Preliminary Engineering Report

APPLEGATE DRIVE

LEWIS AND CLARK COUNTY

RPA Project No. 11502.000

Prepared For:

LEWIS AND CLARK COUNTY
3402 Cooney Drive
Helena, MT 59602

Prepared By:

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February 2012

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Executive Summary

This roadway Preliminary Engineering Report (PER) was developed under contract administered by the Lewis and Clark County Public Works office. The PER is intended to provide an initial evaluation of the approximate 3.0 mile long Applegate Road corridor which is bound by Lincoln Road on the southern end. The PER evaluates road deficiencies and identifies future needs, thereby providing an assessment of improvements necessary to meet or exceed current County road standards. This report is also intended to provide base reconstruction cost estimates used to aid the county in funding development to meet the purpose and need for the desired road improvements.

ES.1. Summary of Findings

The existing roadway does not meet several minimum design criteria presented as guidance by the American Association of State Highway and Transportation Officials (AASHTO), or the minimum standards set by Lewis and Clark County. Likewise, the current surfacing, which is gravel (not paved) for the project corridor, is deficient to meet the needs of the projected loadings it will experience within the study's design life evaluation period. Although the horizontal and vertical alignments are within minimum accepted standards, the roadside ditches do not meet current county depths. Hence, to increase ditch depth which would benefit the road by providing an increase runoff flow capacity would require roadside cuts beyond the existing template.

Based on the evaluation presented herein, we estimate the cost to reconstruct the road to meet assigned design criteria to be approximately **$1.0 million per mile** average. This cost estimate includes further engineering, traffic control during construction, right-of-way acquisition and other contingencies. Base construction cost is estimated to be approximately $670,000 per mile, excluding costs for additional right-or-way, final engineering etc.

1. Introduction

This roadway Preliminary Engineering Report (PER) was prepared by Robert Peccia and Associates (RPA) under contract with Lewis and Clark County, Montana. The contract is administered by the Lewis and Clark County Public Works office. The study segment is a portion of Applegate Drive starting at the intersection with Lincoln Road (Secondary Highway 279) and continuing north for approximately 3.0 miles. The study corridor is further described in the following section.

This segment of Applegate Drive is considered a high-priority road by County staff to receive reconstructive improvements. The prioritization is in some part due to increasing roadway maintenance needs indicative of the impacts caused by current traffic use. In addition, when compared to other portions of the County, this area has experienced a high amount of residential subdivision construction in recent years. Development has added a proportional amount of new traffic, which will continue to contribute to the road's deterioration.

This PER is prepared as an initial task to analyze the deficiencies of the roadway. By evaluating the road's structural and geometric deficiencies or needs, and obtaining an initial snapshot of what improvements are necessary to meet or exceed County road standards, Lewis and Clark County can then better identify funding requirements, and begin subsequent planning for engineering and construction.

In accordance with Chapter XI of the current December 18, 2007 Lewis and Clark County Subdivision Regulations (Amended May 18, 2010), Part H Streets and Roads, the County will also utilize this document to calculate the pro-rata cost share of each new development that contributes traffic impacts to this study segment as a part of its impact corridor. The pro-rata share for each impact will then be reserved to help build the funding needed in part to ultimately reconstruct the roadway as a whole or in separate phases.

RPA has prepared this report with services rendered to meet or exceed those of the practicing consulting engineering industry under similar budget conditions. No warranty, expressed or implied, is made.

1.1. Location and Description

Applegate Drive lies within the northern portion of what is locally known as the Helena Valley. The study area begins at the intersection of Lincoln Road. The project corridor extends northerly for approximately 3.0 miles ending just north of the intersection with Brownstone Road. For the purpose of this study, Milepost [MP] 0.00 is considered as the start of the project corridor at the intersection with Lincoln Road. The mileposts increase in a south to north direction. From Milepost 0.00, Applegate Drive continues due north along the section lines. Refer to the project area map, **Figure 1.1**.

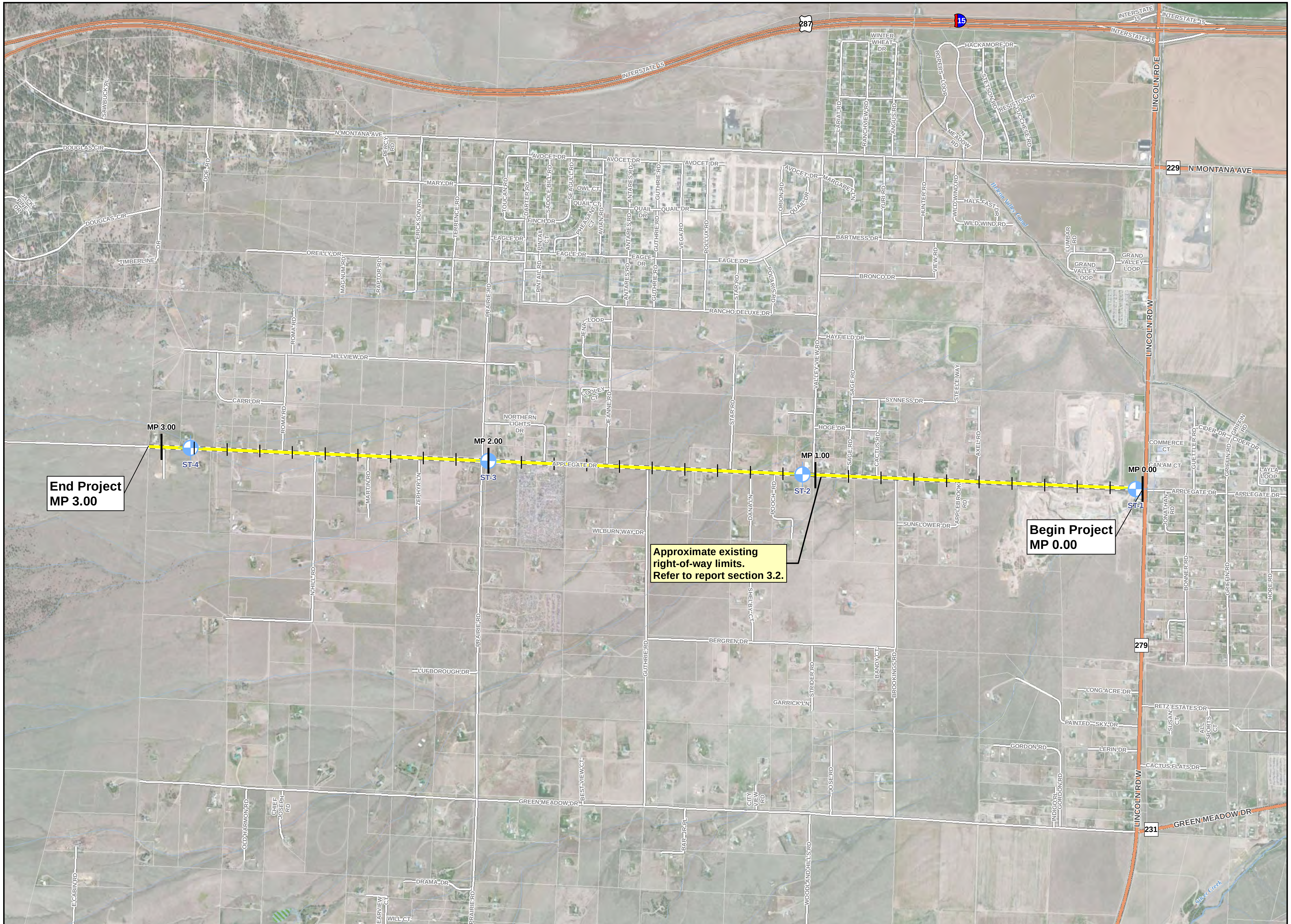
1.2. Methodology to Develop Report

Various field methods were used to obtain existing geometric information to aid in the development of this report. The work conducted is indicative of the preliminary nature of this project's current status and level of design and development. Explicitly, formal survey work of setting control and then completing instrumental topographical survey was not completed. As such, CADD based design work has not been undertaken, except for some basic diagramming.

Field reviews were completed in March 2011. For on-site field reviews, most measurements were taken with a steel tape. Longer measurements were obtained using a wheel tape. For slope or grade estimates, a four-foot long digital smart level was used to record the information in degrees or percent format. This then was converted to approximate slope rates, such as horizontal:vertical (h:v) for describing existing road fill or cut slope rates to compare to design guidelines expressed in that format. For longer measurements, such as checking sight distances, a hand-held laser rangefinder was used. GIS information was used to supplement the field data collection effort as well as minimizing walking and windshield review time.

1.3. Reference Standards

The reference standards used in this study are those specified by the Lewis and Clark County Subdivision Regulations. Specifically, in the regulation's Appendix J, Road Standards, reference documents include American Association of State Highway and Transportation Officials (AASHTO) and Montana Department of Transportation (MDT) publications amongst others. These standards were followed, with the County standards governing all others if design information is provided for the specific item being evaluated. If we deemed it appropriate to use other reference materials, then those materials are documented in this report.



APPLEGATE DR PROJECT AREA

Preliminary Engineering Report

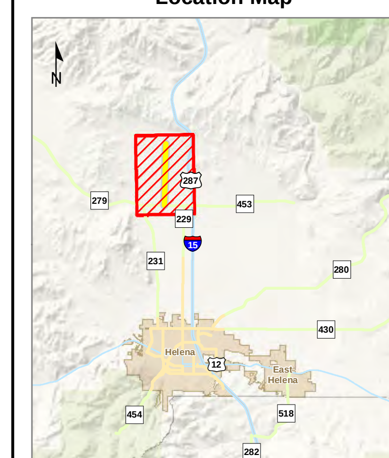
500 0 500 1,000 1,500 Feet

1 inch = 1,500 feet

Map Legend

- Approximate Existing Right-of-Way
- Soil Boring Location
- Local Route
- On-System Route
- Ownership
- City Boundary
- Wetland
- Waterbody
- Canal / Ditch
- Stream / River - Intermittent
- Stream / River - Perennial

Location Map



Map Created by:
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Project: 11502.000 Applegate Road PER

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File Location: F:\highways\11502_000_Applegate_Road_PER\GIS\Maps\ApplegateDrive_ProjectArea.mxd

Figure 1.1

2. Background Data

Background data was collected for the project corridor from various sources and was used to supplement the field data collection efforts discussed later in this report. The background data was used in conjunction with the field collected data to help establish baseline conditions and to assess potential problem areas. This section of the report provides a summary and analysis of the available background data.

2.1. Traffic

Lewis and Clark County completes annual traffic counts for roads under their jurisdiction. The County recognizes the importance of methodically collecting traffic data to analyze traffic growth characteristics and help assess each road's maintenance needs.

Abelin Traffic Services (ATS) of Helena has in the recent years been contracted with the County to complete their Traffic Count Program. 2009 traffic counts for segments of this road study were completed by ATS in August 2009. ATS converts the raw data traffic counts into Average Annual Daily Traffic (AADT) to provide an accurate traffic volume regardless of which month, day or hours the counts were performed. The 2009 traffic data was the most current available data posted on the Lewis and Clark County website during the development of this report.

In addition to existing counts, Lewis and Clark County also provided RPA with the historical traffic counts for Applegate Drive. The AADT counts provide historic data used to develop a baseline of information to characterize traffic growth. RPA plotted the historical counts to assess the annual growth rate. An exponential growth trend line was established to represent historic traffic conditions and to project out to a future 20-year evaluation period to year 2031. The historic traffic counts, as well as the trend line evaluation, are included in **Appendix A** of this report.

2011 AADT values, along with projected 2031 values, were estimated using the exponential growth trend calculated based on the historical traffic data discussed previously. In addition to showing existing and projected AADT traffic values, **Table 2.1** gives the estimated exponential growth rates experienced along each road segment based on the linear trend analysis. A weighted average growth rate combining all traffic count locations along the project corridor is also provided in the table.

For the purpose of this study, a 3.0% heavy vehicle factor was assumed for Applegate Drive north of Brookings Road and a 5.0% heavy vehicle factor was assumed for the section south of Brookings Road. The heavy vehicle factors are conservative estimates based on field reviews and values for similar type roads. These values were used to complete a road surfacing evaluation as a part of this PER. A more thorough discussion of the surfacing design procedures is contained in **Appendix C**.

Table 2.1: Average Annual Daily Traffic (AADT)

Applegate Drive		AADT			
Site ID	**Location**	**2009**	**2011[1]**	**2031[1]**	**Growth[2]**
7A-43	N. of Prairie Rd	309	300	756	4.72%
7A-45	S. of Prairie Rd	313	378	1744	7.94%
7A-46	N. of Valley View	581	647	1608	4.66%
7A-47	S. of Brookings Rd	877	1051	2763	4.95%
7A-48	N. of Lincoln Rd	1240	1175	3035	4.86%
Weighted Average:					**5.17%**

*(1) AADT was projected based on 20-year historical counts using an exponential yearly growth rate of historical data (**Appendix A**).*
(2) Estimated exponential growth rate based on historical traffic count data.

2.2. Crash History

The MDT Traffic and Safety Bureau provided crash information and data for the approximate 3.0 mile section of Applegate Drive north of Lincoln Road (S-279). The crash information covers a 5-year time period from January 1, 2006 to December 31, 2010. A total of eighteen crashes were investigated on this segment of roadway. The crash information was analyzed to identify general crash characteristics and trends.

The crash analysis shows crash clusters occurring at four general locations. The first cluster occurred at the intersection with Lincoln Road. At this location five crashes were reported, four of which involved multiple vehicles. Of the five, three resulted in injuries with no fatalities. The single-vehicle crash involved a pedestrian.

The second identified crash cluster occurred near the Valley Excavating, Sand & Gravel entrances. There were three reported crashes in and around the business entrances during the analysis period. Of these three crashes, only one involved multiple vehicles while no injuries were reported.

The third identified crash cluster occurred near the intersection with Valley View Road. Three crashes were reported at this location, only one of which involved multiple vehicles. One crash resulted in an injury, and one crash resulted in a fatality. The crash that resulted in a fatality was a single vehicle overturn accident.

The fourth identified crash cluster occurred near the intersection with Prairie Road. Four crashes occurred at this location, two of which involved multiple vehicles. No injuries were reported for these crashes.

In total, eighteen crashes were reported for the 3.0-mile segment of Applegate Drive. Ten of the eighteen crashes involved multiple vehicles. The single-vehicle crashes generally appear to be the result of driver error and do not indicate a specific road deficiency. Six injuries and one fatality were reported during the crash analysis period.

3. Existing Conditions

Existing conditions for the Applegate Drive corridor are based on background data and a field review conducted on March 10th, 2011. During the field review, existing physical characteristics were analyzed and recorded to help establish existing conditions along the project corridor. Weather conditions were favorable during the field review.

3.1. Physical Characteristics

Design criteria for assessing proposed roadway improvements are in some part governed by the terrain that the roadway traverses. Terrain classifications are level, rolling and mountainous. The terrain of this roadway is generally level from the beginning of the project (MP 0.00) northerly to approximately the intersection with Prairie Road (MP 2.00). Road grades for this segment vary slightly from 0.5% at the project beginning to approximately 1.5% at the intersection with Prairie Road. North of Prairie Road, the terrain is generally rolling with the grade rising on a steeper slope as the roadway climbs out of the Helena valley. Road grades along this section of roadway start at 1.5% on the southern end (MP 2.00) and climb to approximately 5.0% at the end of project (MP 3.00).

The area is a mix of irrigated and dry land agricultural tracts between parcels of developed suburban residential subdivisions. According to the Greater Helena Area Transportation Plan – 2004 Update, Applegate Drive is functionally classified a Minor Collector between Lincoln Road (MP 0.00 and Valley View (MP 1.00) and as a Local Road north of Valley View. However, the future traffic projections discussed previously indicate that Applegate Drive will function as a Minor Collector between Lincoln Road and Prairie Road (MP 0.00 – MP 2.00) and as a Local Road between Prairie Road and the end of project (MP 2.00 – MP 3.00).

A Minor Collector serves to collect traffic from abutting properties via local road intersections, and distribute to other roads of equal or higher classification. A Local Road provides direct access to abutting lands and provides connections to higher classification systems. For the purposes of this report, Applegate Drive is considered to be a Minor Collector from MP 0.00 to MP 2.00 and as a Local Road from MP 2.00 to MP 3.00 to take into considerations a design that would reflect future need.

3.2. Existing Right-of-Way

Existing right-of-way was determined based on field review and GIS data. During the field review, measurements were taken where right-of-way fence exists. This information supplemented available Cadastral GIS data. Based on the field review and existing data, the right-of-way is approximately 60 feet wide for the entire project corridor. According to Lewis and Clark County standards, the minimum right-of-way width is 80 feet for a Minor Collector and 60 feet for a Local Road. It should be noted that the existing right-of-way values are estimations and are only intended to provide a planning-level assessment to help determine potential roadway reconstruction costs and impacts that may occur due to reconstruction to widen the road template beyond existing right-of-way.

3.3. Design Speed

Design speed is a selected speed used to determine multiple aspects of roadway design criteria. Design speed is selected in relation to topography, vehicle operating speeds, roadside development, and the functional classification of the road or highway. The American Association of State Highway and Transportation Officials (AASHTO) publication "A Policy on Geometric Design of Highways and Streets - 2004" (the Green Book as commonly referred to by the industry) states that the selection of the design speed for roads other than constrained local streets, should be made to use the speed that is the highest practical to attain the desired degree of safety, mobility, and efficiency subject to environmental, economic and other social, political or aesthetic constraints. Further, the design speed should be higher than the running speed of a large proportion of drivers.

As noted previously, based on future projected traffic levels the functional classification of this road is a Minor Collector from MP 0.00 to MP 2.00 and a Local Road from MP 2.00 to MP 3.00. Appendix J, Table A, Road Standards, of the Lewis and Clark County Subdivision Regulations specifies a design speed of 50 mph for a Minor Collector traversing level terrain and 30 mph for a Local Road traversing level terrain. A copy of Table A is included in **Appendix B**.

Exhibit 6-1 of the AASHTO Green Book, reproduced in **Appendix B**, is a table of suggested minimum design speeds for Rural Collectors. We referred to this as a matter of comparison to County design speeds previously discussed. For over 2000 vehicles per day (vpd), AASHTO's minimum design speeds are 60 mph for level terrain; for 400 to 2000 vpd, AASHTO's minimum design speeds are 50 mph for level terrain and 40 mph for rolling terrain. Exhibit 5-1 of the AASHTO Green Book, also reproduced in **Appendix B**, provides suggested minimum design speeds for Local Rural Roads. For 400 to 1500 vpd, AASHTO's minimum design speeds are 50 mph for level terrain and 40 mph for rolling terrain. AASHTO guidance states that designs should exceed their criteria where practical. Every effort should be made to obtain the best possible alignment, grade, sight distance, and improved road cross-sectional elements that are consistent with terrain, present and anticipated development, safety and available funds.

Exhibit 6-4 of the Green Book, contained in **Appendix B**, specifies maximum suggested grades, in percent (%), for specified design speeds of Rural Collector highways. For 50 mph design speeds, level terrain can have a highway grade of up to 6%. For 60 mph in the same terrain, the maximum grade is 5%. Exhibit 5-4 of the Green Book specifies maximum suggested grades for Local Rural Roads. For 50 mph design speeds, a maximum grade of 6% is recommended for level terrain. For the project corridor, there are no existing grades exceeding those recommended in the Green Book based on the terrain criteria.

The County has established a regulatory speed limit of 35 mph for the project corridor. The regulatory speed is less than the County standard design speeds for a Minor Collector, and higher than the County standard design speeds for a Local Road. The current regulatory speed limit is deemed appropriate by the County based on terrain, the road's surfacing condition, geometrics, and level and proximity of roadside development.

Based on the above comparisons, we believe that a design speed of 50 mph, as specified by the County for a Minor Collector, is appropriate for the project corridor. The design speed is higher than the current regulatory speed, which is indicative of improving conditions to those of highest practical to attain the desired degree of safety, mobility, and efficiency subject to environmental, economic and other social, political or aesthetic constraints. The 50 mph design speed was also used for the Local Road portion of the corridor for continuity and as a conservative estimate. It should be noted that the County does not intend to change the regulatory speed limit of 35 mph for the project corridor at this time. However, if the road surfacing is improved to a paved condition, then conditions would presumably be more favorable for safely increasing the regulatory speed limit.

3.4. Alignment

The horizontal road alignment of Applegate Drive is tangential in a north/south direction. The tangent sections of the road are primarily a result of the road following the section lines. There are no horizontal curves along the project corridor. The vertical alignment of Applegate Drive is generally flat with grades much less than those identified as maximum allowable in the County road regulations.

Notwithstanding other geometric features related to the alignment, no substantial adjustments to the horizontal and vertical alignments are expected when this road's design for reconstruction is to be undertaken.

3.5. Sight Distance

Applicable to horizontal and vertical alignment geometric features is the design element of sight distance. The measure of a driver's sight distance is critical to safely avoid collisions with objects. This is measured by stopping sight distance in both horizontal and vertical planes. In addition, to promote efficiency of the road facility relative to its functional classification, an amount of passing sight distance for drivers to enter the opposing lane to pass vehicles is desired.

As noted previously, the roadway lies on straight tangent sections for the entire project length. There do not appear to be any issues related to sight distance along vertical or horizontal curves. Therefore we do not envision any substantial improvements to be required to the present road grade and its associated sight distance.

3.6. Existing Roadway Surfacing

A surfacing evaluation for the Applegate Drive corridor was initiated in April 2011 with field work, soil borings, and laboratory analysis. The evaluation concluded with a surfacing design and evaluation report completed in May, 2011. A detailed surfacing evaluation report is contained in **Appendix C.** This section provides a summary of the findings of the pavement evaluation.

The Applegate Drive corridor is gravel surfaced throughout the entire project length. Four soil borings were completed along this section. The borings, identified as ST-1, ST-2, ST-3, and ST-4 were completed in approximately one-mile intervals. The thickness of the gravel surfacing varies between the four samples from 0 inches to 9 inches. All four samples indicate good quality existing fill material. The existing clayey sand fill thickness varies from 15 inches to 54 inches.

With each boring, soil samples were also obtained for subgrade material. The subgrade soil consists of silty gravel at three locations, and clayey gravel at the fourth. The moisture content is considered to below to near optimum at three locations, and below optimum at the other location. The risk of subgrade failure at all four locations is considered to be low. **Table 3.1** gives a summary of the surfacing evaluation soil boring results.

Table 3.1: Summary of Soil Boring Conditions

	ST-1	**ST-2**	**ST-3**	**ST-4**
Approximate Location	MP 0.00	MP 1.00	MP 2.00	MP 3.00
Existing Gravel Surfacing	0"	8"	9"	0"
Existing Clayey Sand Fill Thickness	54"	22"	15"	30"
Existing Fill Quality	Good	Good	Good	Good
Subgrade	Silty Gravel	Silty Gravel	Clayey Gravel	Silty Gravel
BPF in Clayey Sand Fill	42, 41	24, 18	24	20
Moisture Condition	Below to near	Below to near	Below	Below to near
Risk of Subgrade Failure	Low	Low	Low	Low

<u>Summary</u>:

- The existing gravel surfacing ranges in thickness from 0 inches to 9 inches;
- Existing fill quality is good and ranges in thickness from 15 inches to 54 inches;
- The subgrade has a low risk of failure.
- A bag sample of base course material sampled at 0 to 8 inches deep at boring ST-2 indicates the existing base course does not meet County gradation or Plasticity Index specifications.

3.7. Existing Roadway Typical Section

This section of the report discusses the primary features of each road segment's existing typical section characteristics. The project corridor is comprised of two distinct sections as discussed below. Cross-sectional measurements of Applegate Drive were taken to include surfacing widths, cut and fill slope rates, ditch widths and depth of the roadside ditch.

3.7.1. Existing Typical Section E.1: MP 0.00 to MP 0.25

Existing Typical Section E.1 runs from Lincoln Road (MP 0.00) to MP 0.25. The overall top surface of this section measured to be approximately 32 feet wide, with two 12-foot travel lanes and 4-foot shoulders on each side. The roadside ditch foreslopes were measured to be approximate 4:1 (horizontal : vertical, i.e. four feet horizontal distance for each one foot vertical drop) on each side of the roadway. The ditch backslopes were measured to be approximately 6:1 on each side. The roadside ditch depths were approximately 18 inches deep on each side and therefore do not meet current county standards for ditch depth.

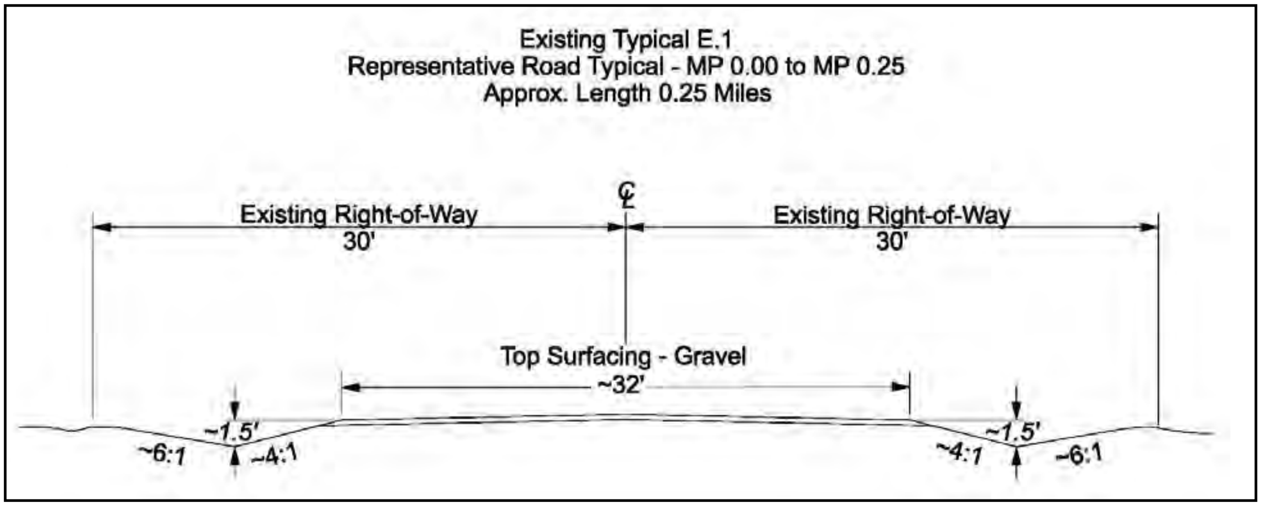


Figure 3.1: Existing Typical Section E.1 (MP 0.00 – MP 0.25) – Looking North

Photo 3.1: Existing Typical Section E.1 looking south.

3.7.2. Existing Typical Section E.2: MP 0.25 to MP 3.00

Existing Typical Section E.2 runs from MP 0.25 to MP 3.00. The overall top surface of this section measured to be approximately 24 feet wide, with two 12-foot travel lanes. There were no discernable shoulders along this section of roadway. The roadside ditch foreslopes were measured to be approximate 4:1 on each side of the roadway. The ditch backslopes were measured to be approximately 6:1 on each side. The roadside ditch depths were approximately 24 inches deep on each side and therefore do not meet current county standards for depth.

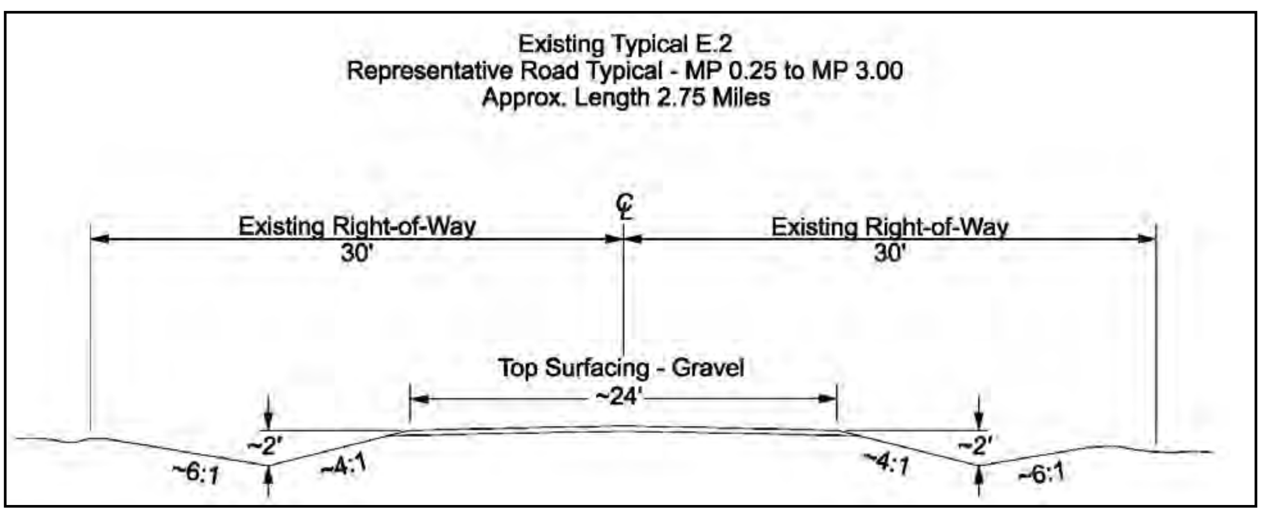


Figure 3.2: Existing Typical Section E.2 (MP 0.25 - MP 3.00) – Looking North

Photo 3.2: Existing Typical Section E.2 looking north.

4. Proposed Conditions

This section discusses the proposed future conditions of the Applegate Drive corridor. Proposed conditions are determined based on existing Lewis and Clark County standards as well as information collected during the field review process.

4.1. Proposed Roadway Typical Sections

The proposed design typical sections are based on the design methodology previously discussed herein. The County Road Standards serve as the basis which was supplemented by AASHTO guidance as needed. The following sections provide detail as to how the proposed typical sections are developed.

4.1.1. Preliminary Surfacing Design

For this study, a preliminary surfacing section was developed based on the four soil borings and projected traffic data. This pavement design is used within this study to estimate reconstruction impacts and costs. As such, the preliminary surfacing design is developed to also meet or exceed the surfacing requirements of the Lewis and Clark County Road Regulations.

Based on the input parameters such as soil conditions and projected traffic conditions, and the approach of analyzing the pavement designs to be in accordance with the County Subdivision Regulations, the recommended reconstruction should have a new pavement section meeting or exceeding the structural integrity as shown in **Table 4.1** below (refer to **Appendix C** for the full pavement design evaluation).

Table 4.1: Recommended Pavement Section

Item	Pavement Section 1	Pavement Section 2
Approximate Location	MP 0.00 - MP 0.75	MP 0.75 - MP 3.00
New Asphalt Pavement	3"	3"
Crushed Top Surfacing	3"	3"
Select Base Course (3-inch minus)	6"	6"
Subbase Course (3-inch minus)	4"	0"
TOTAL	**16"**	**12"**

As discussed previously, the soil borings taken along the project corridor indicated that the existing subgrade has a “low” risk of failure during construction. However, some areas may still require some subgrade stabilization. A discussion about identification of areas and methods for subgrade stabilization is contained in the surfacing evaluation in **Appendix C**.

4.1.2. Design Clear Zone

Typical crashes either involve incidents on the road, or collisions with fixed features off of the road, such as bridge piers, sign supports, overhead utility poles, culverts, and non-traversable ditches or embankments. To counteract the effects of off-road errant vehicles, agencies implement a traversable and unobstructed roadside area beyond the edge of the traveled way for higher volume, rural facilities. Obstacles within the "clear zone" are evaluated to be removed, relocated, redesigned or shielded. The basic parameters to establish the appropriate design clear zone is the road's design speed, design traffic volume, and design roadside cut and fill slope rates.

Lewis and Clark County Road Standards references roadside clear zone requirements to those recommended by AASHTO. A portion of Table 3.1 of the AASHTO 2006 Roadside Design Guide is reproduced in **Table 4.2**. This shows the recommended clear zones based on the design speed and traffic volume parameters for Applegate Drive. The clear zones shown below are measured in feet from the edge of the traveled way.

Table 4.2: Roadside Clear Zone Recommendations (Feet)

		Foreslopes			Backslopes		
Design Speed	Design ADT	6H:1V or Flatter	5H:1V to 4H:1V	3H:1V	3H:1V	5H:1V to 4H:1V	6H:1V or Flatter
45 - 50 mph	Under 750	10 - 12	12 - 14	-	8 - 10	10 - 12	10 - 12
45 - 50 mph	750 - 1500	14 - 16	16 - 20	-	10 - 12	12 - 14	14 - 16
45 - 50 mph	1500 - 6000	16 - 18	20 - 26	-	12 - 14	14 - 16	16 - 18

Based on the values shown in the table above, a minimum clear zone of 20 feet is desired along the roadside foreslope for areas with a design ADT of 1500 to 6000 based on County standard 4:1 minimum foreslope rates. This applies to the section of Applegate Drive between Lincoln Road (MP 0.00) and Prairie Drive (MP 2.00). Similarly, a clear zone of 12 to 14 feet is desired for areas with under 750 vpd; this applies to the section of Applegate Drive between Prairie Drive (MP 2.00) and the end of project (MP 3.00) if 4:1 foreslopes are constructed. Use of an appropriate clear zone distance amounts to a compromise between maximizing safety and minimizing construction costs.

Pursuant to County standards, the 50 mph design speed is applicable to Applegate Drive traversing level terrain. A minimum foreslope rate of 4:1 for a Minor Collector, and 3:1 for a Local Road, is required as shown in Figure 3 and 2, respectively, of Appendix J of the County's Subdivision Regulations. It should be noted that the Roadside Design Guide recommends a minimum foreslope rate of 4:1. This differs from the County standards which call for a minimum 3:1 foreslope for a paved Local Road. This does not make 3:1 rates wrong, but instead allows the County to utilize their minimum standard to work well with multiple roadside constraints. Currently, foreslope rates for the Local Road portion of Applegate Drive are at 4:1 so the proposed design will continue to utilize this flatter slope which maximizes roadside safety within appropriate allowances. County standards also require a minimum ditch depth of 36 inches. In order to achieve this depth, a ditch width of at least 12 feet is needed assuming a 4:1

foreslope. The 12-foot ditch width, combined with a 2-foot paved shoulder, create a 14-foot clear zone for the Local Road typical section, thereby meeting AASHTO safety recommendations for the road segment carrying less that 750 ADT.

Based on these factors, we utilized a 4:1 minimum foreslope rate for the entire project corridor. For the purposes of this study, we are applying a 20-foot clear zone for the Minor Collector typical section (MP 0.00 – MP 2.00) and a 14-foot clear zone for the paved Local Road section (MP 2.00 – MP 3.00).

4.1.3. Surfacing Width

Appendix J of Lewis and Clark County's Subdivision Regulations depict the County's minimum roadway standards. For a Minor Collector, each travel lane is to be 12-feet wide. The shoulder width can vary between 2 feet and 4 feet, as measured between the edge of the travel lane to the edge of the surfacing. For a Local Road, minimum County standards call for 10-foot travel lanes with 2-foot shoulders on each side.

In addition to the County standards, we referred to the AASHTO Green Book for guidance and comparison. Exhibit 6-5 of the Green Book (included in **Appendix B**) specifies that the minimum traveled way for a Rural Collector with a design speed of 50 mph is 22 feet for 1500 – 2000 vpd and 24 feet for over 2000 vpd. The exhibit also recommends a shoulder width on each side of 6 feet for 1500 – 2000 vpd and 8 feet for over 2000 vpd. Exhibit 5-5 of the Green Book (also included in **Appendix B**) specifies that for a Local Road with 400 to 1500 vpd, the minimum recommended width of traveled way is 22 feet for a 50 mph design speed with 5-foot shoulders on each side.

Based on the Green Book recommendations they are substantially greater than County standards (i.e. 6 to 8 foot shoulders on each side). In conforming to County standards in which roadside development and construction cost is a factor, a recommended overall road surfacing width for reconstruction to accommodate two travel lanes and shoulders is 32 feet for the Minor Collector portion of the corridor. This surfacing width accounts for two 12-foot travel lanes and two 4-foot shoulders. That is, the shoulder widths are to County standards but less than AASHTO recommendations. For the Local Road portion of the corridor, a surfacing width of 24 feet was utilized which accounts for two 10-foot travel lanes and two 2-foot shoulders.

4.1.4. Proposed Typical Section P.1

Proposed Typical Section P.1 (**Figure 4.1**) is for the portion of Applegate Drive from Lincoln Road (MP 0.00) to Brookings Road (MP 0.75).This proposed typical section meets minimum Minor Collector standards as defined by the County. Projected future traffic forecasts along this section call for approximately 3000 AADT in 2031 with approximately 5% heavy vehicles. Based on the discussion provided in **Section 4.1.2**, a minimum clear zone of 20 feet is recommended. The proposed typical section would require approximately 10 feet of additional right-of-way along both sides of the roadway. For planning purposes for future need, the 10 feet of additional right-of-way on both sides would convert the current corridor of 60 feet right-of-way to 80 feet which would then meet current minimum County standards. It should be noted that a 16-inch thick surfacing section is recommended for this portion of the project corridor as discussed previously in **Section 4.1.1**.

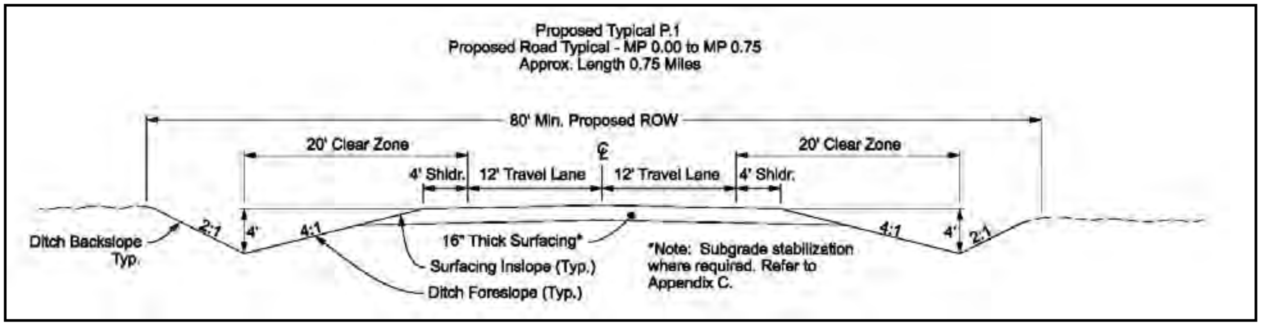


Figure 4.1: Proposed Typical Section P.1 (MP 0.00 - MP 0.75) – Looking North

4.1.5. Proposed Typical Section P.2

Proposed Typical Section P.2 (**Figure 4.2**) was developed for the portion of Applegate Drive from Brookings Road (MP 0.75) to Prairie Road (MP 2.00).This proposed typical section meets minimum Minor Collector standards as defined by the County.

Projected future AADT along this section is expected to be approximately 1750 vpd with approximately 3% heavy vehicles. As with Typical Section P.1, a minimum clear zone of 20 feet is recommended. The proposed typical section would require approximately 10 feet of additional right-of-way along both sides of the roadway. This recommended typical section requires a 12-inch surfacing section as described previously in **Section 4.1.1**.

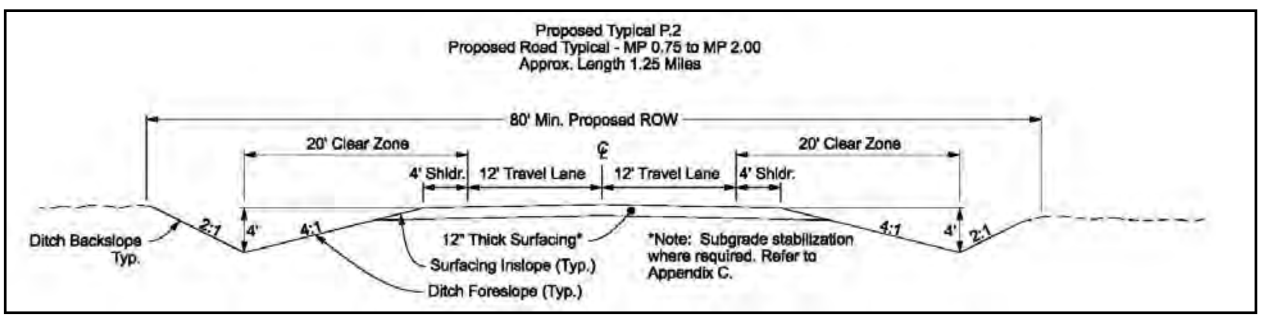


Figure 4.2: Proposed Typical Section P.2 (MP 0.75 - MP 2.00)

4.1.6. Proposed Typical Section P.3

Proposed Typical Section P.3 is shown below in **Figure 4.3**. This section was developed for the portion of Applegate Drive between from Prairie Road (MP 2.00) to the end of project (MP 3.00). This proposed typical section meets minimum paved Local Road standards as defined by the County.

Projected future AADT along this section is expected to be approximately 750 vpd with approximately 3% heavy vehicles. This typical section requires a minimum clear zone width of 14 feet. The proposed typical section would not require any additional right-of-way along either side of the roadway. This recommended typical section requires a 12-inch surfacing section as described previously in **Section 4.1.1**.

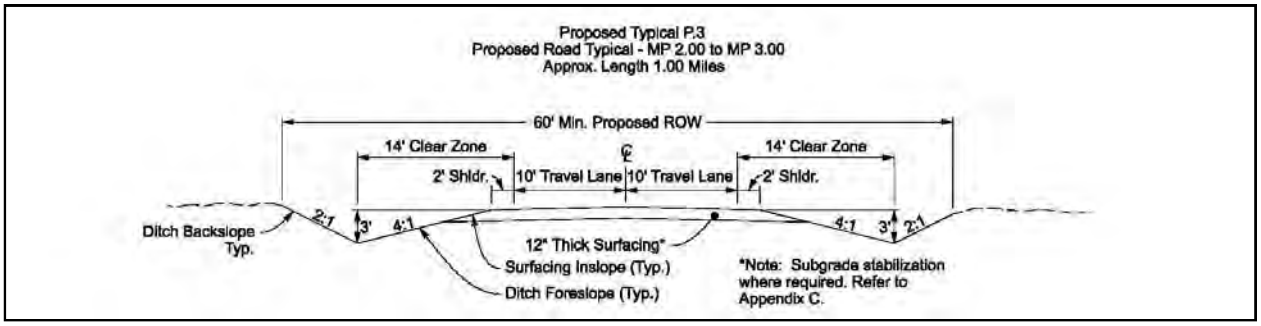


Figure 4.3: Proposed Typical Section P.3 (MP 2.00 - MP 3.00)

4.1.7. Miscellaneous Grading, Cut and Fill Slopes

To estimate earthwork and miscellaneous other feature impacts to reconstruct the roadway in level terrain, we applied the design typical sections, shown in **Figures 4.1** through **4.3** over the existing road templates estimated from field measurements, **Figures 3.1** through **3.2**. The estimate is based on proposed roadway centerlines following existing centerlines. The superimposed typical sections are shown in **Figure 4.4**.

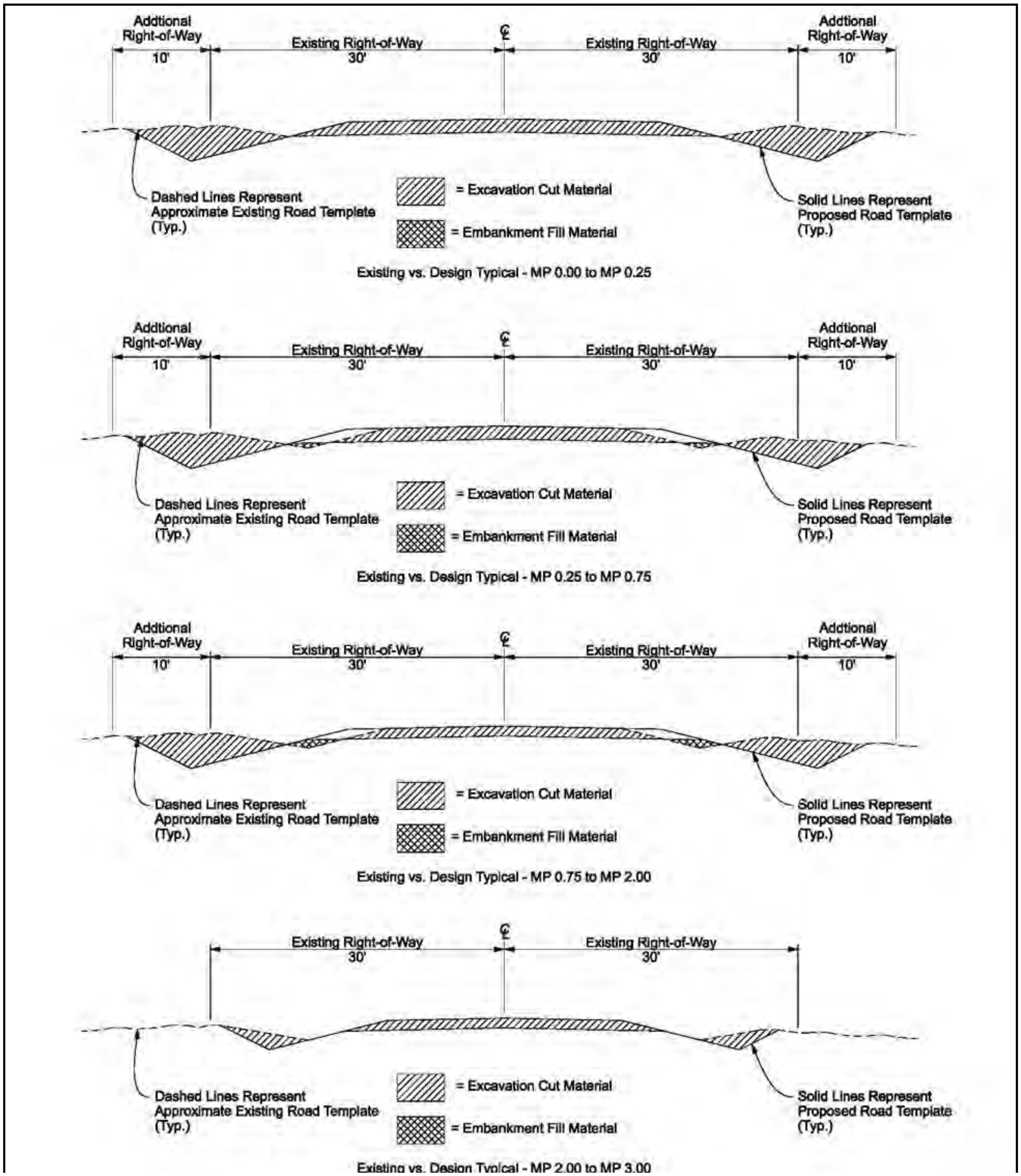


Figure 4.4: Estimated Reconstruction Cut / Fill Impacts

4.1.8. Geotechnical Considerations

Geotechnical evaluations were not undertaken other than the soil borings and laboratory analysis needed to develop a preliminary pavement design. When further design engineering is undertaken in subsequent tasks to develop the roadway reconstruction project(s), additional geotechnical engineering is recommended to confirm such items as subgrade stabilization needs, limits and techniques.

During the course of developing the pavement designs, all four borings completed along the project corridor indicated “good” quality existing base/subbase. The geotechnical engineer evaluated these locations to have “low” risks of subgrade failure during construction. The low risk assessment was based on the fact that the borings encountered clayey sand subgrade that was below or near optimum moisture content. The preliminary indications therefore are that subgrade stabilization is not anticipated. However, construction during unseasonably wet conditions could dictate otherwise.

4.2. Property Values

Previously in this report, we estimated the existing road right-of-way widths based on field review and GIS data. The section of the report addresses how land valuations were estimated.

The predominant land use along this study segment is currently residential or agricultural. We presume the highest and best use of the current agricultural property is that to be developed into a residential subdivision.

To assign fully defendable and accountable costs to right-of-way impacts is outside the scope of this document. To do so would require the preparation of multiple appraisals. By virtue of the amount of parcels adjoining this road’s right-of-way, the appraiser fee to complete this work could amount to several thousand dollars based on industry rates. Instead, to obtain a reasonable estimate of right-of-way acquisition costs, we utilized rates contained in the Lake Helena Drive PER completed in December 2009. These rates were based on the brief research of a local appraiser for recent comparable sales in the Helena Valley for similar size parcels.

In his brief research, the appraiser found that residential tracts of 1- 5 acres sold for $18,000 to $40,000 per acre for similar properties in mixed- use areas with no zoning. For this estimate, we are basing all costs on a per acre basis with no impacts to property improvements such as landscaping, fencing, lawn, sprinkler irrigation, wells, septic drain fields, etc. With that, it is likely that actual acquisition costs could be substantially higher should residential developments be impacted.

Based on the above, we assumed for this estimate that the cost to acquire land for right-of-way from a parcel to be about $32,000 per acre. To acquire the necessary right-of-way, the property must first be appraised. We estimate the appraiser fees for researching comparable sales history, preparing the property valuations, and obtaining title evidence will cost approximately $2,000 per parcel. An assigned land acquisition agent would then use the appraisals to negotiate and procure the necessary right-of-way. We assigned a cost of $1,500 per parcel for the fees that would be charged by a right-of-way acquisition agent. We used web-based information to estimate the number of properties impacted per segment of road. Overall, we estimate that approximately 37 properties could be impacted in order to acquire additional right-of-way during the course of reconstructing 3.0 miles of this road.

4.3. Drainage and Hydraulics

4.3.1. Mainline Cross Drains

The project corridor traverses level terrain following the direction of the north-to-south natural drainage patterns. One existing mainline cross drain was identified during the field review. The cross drain is located approximately at MP 1.44 and appears to serve an existing intermittent stream based on a GIS data review. The existing diameter of this drain is 24 inches.

A review of Flood Insurance Rate Maps (FIRM) and GIS data indicates that there are additional intermittent streams crossing the project corridor. No cross drains were observed in the field at these locations, however. In addition, no flood plains were identified along the project corridor based on FIRM number 3000381415C. Based on this review, and as a conservative approach, it was assumed that a 24-inch diameter culvert would be placed at each intermittent stream location.

The project corridor appears to require very little drainage upgrading other than that discussed previously and development of adequate roadside ditches. Runoff picked up in this area is conveyed primarily along the roadside, crossing under roads that intersect Applegate Drive by the means of small-diameter approach drains. As previously discussed, the roadside ditches in this segment are very shallow with issues of not having adequate cover between the top of the pipe and the approach surfacing. Widening the roadside ditch in this area will provide not only an improved clear recovery area for motorists, but will also increase the ditch depth to allow for improved installation of culverts and ditch flow capacity. Culverts with adequate depth of cover will experience less structural damage from vehicles crossing over the culvert, and lessen crushing the ends of the pipes due to running over the inlets and outlets while turning in or out of approaches.

4.3.2. Approach Culverts

As noted previously, the terrain that runs north-to-south parallel to the road governs much of this road's drainage characteristic. As such, approach culverts play an important role. Improving the roadside ditches as a part of the reconstruction effort will allow for both an increased ditch capacity, and upsizing small diameter culverts as needed while still providing adequate structural cover. For the purposes of this preliminary study, we estimated the number of new approach pipes needed based on a limited windshield review of quantifying the number of approaches within each road segment. The windshield review was supplemented by review of aerial photography and GIS data. We presume that most culverts will require replacement due to abundance of crushed ends and other defects observed at approaches. The lengths of new approach culverts were estimated by applying a road approach width of 24 feet, with additional inlet and outlet lengths calculated based on ditch elevation and slope.

4.3.3. Drainage Summary

Existing culverts that were observed in the field review are included with the assumption that these will require replacement due to modified construction limits. In addition, a nominal amount of new approach culverts will likely be necessary based on the unusable condition for many pipes observed in the field.

Due to the scope of this report, the majority of notable crossings were inspected, but a substantial amount of review was also “windshield.” In addition, FIRMs were reviewed to determine if there were existing floodplains along the project corridor. **Table 4.2** below summarizes hydraulic conveyance features within the study area.

Table 4.3: Existing Cross Drain Summary

	Existing		Replacement		
Location	Diameter	Length	Diameter	Length	Comments
MP 0.90	-	-	24"	56'	Intermittent Stream
MP 1.20	-	-	24"	56'	Spring Gulch - Intermittent Stream
MP 1.44	24"	50'	24"	56'	Intermittent Stream
MP 1.90	-	-	24"	56'	Intermittent Stream
MP 2.50	-	-	24"	56'	Intermittent Stream
MP 2.90	-	-	24"	56'	Intermittent Stream

4.4. Pedestrian and Bicycle Facilities

There are currently no facilities to accommodate pedestrians or bicyclists within this corridor. As such under this study, no costs are being attributed to constructing a shared-use bicycle/pedestrian path as part of the base cost of rebuilding the road. However, an alternative cost of constructing a path on a per-mile basis is included in this report for planning purposes. The estimated cost presented later in this report is for a 10-foot wide asphalt surfaced path.

According to the Greater Helena area Transportation Plan – 2004 Update, an overriding goal for non-motorized transportation in the greater Helena Area is:

> *To develop a living plan for the Greater Helena Area to create and maintain corridors for cyclists and other non-motorized modes of travel and recreation that are safe and effective for their transportation and enjoyment, and to inform and educate motorists, cyclists, and pedestrians in how to safely and respectfully share our roads and other corridors as citizens transport themselves about the community.*

4.5. Auxiliary Turn Lanes

The scope of this work does not include completing definitive turn lane warrant studies at key intersections. However, when the road design is initiated, it can be reasonably ascertained that one or more turn lanes may be warranted. Therefore for the benefit of this study, we have included an estimated cost, not as a base cost, but as an alternate if needed, to construct a left-turn lane serving an approach in a non-signalized intersection. The discussion on traffic control signals follows this section. Turn lanes should be considered at each signalized intersection.

We based the estimated turn lane geometrics for a left-turn lane on the guidelines presented by MDT in their Traffic Engineering Manual. We assume that the shoulder widths in the location of a turn lane will be maintained at 4-feet wide. Using 50 mph design speed criteria, the lane shift bay taper rate will be 50:1 to shift the through lanes outward. An interior bay taper rate of 10:1 is used for vehicles entering the left turn lane. From the left turn bay entry, the recommended deceleration distance is 435 feet. The deceleration is assumed to initiate at the beginning of the left turn bay taper. Since intersection turning movement counts have not been completed as a part of this study, we assume the storage length needed is minimal and left-turning vehicles will complete the maneuver with adequate gaps present in the opposing traffic stream without coming to a stop in most instances. Based on the above, the minimum length left turn lane will require approximately 600 feet of total length for lane shift tapers entering and exiting the left turn area, and 435 feet of auxiliary lane including its bay taper. The total length of road widening for a minimum length left turn lane would then be about 1035 feet.

4.6. Traffic Signals

A signal warrant analysis was not completed under this study. For purposes of estimating the full potential reconstruction cost of the study area, we presume that signal warrants could eventually be met to consider a signal installation, particularly at the intersection with Lincoln Road, within the design life of Applegate Drive. Therefore, an estimated cost to install signal hardware has been included later in this report.

5. Reconstruction Cost Estimates

This section summarizes the process used to develop cost estimates for the reconstruction of Applegate Drive from Lincoln Road north approximately 3 miles. For cost estimating purposes, the Applegate Drive corridor was broken out into four distinct typical sections as listed below. Each typical section has individually unique characteristics that played a role in developing the cost estimates.

- **Typical Section A** – Lincoln Road (MP 0.00) to MP 0.25
- **Typical Section B** – MP 0.25 to Brookings Road (MP 0.75)
- **Typical Section C** – Brookings Road (MP 0.75) to Prairie Road (MP 2.00)
- **Typical Section D** – Prairie Road (MP 2.00) to MP 3.00

Table 5.1 summarizes the estimated cost to reconstruct the Applegate Drive project corridor. **Appendix D** provides a detailed cost estimate consisting of a breakout of major work features, quantities, and unit costs. The following sections briefly discuss how the quantities used in the cost estimate were estimated. The estimated quantities were then multiplied by average unit costs. Average unit costs were based on values used in the Lake Helena Drive PER completed in January 2010. Those average unit costs were based on a review of the bid history of four road projects under construction in the Helena Valley at the time. These projects ranged from full road reconstructions to spot safety improvement projects. It should be noted that the County could similarly improve Applegate Drive by either several smaller spot improvements projects, or larger-length reconstructions.

Table 5.1: Reconstruction Cost Estimate

Applegate Drive	Typical A	Typical B	Typical C	Typical D	Total
Construction Subtotal	$184,022	$369,778	$893,602	$563,845	**$2,011,247**
Total Estimated Cost	$278,324	$576,488	$1,383,833	$761,191	**$2,999,835**
Length (miles)	0.25	0.50	1.25	1.00	**3.00**

5.1. Estimating Procedure

5.1.1. Grading

- The Excavation – Unclassified quantity is estimated from **Figure 4.4** by calculating the end section cut areas and multiplying by the applied length to generate a volume. Consideration is given that the figures are likely worst-case scenarios and intermittent locations will likely balance with lesser cuts and fills.
- Topsoil Salvage and Placing is calculated based on **Figure 4.4** assuming 3 inches of topsoil depth.

5.1.2. Surfacing

- The miscellaneous road surfacing quantities such as the crushed top surfacing, select base, subbase, plant mix asphalt paving, prime, and seal coat is estimated based on the recommended pavement design and the proposed surfacing widths as shown in **Figures 4.1** through **4.3**.
- A nominal amount of Traffic Gravel is included to allow for a temporary wearing course for traffic driving on the unfinished subgrade.
- Interim paint quantities are included to delineate the road centerline and shoulder lines prior to the road receiving a chip seal. Final paint quantities would then be applied after the chip seal.

5.1.3. Drainage

- The summarized length of approach pipe lengths is estimated based on the number approaches and their assumed cross-sectional characteristics such as slope rate and depth of cover. Approach top widths are estimated as being an average of 24 feet. The amount of access approaches intersecting the roadway in each applicable segment is based on GIS aerial photographs and limited windshield survey. The approach pipes would be 15-inch diameter at minimum to meet the County's requirements for a Minor Collector. Other major drainage features are listed as observed in the field. Their new installation lengths are estimated based on the dimensions generated from the proposed road templates.

5.1.4. Fencing

- It was assumed that new right-of-way fencing would be required along the entire project length. To re-fence the right-of-way, we assume using a typical 5-strand barbwire fence with metal posts.
- It was also assumed that fence panel would be needed for every 330 feet of new fence.

5.1.5. Roadside Revegetation

- Quantifying seeding, fertilizer and seedbed conditioning is based on sectional measurements taken from the finished slopes shown in **Figure 4.4**.

5.1.6. Subgrade Stabilization

- The preliminary pavement designs included with this report identifies all areas as having good quality subgrade material with low risk of failure. To not be overly conservative in cost estimating, we did not include base costs for subgrade stabilization.

5.1.7. Right-of-Way

- To estimate appraisal costs for right-of-way acquisition, we applied a $2,000 per parcel fee for an assumed 37 parcels. A similar approach is taken to estimate fees for an agent to prepare closing documents, negotiate the right-of-way, and file documents for record.
- The existing right-of-way width appears to generally be 60 feet wide along the entire project corridor. In order to accommodate the proposed typical sections, a minimum of 10 feet of additional right-of-way acquisition would be needed along each side of the roadway from Lincoln Road (MP 0.00) to Prairie Drive (MP 2.00). It appears that there is adequate existing right-of-way from Prairie Drive to the end of the project corridor (MP 3.00) to accommodate the proposed Local Road typical section.
- $32,000 per acre land valuation is used to estimate the cost to acquire land for right of way purposes. This valuation is based on limited coordination with a local appraiser whom completed a brief research of the area to obtain comparable sales history during development of the 2009 PERs. The economic situation and housing industry is assumed to be still very similar. The comparable sales research yielded transactions amounting to $18,000 to $40,000 per Acre for residential tracts from 1/4 – 4 Acres in size. In some cases, highly sought after tracts were much higher in per acre price. We apply the assumption that agricultural tracts will be negotiated by the owner at residential land values (given the opportunity to subdivide as the highest and best use), and that the cost per acre is based on all similar size parcels.

5.2. Alternate Costs

A number of additional alternative costs were included as part of the project cost estimate. These costs are separate from those developed for the roadway reconstruction. These costs are provided in the event that separate alternative features are needed from those necessary for standard roadway reconstruction. **Table 5.2** provides a summary of the additional alternative cost estimates. The following sections provide information as to how these costs were derived.

Table 5.2: Additional Alternate Costs Estimate

Major Work Feature	Unit	Unit Cost	Number of Units	Total Cost
Traffic Signal	LS	$68,000.00	1	**$68,000**
Turn Lane	LS	$100,000.00	1	**$100,000**
Sanitary Sewer Main	MI	$211,200.00	3.00	**$633,600**
Water Main	MI	$396,000.00	3.00	**$1,188,000**
Bicycle/Ped. Path Reconstruction	MI	$77,825.00	3.00	**$233,475**

5.2.1. Traffic Signal

- The estimated cost to install traffic signal hardware for one intersection is based on the bid history of components currently being installed by MDT around the Helena area.

5.2.2. Left-Turn Lane Widening

- The estimated cost to widen the roadway to install a single turn lane is based on proportion to that cost to construct the roadway with no turn lane.

5.2.3. Miscellaneous

- The estimate includes a per mile cost to install an 8-inch water main and an 8-inch sanitary sewer main for future services. The estimate is based on an installed cost of $75 per linear foot for the water main, and $40 per linear foot for the sewer main. For planning purposes, the County desires to include an estimate since installing a water main and/or sanitary sewer main would likely be cost-effective to complete at the time the roadway is being reconstructed.
- A per mile estimate is included to construct an alternate 10 foot wide shared-use bicycle/pedestrian path. The estimate uses 2-inch thick plant mix asphalt surfacing over 4 inches of crushed top surfacing aggregate base. Note that if a pathway is included, land needed for right-of-way could increase beyond the minimum 80 feet assumed by a proportional amount equal to the width of the path plus a desirable offset from the edge of the road's construction limits.

Appendix A

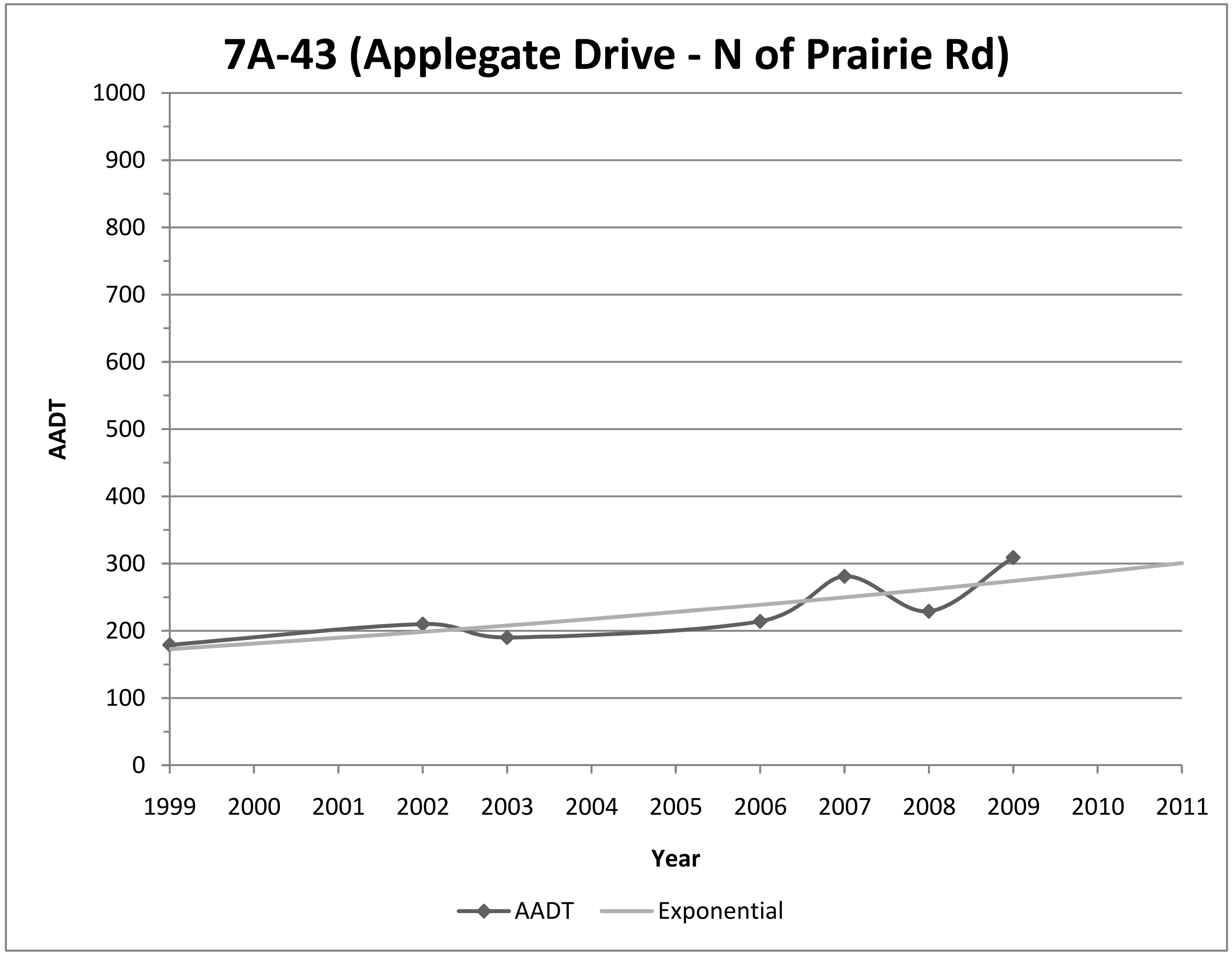
Background Data

#

7A-43 (Applegate Drive - N of Prairie Rd)

Year	AADT	Exponential
1999	179	173
2002	210	198
2003	190	208
2006	214	239
2007	281	250
2008	229	262
2009	309	274
2011	-	300
2031	-	756
i	-	**4.72%**

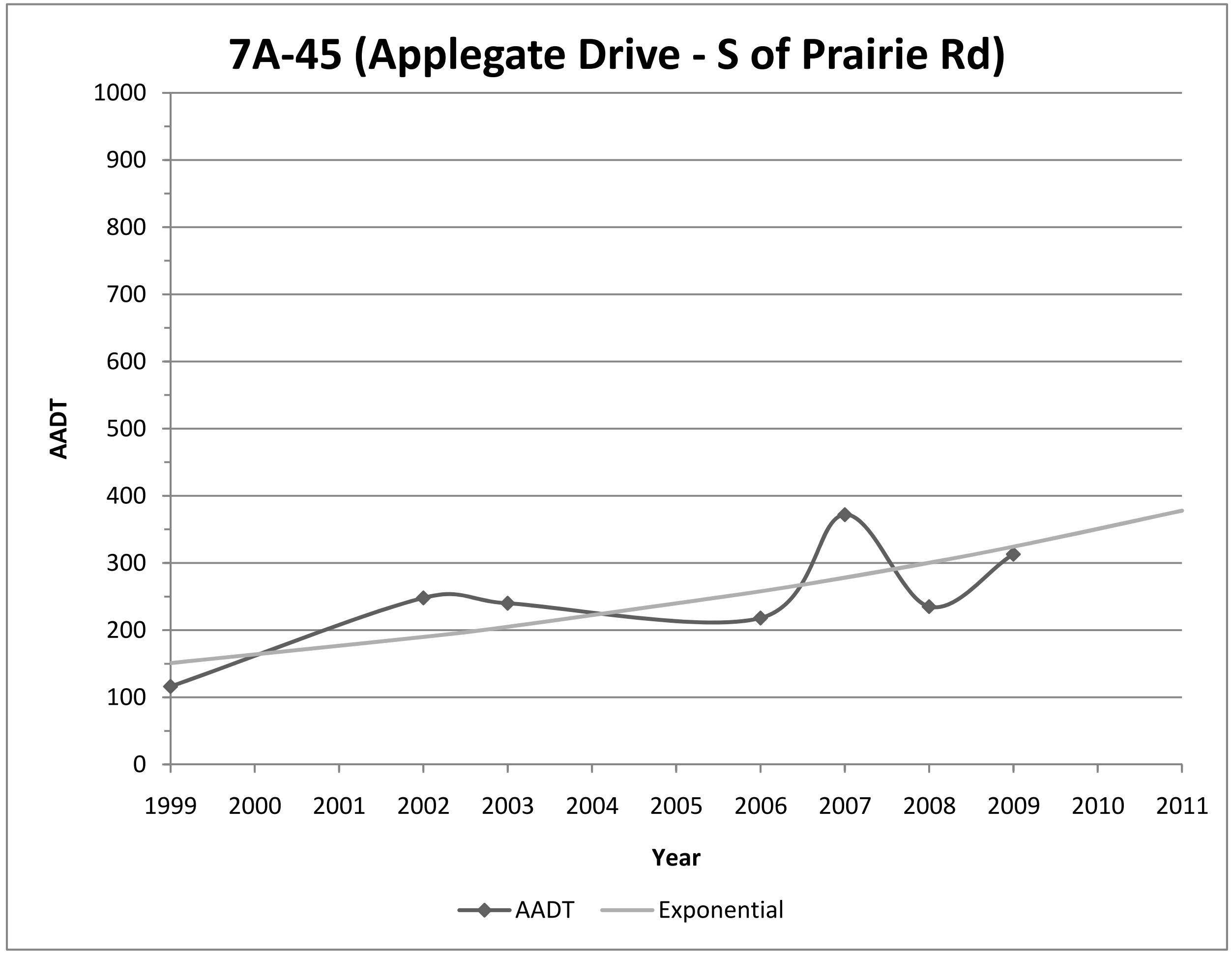
Note: Traffic data was unavailable before 1999



7A-45 (Applegate Drive - S of Prairie Rd)

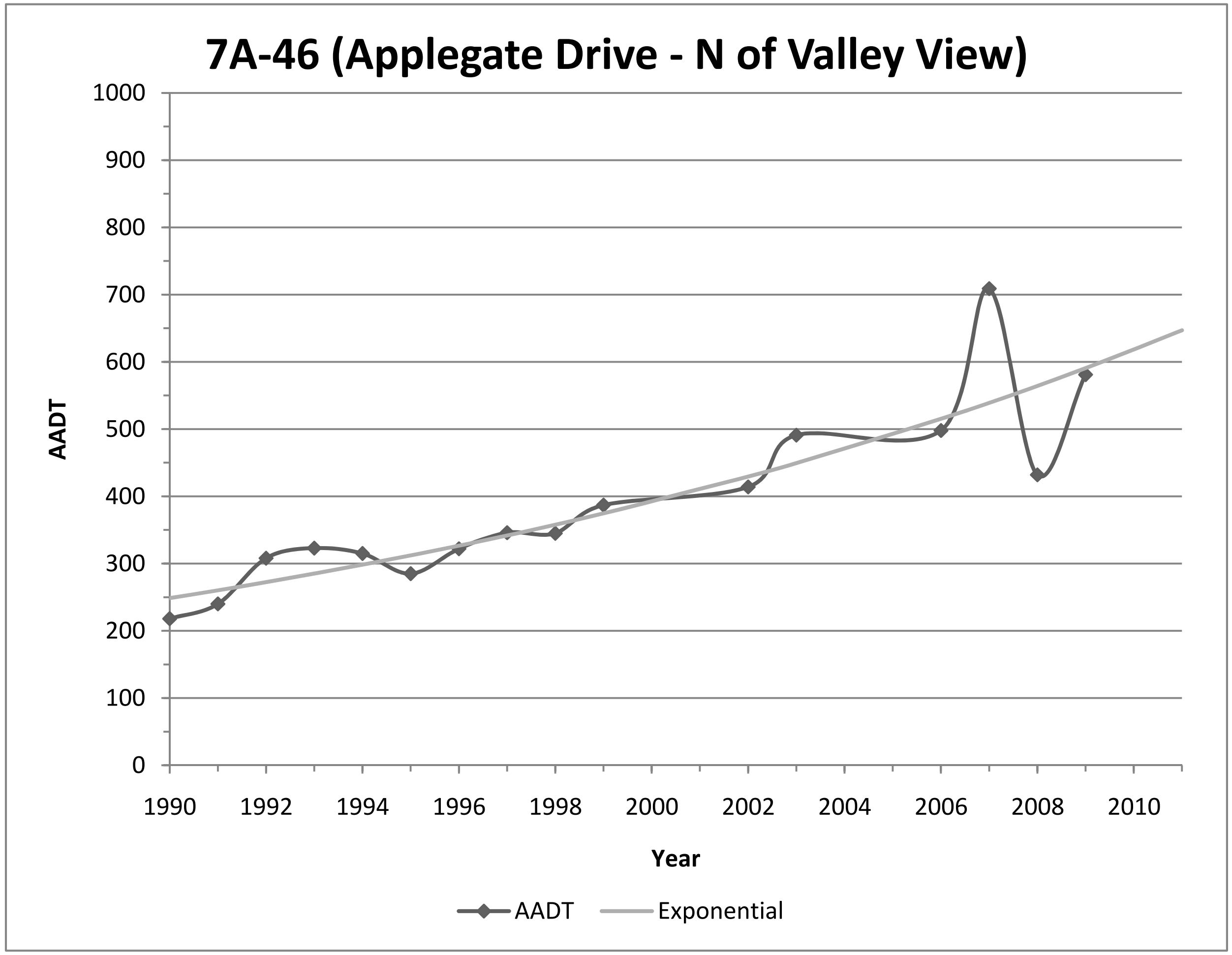
Year	AADT	Exponential
1999	116	151
2002	248	190
2003	240	205
2006	218	258
2007	372	278
2008	235	301
2009	313	324
2011	-	378
2031	-	1744
i	-	**7.94%**

Note: Traffic data was unavailable before 1999



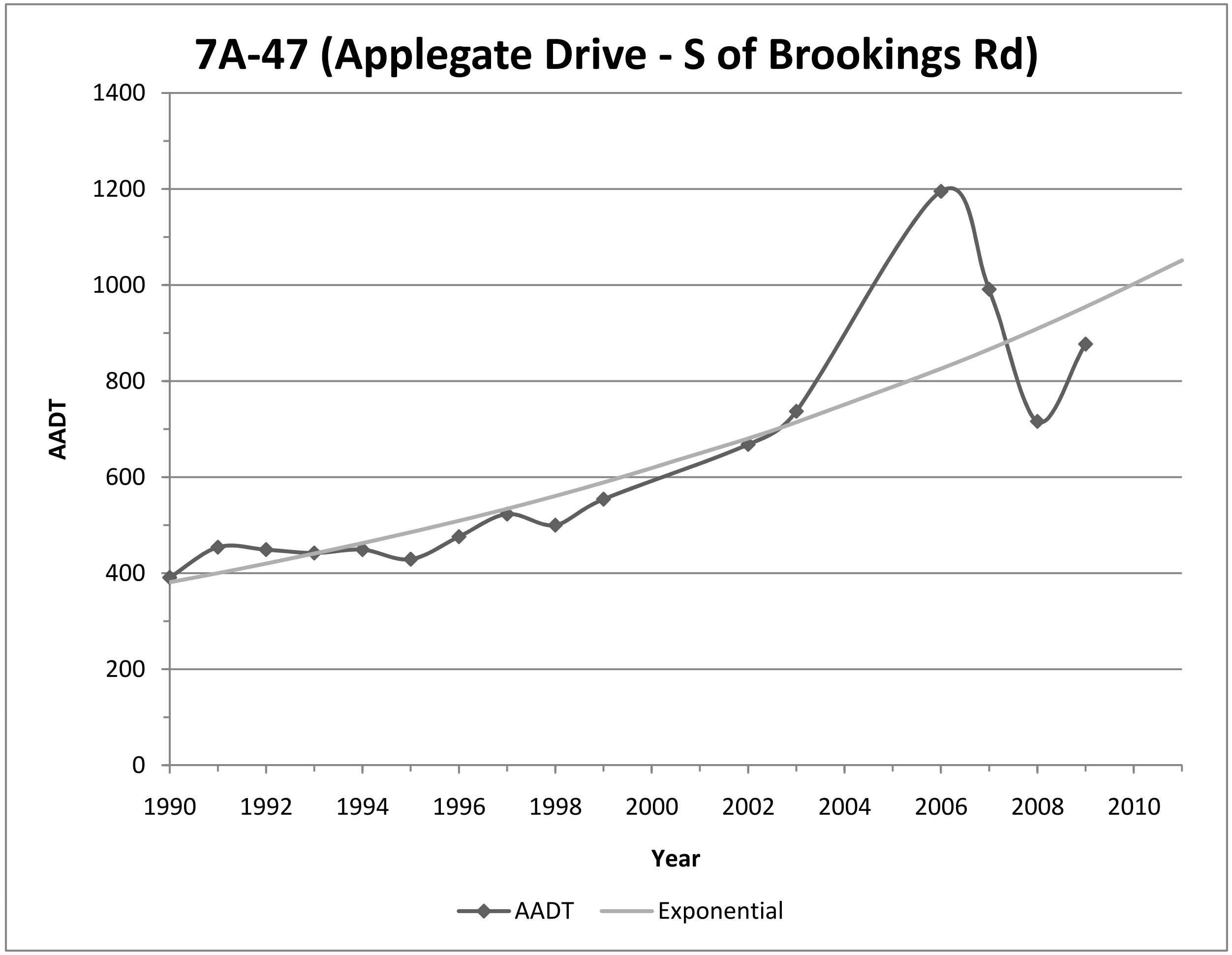
7A-46 (Applegate Drive - N of Valley View)

Year	AADT	Exponential
1990	218	249
1991	240	260
1992	308	272
1993	323	285
1994	315	298
1995	285	312
1996	322	327
1997	346	342
1998	345	358
1999	387	375
2002	414	429
2003	491	449
2006	498	515
2007	709	539
2008	432	564
2009	581	591
2011	-	647
2031	-	1608
i	-	**4.66%**



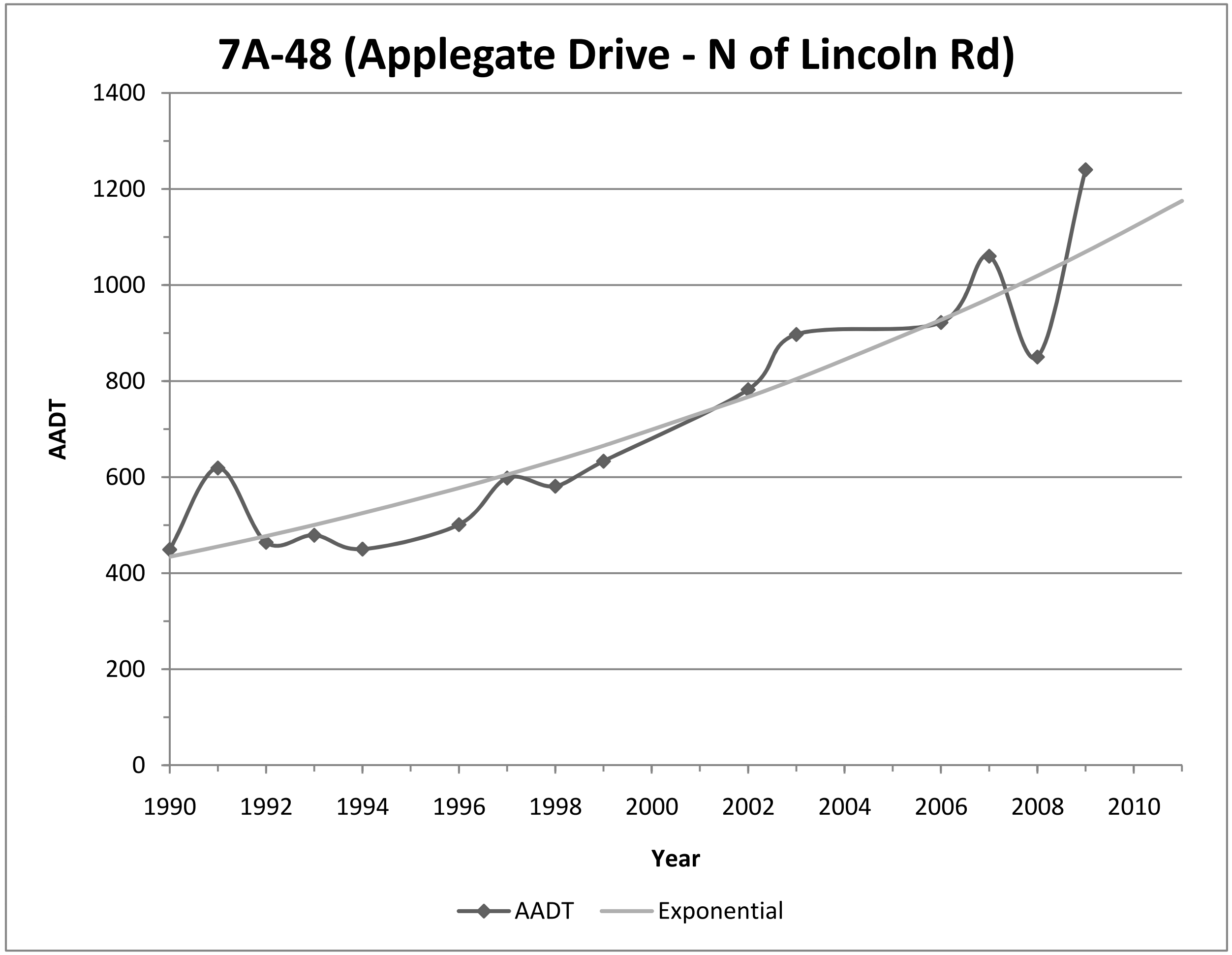
7A-47 (Applegate Drive - S of Brookings Rd)

Year	AADT	Exponential
1990	391	381
1991	454	400
1992	449	420
1993	442	441
1994	449	462
1995	429	485
1996	476	509
1997	523	534
1998	500	561
1999	554	589
2002	668	680
2003	737	714
2006	1195	826
2007	991	866
2008	716	909
2009	877	954
2011	-	1051
2031	-	2763
i	-	**4.95%**



7A-48 (Applegate Drive - N of Lincoln Rd)

Year	AADT	Exponential
1990	449	434
1991	619	455
1992	464	477
1993	479	501
1994	450	525
1996	501	577
1997	598	605
1998	581	634
1999	633	665
2002	782	767
2003	897	804
2006	922	927
2007	1060	972
2008	850	1019
2009	1240	1069
2011	-	1175
2031	-	3035
i	-	**4.86%**



Applegate Drive		AADT			
Site ID	**Location**	**2009**	**2011**	**2031**	**Growth**
7A-43	N. of Prairie Rd	309	300	756	4.72%
7A-45	S. of Prairie Rd	313	378	1744	7.94%
7A-46	N. of Valley View	581	647	1608	4.66%
7A-47	S. of Brookings Rd	877	1051	2763	4.95%
7A-48	N. of Lincoln Rd	1240	1175	3035	4.86%
Weighted Average:					**5.17%**

Appendix B

Design Reference Exhibits

#

TABLE A
COUNTY ROAD DESIGN CRITERIA

	Terrain	Major Collector	Minor Collector	Local Road
Design Speed (MPH)	Level	55	50	30
	Rolling	45	40	25
	Mountainous	45	30	20
Curvature - Minimum at Centerline (feet)	Level	575	575	250
	Rolling	440	440	175
	Mountainous	330	300	110
Minimum Stopping Sight Distance (feet)	Level	per AASHTO	425	200
	Rolling	"	305	150
	Mountainous	"	200	110
Maximum Grade	Level	per AASHTO	6%	6%
	Rolling	"	8%	9%
	Mountainous	"	10%	11%
Length of Maximum Grade (feet)		per AASHTO	per AASHTO	per AASHTO
Minimum Grade		0.5%	0.5%	0.5%
Superelevation		per AASHTO	per AASHTO	N/A
Minimum Intersection Spacing (feet)		500	275	150
Driveway Spacing (feet)		45	45	40
Maximum Length of Cul-de-Sac (feet)		Not Allowed	Not Allowed	See Chapter XI.H.11
Minimum Radius of Cul-de-Sac (feet)		Not Allowed	Not Allowed	48
Sight Distance Triangle (feet)	Level	300	255	120
	Rolling	210	170	95
	Mountainous	210	120	80
Minimum Right of Way Width		100	80	60
Minimum Right of Way Radius for Cul-de-sac (feet)		NA	NA	48
Vertical Clearance (feet)		16.5	16.5	14.5
Intersection Curb Return Radii (feet)		25	25	15
Minimum Sidewalk Width (feet)		5	5	5
Sidewalk Offset From Back of Curb (feet)		5-10	5-10	5
Bike Lane Width (feet)		4-8	4-8	N/A
Minimum Culvert Diameter (inches)		18	15	15
Minimum Culvert Cover		Meet or exceed suppliers recommendations	Meet or exceed suppliers recommendations	Meet or exceed suppliers recommendations
Minimum Culvert Grade		0.5%	0.5%	0.5%
Culvert Material		Support HS-20 Loading	Support HS-20 Loading	Support HS-20 Loading

Type of terrain	Metric: Design speed (km/h) for specified design volume (veh/day)						US Customary: Design speed (mph) for specified design volume (veh/day)					
	under 50	50 to 250	250 to 400	400 to 1500	1500 to 2000	2000 and over	under 50	50 to 250	250 to 400	400 to 1500	1500 to 2000	2000 and over
Level	50	50	60	80	80	80	30	30	40	50	50	50
Rolling	30	50	50	60	60	60	20	30	30	40	40	40
Mountainous	30	30	30	50	50	50	20	20	20	30	30	30

Exhibit 5-1. Minimum Design Speeds for Local Rural Roads

Metric				US Customary			
Initial speed (km/h)	Design stopping sight distance (m)	Rate of vertical curvature, K^a (m/%) Crest	Rate of vertical curvature, K^a (m/%) Sag	Initial speed (mph)	Design stopping sight distance (ft)	Rate of vertical curvature, K^a (ft/%) Crest	Rate of vertical curvature, K^a (ft/%) Sag
20	20	1	3	15	80	3	10
30	35	2	6	20	115	7	17
40	50	4	9	25	155	12	26
50	65	7	13	30	200	19	37
60	85	11	18	35	250	29	49
70	105	17	23	40	305	44	64
80	130	26	30	45	360	61	79
90	160	39	38	50	425	84	96
100	185	52	45	55	495	114	115
				60	570	151	136

[a] Rate of vertical curvature, K, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Chapter 3 for details.)

Exhibit 5-2. Design Controls for Stopping Sight Distance and for Crest and Sag Vertical Curves

Metric			US Customary		
Design speed (km/h)	Design passing sight distance (m)	Rate of vertical curvature, K^{a} (m/%)	Design speed (mph)	Design passing sight distance (ft)	Rate of vertical curvature, K^{a} (ft/%)
30	200	46	20	710	180
40	270	84	25	900	289
50	345	138	30	1090	424
60	410	195	35	1280	585
70	485	272	40	1470	772
80	540	338	45	1625	943
90	615	438	50	1835	1203
100	670	520	55	1985	1407
			60	2135	1628

[a] Rate of vertical curvature, K, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Chapter 3 for details.)

Exhibit 5-3. Design Controls for Crest Vertical Curves Based on Passing Sight Distance

Grades

Suggested maximum grades for local rural roads are shown in Exhibit 5-4.

	Metric									US Customary								
	Maximum grade (%) for specified design speed (km/h)									Maximum grade (%) for specified design speed (mph)								
Type of terrain	20	30	40	50	60	70	80	90	100	15	20	25	30	40	45	50	55	60
Level	9	8	7	7	7	7	6	6	5	9	8	7	7	7	7	6	6	5
Rolling	12	11	11	10	10	9	8	7	6	12	11	11	10	10	9	8	7	6
Mountainous	17	16	15	14	13	12	10	10	—	17	16	15	14	13	12	10	10	—

Exhibit 5-4. Maximum Grades for Local Rural Roads

Alignment

Alignment between control points should be designed to be as favorable as possible consistent with the environmental impact, topography, terrain, design traffic volume, and the amount of reasonably obtainable right-of-way. Sudden changes between curves of widely different radii or between long tangents and sharp curves should be avoided. Where practical, the design should include passing opportunities. Where crest vertical curves and horizontal curves occur together, there should be greater than minimum sight distance to ensure that the horizontal curves are visible to approaching drivers.

Metric

Design speed (km/h)	Minimum width of traveled way (m) for specified design volume (veh/day): under 400	400 to 1500	1500 to 2000	over 2000
20	5.4	6.0[a]	6.0	6.6
30	5.4	6.0[a]	6.6	7.2[c]
40	5.4	6.0[a]	6.6	7.2[c]
50	5.4	6.0[a]	6.6	7.2[c]
60	5.4	6.0[a]	6.6	7.2[c]
70	6.0	6.6	6.6	7.2[c]
80	6.0	6.6	6.6	7.2[c]
90	6.6	6.6	7.2[c]	7.2[c]
100	6.6	6.6	7.2[c]	7.2[c]
	Width of graded shoulder on each side of the road (m)			
All speeds	0.6	1.5[a,b]	1.8	2.4

US Customary

Design speed (mph)	Minimum width of traveled way (ft) for specified design volume (veh/day): under 400	400 to 1500	1500 to 2000	over 2000
15	18	20[a]	20	22
20	18	20[a]	22	24[c]
25	18	20[a]	22	24[c]
30	18	20[a]	22	24[c]
40	18	20[a]	22	24[c]
45	20	22	22	24[c]
50	20	22	22	24[c]
55	22	22	24[c]	24[c]
60	22	22	24[c]	24[c]
	Width of graded shoulder on each side of the road (ft)			
All speeds	2	5[a,b]	6	8

[a] For roads in mountainous terrain with design volume of 400 to 600 veh/day, use 5.4-m [18-ft] traveled way width and 0.6-m [2-ft] shoulder width.

[b] May be adjusted to achieve a minimum roadway width of 9 m [30 ft] for design speeds greater than 60 km/h [40 mph].

[c] Where the width of the traveled way is shown as 7.2 m [24 ft], the width may remain at 6.6 m [22 ft] on reconstructed highways where alignment and safety records are satisfactory.

See text for roadside barrier and offtracking considerations.

Exhibit 5-5. Minimum Width of Traveled Way and Shoulders

Type of terrain	Metric: Design speed (km/h) for specified design volume (veh/day)			US Customary: Design speed (mph) for specified design volume (veh/day)		
	0 to 400	400 to 2000	over 2000	0 to 400	400 to 2000	over 2000
Level	60	80	100	40	50	60
Rolling	50	60	80	30	40	50
Mountainous	30	50	60	20	30	40

Note: Where practical, design speeds higher than those shown should be considered.

Exhibit 6-1. Minimum Design Speeds for Rural Collectors

Metric: Design speed (km/h)	Design stopping sight distance (m)	Rate of vertical curvature, K^a (m/%)		US Customary: Design speed (mph)	Design stopping sight distance (ft)	Rate of vertical curvature, K^a (ft/%)	
		Crest	Sag			Crest	Sag
20	20	1	3	15	80	3	10
30	35	2	6	20	115	7	17
40	50	4	9	25	155	12	26
50	65	7	13	30	200	19	37
60	85	11	18	35	250	29	49
70	105	17	23	40	305	44	64
80	130	26	30	45	360	61	79
90	160	39	38	50	425	84	96
100	185	52	45	55	495	114	115
				60	570	151	136

[a] Rate of vertical curvature, K, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Chapter 3 for details.)

Exhibit 6-2. Design Controls for Stopping Sight Distance and for Crest and Sag Vertical Curves

Metric			US Customary		
Design speed (km/h)	Design passing sight distance (m)	Rate of vertical curvature, K^a (m/%)	Design speed (mph)	Design passing sight distance (ft)	Rate of vertical curvature, K^a (ft/%)
30	200	46	20	710	180
40	270	84	25	900	289
50	345	138	30	1090	424
60	410	195	35	1280	585
70	485	272	40	1470	772
80	540	338	45	1625	943
90	615	438	50	1835	1203
100	670	520	55	1985	1407
			60	2135	1628

[a] Rate of vertical curvature, K, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Chapter 3 for details.)

Exhibit 6-3. Design Controls for Crest Vertical Curves Based on Passing Sight Distance

	Metric								US Customary								
	Maximum grade (%) for specified design speed (km/h)								Maximum grade (%) for specified design speed (mph)								
Type of terrain	30	40	50	60	70	80	90	100	20	25	30	35	40	45	50	55	60
Level	7	7	7	7	7	6	6	5	7	7	7	7	7	7	6	6	5
Rolling	10	10	9	8	8	7	7	6	10	10	9	9	8	8	7	7	6
Mountainous	12	11	10	10	10	9	9	8	12	11	10	10	10	10	9	9	8

Note: Short lengths of grade in rural areas, such as grades less than 150 m [500 ft] in length, one-way downgrades, and grades on low-volume rural collectors may be up to 2 percent steeper than the grades shown above.

Exhibit 6-4. Maximum Grades for Rural Collectors

Metric					US Customary				
Design speed (km/h)	Minimum width of traveled way (m) for specified design volume (veh/day)[a]				Design speed (mph)	Minimum width of traveled way (ft) for specified design volume (veh/day)[a]			
	under 400	400 to 1500	1500 to 2000	over 2000		under 400	400 to 1500	1500 to 2000	over 2000
30	6.0[b]	6.0	6.6	7.2	20	20[b]	20	22	24
40	6.0[b]	6.0	6.6	7.2	25	20[b]	20	22	24
50	6.0[b]	6.0	6.6	7.2	30	20[b]	20	22	24
60	6.0[b]	6.6	6.6	7.2	35	20[b]	22	22	24
70	6.0	6.6	6.6	7.2	40	20[b]	22	22	24
80	6.0	6.6	6.6	7.2	45	20	22	22	24
90	6.6	6.6	7.2	7.2	50	20	22	22	24
100	6.6	6.6	7.2	7.2	55	22	22	24	24
					60	22	22	24	24
	Width of shoulder on each side of road (m)					Width of shoulder on each side of road (ft)			
All speeds	0.6	1.5[c]	1.8	2.4	All speeds	2.0	5.0[c]	6.0	8.0

[a] On roadways to be reconstructed, a 6.6-m [22-ft] traveled way may be retained where the alignment and safety records are satisfactory.

[b] A 5.4-m [18-ft] minimum width may be used for roadways with design volumes under 250 veh/day.

[c] Shoulder width may be reduced for design speeds greater than 50 km/h [30 mph] as long as a minimum roadway width of 9 m [30 ft] is maintained.

See text for roadside barrier and offtracking considerations.

Exhibit 6-5. Minimum Width of Traveled Way and Shoulders

Drivers who inadvertently leave the traveled way can often recover control of their vehicles if foreslopes are 1V:4H or flatter and shoulders and ditches are well rounded or otherwise made traversable. Such recoverable slopes should be provided where terrain and right-of-way conditions allow.

Where provision of recoverable slopes is not practical, the combinations of rate and height of slope provided should be such that occupants of an out-of-control vehicle have a good chance of survival. Where high fills, right-of-way restrictions, watercourses, or other problems render such designs impractical, roadside barriers should be considered, in which case the maximum rate of fill slope may be used. Reference should be made to the current edition of the AASHTO *Roadside Design Guide* (**3**). For further information, see the section on "Traffic Barriers" in Chapter 4.

Cut sections should be designed with adequate ditches. Preferably, the foreslope should not be steeper than 1V:3H and, where practical, should be 1V:4H or flatter. The ditch bottom and slopes should be well rounded, and the backslope should not exceed the maximum needed for stability.

TABLE 3.1 (Cont'd)

[U.S. Customary Units]

DESIGN SPEED	DESIGN ADT	FORESLOPES			BACKSLOPES		
		1V:6H or flatter	1V:5H TO 1V:4H	1V:3H	1V:3H	1V:5H TO 1V:4H	1V:6H or flatter
40 mph or less	UNDER 750	7 – 10	7 – 10	**	7 – 10	7 – 10	7 – 10
	750 – 1500	10 – 12	12 – 14	**	10 – 12	10 – 12	10 – 12
	1500 – 6000	12 – 14	14 – 16	**	12 – 14	12 – 14	12 – 14
	OVER 6000	14 – 16	16 – 18	**	14 – 16	14 – 16	14 – 16
45–50 mph	UNDER 750	10 – 12	12 – 14	**	8 – 10	8 – 10	10 – 12
	750 – 1500	14 – 16	16 – 20	**	10 – 12	12 – 14	14 – 16
	1500 – 6000	16 – 18	20 – 26	**	12 – 14	14 – 16	16 – 18
	OVER 6000	20 – 22	24 – 28	**	14 – 16	18 – 20	20 – 22
55 mph	UNDER 750	12 – 14	14 – 18	**	8 – 10	10 – 12	10 – 12
	750 – 1500	16 – 18	20 – 24	**	10 – 12	14 – 16	16 – 18
	1500 – 6000	20 – 22	24 – 30	**	14 – 16	16 – 18	20 – 22
	OVER 6000	22 – 24	26 – 32 *	**	16 – 18	20 – 22	22 – 24
60 mph	UNDER 750	16 – 18	20 – 24	**	10 – 12	12 – 14	14 – 16
	750 – 1500	20 – 24	26 – 32 *	**	12 – 14	16 – 18	20 – 22
	1500 – 6000	26 – 30	32 – 40 *	**	14 – 18	18 – 22	24 – 26
	OVER 6000	30 – 32 *	36 – 44 *	**	20 – 22	24 – 26	26 – 28
65–70 mph	UNDER 750	18 – 20	20 – 26	**	10 – 12	14 – 16	14 – 16
	750 – 1500	24 – 26	28 – 36 *	**	12 – 16	18 – 20	20 – 22
	1500 – 6000	28 – 32 *	34 – 42 *	**	16 – 20	22 – 24	26 – 28
	OVER 6000	30 – 34 *	38 – 46 *	**	22 – 24	26 – 30	28 – 30

* Where a site specific investigation indicates a high probability of continuing crashes, or such occurrences are indicated by crash history, the designer may provide clear-zone distances greater than the clear-zone shown in Table 3.1. Clear zones may be limited to 30 ft for practicality and to provide a consistent roadway template if previous experience with similar projects or designs indicates satisfactory performance.

** Since recovery is less likely on the unshielded, traversable 1V:3H slopes, fixed objects should not be present in the vicinity of the toe of these slopes. Recovery of high-speed vehicles that encroach beyond the edge of the shoulder may be expected to occur beyond the toe of slope. Determination of the width of the recovery area at the toe of slope should take into consideration right-of-way availability, environmental concerns, economic factors, safety needs, and crash histories. Also, the distance between the edge of the through traveled lane and the beginning of the 1V:3H slope should influence the recovery area provided at the toe of slope. While the application may be limited by several factors, the foreslope parameters which may enter into determining a maximum desirable recovery area are illustrated in Figure 3.2.

Appendix C

Pavement Evaluation

SK GEOTECHNICAL

May 10, 2011 Project 11-2762

Mr. Tom Cavanaugh, P.E.
Robert Peccia & Associates
Via Email: *tom@rpa-hln.com*

Dear Tom:

Re: Pavement Evaluation, Applegate Drive, Lewis and Clark County Road Improvement Projects, Helena, Montana

The pavement evaluation for the above-referenced project has been completed. The purpose of the pavement evaluation was to perform soil borings along the alignment and laboratory tests on selected samples to assist Robert Peccia & Associates (RPA) and Lewis and Clark County to complete initial preliminary engineering analysis and planning for Applegate Drive improvements. The pavement evaluation was performed in general accordance with our Subconsultant Agreement dated March 10, 2011.

Project Information

It is our understanding Applegate Drive, north of Lincoln Road, is being considered by Lewis and Clark County to receive reconstructive improvements. Depending on county road prioritizing and funding availability, the intent will be for whole or parts of the road to be reconstructed to meet or exceed minimum County standards. The portion of road being evaluated in this report is from the intersection of Lincoln Road East (State Secondary Highway 279) extending northward for 3 miles to where the existing roadway basically ends at a turnaround area. The Applegate Drive roadway limits considered for this pavement evaluation are shown on the attached Boring Location Sketch. At this time, the engineering evaluation along Applegate Drive is based on a new pavement section to bring the road into compliance of meeting or exceeding the minimum road standards in accordance with the Lewis and Clark Subdivision Regulations dated December 18, 2007 (amended 2009 and 2010).

Field Procedures

On April 4 and 5, 2011, Borings ST-1 through ST-4 were performed along the 3-mile alignment being considered for reconstruction. Therefore, the borings were located about 1 mile apart. Boring locations were selected by our personnel and were generally alternated from the northbound and southbound lanes. The locations of Borings ST-1 through ST-4 are shown on the attached sketch. To complete the borings, minor traffic control was performed while drilling.

The borings were performed with a truck-mounted core and auger drill. Sampling of the borings was performed in accordance with American Society for Testing and Materials (ASTM) Method of Test D 1586, "Penetration Test and Split-Barrel Sampling of Soils." Using this method, we advanced the borehole with hollow-stem auger to the desired test depth. Then a 140-pound hammer falling 30 inches drove a standard, 2-inch OD, split-barrel sampler a total penetration of 1 1/2 to 2 feet below the tip of the hollow-stem auger. The blows for the 1 1/2-foot of penetration are indicated on the boring logs, and are an index of soil strength characteristics. The last 1-foot portion of each penetration test is the N-value, and referred to as blows per foot (BPF) in this report.

While drilling, our engineering assistant attempted to measure the thickness of the existing gravel surfacing to the nearest 1/2 inch. We wish to point out, however, that measuring the existing gravel surfacing thickness to the nearest 1/2 inch can be difficult due to intermixing of the gravel surfacing with the underlying fill. Bag samples of the existing gravel surfacing course and subgrade were collected from select borings. The borings were then backfilled by our drill crew.

The soils encountered in the borings were visually and manually classified in accordance with ASTM D 2488, "Standard Practice for Description and Identification of Soils (Visual – Manual Procedures)." A summary of the ASTM classification system is attached. All samples were then returned to our laboratory for review of the field classifications by a geotechnical engineer. Representative samples will remain in our office for a period of 60 days to be available for your examination.

Results

General. Log of Boring sheets indicating the depth and identification of the various soil strata, the penetration resistance, laboratory test data, and water level information are attached. It should be noted that the depths shown as boundaries between the strata are only approximate. The actual changes may be transitions and the depths of changes vary between borings.

Geologic origins presented for each stratum on the Log of Boring sheets are based on the soil types, blows per foot, and available common knowledge of the depositional history of the site. Because of the complex glacial and post-glacial depositional environments, geologic origins are frequently difficult to ascertain. A detailed evaluation of the geologic history of the roadway as well as review of contour maps and cross sections was not performed.

The general soil profile encountered by the four borings was clayey gravel surfacing course underlain by clayey sand with gravel fill over silty gravel subgrades. Table 1 below summarizes the existing gravel surfacing and subgrade conditions encountered at the four borings.

Table 1. Summary of Boring Conditions – Applegate Drive

Boring	ST-1	ST-2	ST-3	ST-4
Existing Gravel Surfacing	0"	8"	9"	0"
Existing Clayey Sand Fill Thickness	54"	22"	15"	30"
Existing Fill Quality	Good	Good	Good	Good
Subgrade	Silty Gravel	Silty Gravel	Clayey Gravel	Silty Gravel
BPF in Clayey Sand Fill	42, 41	24, 18	24	20
Moisture Condition of Fill	Below to near	Below to near	Below	Below to near
Risk of Subgrade Failure	Low	Low	Low	Low

General Statistical Summary
Existing Fill Quality: 4 of 4 borings (100%) encountered GOOD quality clayey sand fill
Subgrade Conditions: 4 of 4 borings (100%) have LOW risk to become unstable during construction

Existing Clayey Sand Fill. As indicated in Table 1 above, the four borings encountered existing clayey sand fill to depths ranging from 2 to 4 feet. Penetration tests were performed in the clayey sand fill. In general, penetration resistances in the clayey sand typically ranged from 18 to 42 blows per foot (BPF), indicating it was medium dense to dense.

Subgrade. Beneath the existing fill, Borings ST-1, ST-2, and ST-4 encountered silty gravel with sand to the borings' termination depth of 5 1/2 feet. Boring ST-3 encountered clayey gravel with sand. Penetration resistances in the silty gravel and clayey gravel subgrades typically ranged from 25 to 43 BPF, indicating these materials were medium dense to dense.

Moisture content tests were performed on all of the penetration test samples from the borings. The moisture contents are indicated on the boring logs and were either compared to the optimum moisture content determined by our standard Proctor (described below) or typical optimum moisture contents for these types of soils. Based on these moisture content tests, the existing fill in Borings ST-1 through ST-4 are below or near optimum moisture content.

Groundwater. Groundwater was not encountered in the four borings to their termination depth of 5 1/2 feet at the time of our fieldwork. We wish to point out that clayey subgrades were encountered by the borings. Several days may be required for groundwater levels to develop and stabilize in these types of soils. Surface water can also become trapped on top of these clay soils (perched groundwater), and then be encountered during construction.

Laboratory Tests

Two composite subgrade samples and three standard penetration test (SPT) jar samples were selected for laboratory tests. The results are summarized in Table 2 below and are attached to this report.

Table 2. Summary of Laboratory Tests

Sample	Atterberg Limits			P_{200} (%)	Standard Proctor		CBR Value
	LL	PL	PI		MDD (pcf)	OMC (%)	
Composite Sample ST-1 and ST-2, 1' to 4'	26	16	10	27	131.1	8.4	20.8
Composite Sample ST-3 and ST-4, 1' to 4'	33	17	16	30	129.2	9.1	10.0
Jar Sample ST-1, 4' to 5½'	NP	NP	NP	---	---	---	---
Jar Sample ST-2, 4' to 5½'	---	---	---	14	---	---	---
Jar Sample ST-4	---	---	---	18	---	---	---

MDD = Maximum Dry Density (ASTM D 698), pounds per cubic foot (pcf)
OMC = Optimum Moisture Content
NP = Not Plastic

As can be seen above, the clayey sand fill samples tested from Borings ST-1 through ST-4 were plastic, having plasticity indexes of 10 and 16, respectively. The percent-finer-than-a-200-sieve (P_{200}) of these samples were about 27 and 30 percent as well. These results indicate the fill classifies as low plasticity clayey sand, which would be considered a poor quality base course.

Standard Proctors (ASTM D 698) and California bearing ratio (CBR) tests were performed on the two samples indicated above. CBR values for these samples were 10.0 and 20.8.

Pavement Analysis and Recommendations

Available Information. RPA provided us with the Applegate Drive traffic information indicated on the attached traffic chart for a compilation of different sites from 7A-43 through 7A-48 along the roadway. These traffic count sites represent Applegate Drive starting at Lincoln Road and continuing north to the end of the road. As can be seen in the chart, the annual average daily traffic (AADT) changes significantly as the road progresses north. Additionally, gravel operations are located near the south end of Applegate Drive, north of Lincoln Road. Due to the significant changes in AADT and truck traffic associated with the gravel operations, pavement analysis and design was performed from Lincoln Road to Brookings Road and north of Brookings Road. Traffic data from site 7A-48 was used for the Lincoln Road to Brookings Road portion. Traffic data from site 7A-46 was used for that portion north of Brookings Road.

A detailed traffic count used to determine the percent of heavy trucks was not performed for Applegate Drive. In previous pavement evaluations performed by SK Geotechnical in the nearby area, the percent of heavy trucks was typically estimated or measured to be near 3 percent. It is our opinion that 3 percent trucks is a reasonable estimate for truck traffic north of Brookings Road. Between Lincoln Road and Brookings Road, it is our opinion that 5 percent truck traffic should be used for analysis when considering the gravel operations.

Method. Pavement sections for the roadway were evaluated using DARWin™, a computer program based on the *1993 AASHTO Guide for Design of Pavement Structures*. The AASHTO Pavement Design Method is based on numerous input parameters, each affecting the required total pavement thickness for a given road. Based on the traffic information provided by RPA, we were able to perform a simple traffic analysis to determine the design Equivalent Single 18-kip Axle Load (ESAL). The simple traffic analysis is included in the DARWin output. The input parameters and traffic information are summarized in Table 3 below.

Table 3. Summary of Pavement Design Assumptions and Analysis

Parameter	**Lincoln Road to Brookings Road**	**North of Brookings Road**
Road Classification	Minor Collector	Minor Collector/Local Road
2011 AADT	1,175	647
2031 AADT	3,035	1,608
Percent Trucks	5%	3%
Estimated Annual Growth	4.86%	4.66%
Performance Period	20 Years	20 Years
Initial Serviceability	4.2	4.2
Terminal Serviceability	2.5	2.5
Reliability	85%	85%
Number of Lanes in Design Direction	1	1
Percent All Trucks in Design Lane	50	50
Percent Trucks in Design Direction	100	100
18-kip ESALs	435,985	102,280

As can be seen above, we calculated a design ESAL of 435,985 for Applegate Drive from Lincoln Road to Brookings Road, and a design ESAL of 102,280 for Applegate Drive north of Brookings Road. Due to the gravel operations, a truck factor of 1.39 was used for Lincoln Road to Brookings Road, which represents the gravel haul trucks.

The DARWin pavement design uses roadbed soil resilient modulus (M_R) to identify subgrade strength. CBR is another method of representing subgrade strength. Correlations of these subgrade strength parameters are contained in the *1993 AASHTO Design of Pavement Structures* manual. For soils having CBR values of 10 or less, the manual indicates the following equation can be used.

$$M_r \text{ (psi)} = 1{,}500 \times CBR$$

As previously indicated in Table 2, CBR values of 10.0 and 20.8 were determined for subgrade samples along this roadway. A common procedure to select the design CBR is use a value one standard deviation below the mean. In this case, the mean (average) value is 15.4 and the standard deviation is 7.6, resulting in a preliminary design value of 7.8. The CBR values for clayey sands typically range from 10 to 20. It is our opinion the preliminary value of 7.8 is too low, and a design CBR of 10 should be used for pavement design. This results in an M_r equal to 15,000.

Pavement Sections. Pavement sections were developed in general accordance to meet or exceed minimum roadway sections of the Lewis and Clark County Subdivision Regulations. Based on this approach and the above input parameters and design information, our recommended pavement sections are summarized in Table 4 below.

Table 4. Recommended Pavement Section for Applegate Drive

Item	**Lincoln Road to Brookings Road**	**North of Brookings Road**
Asphalt Pavement	3"	3"
Crushed Top Surfacing	3"	3"
Select Base Course*	6"	6"
Subbase Course*	4"	0"
Total	**16"**	**12"**

*Per Table B-4 of Lewis and Clark Subdivision Regulations.

Constructability.

General. A common problem in roadway construction is encountering unstable subgrades. Unstable subgrades are those subgrade soils that are excessively wet and soft, and cannot support heavy rubber-tired construction equipment as well as cannot be compacted to specification. They commonly occur beneath existing roads where surface water has seeped through gravel surfacing and become trapped in the underlying base course and subgrade. This water saturates the clays, reducing their shear strength, and the clay subgrade becomes too soft and wet to support the heavy rubber-tired construction equipment. When this occurs during fast-tracked construction projects, it can cause delays, which then results in change orders.

As previously indicated in Table 1, all of the borings encountered clayey sand subgrade, which has a "low" risk of subgrade failure during construction. However, there could be areas located between the borings, which were about 1 mile apart, that are unstable.

Identification of Unstable Areas. When considering total reconstruction, the best method of determining unstable subgrades is to perform proof rolling observations directly on the exposed subgrade during construction. Proof rolling should be performed with a loaded tandem axle dump truck or equivalent. Unstable areas are those subgrade soils where proof rolling indicates 1/2 inch or more of deflection is occurring. Another method of determining unstable subgrades is whether or not they can be recompacted to specification, typically 95 percent of their standard Proctor maximum dry density. If unstable subgrades are identified, we recommend installing a stabilized pavement section as described below.

Stabilized Pavement Section. Two alternatives for stabilized pavement sections are indicated in Table 5 below. Alternatives 1 and 2 are stabilized pavement sections using geosynthetics.

Table 5. Stabilized Pavement Section for Excessively Soft (Unstable) Subgrade Areas

Item	**Alternative 1**	**Alternative 2**
Asphalt Pavement	3"	3"
Crushed Top Surfacing	3"	3"
Select Base and/or Subbase	20"	20"
Geosynthetic	Tensar BX 1300 over Class 2 Non-woven Fabric	Mirafi RS580i

Other Alternatives. We suggest also contacting Lewis and Clark County personnel and/or discussing these types of stabilized pavement sections with the contractor, who may have other alternatives for constructing pavements on unstable subgrades. Another alternative is to allow unstable subgrades to possibly dry out during construction. For this approach, several weeks of warm, windy weather will likely be needed to allow the exposed conditions to dry out and become more stable. We have found, however, that the construction schedule of most contractors does not allow them to wait for these areas to dry out and become stable.

Some consideration can also be given to specifying that all construction activities are performed with low-pressured ground equipment. In Montana, however, this equipment is generally not readily available by most earthwork and paving contractors.

Specifications

When the Applegate Drive reconstruction project(s) are undertaken, we recommend all earthwork, subgrade preparation, gravel base and subbase, and asphalt pavement be specified and constructed in accordance with Montana Public Works Standard Specifications (MPWSS). The Montana Department of Transportation (MDT) Specifications for Road and Bridge Design can also be used, however, they are slightly more stringent. If geosynthetics are utilized, we recommend they be placed and constructed in accordance with the manufacturer's recommendations.

Observation and Testing

We recommend the pavement subgrades be observed by a geotechnical engineer or an engineering assistant working under the direction of a geotechnical engineer to see if the materials are similar to those encountered by the borings. During construction, we recommend density tests be taken on the recompacted subgrade and compacted crushed top surfacing, select base, and subbase courses. The thicknesses of crushed top surfacing, select base, and subbase should also be checked to confirm they meet specifications.

We also recommend density testing of the asphaltic concrete surface and Marshall tests on asphaltic concrete mix to evaluate strength and air voids. Cores of asphalt concrete should be taken at intervals to evaluate pavement thickness and compaction. Paving observations should also be performed to confirm the specified thickness of asphalt is provided throughout the roadway.

General Recommendations

Basis of Recommendations. The analyses and recommendations submitted in this report are based upon the data obtained from the borings performed at the locations indicated on the attached sketch. Often, variations occur between these borings, the nature and extent of which do not become evident until additional exploration or construction is conducted. A reevaluation of the recommendations in this report should be made after performing on-site observations during construction to note the characteristics of any variations. The variations may result in additional earthwork and construction costs, and it is suggested that a contingency be provided for this purpose.

It is recommended that when the road is reconstructed, we or another qualified geotechnical engineering firm be retained to perform the observations and testing program for the site preparation. This will allow correlation of the soil conditions encountered during construction to the soil borings.

Groundwater Fluctuations. We made water level observations in the borings at the times and under the conditions stated on the boring logs. These data were interpreted in the text of this report. The period of observation was relatively short, and fluctuation in the groundwater level may occur due to rainfall, flooding, irrigation, spring thaw, drainage, and other seasonal and annual factors not evident at the time the observations were made. Design drawings and specifications and construction planning should recognize the possibility of fluctuations.

Use of Report. This report is for the exclusive use of the Robert Peccia & Associates to use in conjunction with the preliminary road reconstruction analysis being completed by them for the County. In the absence of our written approval, we make no representation and assume no responsibility to other parties regarding this report. The data, analyses and recommendations may not be appropriate for other structures or purposes. We recommend parties contemplating other alignments or purposes contact us.

Level of Care. Services performed by SK Geotechnical Corporation personnel for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in this area under similar budget and time restraints. No warranty, expressed or implied, is made.

We appreciate the opportunity to provide these services for you. If we can be of further assistance, please contact us at your convenience.

Sincerely,

Brett M. Warren, EI
Staff Engineer

Professional Certification
I hereby certify that this report was prepared under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Montana.

MONTANA
GREGORY T. STAFFILENO
No. 10798 P.E.
REGISTERED PROFESSIONAL ENGINEER

Gregory T. Staffileno, PE
Principal, Geotechnical Engineer
License Number 10798PE

bmw/gts:khr

Attachments:
Boring Location Sketch
Descriptive Terminology
Log of Boring Sheets ST-1 through ST-4
Laboratory Tests
Applegate Drive Traffic Chart
DARWin Pavement Analysis

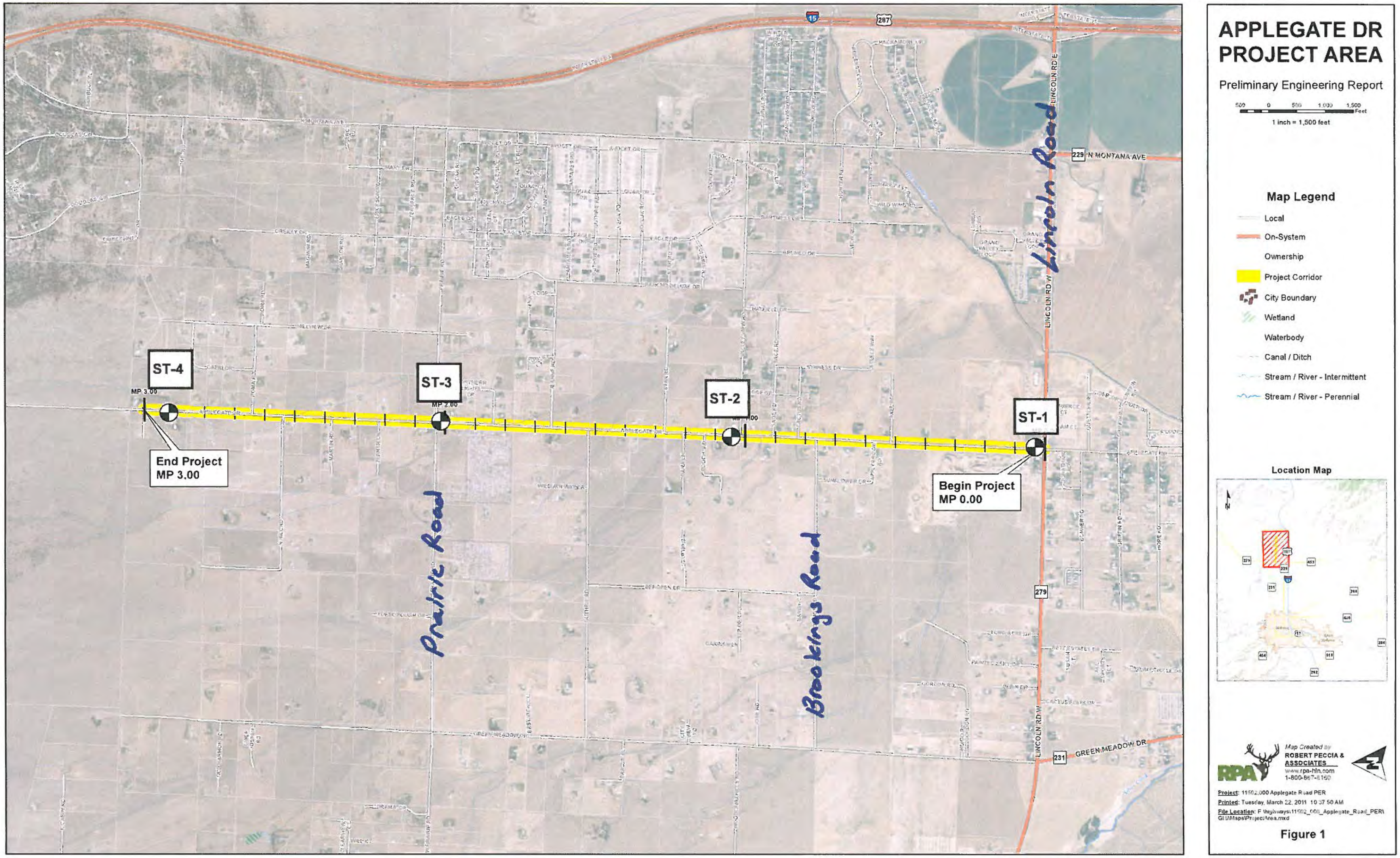


Figure 1

BORING LOCATION SKETCH
Pavement Evaluation
Applegate Drive
N. of Helena, MT

Project: 11-2762
Date: 5/10/2011

Descriptive Terminology

Standard D 2487
Classification of Soils for Engineering Purposes
(Unified Soil Classification System)

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

[A] Based on the material passing the 3" (75 mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay
[D] Sands with 5 to 12% fines require dual symbols.
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay
[E] $C_U = D_{50} / D_{10}$
$C_C = (D_{30})^2 / (D_{10} \times D_{50})$
[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel", whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200 predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.

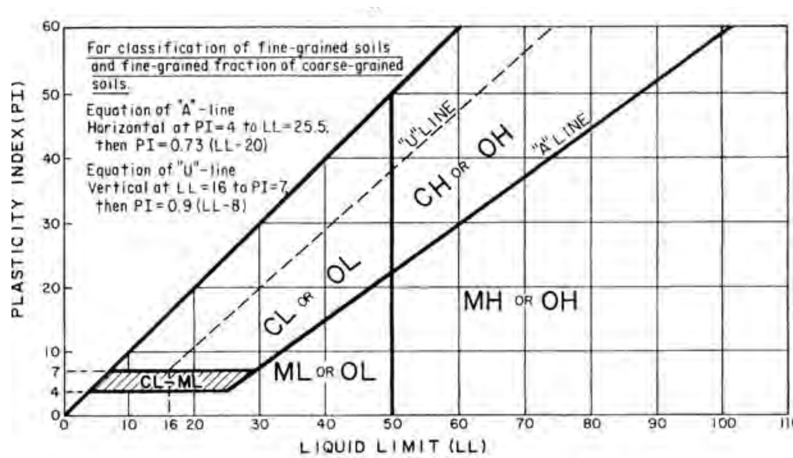


FIG. 4 Plasticity Chart

Laboratory Tests

DD	Dry density, pcf	OC	Organic content, %
WD	Wet density, pcf	P_{200}	% passing 200 sieve
LL	Liquid limit	PL	Plastic limit
PI	Plasticity index	MC	Natural moisture content, %
qu	Unconfined compressive strength, psf		
qp	Pocket penetrometer strength, tsf		

Particle Size Identification

Boulders .... over 12"
Cobbles.................................. 3" to 12"
Gravel
coarse.................................. 3/4" to 3"
fine................................. No. 4 to 3/4"
Sand
coarse......................... No. 4 to No. 10
medium.................... No. 10 to No. 40
fine......................... No. 40 to No. 200
Silt .... No. 200 to .005 mm
Clay .... less than .005 mm

Relative Density of Cohesionless Soils

very loose .... 0 to 4 BPF
loose .... 5 to 10 BPF
medium dense................. 11 to 30 BPF
dense............................... 31 to 50 BPF
very dense........................ over 50 BPF

Consistency of Cohesive Soils

very soft.............................. 0 to 1 BPF
soft...................................... 2 to 3 BPF
rather soft .... 4 to 5 BPF
medium............................... 6 to 8 BPF
rather stiff .... 9 to 12 BPF
stiff .... 13 to 16 BPF
very stiff .... 17 to 30 BPF
hard.................................. over 30 BPF

Moisture Content (MC) Description

rather dry	MC less than 5%, absence of moisture, dusty
moist	MC below optimum, but no visible water
wet	MC over optimum, visible free water, typically below water table
saturated	Clay soils were MC over optimum

Drilling Notes

Standard penetration test borings were advanced by 3¼" or 4¼" ID hollow-stem augers, unless noted otherwise. Standard penetration test borings are designated by the prefix "ST" (split tube). Hand auger borings were advanced manually with a 2 to 3" diameter auger to the depths indicated. Hand auger borings are indicated by the prefix "HA."

Sampling. All samples were taken with the standard 2" OD split-tube sampler, except where noted. TW indicates thin-walled tube sample. CS indicates California tube sample.

BPF. Numbers indicate blows per foot recorded in standard penetration test, also known as "N" value. The sampler was set 6" into undisturbed soil below the hollow-stem auger. Driving resistances were then counted for second and third 6" increments and added to get BPF. Where they differed significantly, they were separated by backslash (/). In very dense/hard strata, the depth driven in 50 blows is indicated.

WH. WH indicates the sampler penetrated soil under weight of hammer and rods alone; driving not required.

Note. All tests were run in general accordance with applicable ASTM standards.

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LOG OF BORING

PROJECT: 11-2762
GEOTECHNICAL EVALUATION
Applegate Drive
Helena, Montana

BORING: **ST-1**

LOCATION:
See attached sketch.

DRILLED BY: C. Larsen | METHOD: 3 1/4" HSA, Automatic | DATE: 4/4/11 | SCALE: 1" = 1'

Elev.	Depth 0.0	Symbol	Description of Materials	BPF	WL	MC (%)	Remarks
			FILL: Clayey Sand with Gravel, fine- to coarse-grained, reddish brown, rather dry to moist.				
				42		5.1	Composite subgrade bag sample, ST-1 and ST-2, 1' to 4': LL=26, PL=16, PI=10 P_{200}=27% MDD=131.1 pcf OMC=8.4%
				41		6.2	
	4.5	GM	SILTY GRAVEL with SAND, fine- to coarse-grained, yellowish brown, rather dry, dense. (Alluvium)	43		2.2	LL=NP,PL=NP,PI=NP
	5.5		END OF BORING Water not observed with 4' of hollow-stem auger in the ground. Water not observed to dry cave-in depth of 2' immediately after withdrawal of auger.				

BORING BPF WL MC 2762.GPJ LAGNNN06.GDT 5/10/11

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LOG OF BORING

PROJECT: 11-2762
GEOTECHNICAL EVALUATION
Applegate Drive
Helena, Montana

BORING: **ST-2**

LOCATION:
See attached sketch.

DRILLED BY: C. Larsen | METHOD: 3 1/4" HSA, Automatic | DATE: 4/4/11 | SCALE: 1" = 1'

Elev.	Depth 0.0	Symbol	Description of Materials	BPF	WL	MC (%)	Remarks
			FILL: 8" Clayey Gravel Surfacing over Clayey Sand with Gravel, low plasticity, reddish brown, moist.				Surfacing bag sample 0" to 8": LL=23, PL=15, PI=8 P_{200}=21%
				15/9		8.2	
							Composite subgrade bag sample, ST-1 and ST-2, 1' to 4': LL=26, PL=16, PI=10 P_{200}=27% MDD=131.1 pcf OMC=8.4%
	2.5	GM	SILTY GRAVEL, fine- to coarse-grained, light brown, rather dry to moist, medium dense. (Alluvium)	9/16		7.4	
				29		3.2	P_{200}=14.8%
	5.5		END OF BORING Water not observed with 4' of hollow-stem auger in the ground. Water not observed to dry cave-in depth of 2' immediately after withdrawal of auger.				

BORING BPF WL MC 2762.GPJ LAGNNN06.GDT 5/10/11

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LOG OF BORING

PROJECT: 11-2762 **GEOTECHNICAL EVALUATION** Applegate Drive Helena, Montana	BORING: **ST-3** LOCATION: See attached sketch.

DRILLED BY: C. Larsen	METHOD: 3 1/4" HSA, Automatic	DATE: 4/5/11	SCALE: 1" = 1'

Elev.	Depth 0.0	Symbol	Description of Materials	BPF	WL	MC (%)	Remarks
			FILL: Clayey Gravel Surfacing, fine- to coarse-grained, gray, reddish brown, moist.				
	0.8		FILL: Clayey Sand with Gravel, low plasticity, reddish brown, moist.	24		5.2	Subgrade bag sample 1' to 1.5': LL=27, PL=17, PI=10 P_{200}=42%
	2.0	GC	CLAYEY GRAVEL with SAND, fine- to coarse-grained, reddish brown, moist, medium dense to dense. (Alluvium)	34		6.3	Composite subgrade bag sample, ST-3 and ST-4, 1' to 4': LL=33, PL=17, PI=16 P_{200}=30% MDD=129.2 pcf OMC=9.1%
				25		5.5	
	5.5		END OF BORING Water not observed with 4' of hollow-stem auger in the ground. Water not observed to dry cave-in depth of 1 1/2' immediately after withdrawal of auger.				

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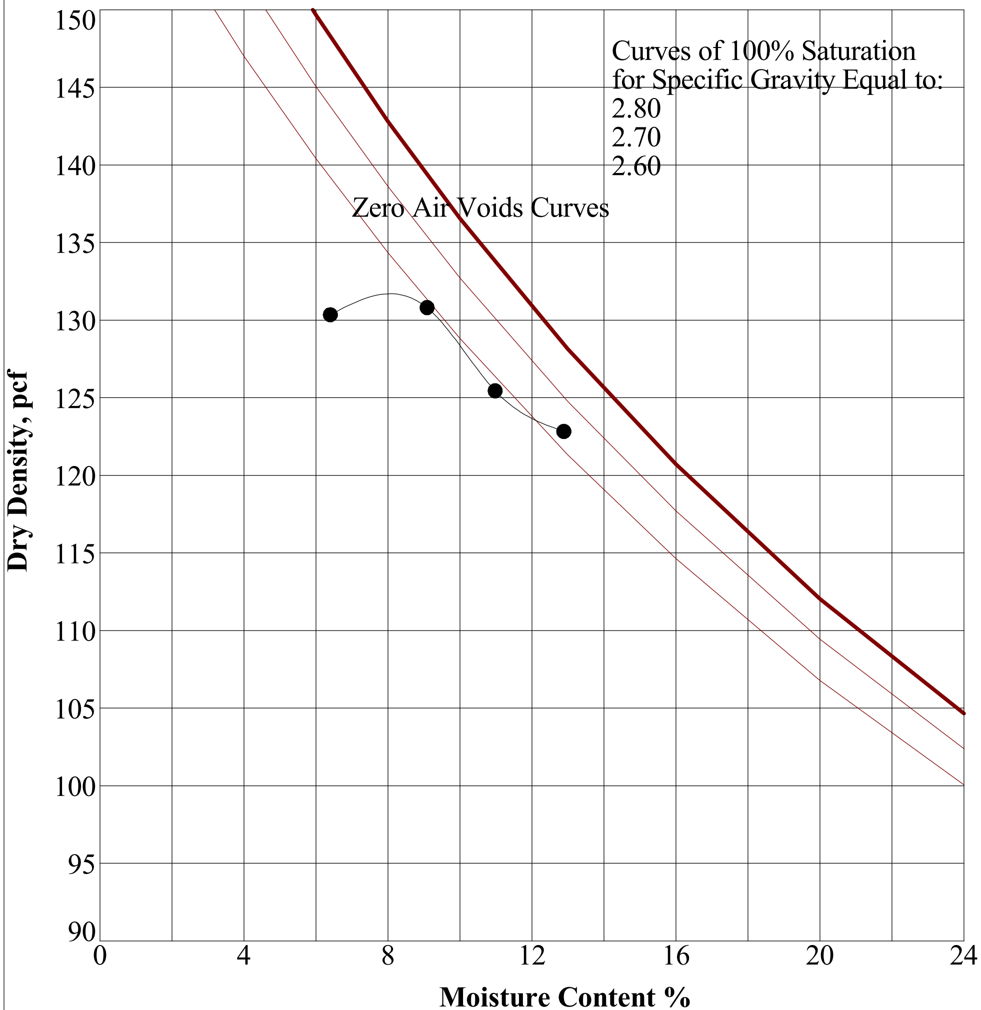
LOG OF BORING

PROJECT: 11-2762 **GEOTECHNICAL EVALUATION** Applegate Drive Helena, Montana	BORING: **ST-4** LOCATION: See attached sketch.

DRILLED BY: C. Larsen	METHOD: 3 1/4" HSA, Automatic	DATE: 4/5/11	SCALE: 1" = 1'

Elev.	Depth 0.0	Symbol	Description of Materials	BPF	WL	MC (%)	Remarks
			FILL: Clayey Sand with Gravel, fine- to coarse-grained, reddish brown, moist.	20		7.6	
	2.5	GM	SILTY GRAVEL with SAND, fine- to coarse-grained, yellowish reddish brown, moist, medium dense to very dense. (Alluvium)	11/23		6.3	Composite subgrade bag sample, ST-3 and ST-4, 1' to 4': LL=33, PL=17, PI=16 P_{200}=30% MDD=129.2 pcf OMC=9.1%
				29/50-4"		7.1	P_{200}=18.1%
	5.5		END OF BORING Water not observed with 4' of hollow-stem auger in the ground. Water not observed to dry cave-in depth of 2' immediately after withdrawal of auger.				

BORING BPF WL MC 2762.GPJ LAGNNN06.GDT 5/10/11



ASTM D 698 Method C

Maximum Dry Density, pcf	Optimum Moisture Content %
131.1	**8.4**

Rammer Type: Mechanical
Preparation Method: Moist

Soil Description (Visual-Manual)

CLAYEY SAND with GRAVEL, fine- to coarse-grained, reddish brown.

Sieve Size	% Retained
1 1/2"	4
3/4"	4.9
3/8"	17
#4	31

Sample No:	---
Lab Sample No:	P-1
Date Sampled:	04/04/2011
Sampled By:	Drill Crew
Date Received:	04/12/2011
Sampled From:	Composite ST-1 and ST-2
Depth:	1' to 4'
Performed by:	MTF, DPH/SKG
Date Performed:	04/14/2011

Comments

Remarks

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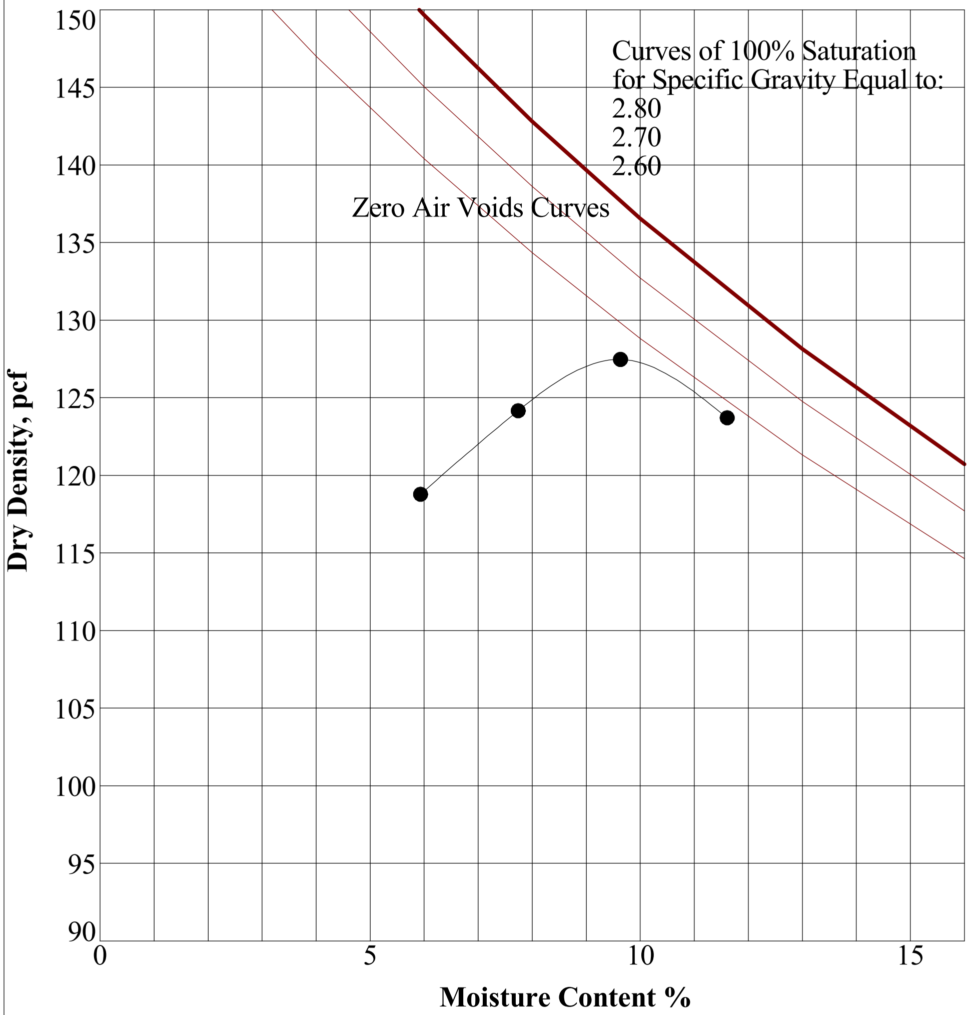
Laboratory Compaction Characteristics of Soil (Proctor)

Project No.: 11-2762
Applegate Drive
Helena, Montana

PROCTOR

P-1

5/10/11



ASTM D 698 Method C

Maximum Dry Density, pcf	Optimum Moisture Content %
127.5	**9.6**

Rammer Type: Mechanical
Preparation Method: Moist

Soil Description (Visual-Manual)

CLAYEY SAND with GRAVEL, fine- to coarse-grained, reddish brown.

Sieve Size	% Retained
1 1/2"	0
3/4"	7
3/8"	23
#4	41

Sample No:	---
Lab Sample No:	P-2
Date Sampled:	04/05/2011
Sampled By:	Drill Crew
Date Received:	04/12/2011
Sampled From:	Composite ST-3 and ST-4
Depth:	1' to 4'
Performed by:	MTF, DPH/SKG
Date Performed:	04/15/2011

Comments

Remarks

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Laboratory Compaction Characteristics of Soil (Proctor)

Project No.: 11-2762
Applegate Drive
Helena, Montana

PROCTOR

P-2

5/10/11

California Bearing Ratio Test

(ASTM D 1883 /AASHTO T 193)

Project: 11-2762 Applegate Drive, Lewis and Clark County
Road Improvements Projects, Helena, Montana

Date: 05/10/11

Boring: ST-1 and ST-2 **Sample:** P-1 **Depth:** 1' to 4'

Sample Description: Clayey Sand with Gravel, fine- to coarse-grained, reddish brown.
(Remolded to 95% relative compaction.)
(Sample was submersed in water and allowed to saturate for 96.0 hours.)

Maximum Dry Density: 131.1 pcf Procedure: ASTM D 698 Method C

Initial			**Final**		
Wt. Specimen + Tare Wet	194.7	gms	Wt. Specimen + Tare Wet	1444.3	gms
Wt. Specimen + Tare Dry	190.0	gms	Wt. Specimen + Tare Dry	1324.5	gms
Wt. Tare	32.6	gms	Wt. Tare	145.2	gms
Moisture Content	3.0%		Moisture Content	10.2%	

Initial Wt. 4629.0 gms Diameter 6.00 in Initial Ht. 4.58 in

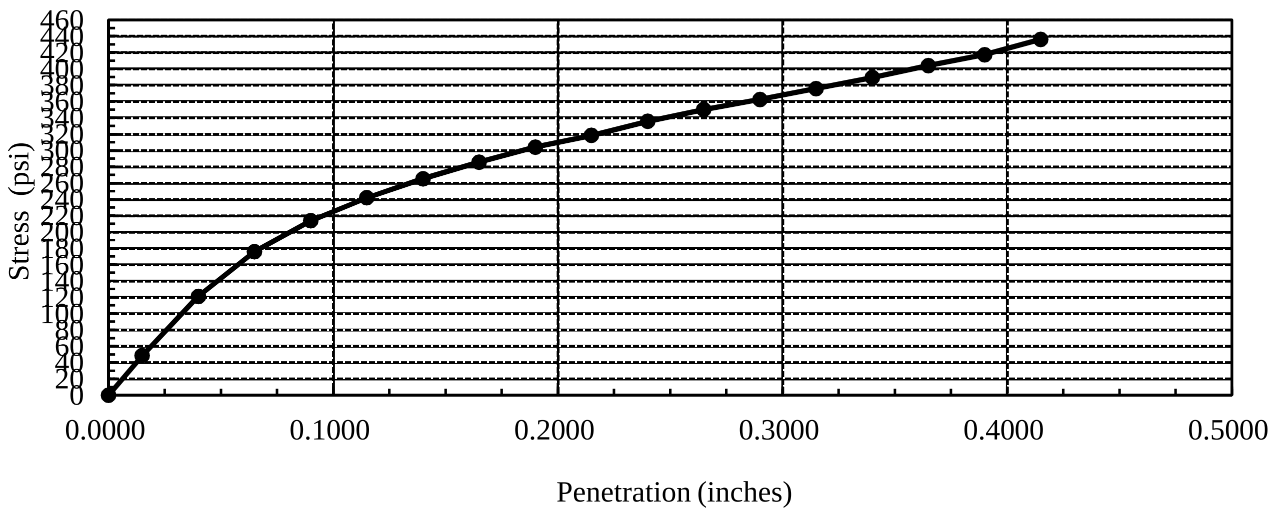
Initial Dry Unit Wt.	132.2	pcf	Initial Relative Compaction	100.9%
Final Dry Unit Wt.	132.1	pcf	Final Relative Compaction	100.8%

Swell Test

Surcharge Weight	22.5	lbs	Surcharge Pressure	133.4	psf		
Initial Dial Rdg.	0.5000		Final Dial Rdg.	0.5050		Swell	0.1%

CBR Test

Surcharge Weight	22.5	lbs	Surcharge Pressure	128.1	psf
CBR @ 0.1 in.	**21.4**		CBR @ 0.2 in	**20.3**	



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California Bearing Ratio Test

(ASTM D 1883 /AASHTO T 193)

Project: 11-2762 Applegate Drive, Lewis and Clark County
Road Improvements Projects, Helena, Montana

Date: 05/10/11

Boring: ST-3 and ST-4 **Sample:** P-2 **Depth:** 1' to 4'

Sample Description: Clayey Sand with Gravel, fine- to coarse-grained, reddish brown.
(Remolded to 95% relative compaction.)
(Sample was submersed in water and allowed to saturate for 96.1 hours.)

Maximum Dry Density: 127.5 pcf Procedure: ASTM D 698 Method C

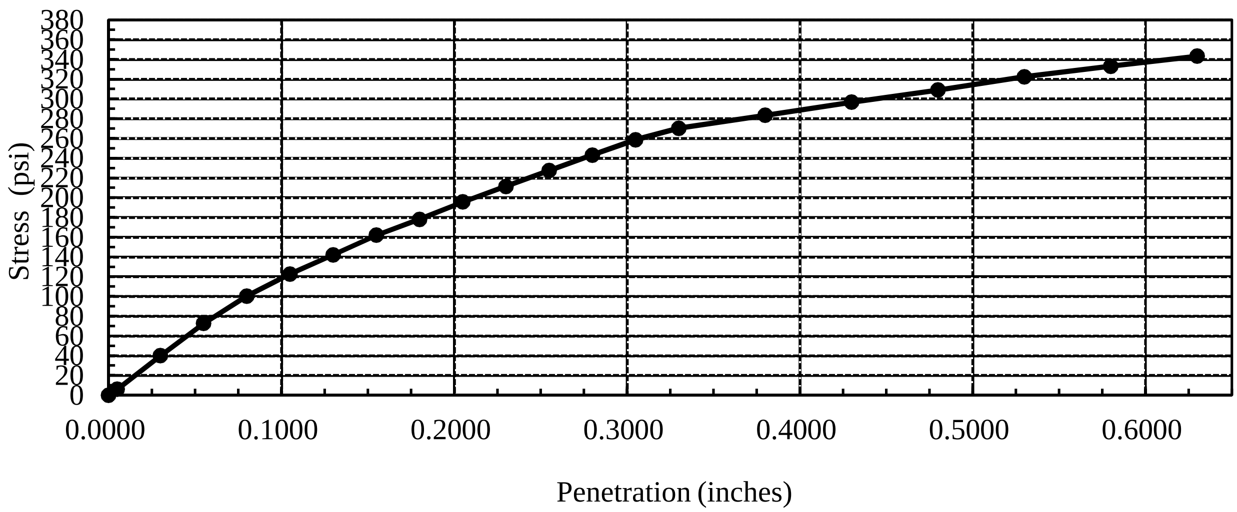
Initial			**Final**		
Wt. Specimen + Tare Wet	977.6	gms	Wt. Specimen + Tare Wet	842.6	gms
Wt. Specimen + Tare Dry	904.8	gms	Wt. Specimen + Tare Dry	775.7	gms
Wt. Tare	149.0	gms	Wt. Tare	160.9	gms
Moisture Content	9.6%		Moisture Content	10.9%	

Initial Wt. 4554.3 gms Diameter 6.00 in Initial Ht. 4.58 in

Initial Dry Unit Wt. 122.2 pcf Initial Relative Compaction 95.8%
Final Dry Unit Wt. 122.2 pcf Final Relative Compaction 95.9%

Swell Test
Surcharge Weight 22.5 lbs Surcharge Pressure 133.4 psf
Initial Dial Rdg. 0.6000 Final Dial Rdg. 0.5990 Swell 0.0%

CBR Test
Surcharge Weight 22.5 lbs Surcharge Pressure 128.1 psf
CBR @ 0.1 in. **10.0** CBR @ 0.2 in **11.9**



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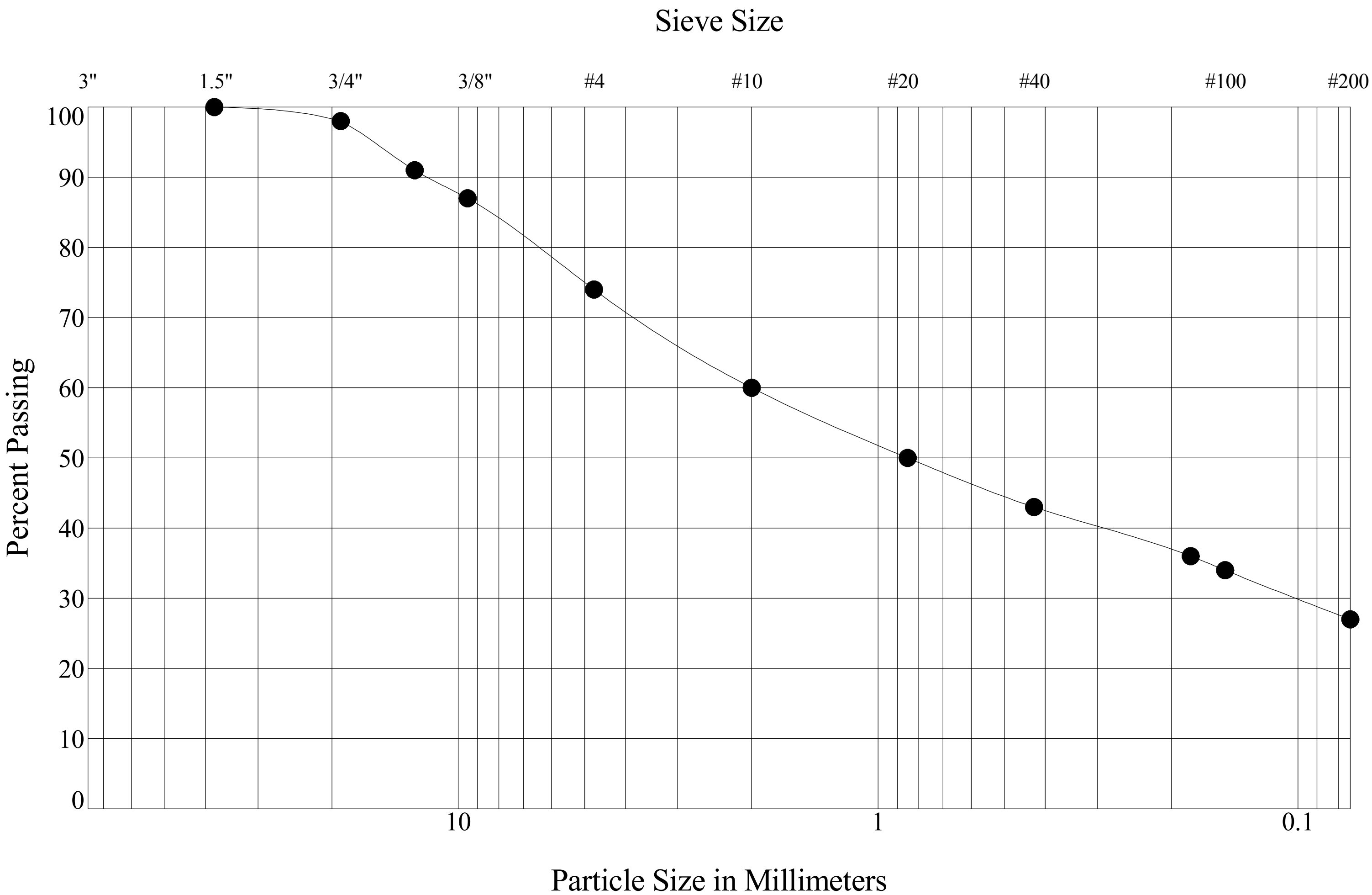

Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
	100	98	87	74	60	50	43	36	34	27

Boring No.: ST-1 & ST-2
Sample No.: P-1
Depth: 1' to 4'

Date Received: 04/12/2011

Liquid Limit:	26
Plastic Limit:	16
Plasticity Index:	10
Classification:	SC
Moisture Content:	3.0%

Percent Gravel: 26.0
Percent Sand: 47.0
Percent Silt + Clay: 27.0
ASTM Group Name: CLAYEY SAND with GRAVEL

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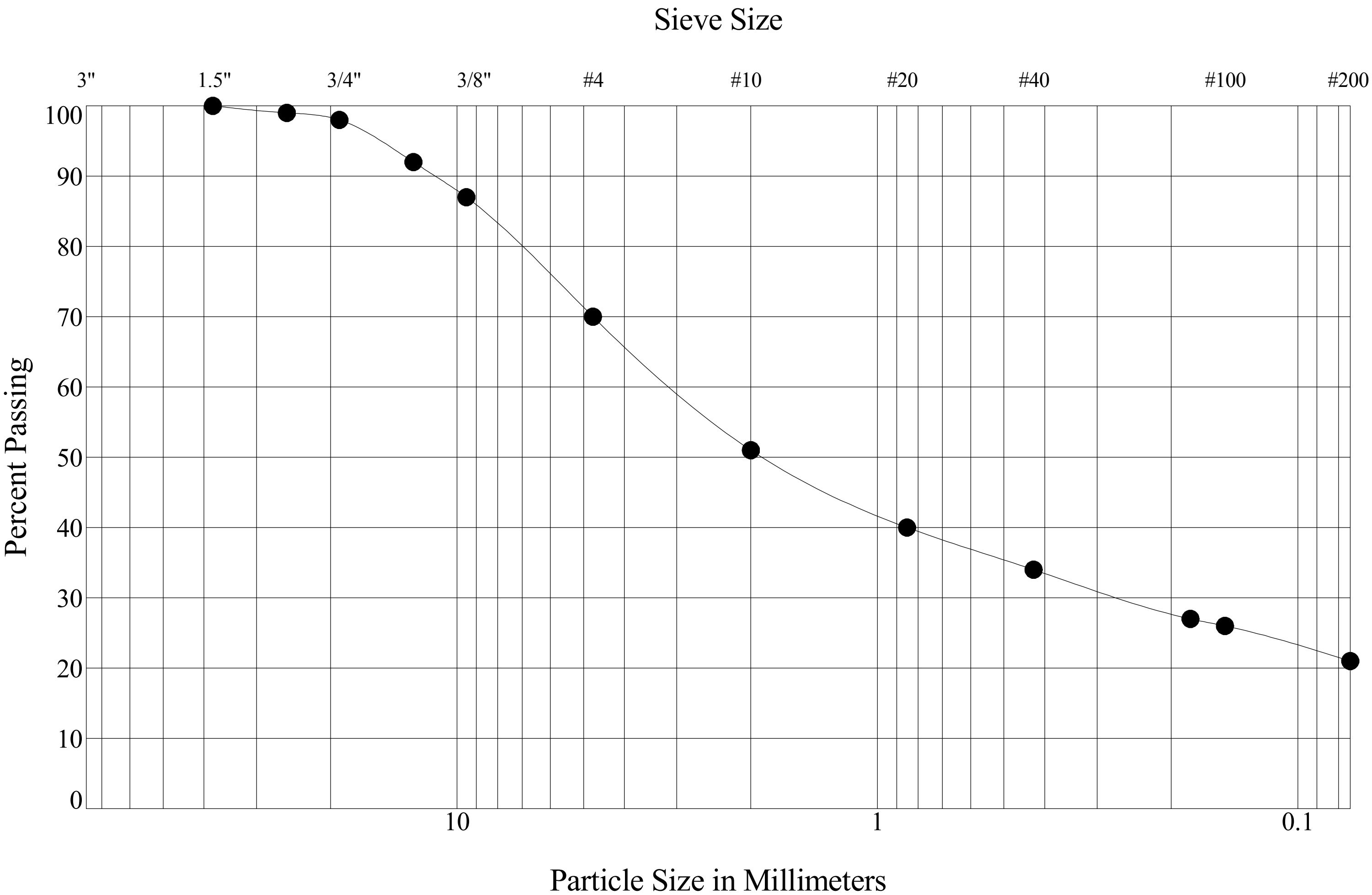
Sieve Analysis
Project Number: 11-2762
Applegate Drive
Helena, Montana

5/10/11



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
	100	98	87	70	51	40	34	27	26	21

Boring No.: ST-2
Sample No.: Bag
Depth: 0" to 8"

Date Received: 04/12/2011

Liquid Limit:	23
Plastic Limit:	15
Plasticity Index:	8
Classification:	SC
Moisture Content:	3.6%

Percent Gravel: 30.0
Percent Sand: 49.0
Percent Silt + Clay: 21.0
ASTM Group Name: CLAYEY SAND with GRAVEL

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Applegate Drive
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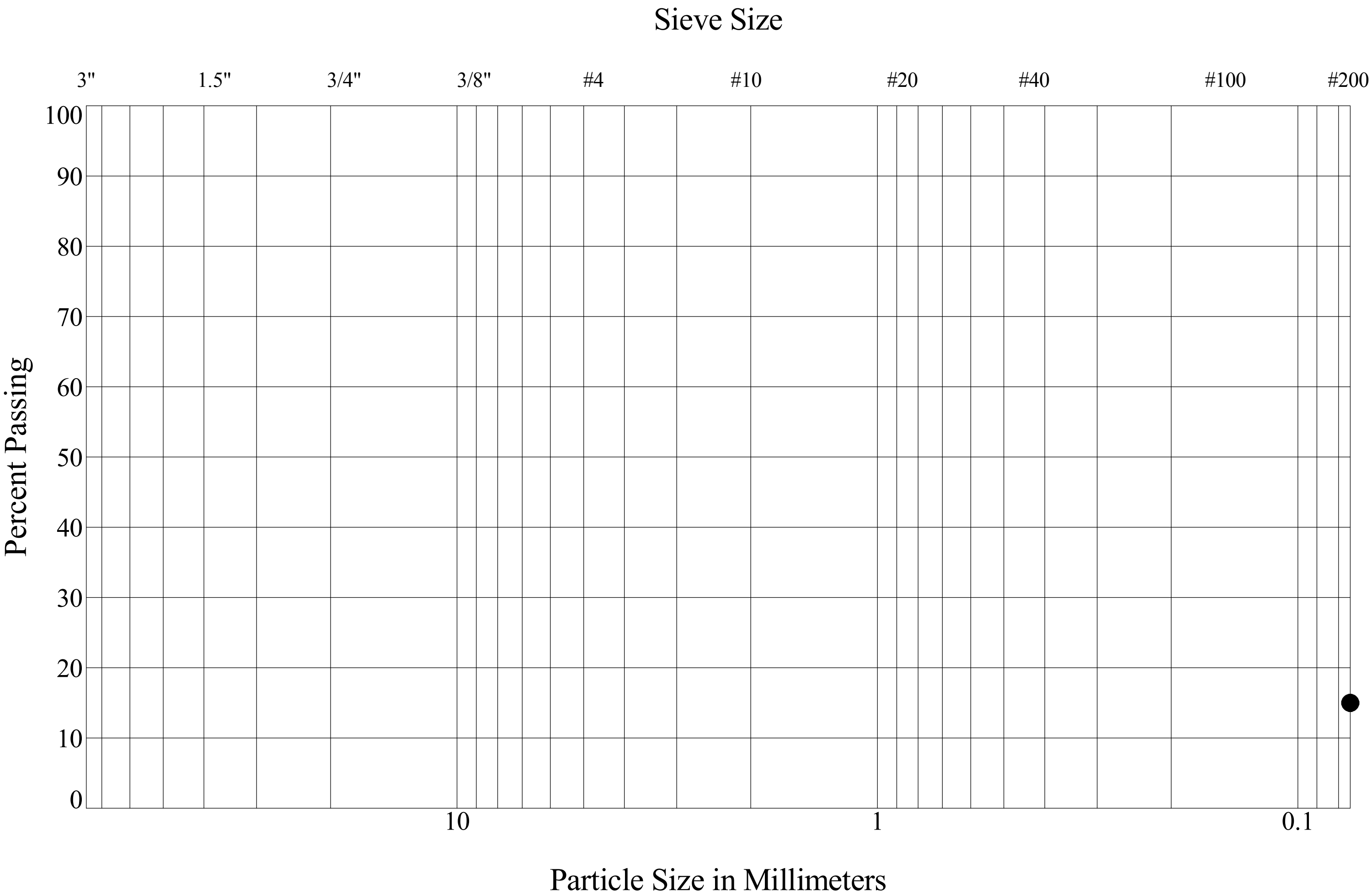
5/10/11



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
										15

Boring No.: ST-2
Sample No.:
Depth: 4.0' to 5.5'

Date Received: 04/12/2011

Percent Gravel: 0.0
Percent Sand: 0.0
Percent Silt + Clay: 15.0
ASTM Group Name:

Liquid Limit:	
Plastic Limit:	
Plasticity Index:	
Classification:	
Moisture Content:	3.2%

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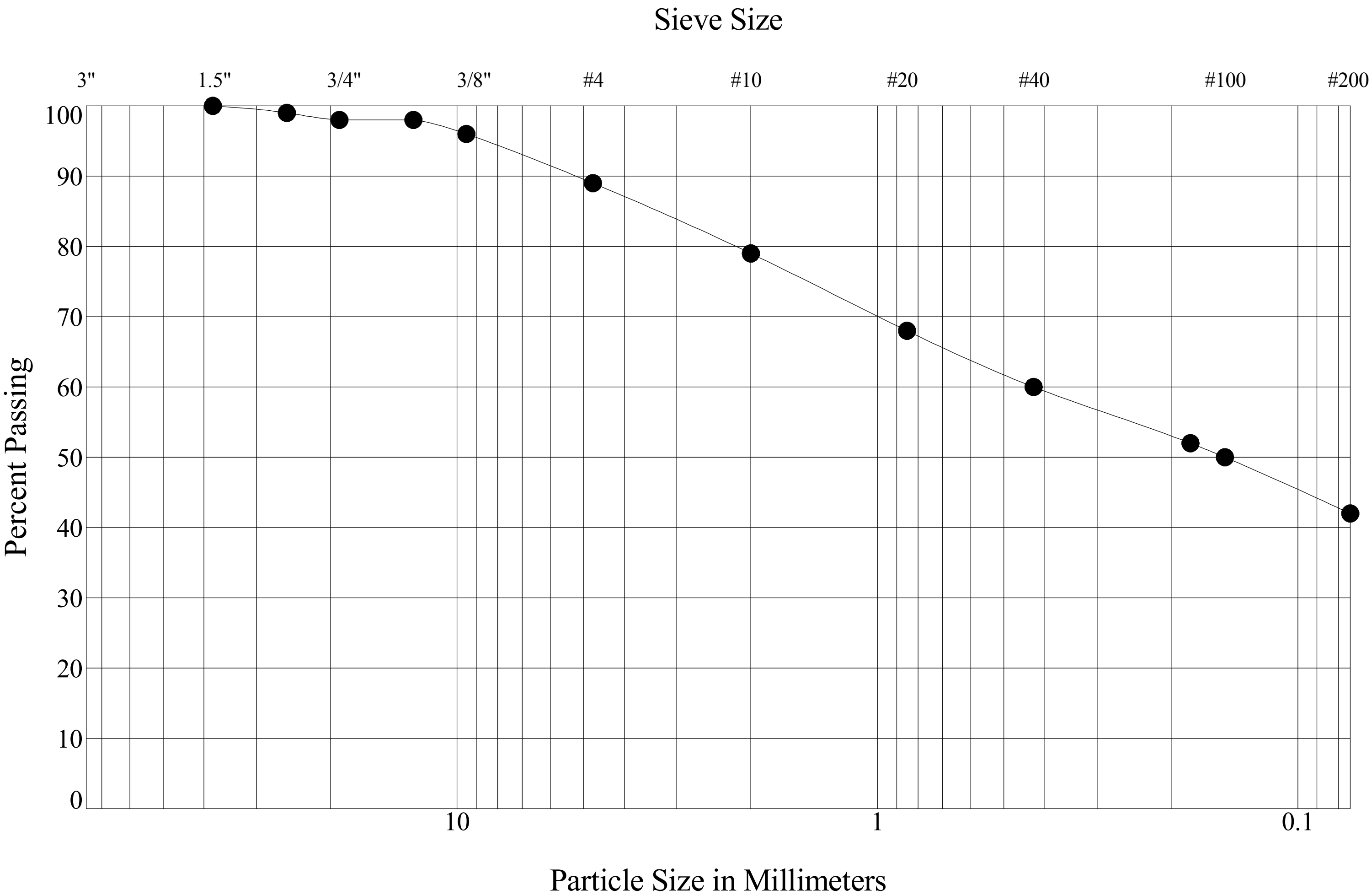
Helena, Montana

5/10/11



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
	100	98	96	89	79	68	60	52	50	42

Boring No.: ST-3
Sample No.: Bag
Depth: 1.0' to 1.5'

Date Received: 04/12/2011

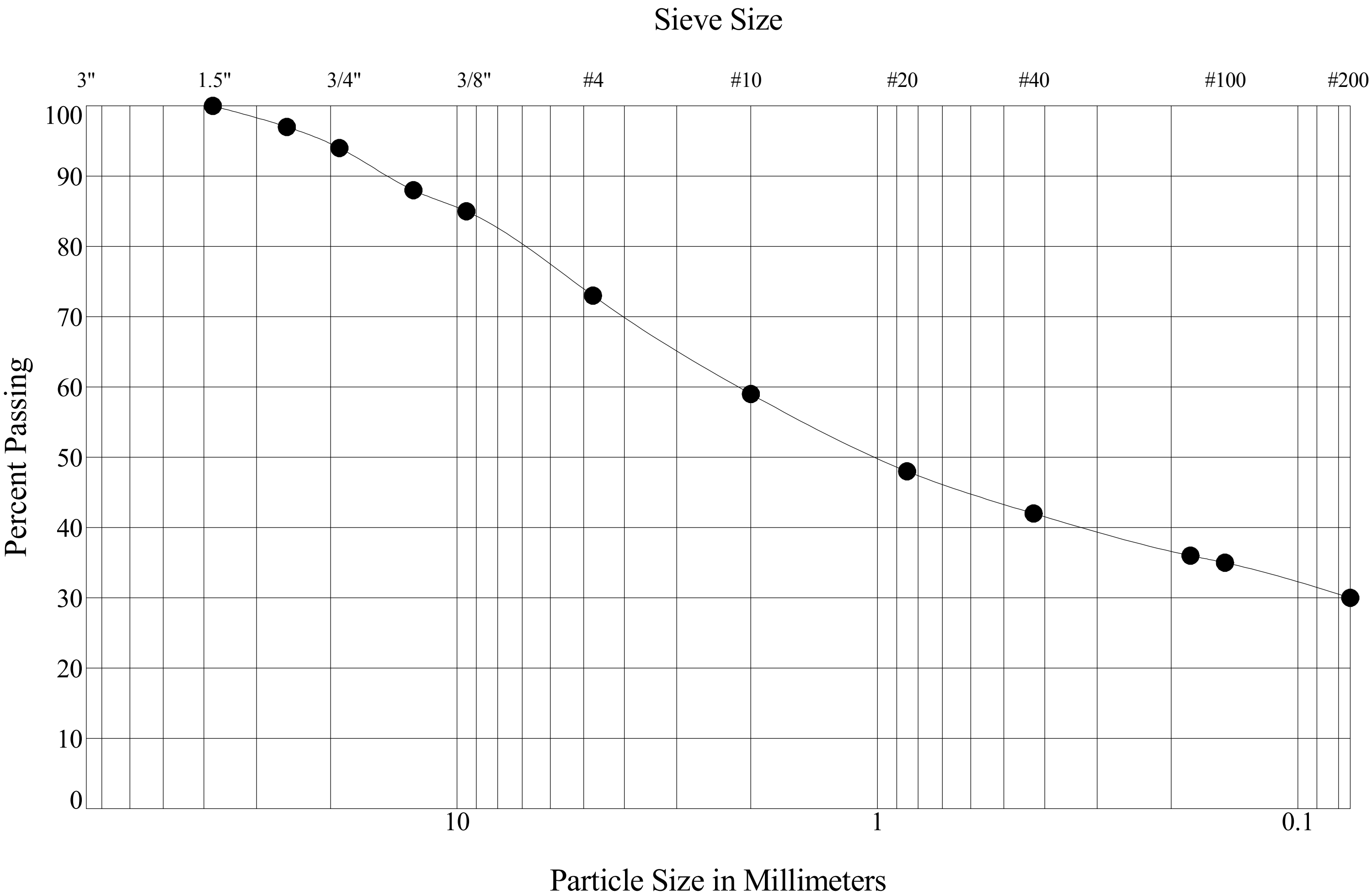
Liquid Limit:	27
Plastic Limit:	17
Plasticity Index:	10
Classification:	SC
Moisture Content:	7.5%

Percent Gravel: 11.0
Percent Sand: 47.0
Percent Silt + Clay: 42.0
ASTM Group Name: CLAYEY SAND

SK GEOTECHNICAL
2611 Gabel Road
P. O. Box 80190
ings, MT 59108-0190
'hone: 406.652.3930
Fax: 406.652.3944

Sieve Analysis
Project Number: 11-2762
Applegate Drive
Helena, Montana

5/10/11



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
	100	94	85	73	59	48	42	36	35	30

Boring No.: ST-3 & ST-4 Date Received: 04/12/2011
Sample No.: P-2
Depth: 1' to 4'

Percent Gravel: 27.0
Percent Sand: 43.0
Percent Silt + Clay: 30.0
ASTM Group Name: CLAYEY SAND with GRAVEL

Liquid Limit:	33
Plastic Limit:	17
Plasticity Index:	16
Classification:	SC
Moisture Content:	4.2%

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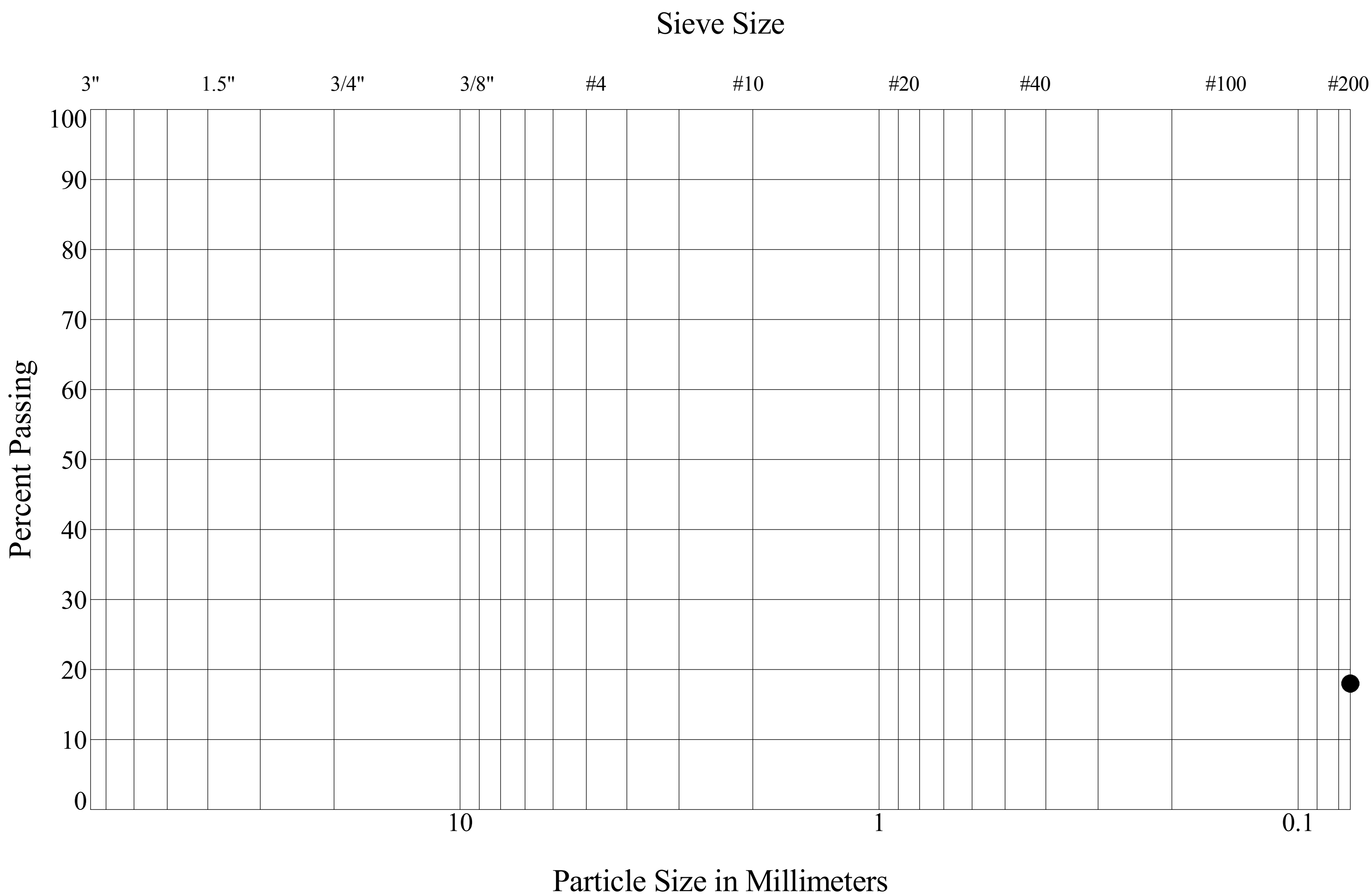
Project Number: 11-2762
Applegate Drive
Helena, Montana

5/10/11



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
										18

Boring No.: ST-4
Sample No.:
Depth: 4.0' - 5.5'

Date Received: 04/12/2011

Percent Gravel: 0.0
Percent Sand: 0.0
Percent Silt + Clay: 18.0
ASTM Group Name:

Liquid Limit:	
Plastic Limit:	
Plasticity Index:	
Classification:	
Moisture Content:	7.1%

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Sieve Analysis
Project Number: 11-2762
Applegate Drive
Helena, Montana

5/10/11

Applegate Drive		AADT			
Site ID	Location	2009	2011	2031	Growth
7A-43	N. of Prairie Rd	309	300	756	4.72%
7A-45	S. of Prairie Rd	313	378	1744	7.94%
7A-46	N. of Valley View	581	647	1608	4.66%
7A-47	S. of Brookings Rd	877	1051	2763	4.95%
7A-48	N. of Lincoln Rd	1240	1175	3035	4.86%
Weighted Average:					**5.17%**

Applegate Drive: Lincoln Road to Brookings Road

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DARWin(tm) - Pavement Design

A Proprietary AASHTOWARE(tm)
Computer Software Product

Flexible Structural Design Module

Project Description
Applegate Drive: Lincoln Road to Brookings Road

Flexible Structural Design Module Data
18-kip ESALs Over Initial Performance Period: 435,985
Initial Serviceability: 4.2
Terminal Serviceability: 2.5
Reliability Level (%): 85
Overall Standard Deviation: .45
Roadbed Soil Resilient Modulus (PSI): 15,000
Stage Construction: 1

Calculated Structural Number: 2.22

Specified Layer Design
Layer: 1
Material Description: Asphalt Pavement
Structural Coefficient (Ai): .41
Drainage Coefficient (Mi): 1
Layer Thickness (Di) (in): 3.00
Calculated Layer SN: 1.23

Layer: 2
Material Description: Crushed Top Surfacing
Structural Coefficient (Ai): .14
Drainage Coefficient (Mi): 1
Layer Thickness (Di) (in): 3.00
Calculated Layer SN: .42

Layer: 3
Material Description: Select Base Course
Structural Coefficient (Ai): .07
Drainage Coefficient (Mi): .9
Layer Thickness (Di) (in): 6.00
Calculated Layer SN: .38

Layer: 4
Material Description: Subbase Course
Structural Coefficient (Ai): .07
Drainage Coefficient (Mi): .9
Layer Thickness (Di) (in): 4.00
Calculated Layer SN: .25

Total Thickness (in): 16.00
Total Calculated SN: 2.28

Simple ESAL Calculation
Initial Performance Period (years): 20
Initial Two-Way Daily Traffic (ADT): 1,175
% Heavy Trucks (of ADT) FHWA Class 5 or Greater: 5
Number of Lanes In Design Direction: 1
Percent of All Trucks In Design Lane (%): 50
Percent Trucks In Design Direction (%): 100
Average Initial Truck Factor (ESALs/truck): 1.39
Annual Truck Factor Growth Rate (%): 0

Annual Truck Volume Growth Rate (%): 4.86
Growth: Simple

Total Calculated Cumulative Esals: 435,985

Applegate Drive: North of Brookings Road.

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DARWin(tm) - Pavement Design

A Proprietary AASHTOWARE(tm)
Computer Software Product

Flexible Structural Design Module

Project Description
Applegate Drive: North of Brookings Road

Flexible Structural Design Module Data
18-kip ESALs Over Initial Performance Period: 102,280
Initial Serviceability: 4.2
Terminal Serviceability: 2.5
Reliability Level (%): 85
Overall Standard Deviation: .45
Roadbed Soil Resilient Modulus (PSI): 15,000
Stage Construction: 1

Calculated Structural Number: 1.73

Specified Layer Design
Layer: 1
Material Description: Asphalt Pavement
Structural Coefficient (Ai): .41
Drainage Coefficient (Mi): 1
Layer Thickness (Di) (in): 3.00
Calculated Layer SN: 1.23

Layer: 2
Material Description: Crushed Top Surfacing
Structural Coefficient (Ai): .14
Drainage Coefficient (Mi): 1
Layer Thickness (Di) (in): 3.00
Calculated Layer SN: .42

Layer: 3
Material Description: Select Base Course
Structural Coefficient (Ai): .07
Drainage Coefficient (Mi): .9
Layer Thickness (Di) (in): 6.00
Calculated Layer SN: .38

Layer: 4
Material Description: Subbase Course
Structural Coefficient (Ai): .07
Drainage Coefficient (Mi): .9
Layer Thickness (Di) (in): .00
Calculated Layer SN: .00

Total Thickness (in): 12.00
Total Calculated SN: 2.03

Simple ESAL Calculation
Initial Performance Period (years): 20
Initial Two-Way Daily Traffic (ADT): 647
% Heavy Trucks (of ADT) FHWA Class 5 or Greater: 3
Number of Lanes In Design Direction: 1
Percent of All Trucks In Design Lane (%): 50
Percent Trucks In Design Direction (%): 100
Average Initial Truck Factor (ESALs/truck): 1
Annual Truck Factor Growth Rate (%): 0

Annual Truck Volume Growth Rate (%): 4.66
Growth: Simple

Total Calculated Cumulative Esals: 102,280

Appendix D

Cost Estimates

Applegate Drive Reconstruction Cost Estimate

Major Work Feature	Unit	Unit Cost	Number of Units					Total Cost
			Typical A	Typical B	Typical C	Typical D	Total	
Survey - Staking and Grade Control	MI	$15,000.00	0.25	0.50	1.25	1.00	3.00	$45,000
Topsoil - Salvage and Place	CY	$4.05	538	1,051	2,628	1,613	5,830	$23,612
Excavation - Unclassified	CY	$5.50	5,386	9,170	20,130	8,413	43,098	$237,039
MPDES Permit Fees	LS	$900.00	1	1	1	1	4	$3,600
Temporary Erosion Control - LS	LS	$4,000.00	1	1	1	1	4	$16,000
Select Base Course	CY	$12.00	1,628	3,255	4,706	2,812	12,400	$148,802
Crushed Top Course	CY	$25.41	510	1,080	2,767	1,749	6,107	$155,176
Aggregate Treatment (Prime)	SY	$0.41	5,013	10,026	25,065	15,951	56,056	$22,983
Asphalt Tack Coat	SY	$0.10	4,853	9,706	24,266	14,549	53,375	$5,337
Chip Seal & Cover	SY	$2.00	4,693	9,387	23,467	14,080	51,627	$103,253
Plant Mix Asphalt Paving	Ton	$81.38	856	1,769	4,508	2,795	9,927	$807,885
Reset Mailbox	Each	$200.83	2	6	16	12	36	$7,129
Traffic Gravel	CY	$19.03	359	717	1,793	1,434	4,302	$81,867
Remove/Reset Signs	Each	$184.30	1	0	2	3	6	$1,106
Interim Striping - Yellow Paint	Gal	$34.18	11	21	53	42	127	$4,331
Final Striping - Yellow Paint	Gal	$34.18	11	21	53	42	127	$4,331
Interim Striping - White Paint	Gal	$34.30	11	21	53	42	127	$4,346
Final Striping - White Paint	Gal	$34.30	11	21	53	42	127	$4,346
Remove Existing Culverts	LF	$6.00	224	616	1,960	1,456	4,256	$25,536
Approach/Relief Drain Pipe - 18/24 In.Dia.	LF	$50.17	224	616	1,792	1,344	3,976	$199,476
Drainage Pipe 24 Inch Dia.	LF	$50.00	0	0	168	112	280	$14,000
Farm Fence - Type Type 5M	LF	$2.25	2,640	5,280	13,200	10,560	31,680	$71,280
Fence Panel	Each	$145.92	8	16	40	32	96	$14,008
Seeding	Acre	$294.16	1.52	3.03	7.58	4.85	16.97	$4,992
Fertilize Seed	Acre	$120.84	1.52	3.03	7.58	4.85	16.97	$2,051
Condition Seedbed Surface	Acre	$221.51	1.52	3.03	7.58	4.85	16.97	$3,759
Subtotal - Construction	**$/Segment**		**$184,022**	**$369,778**	**$893,602**	**$563,845**		**$2,011,247**
Final Engineering, Geotec. & Survey	LS	8.00%	$14,722	$29,582	$71,488	$45,108		$160,900
Construction QA/QC	LS	4.00%	$7,361	$14,791	$35,744	$22,554		$80,450
Contractor Mobilization	LS	5.00%	$9,201	$18,489	$44,680	$28,192		$100,562
Contingency	LS	10.00%	$18,402	$36,978	$89,360	$56,385		$201,125
Traffic Control During Construction	LS	8.00%	$14,722	$29,582	$71,488	$45,108		$160,900
Right-of-Way Appraisals by Agent	Each	$2,000.00	3	11	23	0	37	$74,000
Right-of-Way Acquisition by Agent	Each	$1,500.00	3	11	23	0	37	$55,500
Purchase Right-of-Way	Acre	$32,000.00	0.61	1.21	3.03	0.00	4.85	$155,152
Total Estimated Cost (2011)	**$/Segment**		**$ 278,324**	**$ 576,488**	**$ 1,383,833**	**$ 761,191**		**$2,999,835**

Unit Costs are 2010 Estimates. The County may periodically update unit prices.

Additional Alternate Costs

Major Work Feature	Unit	Unit Cost	Number of Units					Total Cost
			Typical A	Typical B	Typical C	Typical D	Total	
Traffic Signal	LS	$68,000.00					1	$68,000
Turn Lane	LS	$100,000.00					1	$100,000
Sanitary Sewer Main	MI	$211,200.00	0.25	0.50	1.25	1.00	3.00	$633,600
Water Main	MI	$396,000.00	0.25	0.50	1.25	1.00	3.00	$1,188,000
Bicycle/Ped. Path Reconstruction	MI	$77,825.00	0.25	0.50	1.25	1.00	3.00	$233,475