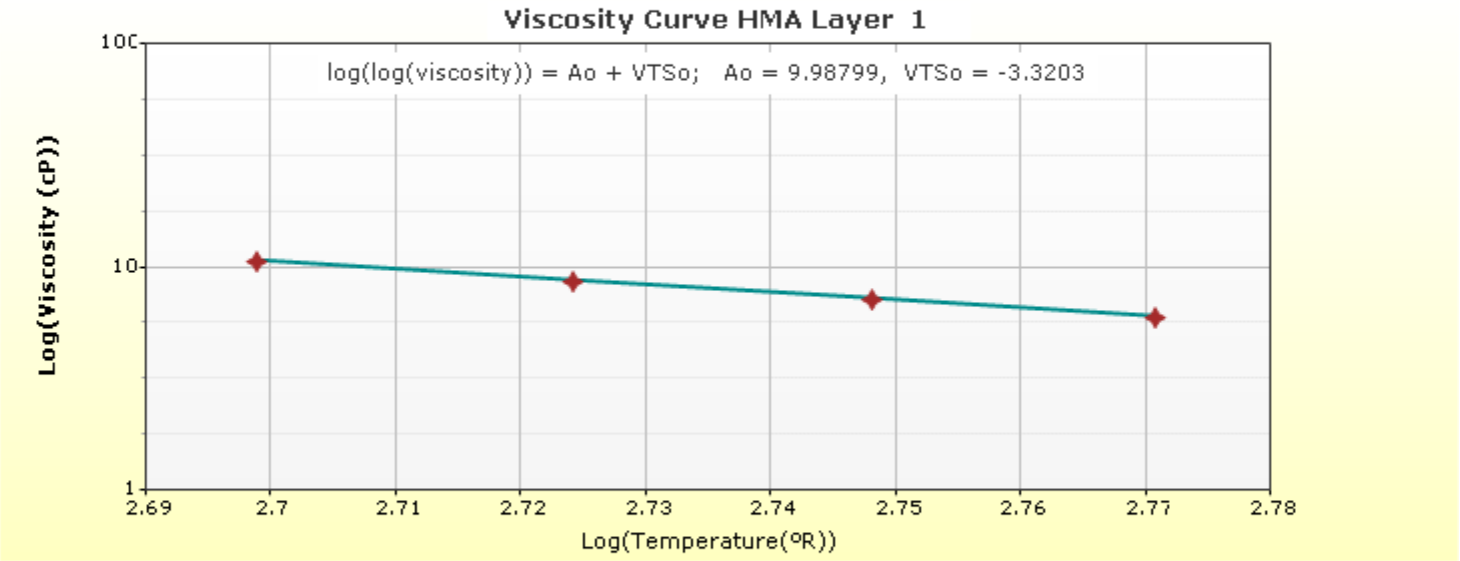
	Creep Compliance (1/psi)		
Loading time (sec)	**-4 ºF**	**14 ºF**	**32 ºF**
1	4.08e-007	6.49e-007	9.01e-007
2	4.53e-007	7.65e-007	1.17e-006
5	5.20e-007	9.51e-007	1.67e-006
10	5.77e-007	1.12e-006	2.17e-006
20	6.40e-007	1.32e-006	2.83e-006
50	7.34e-007	1.64e-006	4.02e-006
100	8.15e-007	1.94e-006	5.23e-006



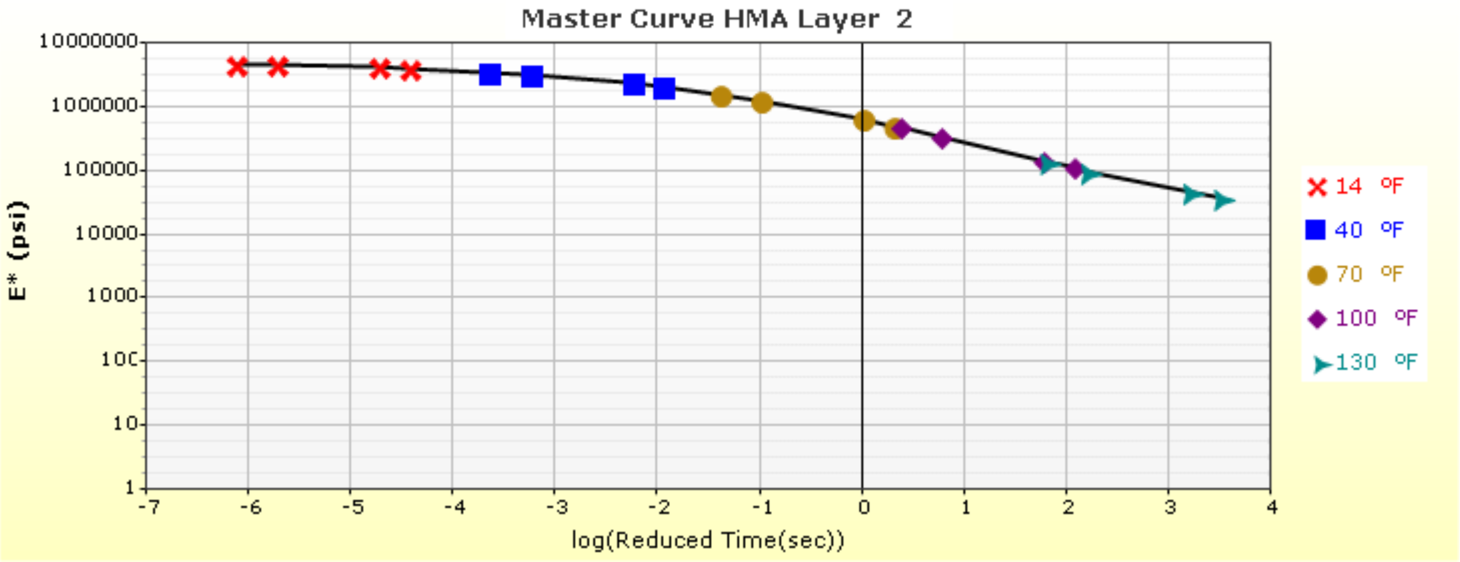
HMA Layer 1: Layer 1 Flexible : 9.5mm Surface PG64

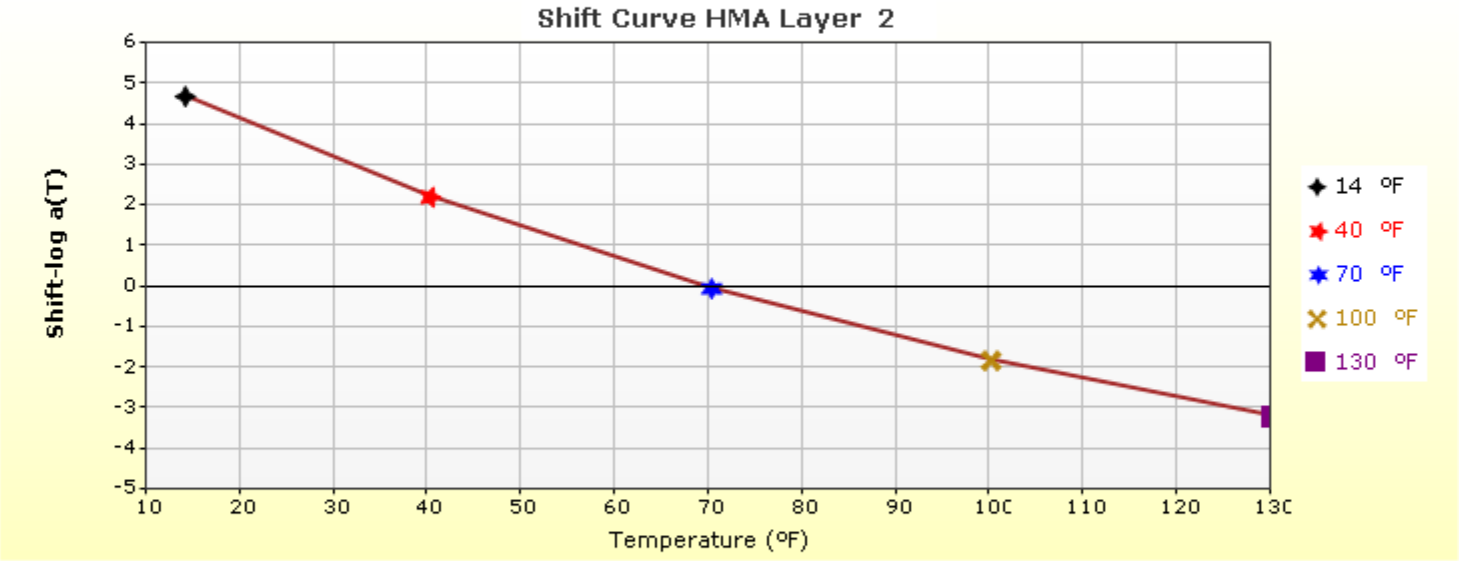


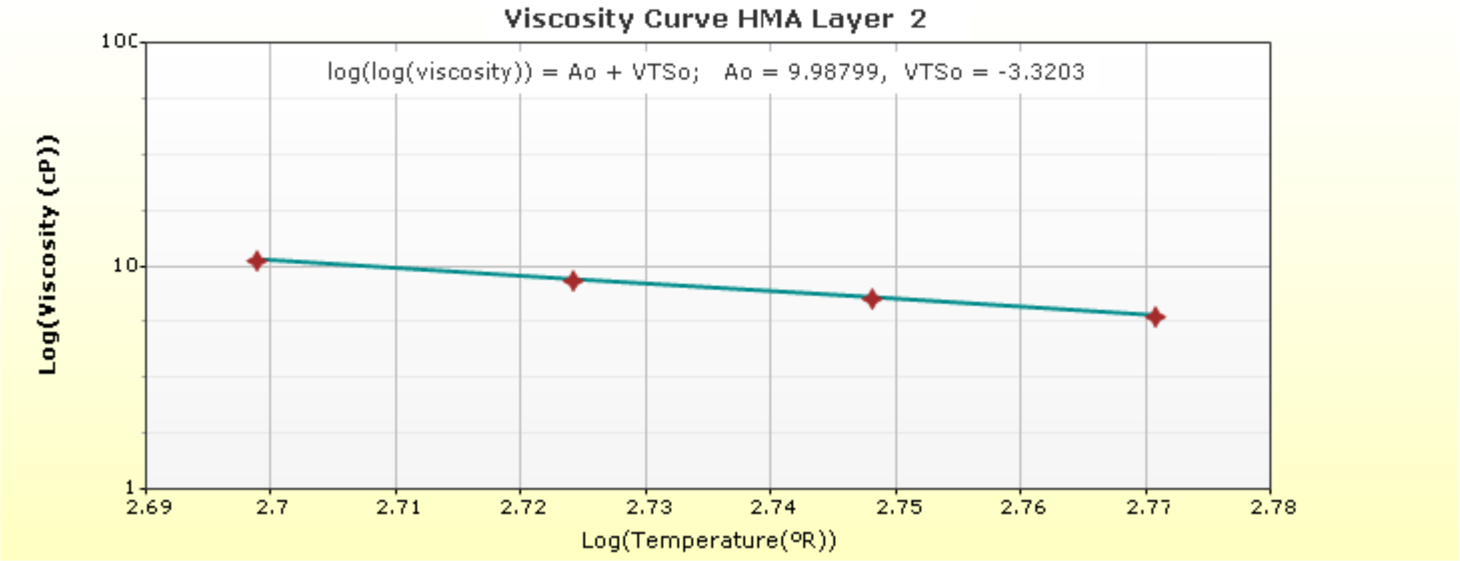




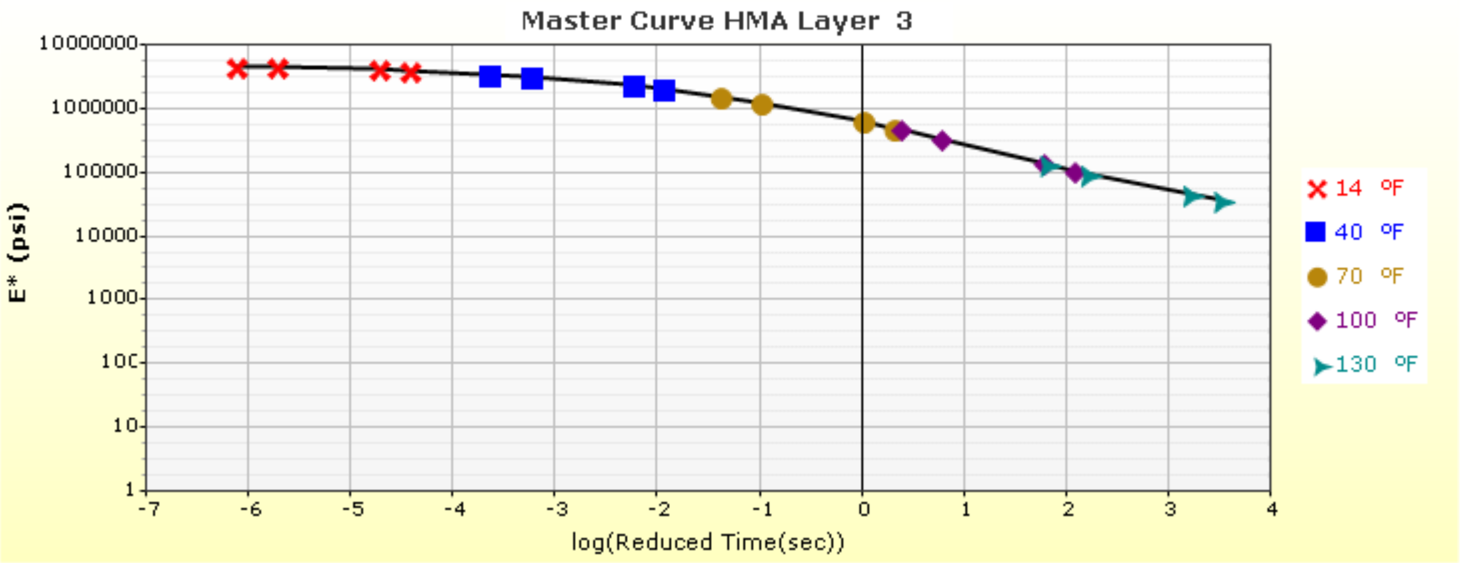
HMA Layer 2: Layer 2 Flexible : 19mm Intermediate PG64

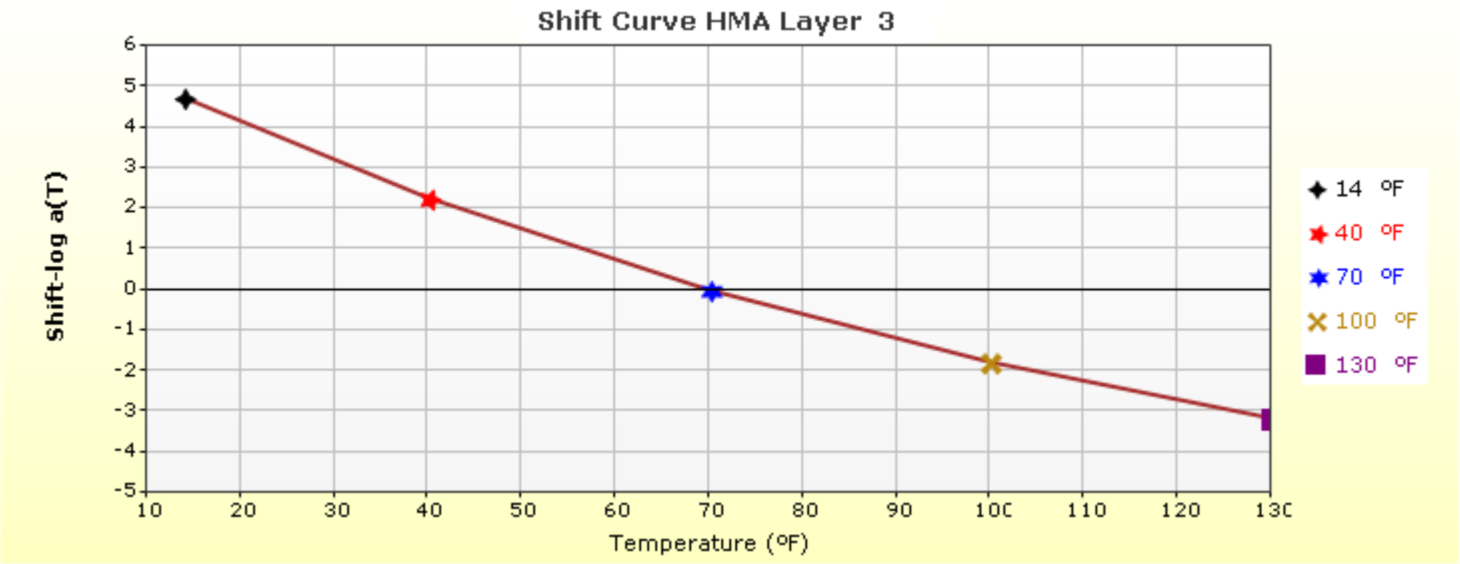


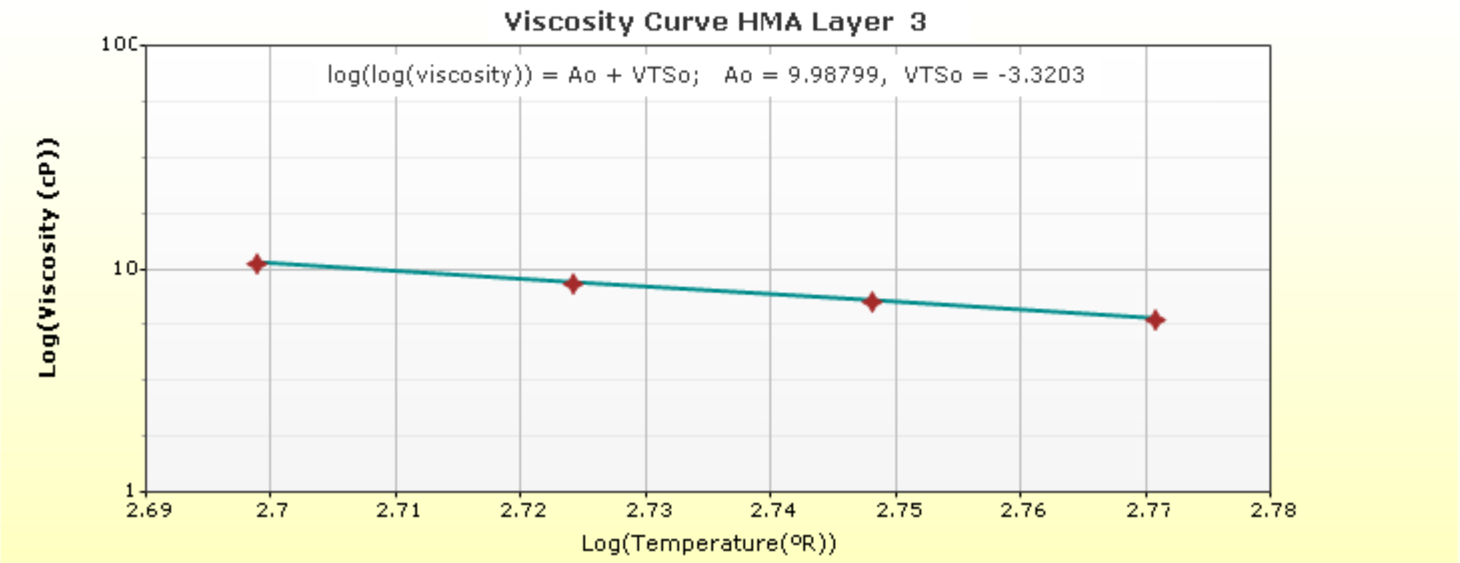




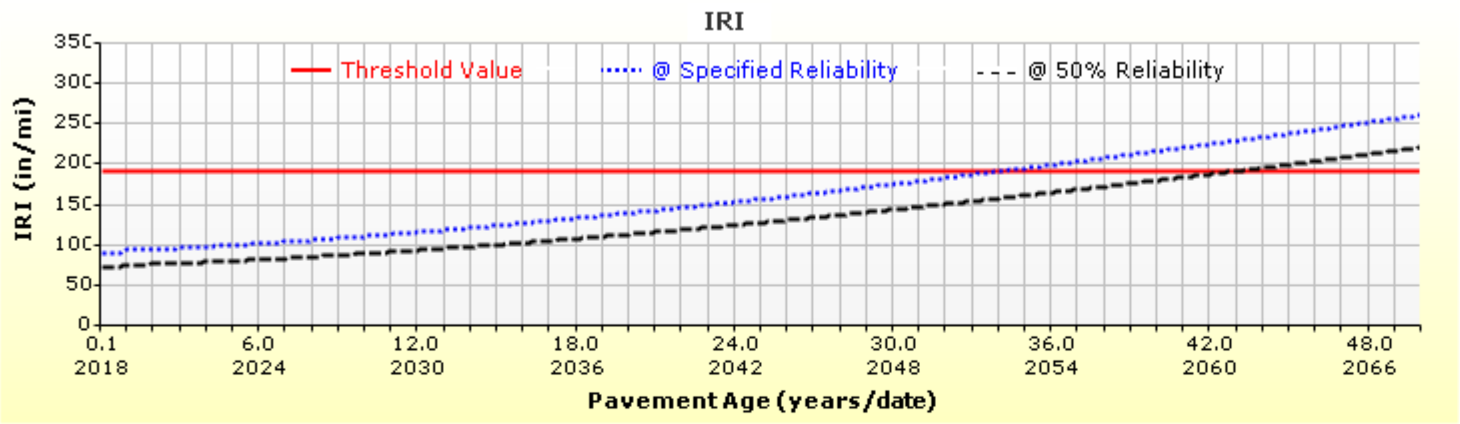
HMA Layer 3: Layer 3 Flexible : 19mm Base PG64

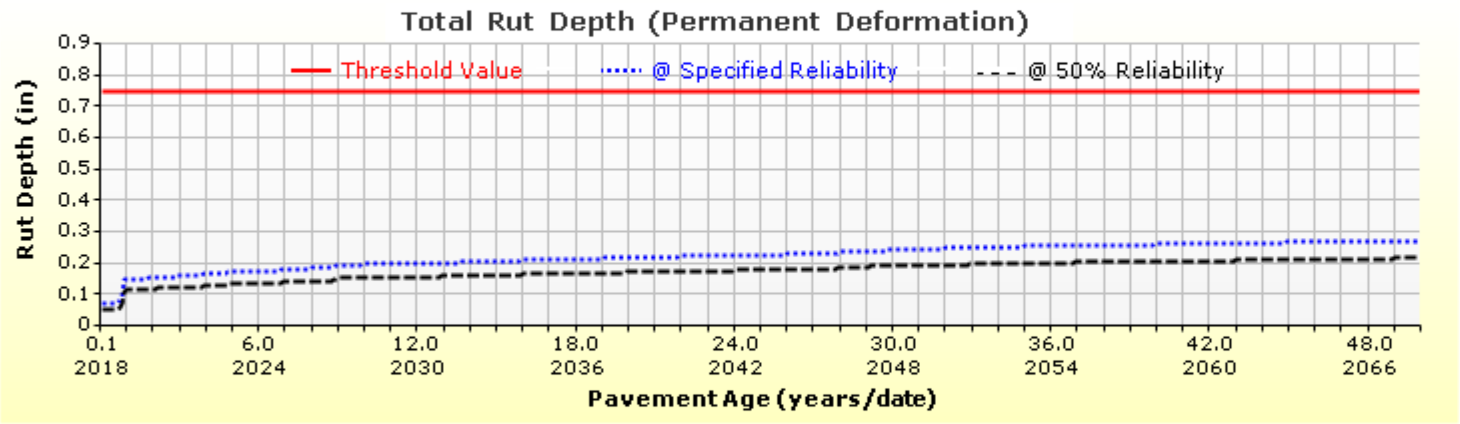


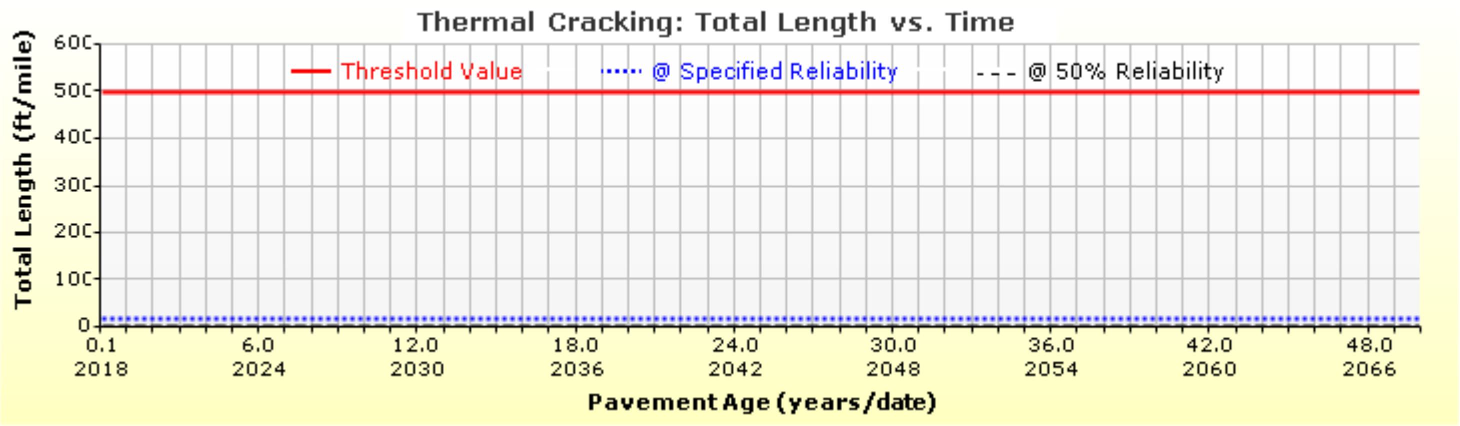


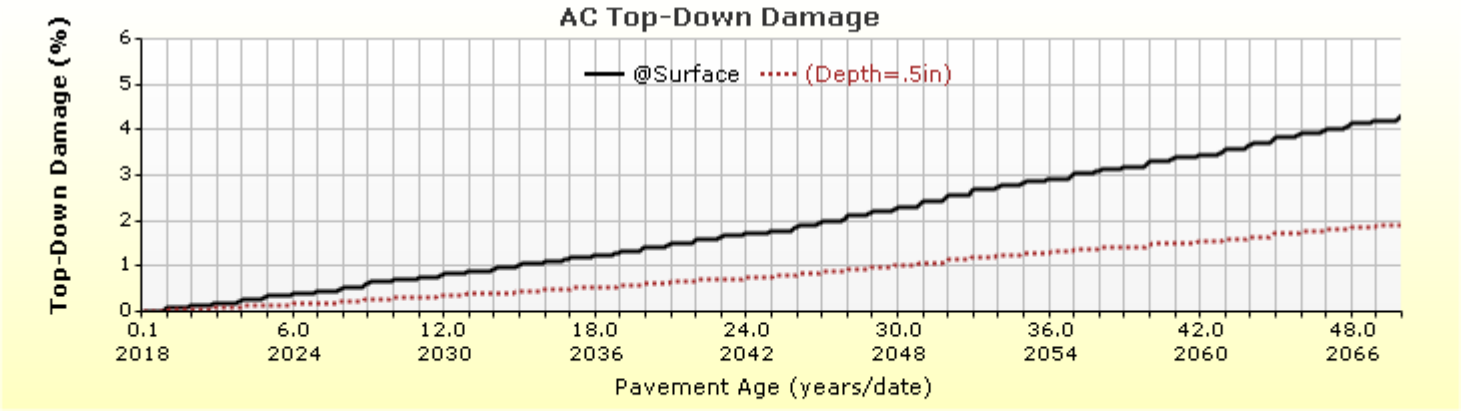


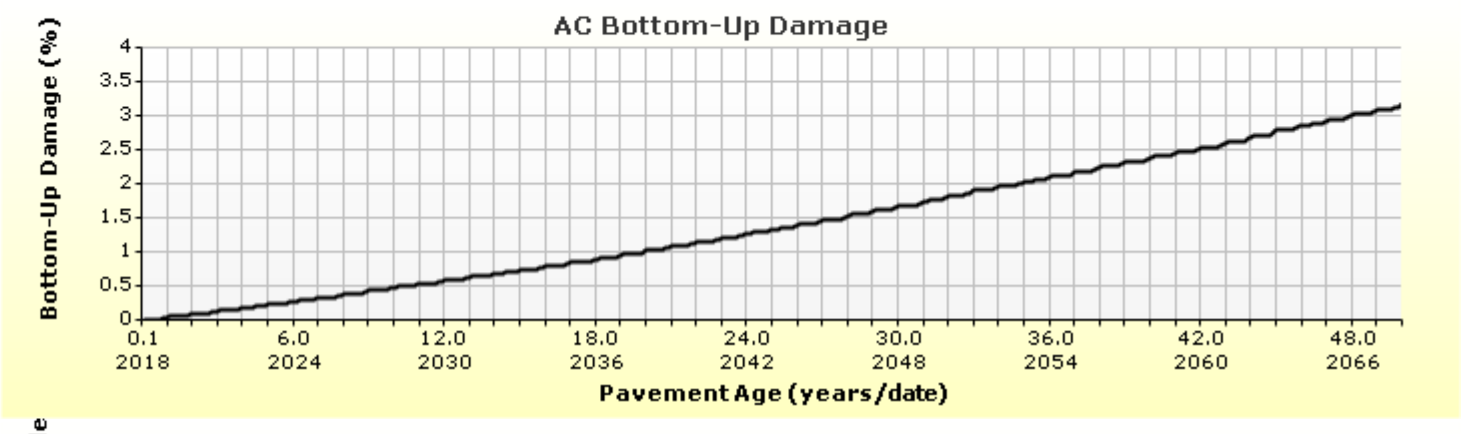
Analysis Output Charts

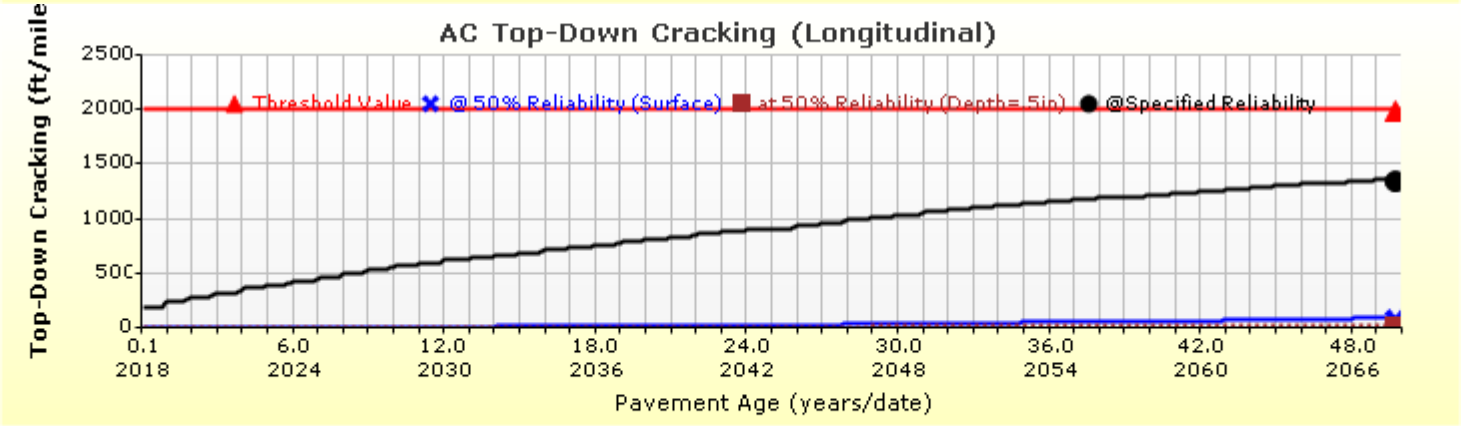


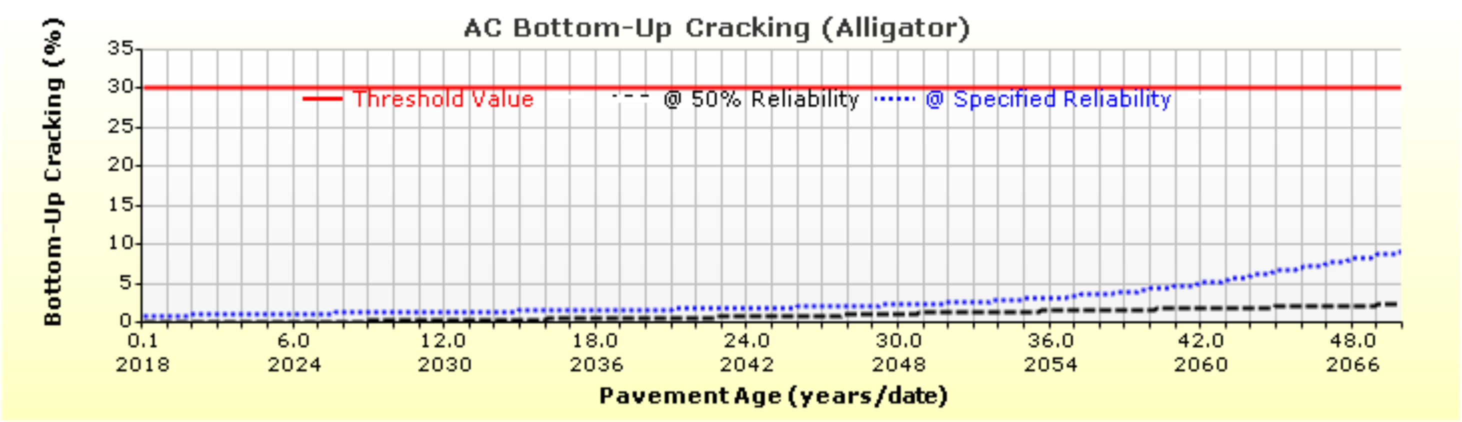


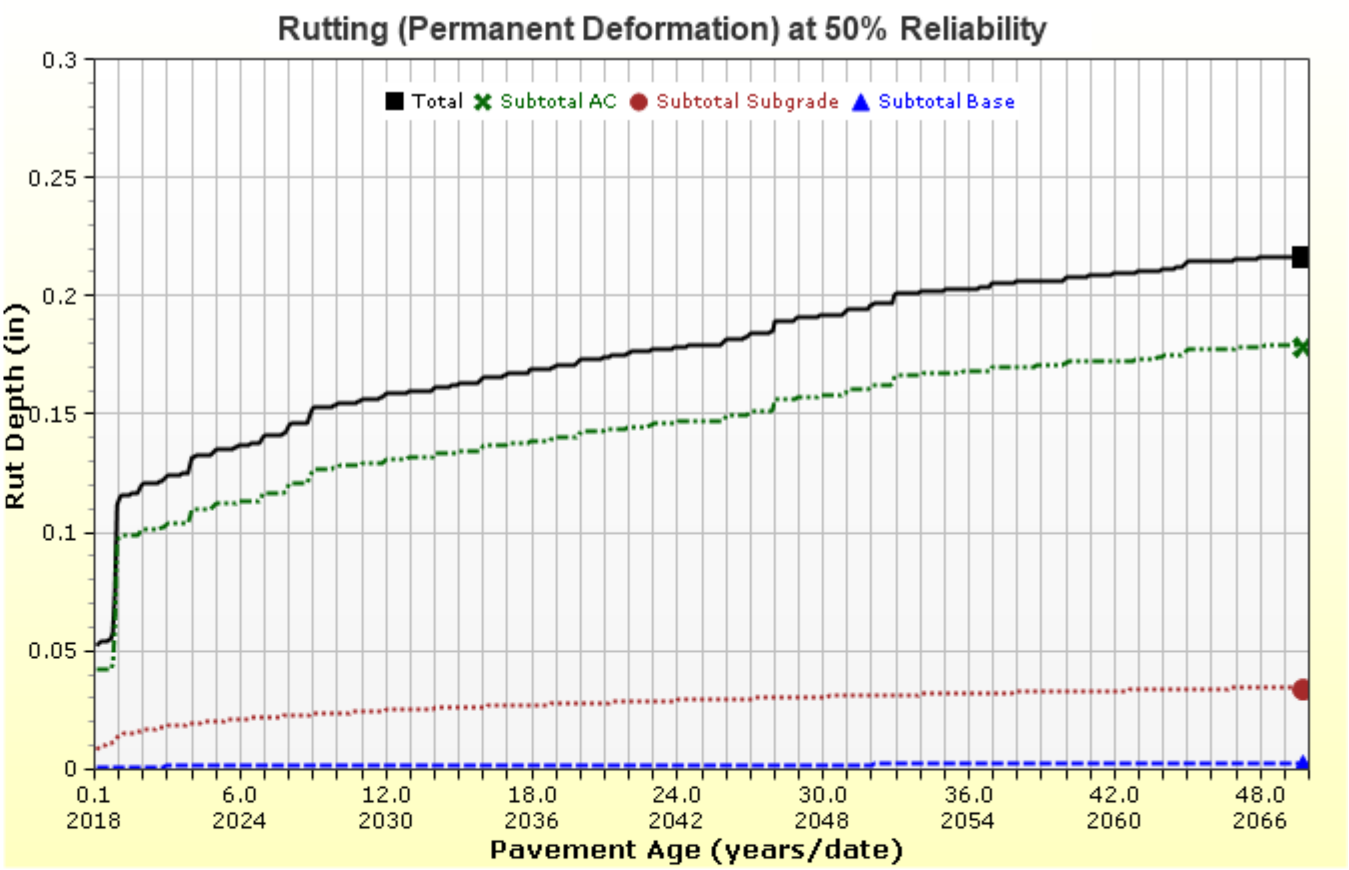



AC Top-Down Damage
@Surface
(Depth=.5in)
Top-Down Damage (%)
0
1
2
3
4
5
6
0.1 2018
6.0 2024
12.0 2030
18.0 2036
24.0 2042
30.0 2048
36.0 2054
42.0 2060
48.0 2066
Pavement Age (years/date)

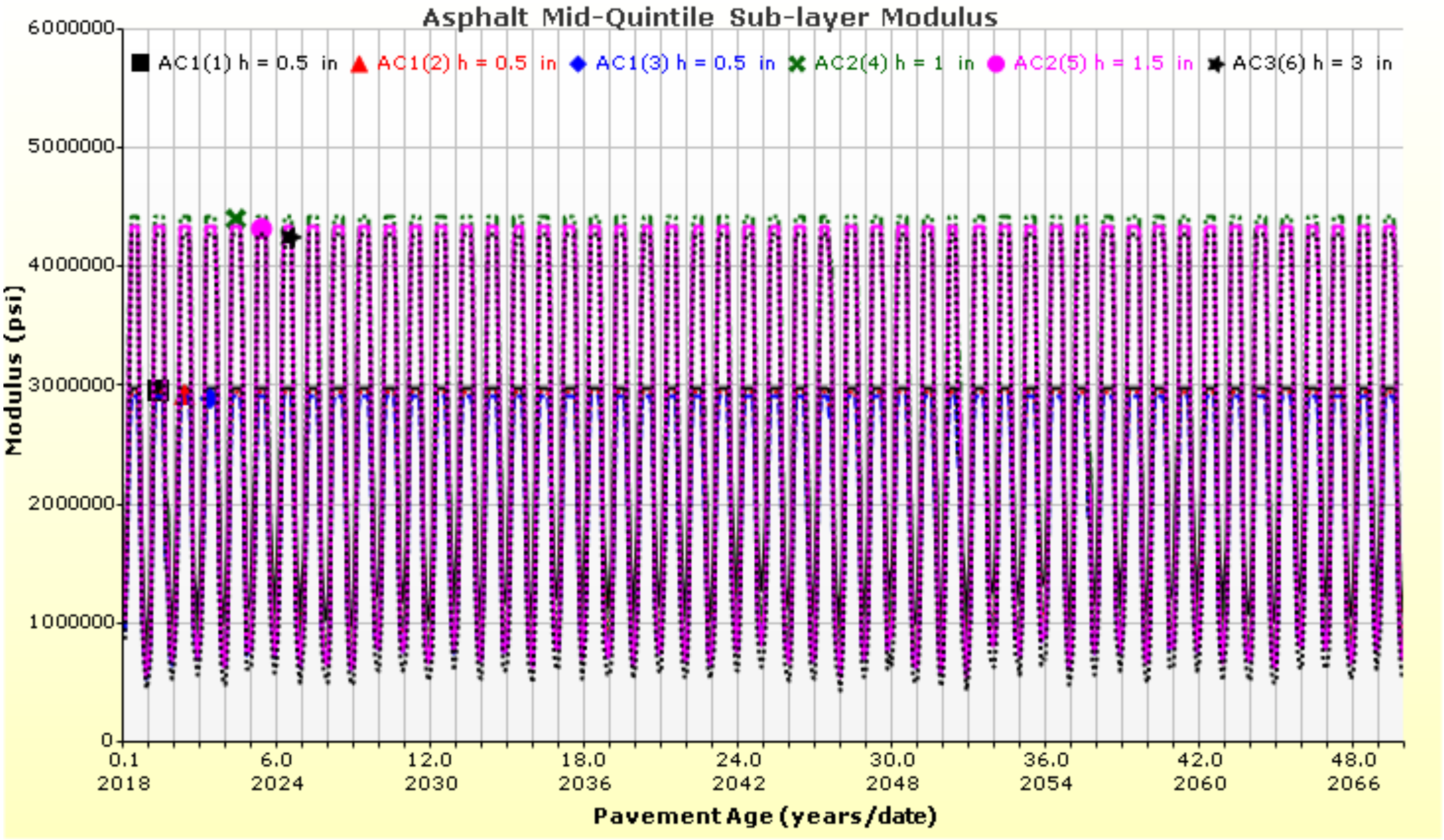

AC Bottom-Up Damage
Bottom-Up Damage (%)
0
0.5
1
1.5
2
2.5
3
3.5
4
0.1 2018
6.0 2024
12.0 2030
18.0 2036
24.0 2042
30.0 2048
36.0 2054
42.0 2060
48.0 2066
Pavement Age (years/date)


AC Top-Down Cracking (Longitudinal)
Threshold Value
@ 50% Reliability (Surface)
at 50% Reliability (Depth=.5in)
@Specified Reliability
Top-Down Cracking (ft/mile
0
500
1000
1500
2000
2500
0.1 2018
6.0 2024
12.0 2030
18.0 2036
24.0 2042
30.0 2048
36.0 2054
42.0 2060
48.0 2066
Pavement Age (years/date)

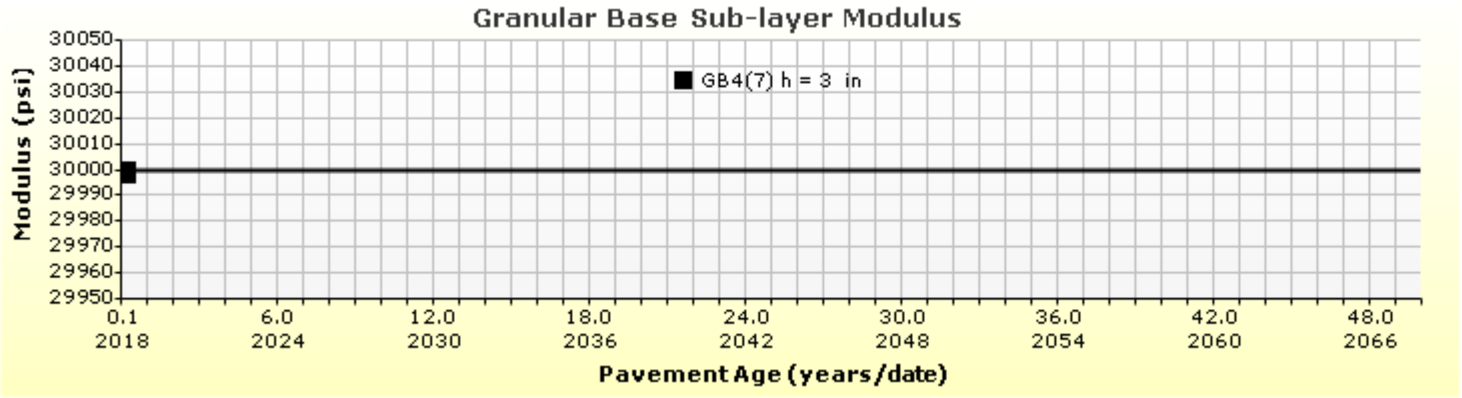

AC Bottom-Up Cracking (Alligator)
Threshold Value
@ 50% Reliability
@ Specified Reliability
Bottom-Up Cracking (%)
0
5
10
15
20
25
30
35
0.1 2018
6.0 2024
12.0 2030
18.0 2036
24.0 2042
30.0 2048
36.0 2054
42.0 2060
48.0 2066
Pavement Age (years/date)



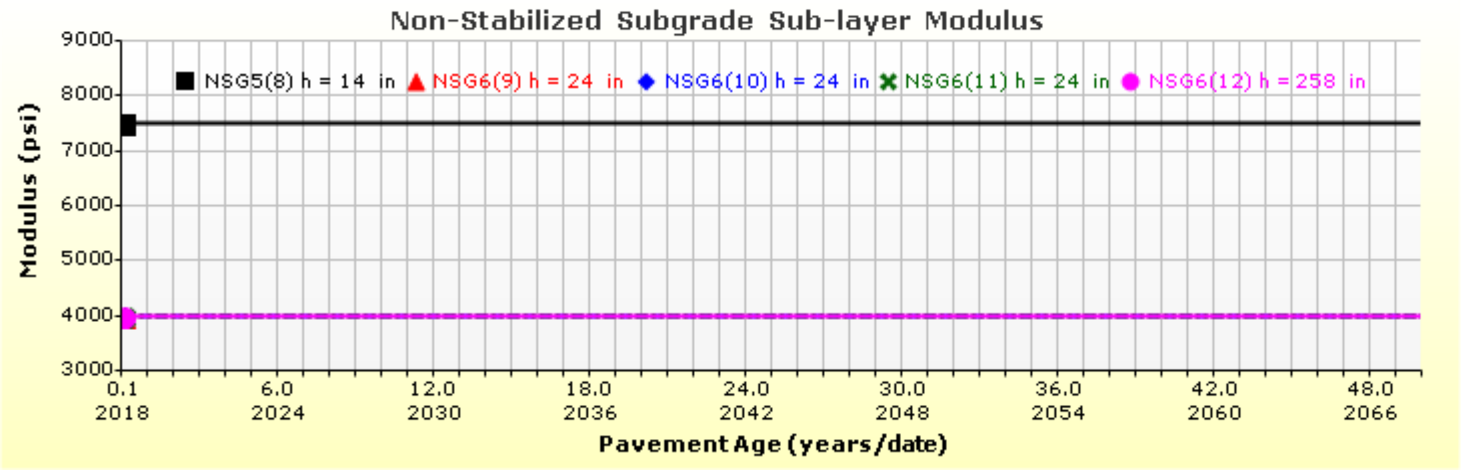
Rutting (Permanent Deformation) at 50% Reliability
■ Total
✖ Subtotal AC
● Subtotal Subgrade
▲ Subtotal Base
Rut Depth (in)
0.3
0.25
0.2
0.15
0.1
0.05
0
0.1
2018
6.0
2024
12.0
2030
18.0
2036
24.0
2042
30.0
2048
36.0
2054
42.0
2060
48.0
2066
Pavement Age (years/date)



Asphalt Mid-Quintile Sub-layer Modulus
AC1(1) h = 0.5 in
AC1(2) h = 0.5 in
AC1(3) h = 0.5 in
AC2(4) h = 1 in
AC2(5) h = 1.5 in
AC3(6) h = 3 in
Modulus (psi)
6000000
5000000
4000000
3000000
2000000
1000000
0
0.1 2018
6.0 2024
12.0 2030
18.0 2036
24.0 2042
30.0 2048
36.0 2054
42.0 2060
48.0 2066
Pavement Age (years/date)



Granular Base Sub-layer Modulus
GB4(7) h = 3 in
Modulus (psi)
30050
30040
30030
30020
30010
30000
29990
29980
29970
29960
29950
0.1 2018
6.0 2024
12.0 2030
18.0 2036
24.0 2042
30.0 2048
36.0 2054
42.0 2060
48.0 2066
Pavement Age (years/date)



Non-Stabilized Subgrade Sub-layer Modulus
NSG5(8) h = 14 in
NSG6(9) h = 24 in
NSG6(10) h = 24 in
NSG6(11) h = 24 in
NSG6(12) h = 258 in
Modulus (psi)
9000
8000
7000
6000
5000
4000
3000
0.1 2018
6.0 2024
12.0 2030
18.0 2036
24.0 2042
30.0 2048
36.0 2054
42.0 2060
48.0 2066
Pavement Age (years/date)

Layer Information

Layer 1 Flexible : 9.5mm Surface PG64

Asphalt		
Thickness (in)	1.5	
Unit weight (pcf)	142.6	
Poisson's ratio	Is Calculated?	False
	Ratio	0.35
	Parameter A	-
	Parameter B	-

Asphalt Dynamic Modulus (Input Level: 1)

T (ºF)	0.1 Hz	1 Hz	10 Hz	25 Hz
10	1998201	2420191	2664791	3288065
40	1337856	1708779	1996975	2273465
70	185430	431985	743934	923021
100	40801	100522	228720	312269
130	21241	38909	85495	119675

Asphalt Binder

Temperature (ºF)	Binder Gstar (Pa)	Phase angle (deg)
40	25339156.98	34.85
55	9930922.35	46.6
70	2585999.72	55.79
85	613653.8	63
100	149927.8	68.71
115	39662.01	73.23
130	11579.02	76.83

General Info

Name	Value
Reference temperature (ºF)	70
Effective binder content (%)	11.6
Air voids (%)	8
Thermal conductivity (BTU/hr-ft-ºF)	0.63
Heat capacity (BTU/lb-ºF)	0.31

Identifiers

Field	Value
Display name/identifier	9.5mm Surface PG64
Description of object	
Author	
Date Created	10/30/2010 1:00:00 AM
Approver	
Date approved	10/30/2010 1:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 1	
User defined field 2	
User defined field 3	
Revision Number	0

Layer 2 Flexible : 19mm Intermediate PG64

Asphalt		
Thickness (in)	2.5	
Unit weight (pcf)	143.8	
Poisson's ratio	Is Calculated?	False
	Ratio	0.35
	Parameter A	-
	Parameter B	-

Asphalt Dynamic Modulus (Input Level: 1)

T (ºF)	0.1 Hz	1 Hz	10 Hz	25 Hz
10	3068424	3745817	4147011	5000000
40	2019038	2605875	3065729	3510185
70	256999	620843	1094369	1370481
100	52936	135667	319812	442511
130	26742	50370	114566	162720

Asphalt Binder

Temperature (ºF)	Binder Gstar (Pa)	Phase angle (deg)
40	25339156.98	34.85
55	9930922.35	46.6
70	2585999.72	55.79
85	613653.8	63
100	149927.8	68.71
115	39662.01	73.23
130	11579.02	76.83

General Info

Name	Value
Reference temperature (ºF)	70
Effective binder content (%)	9.5
Air voids (%)	8
Thermal conductivity (BTU/hr-ft-ºF)	0.63
Heat capacity (BTU/lb-ºF)	0.31

Identifiers

Field	Value
Display name/identifier	19mm Intermediate PG64
Description of object	
Author	
Date Created	9/23/2016 12:00:00 AM
Approver	
Date approved	10/30/2010 1:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 1	
User defined field 2	
User defined field 3	
Revision Number	0

Layer 3 Flexible : 19mm Base PG64

Asphalt		
Thickness (in)	3.0	
Unit weight (pcf)	143.8	
Poisson's ratio	Is Calculated?	False
	Ratio	0.35
	Parameter A	-
	Parameter B	-

Asphalt Dynamic Modulus (Input Level: 1)

T (ºF)	0.1 Hz	1 Hz	10 Hz	25 Hz
10	3096355	3785826	4193128	5000000
40	2029736	2625490	3093190	3546158
70	253654	617549	1093998	1372761
100	51493	133126	316229	438810
130	25826	48970	112241	159924

Asphalt Binder

Temperature (ºF)	Binder Gstar (Pa)	Phase angle (deg)
40	25339156.98	34.85
55	9930922.35	46.6
70	2585999.72	55.79
85	613653.8	63
100	149927.8	68.71
115	39662.01	73.23
130	11579.02	76.83

General Info

Name	Value
Reference temperature (ºF)	70
Effective binder content (%)	9.53
Air voids (%)	8
Thermal conductivity (BTU/hr-ft-ºF)	0.63
Heat capacity (BTU/lb-ºF)	0.31

Identifiers

Field	Value
Display name/identifier	19mm Base PG64
Description of object	
Author	
Date Created	10/30/2010 1:00:00 AM
Approver	
Date approved	10/30/2010 1:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 1	
User defined field 2	
User defined field 3	
Revision Number	0

Layer 4 Non-stabilized Base : CA No 53

Unbound	
Layer thickness (in)	3.0
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5

Modulus (Input Level: 2)

Analysis Type:	Annual representative values
Method:	Resilient Modulus (psi)

Resilient Modulus (psi)
30000.0

Use Correction factor for NDT modulus?	-
NDT Correction Factor:	-

Identifiers

Field	Value
Display name/identifier	CA No 53
Description of object	Separation layer
Author	AASHTO
Date Created	1/1/2011 12:00:00 AM
Approver	
Date approved	1/1/2011 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 1	
User defined field 2	
User defined field 3	
Revision Number	0

Sieve

Liquid Limit	6.0
Plasticity Index	1.0
Is layer compacted?	True

	Is User Defined?	Value
Maximum dry unit weight (pcf)	True	130
Saturated hydraulic conductivity (ft/hr)	False	3.346e-02
Specific gravity of solids	False	2.7
Water Content (%)	True	6

User-defined Soil Water Characteristic Curve (SWCC)	
Is User Defined?	False
af	3.9821
bf	1.7866
cf	0.7431
hr	115.0000

Sieve Size	% Passing
0.001mm	
0.002mm	
0.020mm	
#200	7.5
#100	
#80	
#60	
#50	
#40	
#30	21.0
#20	
#16	
#10	
#8	37.5
#4	47.5
3/8-in.	
1/2-in.	67.5
3/4-in.	80.0
1-in.	90.0
1 1/2-in.	100.0
2-in.	
2 1/2-in.	
3-in.	
3 1/2-in.	

Layer 5 Subgrade : Improved subgrade type IB

Unbound	
Layer thickness (in)	14.0
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5

Modulus (Input Level: 2)

Analysis Type:	Annual representative values
Method:	Resilient Modulus (psi)

Resilient Modulus (psi)
7500.0

Use Correction factor for NDT modulus?	-
NDT Correction Factor:	-

Identifiers

Field	**Value**
Display name/identifier	Improved subgrade type IB
Description of object	Improved subgrade
Author	AASHTO
Date Created	1/1/2011 12:00:00 AM
Approver	
Date approved	1/1/2011 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 1	
User defined field 2	
User defined field 3	
Revision Number	0

Sieve

Liquid Limit	83.0
Plasticity Index	62.0
Is layer compacted?	True

	Is User Defined?	**Value**
Maximum dry unit weight (pcf)	True	100.7
Saturated hydraulic conductivity (ft/hr)	False	1.176e-06
Specific gravity of solids	False	2.7
Water Content (%)	True	20

User-defined Soil Water Characteristic Curve (SWCC)	
Is User Defined?	False
af	163.1526
bf	0.3999
cf	0.0300
hr	500.0000

Sieve Size	**% Passing**
0.001mm	
0.002mm	
0.020mm	
#200	86.4
#100	
#80	
#60	
#50	
#40	95.7
#30	
#20	
#16	
#10	99.2
#8	
#4	
3/8-in.	
1/2-in.	
3/4-in.	
1-in.	
1 1/2-in.	
2-in.	
2 1/2-in.	
3-in.	
3 1/2-in.	

Layer 6 Subgrade : Clay (A-7-6)

Unbound	
Layer thickness (in)	Semi-infinite
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5

Modulus (Input Level: 2)

Analysis Type:	Annual representative values
Method:	Resilient Modulus (psi)

Resilient Modulus (psi)
4000.0

Use Correction factor for NDT modulus?	-
NDT Correction Factor:	-

Identifiers

Field	**Value**
Display name/identifier	Clay (A-7-6)
Description of object	Natural Subgrade
Author	ys
Date Created	9/23/2016 12:00:00 AM
Approver	
Date approved	1/1/0001 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 1	
User defined field 2	
User defined field 3	
Revision Number	0

Sieve

Liquid Limit	83.0
Plasticity Index	62.0
Is layer compacted?	False

	Is User Defined?	**Value**
Maximum dry unit weight (pcf)	False	90.7
Saturated hydraulic conductivity (ft/hr)	False	6.921e-07
Specific gravity of solids	False	2.7
Water Content (%)	False	25.7

User-defined Soil Water Characteristic Curve (SWCC)	
Is User Defined?	False
af	163.1526
bf	0.3999
cf	0.0300
hr	500.0000

Sieve Size	**% Passing**
0.001mm	
0.002mm	
0.020mm	
#200	86.4
#100	
#80	
#60	
#50	
#40	95.7
#30	
#20	
#16	
#10	99.2
#8	
#4	
3/8-in.	
1/2-in.	
3/4-in.	
1-in.	
1 1/2-in.	
2-in.	
2 1/2-in.	
3-in.	
3 1/2-in.	

Calibration Coefficients

AC Fatigue

$$N_f = 0.00432 * C * \beta_{f1} k_1 \left(\frac{1}{\varepsilon_1}\right)^{k_2 \beta_{f2}} \left(\frac{1}{\mathrm{E}}\right)^{k_3 \beta_{f3}}$$

$$C = 10^M$$

$$M = 4.84\left(\frac{V_b}{V_a + V_b} - 0.69\right)$$

k1: 0.007566
k2: 3.9492
k3: 1.281
Bf1: 1
Bf2: 1
Bf3: 1

AC Rutting

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z \beta_{r1} 10^{k_1} T^{k_2 \beta_{r2}} N^{k_3 B_{r3}}$$

$$k_z = (C_1 + C_2 * depth) * 0.328196^{depth}$$

$$C_1 = -0.1039 * H_\alpha^2 + 2.4868 * H_\alpha - 17.342$$

$$C_2 = 0.0172 * H_\alpha^2 - 1.7331 * H_\alpha + 27.428$$

Where:

$H_{ac} = total\ AC\ thickness(in)$

$\varepsilon_p = plastic\ strain\left({}^{in}/_{in}\right)$
$\varepsilon_r = resilient\ strain\left({}^{in}/_{in}\right)$
$T = layer\ temperature\left({}^{\circ}F\right)$
$N = number\ of\ load\ repetitions$

AC Rutting Standard Deviation	0.24*Pow(RUT,0.8026)+0.001	
AC Layer	K1:-3.35412 K2:1.5606 K3:0.4791	Br1:0.07 Br2:1.9 Br3:0.4

Thermal Fracture

$$C_f = 400 * N\left(\frac{\log C / h_{ac}}{\sigma}\right)$$

$$\Delta C = (k * \beta t)^{n+1} * A * \Delta K^n$$

$$A = 10^{(4.389 - 2.52 * \log(E * \sigma_m * n))}$$

$C_f = observed\ amount\ of\ thermal\ cracking(ft/500ft)$
$k = refression\ coefficient\ determined\ through\ field\ calibration$
$N(\ ) = standard\ normal\ distribution\ evaluated\ at(\ )$
$\sigma = standard\ deviation\ of\ the \log\ of\ the\ depth\ of\ cracks\ in\ the\ pavments$
$C = crack\ depth(in)$
$h_{ac} = thickness\ of\ asphalt\ layer(in)$
$\Delta C = Change\ in\ the\ crack\ depth\ due\ to\ a\ cooling\ cycle$
$\Delta K = Change\ in\ the\ stress\ intensity\ factor\ due\ to\ a\ cooling\ cycle$
$A, n = Fracture\ parameters\ for\ the\ asphalt\ mixture$
$E = mixture\ stiffness$
$\sigma_M = Undamaged\ mixture\ tensile\ strength$
$\beta_t = Calibration\ parameter$

Level 1 K: 1.5	Level 1 Standard Deviation: 0.1468 * THERMAL + 65.027
Level 2 K: 0.5	Level 2 Standard Deviation: 0.2841 *THERMAL + 55.462
Level 3 K: 1.5	Level 3 Standard Deviation: 0.3972 * THERMAL + 20.422

CSM Fatigue

$$N_f = 10^{\left(\frac{k_1 \beta_{c1}\left(\frac{\sigma_s}{M_r}\right)}{k_2 \beta_{c2}}\right)}$$

$N_f = number\ of\ repetitions\ to\ fatigue\ cracking$
$\sigma_s = Tensile\ stress(psi)$
$M_r = modulus\ of\ rupture(psi)$

k1: 1	k2: 1	Bc1: 1	Bc2:1

Subgrade Rutting

$$\delta_a(N) = \beta_{s_1} k_1 \varepsilon_v h \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \left\lvert e^{-\left(\frac{\rho}{N}\right)^{\beta}} \right\rvert$$

$\delta_a = permanent\ deformation\ for\ the\ layer$
$N = number\ of\ repetitions$
$\varepsilon_v = average\ veritcal\ strain(in/in)$
$\varepsilon_0, \beta, \rho = material\ properties$
$\varepsilon_r = resilient\ strain(in/in)$

Granular		Fine	
k1: 2.03	Bs1: 0.12	k1: 1.35	Bs1: 0.12
Standard Deviation (BASERUT) 0.1477*Pow(BASERUT,0.6711)+0.001		Standard Deviation (BASERUT) 0.1235*Pow(SUBRUT,0.5012)+0.001	

AC Cracking

AC Top Down Cracking

$$FC_{top} = \left(\frac{C_4}{1 + e^{\left(C_1 - C_2 * \log_{10}(Damage)\right)}}\right) * 10.56$$

c1: 7	c2: 3.5	c3: 0	c4: 1000

AC Cracking Top Standard Deviation

200 + 2300/(1+exp(1.072-2.1654*LOG10 (TOP+0.0001)))

AC Bottom Up Cracking

$$FC = \left(\frac{6000}{1 + e^{\left(C_1 * C_1' + C_2 * C_2' \log_{10}(D * 100)\right)}}\right) * \left(\frac{1}{60}\right)$$

$$C_2' = -2.40874 - 39.748 * (1 + h_{ac})^{-2.856}$$

$$C_1' = -2 * C_2'$$

c1: 1	c2: 1	c3: 6000

AC Cracking Bottom Standard Deviation

1.13+13/(1+exp(7.57-15.5*LOG10 (BOTTOM+0.0001)))

CSM Cracking

$$FC_{ctb} = C_1 + \frac{C_2}{1 + e^{C_3 - C_4(Damage)}}$$

C1: 1	C2: 1	C3: 0	C4: 1000

CSM Standard Deviation

CTB*1

IRI Flexible Pavements

C1 - Rutting
C2 - Fatigue Crack
C3 - Transverse Crack
C4 - Site Factors

C1: 40	C2: 0.4	C3: 0.008	C4: 0.015