Traffic Impact Study

Vista Ridge Commercial
Erie, Colorado

Prepared for:

State Highway 7 Marketplace, Inc.

Kimley»Horn

T R A F F I C   I M P A C T   S T U D Y

Vista Ridge Commercial

Erie, Colorado

Prepared for
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1.0 EXECUTIVE SUMMARY

Vista Ridge Commercial, a retail and service oriented project, is proposed to be located on the northeast corner of the State Highway 7 (SH-7) and Mountain View Boulevard intersection in Erie, Colorado. It is anticipated that this project will consist of several retail establishments, fast-food restaurants, a high turnover sit down restaurant, and a bank. The total square footage is approximately 110,000 square feet with approximately 76,000 square feet of retail shopping, 14,200 square feet of fast-food restaurants, a 7,000 square foot bank, a 5,600 square foot high turnover sit down restaurant, and a 7,000 square foot medical office anticipated. This project is immediately adjacent to another retail development (Erie King Soopers Retail Center) located to the east that includes a proposed 123,000 square foot King Soopers Marketplace, approximately 11,028 square feet of retail space, a gas station with 18 fueling positions, and outlots to include other various retail uses. Analysis was completed for build out of the development in 2018 as well as the 2035 long-term horizon to determine intersection and roadway configurations needed at both planning horizons.

The purpose of this study is to identify project traffic generation characteristics, to identify potential project traffic related impacts on the local street system, and to develop mitigation measures required for the identified impacts. The following five (5) key intersections were included for evaluation within this study:

- SH-7 and Mountain View Boulevard;
- SH-7 and Sheridan Parkway;
- Ridge View Drive and Sheridan Parkway;
- Ridge View Drive and Mountain View Boulevard; and
- Village Vista Drive and Mountain View Boulevard.

The proposed project access intersections along SH-7, Mountain View Boulevard, Sheridan Parkway, and Ridge View Drive were also studied.

Regional access to the project will be provided by Interstate 25 and Northwest Parkway. Primary access to the proposed commercial development will be provided by SH-7 and Mountain View Boulevard. Direct access is proposed at one driveway along SH-7 (to be shared with the Erie King Soopers Retail Center to the east), two driveways along Mountain View Boulevard (one located at the intersection of Village Vista Drive), and two driveways along

Ridge View Drive (with the eastern one shared with Erie King Soopers Retail Center). It is also anticipated that access to this retail development will occur through the Erie King Soopers Retail Center at another driveway along Ridge View Drive and at the full movement access proposed along Sheridan Parkway. The SH-7 access will be restricted to three-quarter movements with the southbound left turn exit restricted. The driveway along Mountain View Boulevard to align with Village Vista Drive will allow full turning movements; while the second access along Mountain View Boulevard, located to the south of the Village Vista Drive intersection, will be restricted to right-in and right-out movements only. All driveways along Ridge View Drive will allow full turning movements.

Full build out of the Vista Ridge Commercial development project is expected to generate approximately 8,808 daily weekday driveway trips. Of these, 660 driveway trips are expected to occur during the morning peak hour, while 825 driveway trips are expected during the afternoon peak hour. Since the project is a commercial development, pass-by trips are expected. These pass-by trips are vehicles already on the street network that will be attracted to the site. The expected pass-by trips to the development results in an anticipated 5,024 weekday daily trips, of which 359 and 489 trips would be new (non pass-by) during the weekday morning and afternoon peak hours, respectively.

Distribution of site traffic on the street system was based on the area street system characteristics, existing traffic patterns, demographic information, anticipated surrounding development areas, and the proposed access system for the project. Assignment of project traffic was based upon the trip generation described previously and the distributions developed. The traffic assignment was added to the background traffic volumes to determine future traffic with the project.

Based on the analysis presented in this report, Kimley-Horn believes the proposed Vista Ridge Commercial project will be successfully incorporated into the existing and future roadway network. Analysis of the existing street network, the proposed project development, and expected traffic volumes resulted in the following recommendations:

2018 Year Improvement Recommendations

- It is recommended that the southbound left-turn lane length at the SH-7/Sheridan Parkway intersection be reduced from 425 feet to 325 feet so that back-to-back left turn storage will be available along Sheridan Parkway between SH-7 and the proposed full movement project access. This length is anticipated to be sufficient to accommodate future left turning traffic volumes.

- It is recommended that a 100-foot northbound left-turn lane be designated along Sheridan Parkway for the proposed full movement access. Since there is approximately 450 feet of back-to-back available storage available between SH-7 and the project driveway, it is recommended that the taper between the left-turn lanes be 25 feet to allow for the recommended 325-foot southbound left-turn lane at SH-7.
- It is recommended that the full movement access on Sheridan Parkway be designated with stop control with a R1-1 “STOP” sign installed on the eastbound exiting approach. The eastbound exiting approach is recommended to be constructed with separate left and right turn lanes. The left-turn lane length recommended is the standard driveway throat depth of 75 feet.
- It is recommended that the northbound left-turn lane at the Ridge View Drive and Sheridan Parkway intersection also be reduced due to the proposed project access location along Sheridan Parkway. This left-turn lane is recommended to be reduced from 350 feet to 150 feet. This turn bay length is anticipated to be sufficient to accommodate future left turning traffic volumes.
- With construction of the project, the east leg of the Village Vista Drive and Mountain View Boulevard intersection will be improved. When the project is constructed, it is recommended that the existing striped full lane width median be redesignated with a 175-foot westbound left turn lane. If possible, it is encouraged that this westbound left turn lane be constructed so that the future 250-foot westbound left turn lane can be designated to accommodate 2035 traffic volumes.
- It is recommended that an eastbound left-turn lane be designated within the full width striped median along SH-7 at the proposed three-quarter movement access. It is recommended that this left-turn lane be designated with a length of 655 feet plus a 220-foot taper (875-foot total length).
- A continuous westbound auxiliary acceleration/deceleration lane exists along State Highway 7 between Sheridan Parkway and Mountain View Boulevard. This existing lane will serve as both an acceleration and deceleration lane for the proposed three-quarter SH-7 project access.
- At the proposed SH-7 three quarter movement access, it is recommended that a R3-2 No Left Turn sign be installed for the southbound approach for motorists exiting the

development. This sign can be installed under the R1-1 “STOP” or R1-2 “YIELD” sign if desired.

- Both access approaches to Ridge View Drive are recommended to be designated with R1-1 “STOP” signs installed on the northbound approach out of the development. The eastern access is anticipated to receive the most traffic and is therefore recommended to have separate left and right lanes. The western access on Ridge View Drive is believed to operate acceptably with shared northbound left turn/right turn lanes.
- It is recommended that the full lane width median along Ridge View Drive be restriped to include a two-way left-turn lane through the proposed project accesses. It is recommended that this be coordinated with Montex North and South developments to provide a coordinated plan for Ridge View Drive.
- The westbound approach exiting the project at the right-in/right-out access along Mountain View Boulevard is recommended to operate with stop control. Therefore it is recommended that a R1-1 “STOP” sign be installed for this approach. In addition, a R3-2 No Left Turn Sign should be installed underneath the STOP sign to identify the turn movement restriction at this access.

2035 Long Term Twenty Year Planning Horizon Improvement Recommendations

- SH-7 may need to be a six-lane roadway by 2035. It is recommended that the westbound right turn deceleration and acceleration lanes from the three-quarter movement project driveway along SH-7 be reconstructed in addition to the three westbound through lanes. Sheridan Parkway may need to be a four-lane (or six-lane) roadway by 2035 as identified within the Amendment to the SH 7 Access Control Plan.
- The intersection of State Highway 7 with Sheridan Parkway is recommended to have dual left-turn lanes on all approaches and right turn lanes for the northbound and southbound directions.
- Upon construction of the dual southbound left-turn lanes at the SH-7 and Sheridan Parkway intersection, it is believed that the turn lane storage bay length can be reduced to 200 feet. This will allow for a 150-foot northbound left-turn lane at the proposed Sheridan Parkway access with a standard 100-foot taper between the back-to-back left-turn lanes along Sheridan Parkway between the proposed full movement access and SH-7.
- If future traffic volumes are realized along Mountain View Boulevard, the intersection of Village Vista Drive and Mountain View Boulevard will warrant and require signalization.

Therefore, the Town of Erie should monitor traffic volumes in the future to determine if and when this improvement is needed.

General Recommendations

- All on-site and off-site roadway improvements should be incorporated into the Civil Drawings, and conform to standards of the Town of Erie, State of Colorado Department of Transportation (CDOT), American Association of State Highway and Transportation Officials (AASHTO) Geometric Design of Highways and Streets, Institute of Transportation Engineers (ITE), and/or the Manual on Traffic Control Devices (MUTCD) – 2009 Edition as appropriate.

2.0 INTRODUCTION

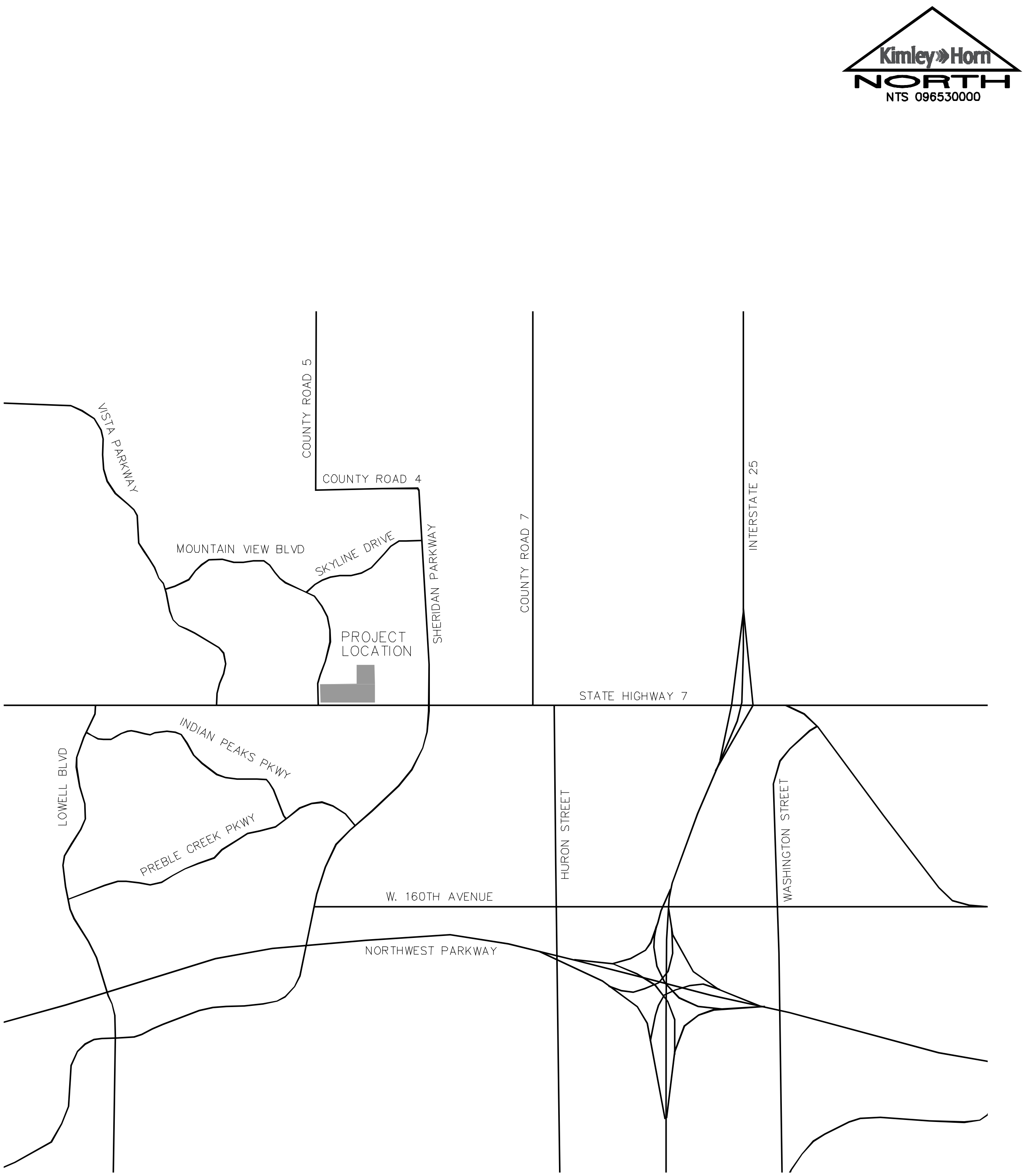
Kimley-Horn and Associates, Inc. (Kimley-Horn) has prepared this report to document the results of a Traffic Impact Study of future traffic conditions associated with the proposed Vista Ridge Commercial project to be located on the northeast corner of the State Highway 7 (SH-7) and Mountain View Boulevard intersection in Erie, Colorado. A vicinity map illustrating the project location with respect to the surrounding area is shown in **Figure 1**.

It is anticipated that this project will consist of several retail establishments, fast-food restaurants, a high turnover sit down restaurant, and a bank. The total square footage is approximately 110,000 square feet with approximately 76,000 square feet of retail shopping, 14,200 square feet of fast-food restaurants, a 7,000 square foot bank, a 5,600 square foot high turnover sit down restaurant, and a 7,000 square foot medical office anticipated. A site plan illustrating the proposed development is provided in **Appendix F**.

Analysis was completed for the anticipated build out of the development in 2018 as well as the 2035 long-term horizon to determine intersection and roadway configurations needed at both planning horizons. The purpose of this study is to identify project traffic generation characteristics, to identify potential project traffic related impacts on the local street system, and to develop mitigation measures required for the identified impacts. The following five (5) key intersections were included for evaluation within this study:

- SH-7 and Mountain View Boulevard;
- SH-7 and Sheridan Parkway;
- Ridge View Drive and Sheridan Parkway;
- Ridge View Drive and Mountain View Boulevard; and
- Village Vista Drive and Mountain View Boulevard.

The proposed project access intersections along SH-7, Mountain View Boulevard, Sheridan Parkway, and Ridge View Drive were also studied.



VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
VICINITY MAP

FIGURE 1

3.0 EXISTING CONDITIONS

The following sections outline existing conditions in the vicinity of the Vista Ridge Commercial project.

3.1 Existing Study Area

The existing project site consists of vacant, undeveloped land. The land directly west of the project site includes retail, restaurants, and a gas station. South of the project site are single family homes, and directly north of the project site is an existing school, Vista Ridge Academy, and vacant land.

The future Erie King Soopers Retail Center development is located immediately east of the project site, and a proposed apartment complex will be located adjacent to the project within the northern area. The apartment complex will include two separate driveways along Ridge View Road, not shared with this project development. The land uses in the general surroundings of the site to the north and west are primarily residential and to the south are vacant/agricultural. Land uses to the east are currently mostly vacant, but residential and commercial uses exist as well.

3.2 Existing Roadway Network

Regional access to the project will be provided by Interstate 25 and Northwest Parkway. Primary access to the proposed commercial development will be provided by SH-7 and Mountain View Boulevard. The roadways adjacent to the proposed project are described within the following paragraphs.

<u>State Highway 7 (SH-7)</u>

State Highway 7 is a four-lane roadway with a striped median and a 55 mile per hour speed limit adjacent to the site. This segment of the roadway travels east-west. Separate left-turn and right-turn lanes have been constructed along SH-7 at both signalized intersections with Mountain View Boulevard and Sheridan Parkway.

<u>Sheridan Parkway</u>

Sheridan Parkway is a two-lane roadway with a double yellow centerline and a 45 mile per hour speed limit adjacent to the site. This segment of the roadway runs north-south. The intersection with SH-7 is signalized, and the intersection with Ridge View Drive operates with stop control on the eastbound Ridge View Drive approach.

Mountain View Boulevard

Mountain View Boulevard is a four-lane roadway with a landscaped median between SH-7 and Ridge View Drive. The roadway primarily provides residential access for areas to the north of the site. This segment of the roadway travels north-south and has a 35 mile per hour speed limit. The intersection of Mountain View Boulevard with SH-7 is a signalized “T”-intersection.

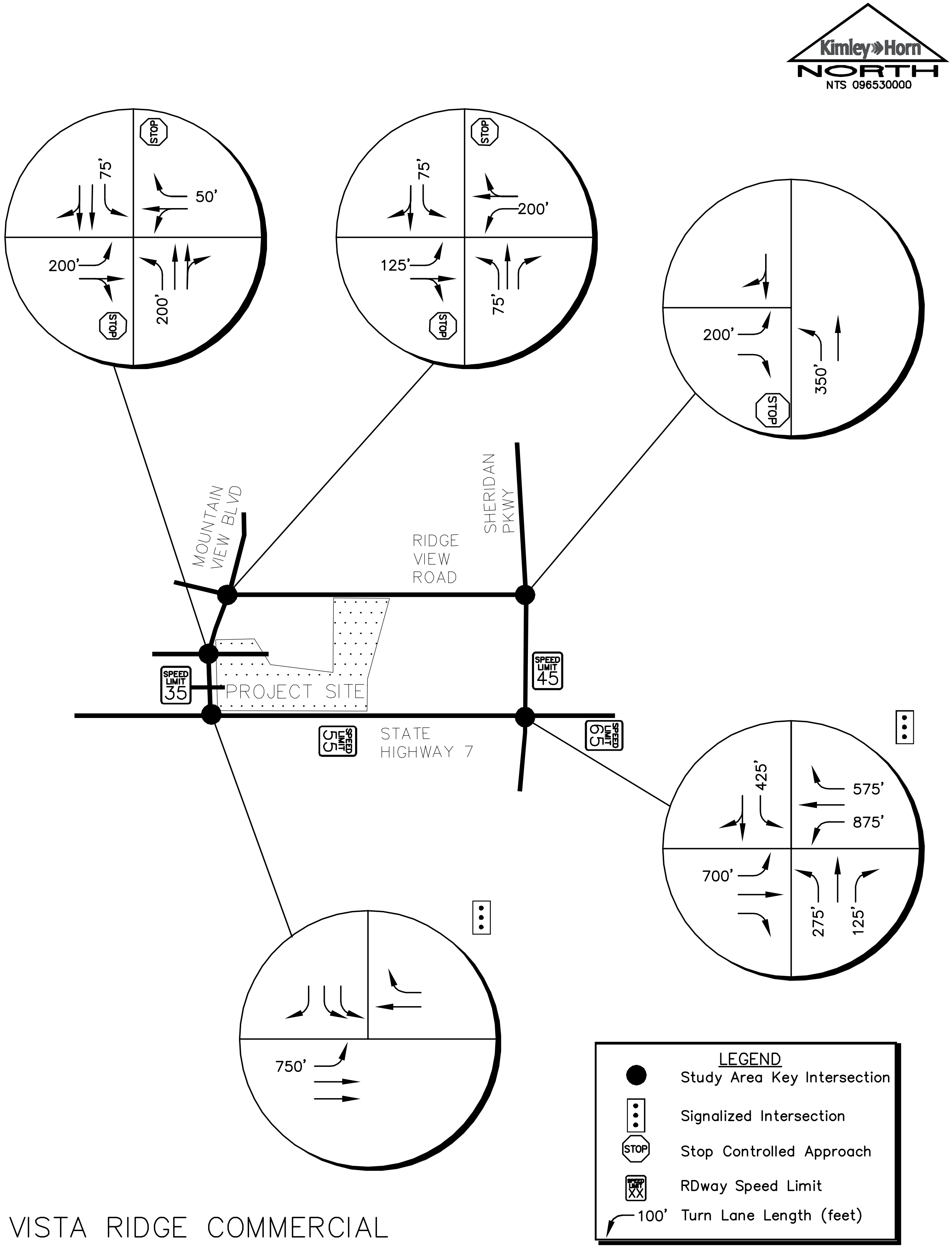
Ridge View Drive

Ridge View Drive is a two-lane, east-west, roadway with a full lane width striped median and a 35 mile per hour speed limit adjacent to the site. The intersection with Sheridan Parkway is a “T”-intersection and operates with stop control on the eastbound approach. Ridge View Drive’s intersection with Mountain View Boulevard is a four-legged intersection with stop control on both the eastbound and westbound approaches. The west leg of this intersection is named Fairway Pointe Drive. For purposes of this study, this intersection is referenced by the name Ridge View Drive and Mountain View Boulevard.

Existing intersection lane configurations and control for the study area are shown in **Figure 2**.

3.3 Existing Traffic Volumes

Existing peak hour turning movement counts were conducted at the study key intersections on Wednesday, October 15, 2014 and September 24, 2015 for the morning (AM) and afternoon (PM) peak hours. The October 2014 counts were obtained from the “Erie King Soopers #129 Retail Center Traffic Impact Study”, prepared by Kimley-Horn and Associates, Inc. in March 2015. All counts were conducted in 15-minute intervals during the AM peak hour and PM peak hour of adjacent street traffic from 7:00 AM to 9:00 AM and 4:00 PM to 6:00 PM, respectively. The peak hour volumes from these counts are shown in **Figure 3**, and the raw data count sheets are provided in **Appendix A**.



VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
EXISTING LANE CONFIGURATIONS

FIGURE 2

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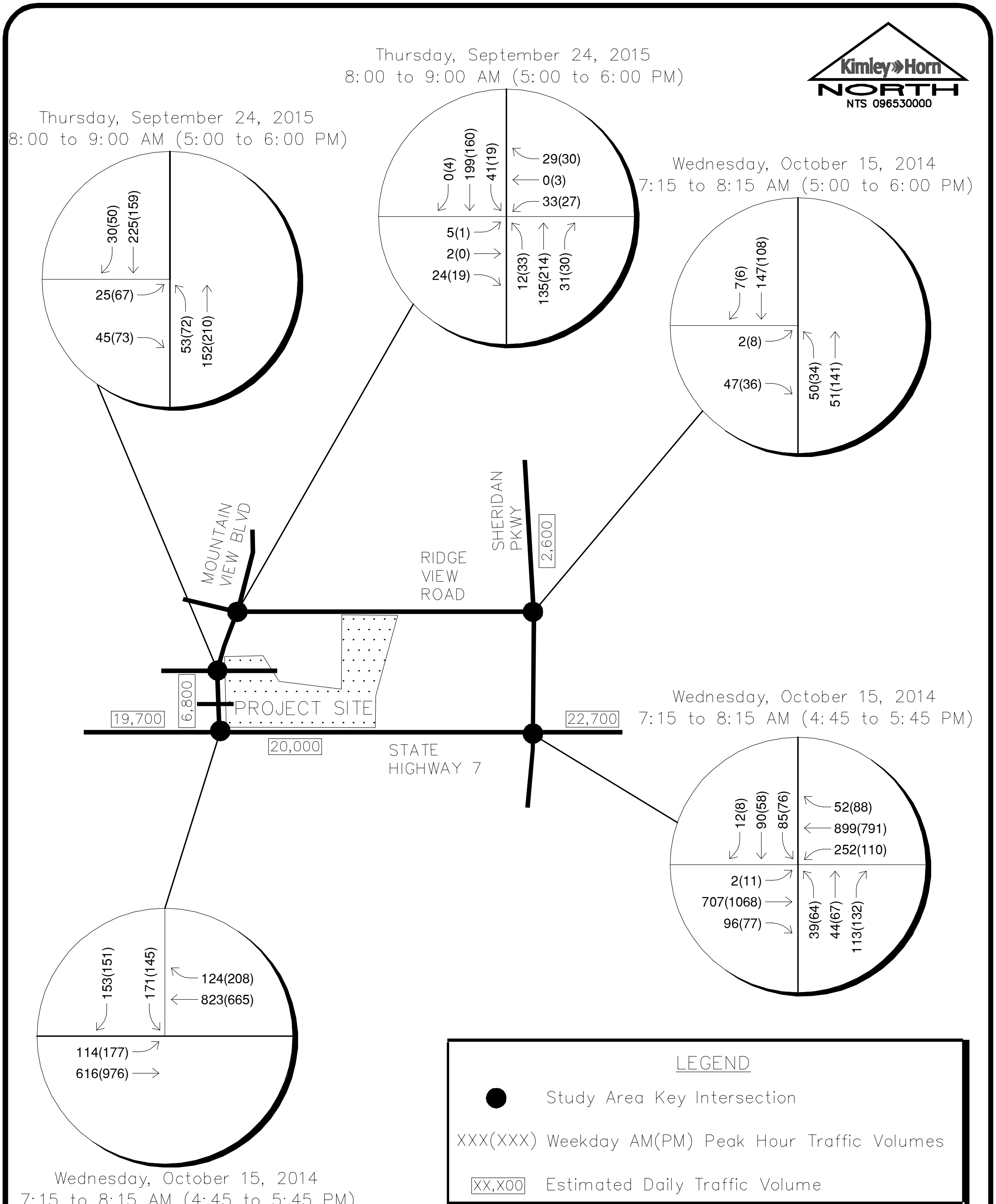

VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
EXISTING TRAFFIC VOLUMES

FIGURE 3

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4.0 FUTURE CONDITIONS

This section of the report details conditions that are expected with the development of the Vista Ridge Commercial project for both the build out (2018) and twenty-year (2035) horizon years.

4.1 Future Roadway Network

Both SH-7 and Sheridan Parkway are anticipated to be improved in the future. According to the Town of Erie's 2008 Transportation Master Plan, both roadways will need additional through lanes for increased capacity in the future. SH-7 was identified as a six-lane principal arterial with a raised median adjacent to the site on the Capacity Improvements Map (2030 to Build out) within the Transportation Master Plan. Sheridan Parkway was also identified as a six-lane principal arterial with a raised median.

4.2 Proposed Project Access

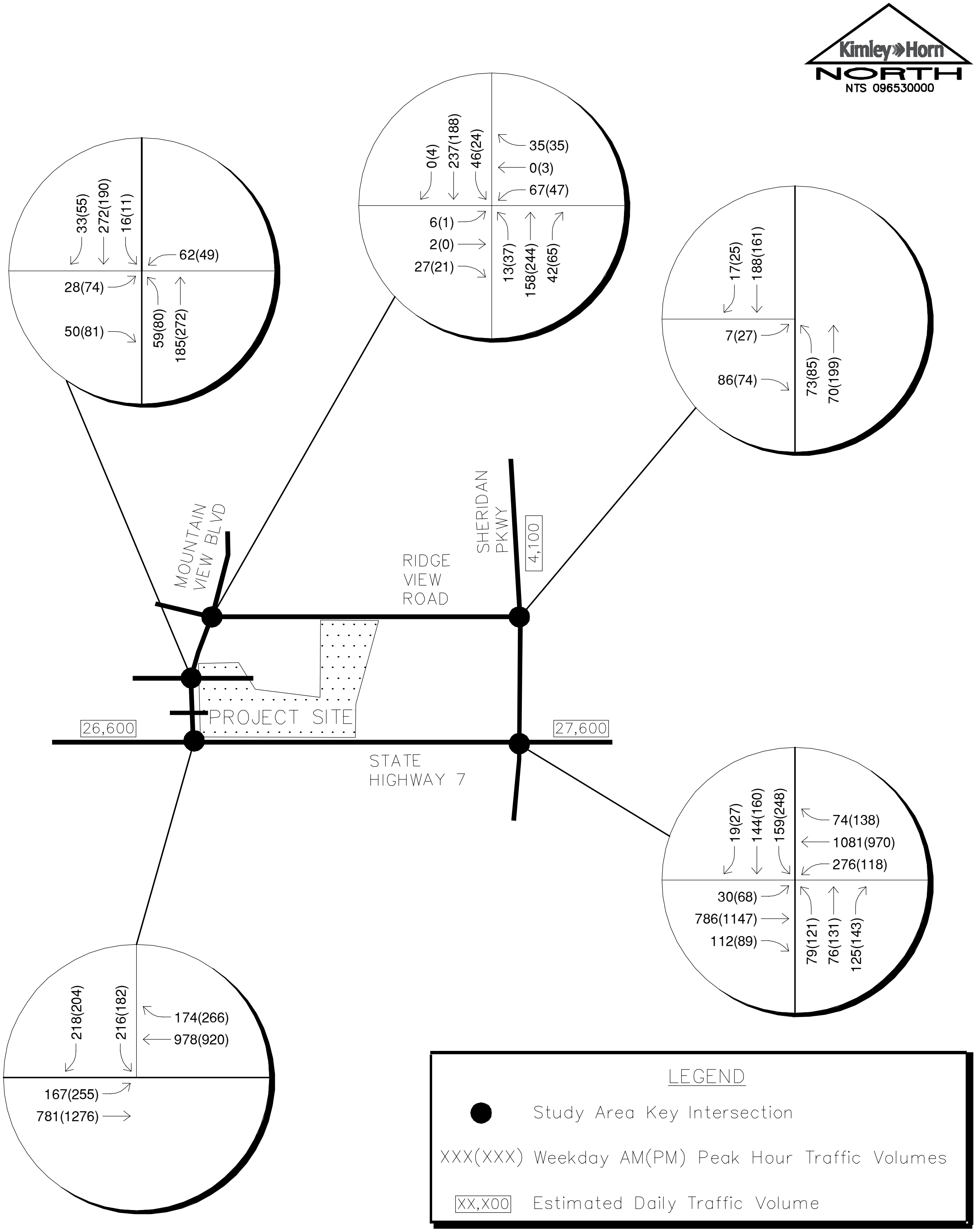
Direct access to Vista Ridge Commercial is proposed at one driveway along SH-7 (to be shared with the Erie King Soopers Retail Center to the east), two driveways along Mountain View Boulevard (one located at the intersection of Village Vista Drive), and two driveways along Ridge View Drive (with the eastern one shared with Erie King Soopers Retail Center). It is also anticipated that access to this retail development will occur through the Erie King Soopers Retail Center at another driveway along Ridge View Drive and at the full movement access proposed along Sheridan Parkway. The SH-7 access will be restricted to three-quarter movements with the southbound left turn exit restricted. The driveway along Mountain View Boulevard to align with Village Vista Drive will allow full turning movements; while the second access along Mountain View Boulevard, located to the south of the Village Vista Drive intersection, will be restricted to right-in and right-out movements only. All driveways along Ridge View Drive will allow full turning movements.

With completion of the project, it is proposed that the private east-west access roadway through the site be a two-lane roadway. Limitations exist to widen the roadway to provide a three-lane section due to setbacks from the apartment complex to the north and State Highway 7 to the south. It is anticipated that a two-lane roadway will operate acceptably, especially since the apartment complex along the north side of the roadway will not have access along this private street. Therefore, left turns will primarily occur from the westbound direction into the Vista Ridge Commercial project outparcels along the SH-7 frontage. The average left turn volume into the accesses along the private street to the north and south is anticipated to be approximately 30 vehicles per hour during the peak with an opposing through volume of approximately 125 vehicles per hour. Based on the “Transportation and Land Development”, 2nd edition, by the Institute of Transportation Engineers (ITE), Figure 5-21 - Suggested Warrants for Isolated Left-Turn Bays, left turn lanes are not warranted along this roadway with an anticipated posted speed limit of 35 miles per hour or less. The opposing through volume along the roadway would need to approximately 220 vehicles per hour to warrant a separate left turn lane based on the projected left turn volumes at the accesses. Therefore, the internal access roadway is believed to be sufficient providing a single through lane in each direction without a left turn lane.

4.3 Future Traffic Volumes

According to the information provided on the website for the Colorado Department of Transportation (CDOT), the 20-year growth factor along SH-7 adjacent to the site is 1.97. These values equate to an annual growth rate of approximately 3.4 percent. SH-7 traffic information from the CDOT Online Transportation Information System (OTIS) website is included in **Appendix B**. The annual growth rate was used to estimate near-term 2018 and long-term 2035 traffic volume projections at the study key intersections.

In addition to this growth rate application, project traffic volumes from the adjacent Erie King Soopers #129 Retail Center project to the east, the 144 apartment units within the northern project site area, and the 11,976 square foot Les Schwab tire and 3,000 square foot fast food restaurant project directly on the northeast corner of the SH-7 and Mountain View Boulevard were included. It should be noted that the Erie King Soopers #129 traffic study included some of this project development in that project. This traffic was removed from the background traffic calculations for these duplicate uses that are now part of this project so that it could be applied as project traffic with this Vista Ridge Commercial project. The calculated background traffic volumes for both 2018 and 2035 are shown in **Figures 4** and **5**, respectively.



VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
2018 BACKGROUND TRAFFIC VOLUMES

FIGURE 4

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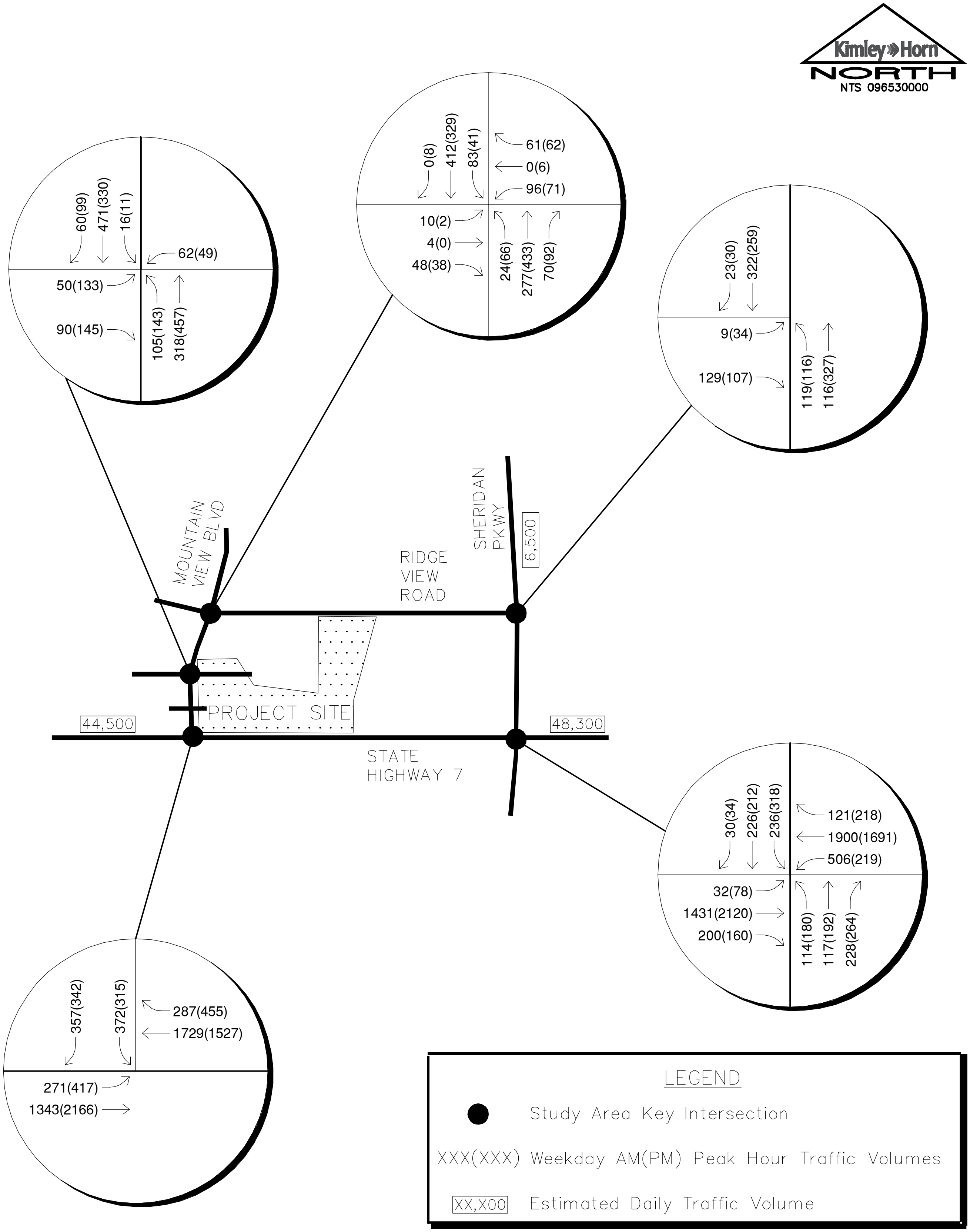

VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
2035 BACKGROUND TRAFFIC VOLUMES

FIGURE 5

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5.0 PROJECT TRAFFIC CHARACTERISTICS

5.1 Trip Generation

Site-generated traffic estimates are determined through a process known as trip generation. Rates and equations are applied to the proposed land use to estimate traffic generated by the development during a specific time interval. The acknowledged source for trip generation rates is the *Trip Generation*[1] report published by the Institute of Transportation Engineers (ITE). ITE has established trip rates in nationwide studies of similar land uses. For this study, the ITE Trip Generation fitted curve and average trip rates that apply to Shopping Center (ITE Land Use Code 820), Drive-In Bank (912), Medical-Dental Office Building (720), High Turnover Sit-Down Restaurant, and Fast-Food Restaurant with Drive-Thru (934) were used to estimate traffic generated by the proposed development.

Since a mix of uses, shopping center (retail), bank, fast-food, and tire store, is proposed within the same development, it is anticipated that traffic will be shared between each use. This internal trip generation, or capture, is most specifically expected to occur between the bank, restaurants, and shopping center (retail) uses. Therefore, the ITE internal capture procedure was used to determine the amount of traffic that may be shared between uses, which thereby reduces the number of external trips.

Based on this, full build out of the Vista Ridge Commercial project is expected to generate approximately 8,808 daily weekday driveway trips. Of these, 660 driveway trips are expected to occur during the morning peak hour, while 825 driveway trips are expected during the afternoon peak hour. Since the project is a commercial development, pass-by trips are expected. These pass-by trips are vehicles already on the street network that will be attracted to the site. The expected pass-by trips to the development results in an anticipated 5,024 weekday daily trips, of which 359 and 489 trips would be new (non pass-by) during the weekday morning and afternoon peak hours, respectively. The internal capture methodology and procedure as well as the pass-by percentages for each use were obtained from the ITE “Trip Generation Manual, Ninth Edition Volume 1, Users Guide and Handbook” 2012. Of note, the afternoon peak hour internal capture and pass-by rates were applied to the morning peak hour and daily as needed

[1] Institute of Transportation Engineers, *Trip Generation: An Information Report*, Ninth Edition, Washington DC, 2012.

as these rates are anticipated to be similar throughout the day. **Table 1** summarizes the estimated traffic generation for proposed development. The trip generation worksheets are included in **Appendix C**. These calculations illustrate the equations used, directional distribution of trips, and number of daily trips based on the published ITE *Trip Generation Report*.

Table 1 – External Project Trip Generation

	Vehicles Trips						
	Daily	Weekday AM Peak Hour			Weekday PM Peak Hour		
		In	Out	Total	In	Out	Total
Non Pass-By Trips							
Shopping Center (820)	2,330	30	9	39	114	124	238
Fast-Food Restaurant with Drive-Thru Window (934)	2,446	151	138	289	86	77	163
High-Turnover Sit-Down Restaurant (932)	54	2	2	4	7	6	13
Drive-In Bank (912)	122	7	3	10	25	23	48
Medical Offices (720)	72	13	4	17	8	19	27
Total	**5,024**	**203**	**156**	**359**	**240**	**249**	**489**
Pass-By Trips							
Shopping Center (820)	1,198	10	3	13	59	64	123
Fast-Food Restaurant with Drive-Thru Window (934)	2,446	145	132	277	86	77	163
High-Turnover Sit-Down Restaurant (932)	32	1	1	2	4	3	7
Drive-In Bank (912)	108	7	2	9	22	21	43
Total	**3,784**	**163**	**138**	**301**	**171**	**165**	**336**
Total Trips	**8,808**	**366**	**294**	**660**	**411**	**414**	**825**

5.2 Trip Distribution

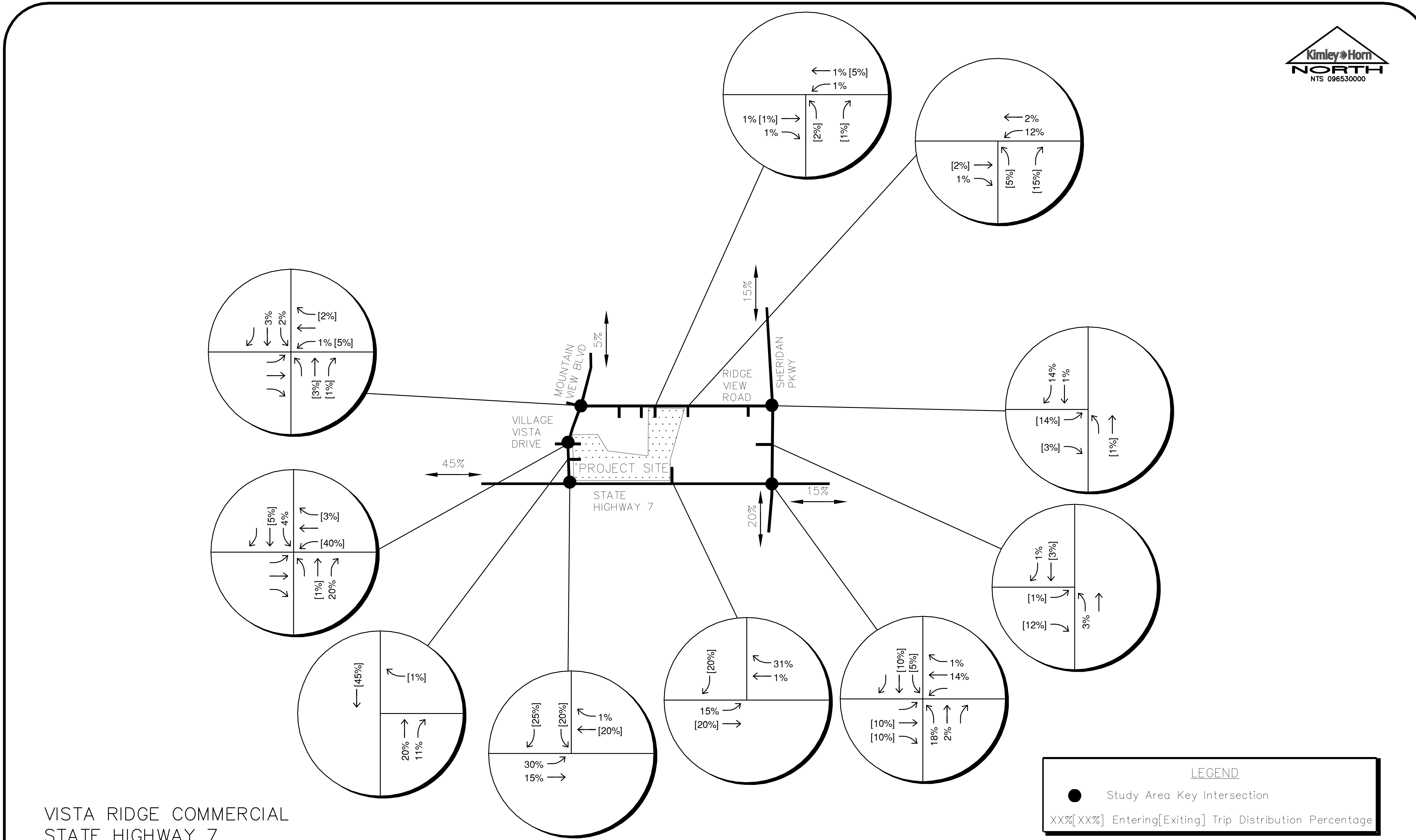
Distribution of site traffic was based on the area street system characteristics, existing traffic patterns and volumes, existing demographic information, and the proposed access system for the project. The non-pass-by directional distribution of traffic is a means to quantify the percentage of site-generated traffic that approaches the site from a given direction and departs the site back to the original source direction. **Figure 6** illustrates the expected non pass-by trip distribution for the site. Due to the nature of the proposed uses, both new (non-pass-by) and pass-by trips are anticipated to be generated by this project. Pass-by distributions capture the route of the vehicle, which is a percentage of traffic driving by the site, arriving from a direction and then continuing in that original direction when leaving. Pass-by distributions are prepared directly based on existing traffic volume counts along the adjacent streets. **Figures 7** and **8**, illustrate the pass-by traffic, calculated separately for the morning and afternoon peak hours, respectively, due to the directional differences of traffic during the peak hours.

5.3 Traffic Assignment

Traffic assignment was obtained by applying the distributions from **Figures 6** through **8** to the estimated traffic generation of the project shown in **Table 1**. The non-pass-by traffic assignment is shown in **Figure 9**. Pass-by traffic assignment is shown in **Figure 10**.

5.4 Total (Background Plus Project) Traffic

Project traffic volumes were added to the background volumes to represent estimated traffic conditions for the short-term 2018 project build out horizon and long-term 2035 horizon. **Figure 11** illustrates the background plus project traffic volumes for the 2018 horizon at the study key intersections and the access intersections proposed with the project. The 2035 background plus project traffic volumes are shown in **Figure 12**.



VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
NON PASS-BY TRIP DISTRIBUTION

FIGURE 6

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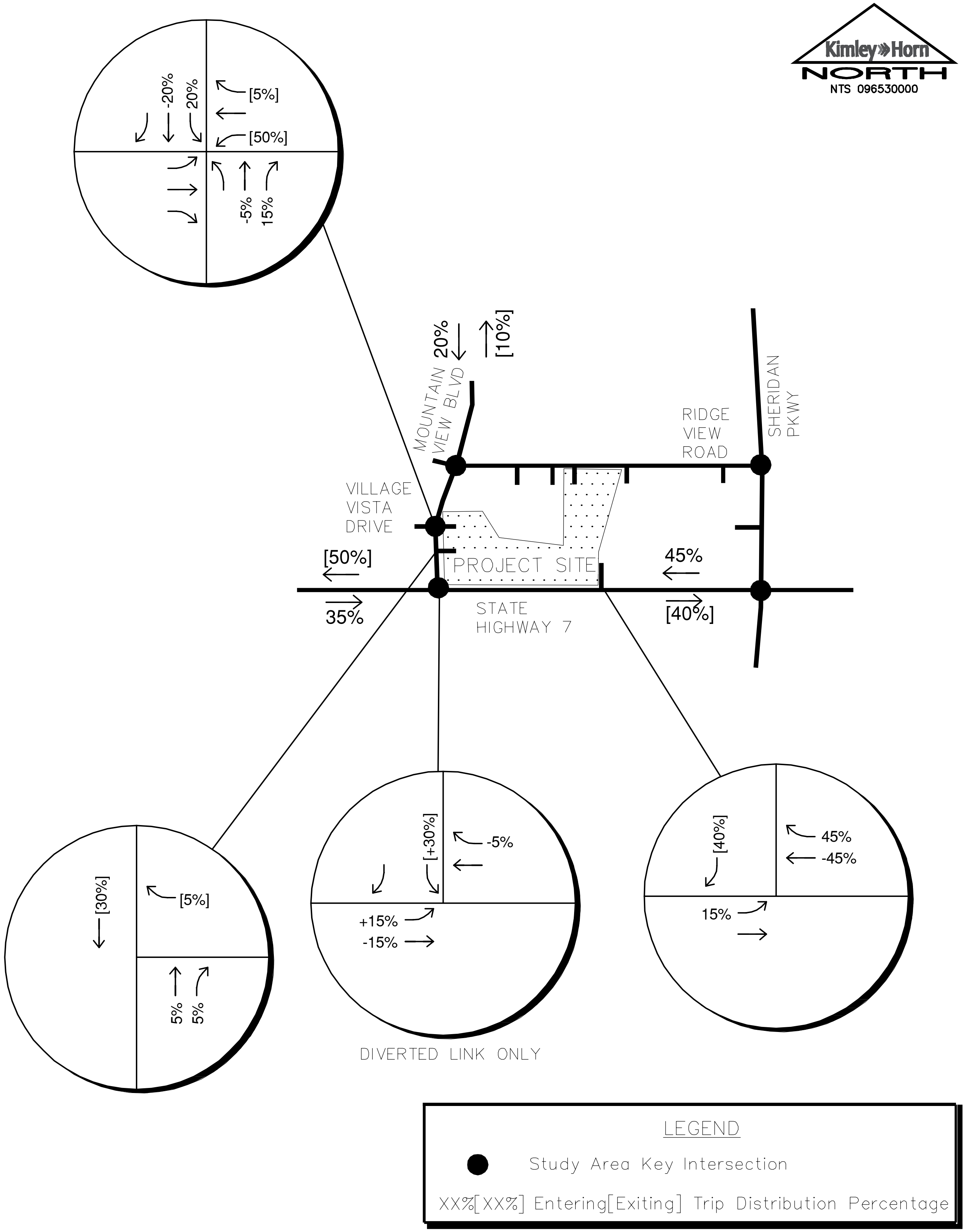

VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
AM PEAK PASS-BY TRIP DISTRIBUTION

FIGURE 7

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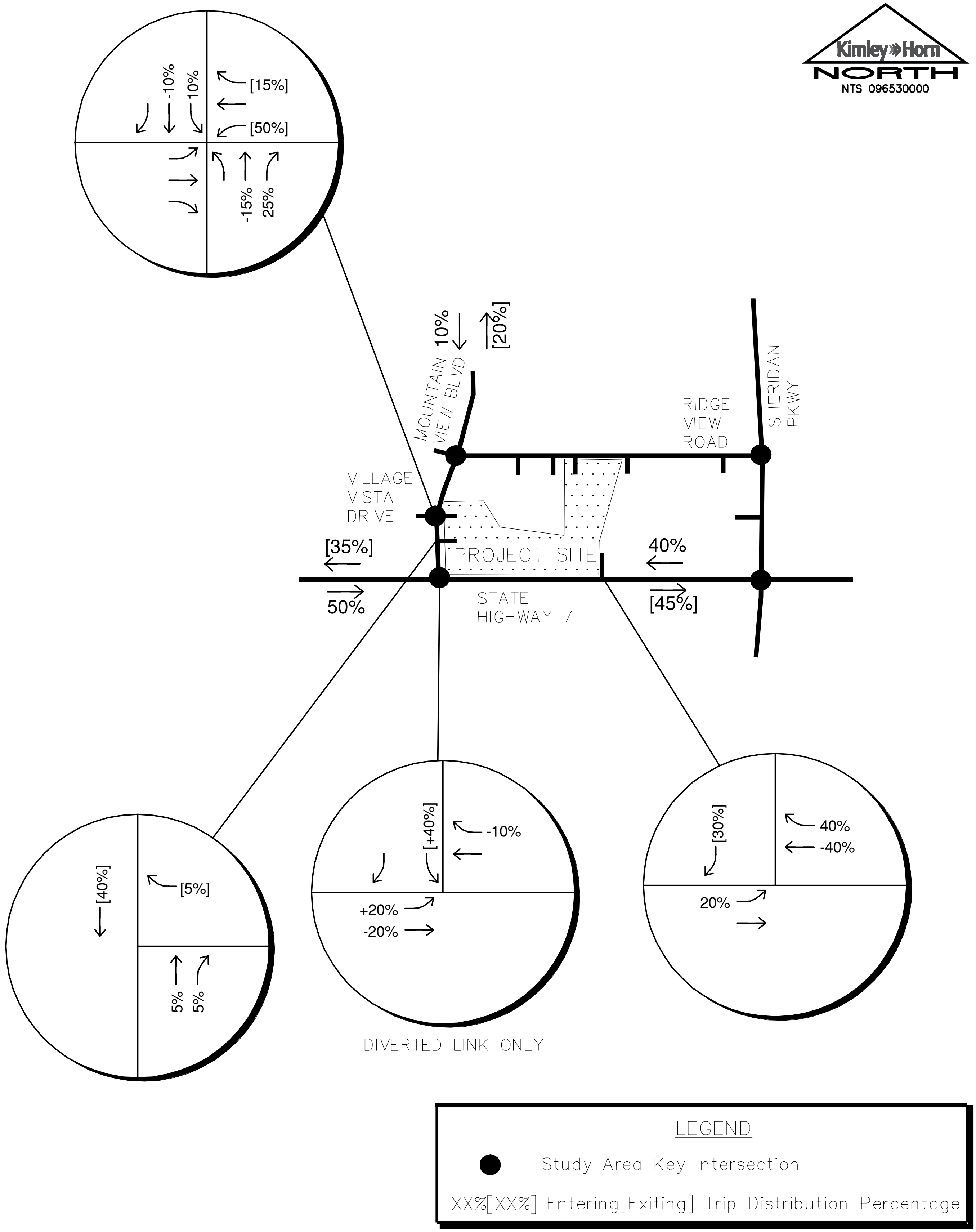
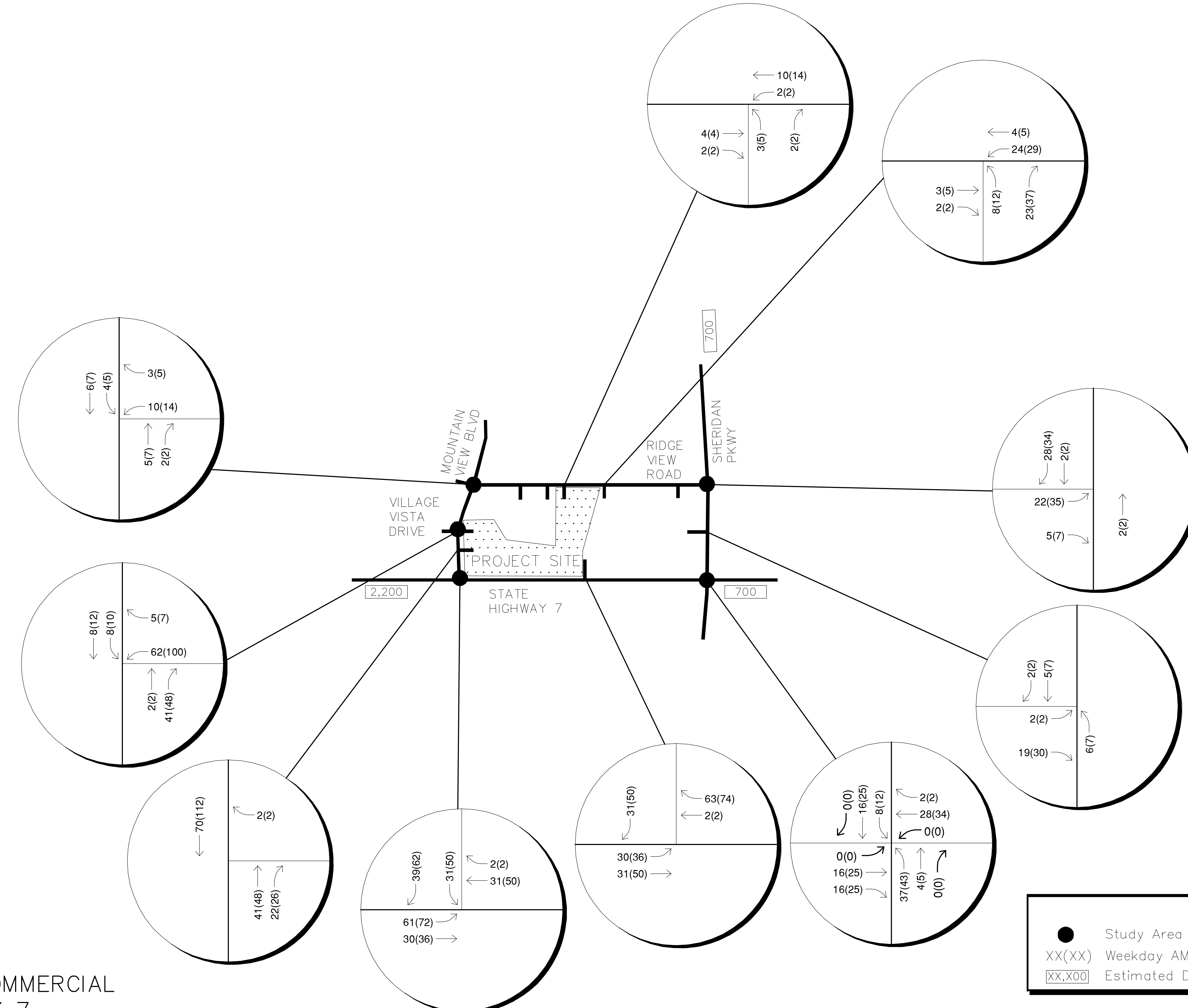


VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
PM PEAK PASS–BY TRIP DISTRIBUTION

FIGURE 8

Kimley»Horn

NORTH
NTS 096530000



LEGEND

●	Study Area Key Intersection
XX(XX)	Weekday AM(PM) Peak Hour Traffic Volumes
XX,X00	Estimated Daily Traffic Volume

VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
PROJECT TRAFFIC ASSIGNMENT

FIGURE 9

Kimley»Horn

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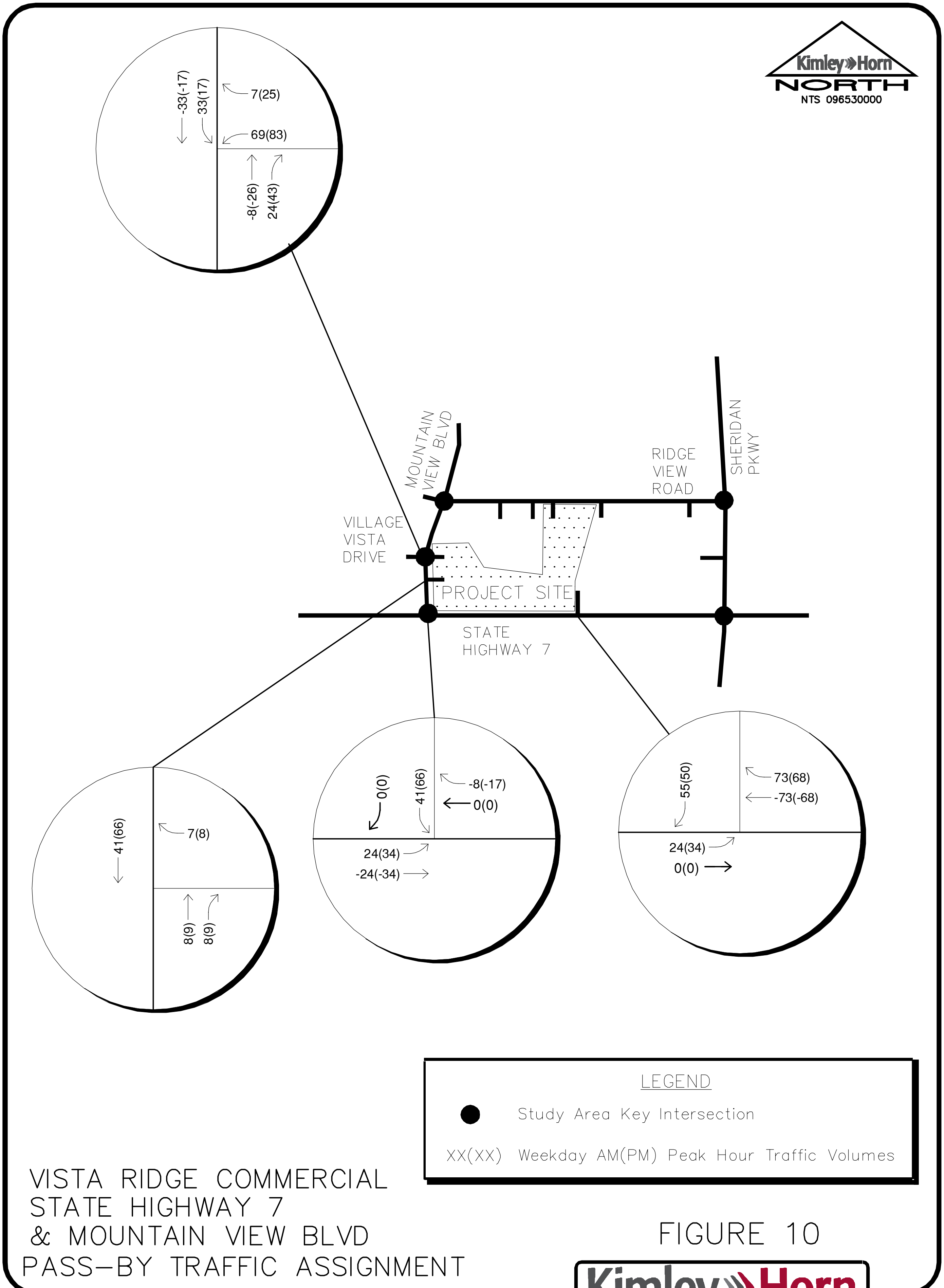

VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
PASS–BY TRAFFIC ASSIGNMENT

FIGURE 10

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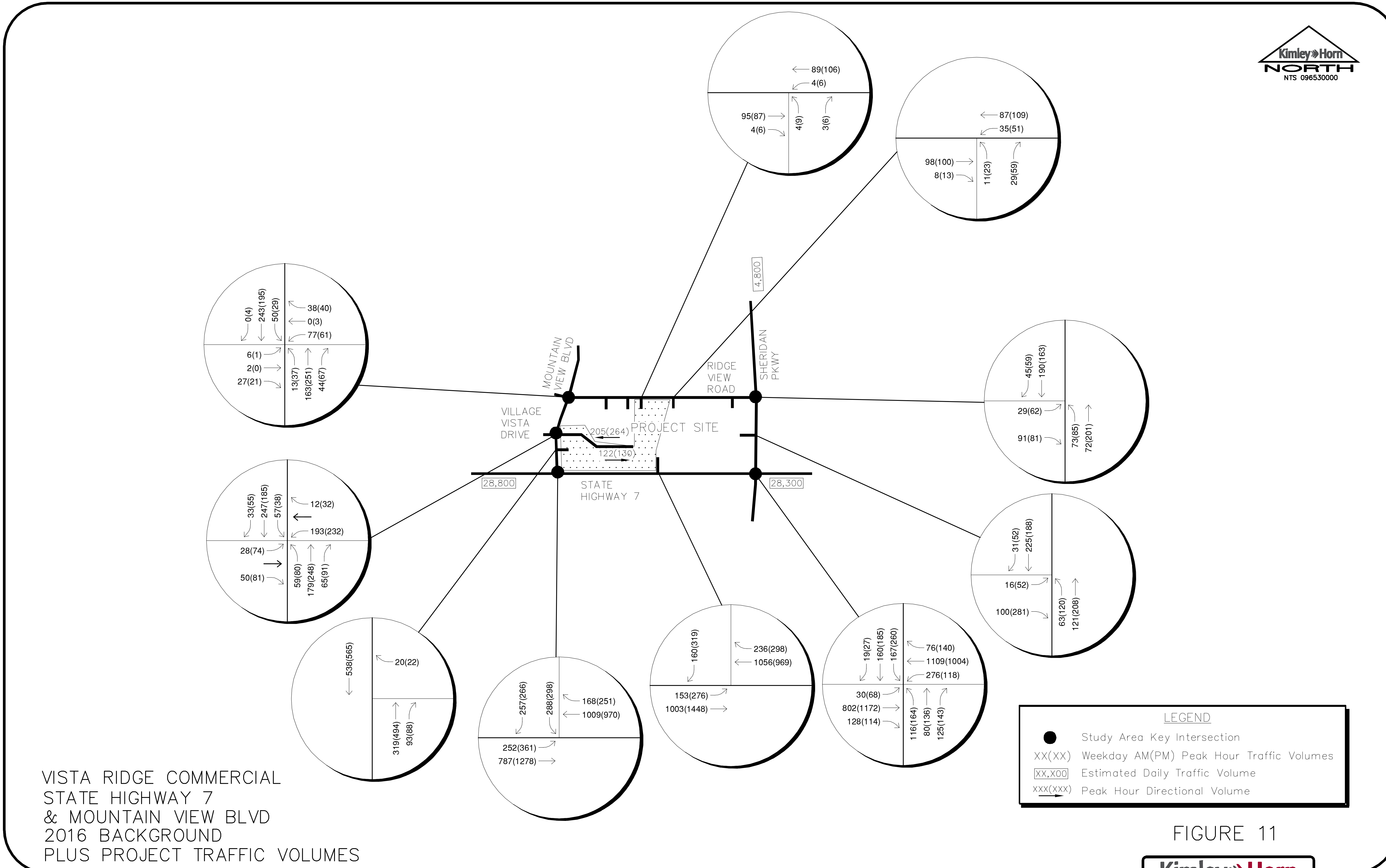

VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
2016 BACKGROUND
PLUS PROJECT TRAFFIC VOLUMES

FIGURE 11

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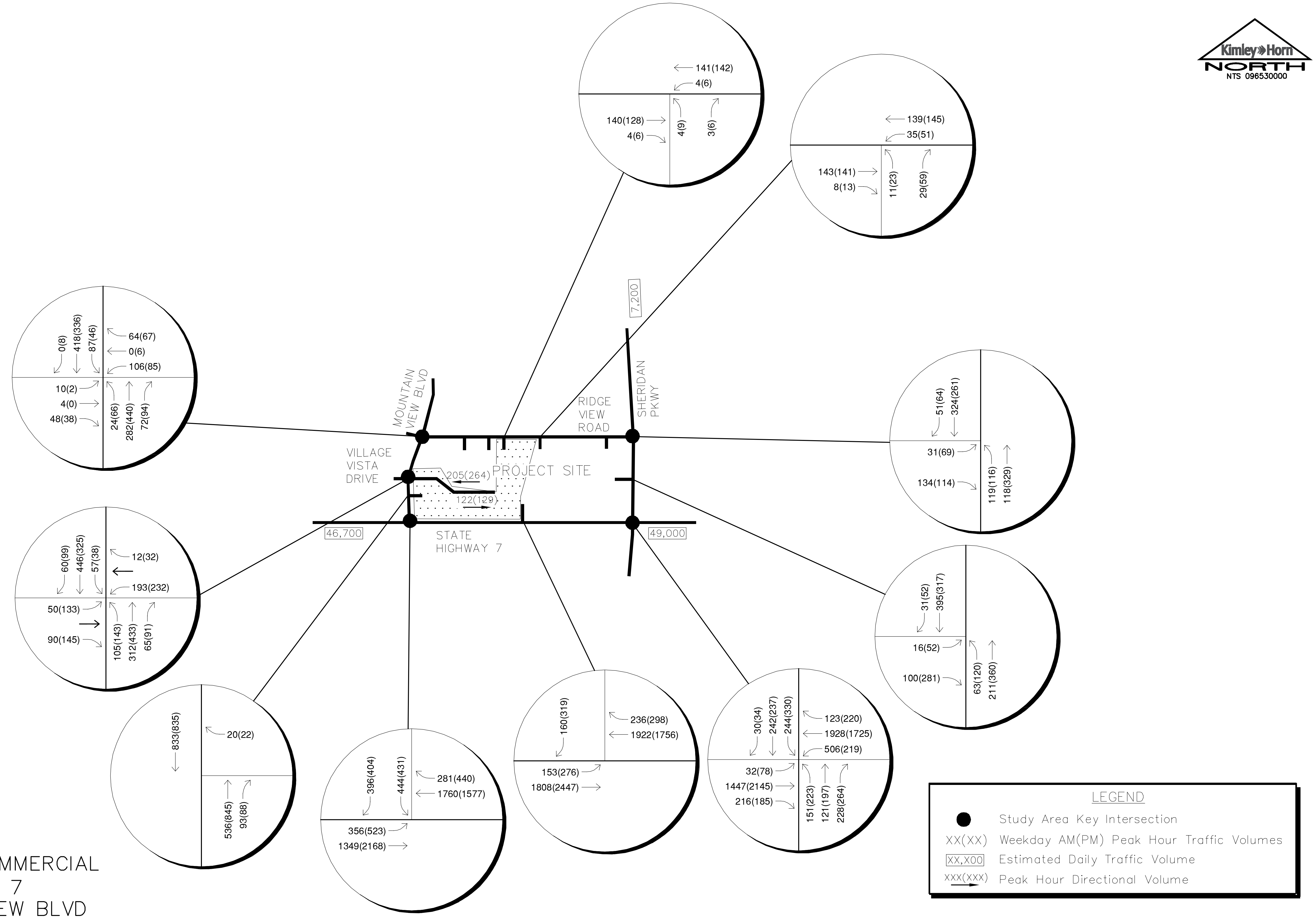

VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
2035 BACKGROUND
PLUS PROJECT TRAFFIC VOLUMES

FIGURE 12

Kimley»Horn

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6.0 TRAFFIC OPERATIONS ANALYSIS

Kimley-Horn's analysis of traffic operations in the vicinity of the site was conducted to determine potential capacity deficiencies in the 2018 and 2035 development horizons at the identified key intersections. The acknowledged source for determining overall capacity is the current edition of the *Highway Capacity Manual*[2].

6.1 Analysis Methodology

Capacity analysis results are listed in terms of Level of Service (LOS). LOS is a qualitative term describing operating conditions a driver will experience while traveling on a particular street or highway during a specific time interval. It ranges from A (very little delay) to F (long delays and congestion). For intersections and roadways in this study area, common traffic engineering practice recommends intersection LOS D and movement LOS E as the minimum desirable threshold for acceptable operations. **Table 2** shows the definition of LOS for signalized and unsignalized intersections.

Table 2 – Level of Service Definitions

Level of Service	Signalized Intersection Average Total Delay (sec/veh)	Unsignalized Intersection Average Total Delay (sec/veh)
A	≤ 10	≤ 10
B	> 10 and ≤ 20	> 10 and ≤ 15
C	> 20 and ≤ 35	> 15 and ≤ 25
D	> 35 and ≤ 55	> 25 and ≤ 35
E	> 55 and ≤ 80	> 35 and ≤ 50
F	> 80	> 50

Definitions provided from the Highway Capacity Manual, Special Report 209, Transportation Research Board, 2010.

Study area intersections were analyzed based on average total delay analysis for signalized and unsignalized intersections. Under the unsignalized analysis, the LOS for a two-way stop controlled intersection is determined by the computed or measured control delay and is defined for each minor movement. LOS for a two-way stop-controlled intersection is not defined for the intersection as a whole. LOS for a signalized and four-way stop controlled intersection is defined for each approach and for the intersection. The intersection analysis was conducted using Synchro software with results reported using the Highway Capacity Manual (HCM) procedure.

[2] Transportation Research Board, *Highway Capacity Manual*, Special Report 209, Washington DC, 2010.

6.2 Intersection Operational Analysis

Calculations for the LOS at the study key intersections are provided in **Appendix D**. The analyses are based on the lane geometry and intersection control shown in **Figure 2**. The existing peak hour factors are also used in the existing and short term horizon (2018) analysis. The analysis determines what improvements may be needed at the intersections and accesses to accommodate background growth and project related traffic in the two study horizons.

<u>SH-7 and Mountain View Boulevard</u>

The existing "T"-intersection of SH-7 and Mountain View Boulevard operates with signal control and a 110-second cycle length. The southbound approach includes dual left-turn lanes and a single right turn lane. The westbound approach contains separate through and right turn lanes. The eastbound approach has one designated left-turn lane with protected permissive phasing and two through lanes. As such, the intersection operates acceptably for the morning and afternoon peak hours. In 2018, the intersection is anticipated to continue operating acceptably, with or without the addition of Vista Ridge Commercial project traffic. By 2035, it is believed that SH-7 through this Mountain View Boulevard intersection would be improved to accommodate future traffic. The SH-7 Access Control Plan identifies SH-7 to be improved to be a six-lane roadway. In addition, it is recommended that the southbound right turn operate with overlap phasing. With this configuration, the intersection is anticipated to continue operating acceptably through the 2035 horizon, with or without the addition of project traffic. **Table 3** provides the results of the level of service analysis conducted at this intersection.

Table 3 – SH-7 and Mountain View Boulevard LOS Results

Scenario	AM Peak Hour		PM Peak Hour	
	Delay (sec/veh)	**LOS**	**Delay (sec/veh)**	**LOS**
2014 Existing	20.5	C	13.5	B
2018 Background	46.1	D	40.9	D
2018 Background Plus Project	54.9	D	53.7	D
2035 Background #	17.8	B	18.3	B
2035 Background Plus Project #	41.4	D	47.8	D

Three Through Lanes EB and WB and SB Right Turn Overlap Phasing

SH-7 and Sheridan Parkway

The existing signalized intersection at SH-7 and Sheridan Parkway operates with traffic signal control and a 110-second cycle length. The eastbound, westbound, and northbound approaches have designated left turn, through, and right turn lanes with protected/permissive phasing for left turn movements. The southbound approach has a shared through/right turn lane and a designated left-turn lane with protected/permissive phasing. With this configuration, the existing intersection operates acceptably at LOS C during the morning peak hour and LOS D during the afternoon peak hour. With the existing configuration, the intersection is anticipated to continue operating acceptably at LOS D during the morning peak hour and LOS E during the afternoon peak hour in 2018 prior to the addition of the proposed development traffic. With the addition of the proposed development traffic in 2018, the intersection is anticipated to operate at a LOS F for the morning and afternoon peak hours. The through volume of traffic utilizing SH-7 is currently nearing capacity with just a single through lane in each direction prior to the addition of project traffic. The roadway will likely need to be improved by CDOT in the near term future to accommodate future traffic with at least two through lanes in each direction. It is recommended that in the interim CDOT consider converting the eastbound and westbound approaches to include two through lanes. This could be accomplished by redesignating and restriping the eastbound and westbound separate right turn lanes to shared through/right turn lanes. With this modification and the addition of project traffic, the intersection is anticipated to operate at LOS C during the morning peak hour and LOS D during the afternoon peak hour. The improvement offered by converting the eastbound and westbound right turn lanes to through lanes demonstrates the capacity issues for traffic traveling along SH-7.

As previously described, it is anticipated that SH-7 will be improved to be six-lane roadway by the 2035 horizon. In addition, Sheridan Boulevard may be a four-lane roadway in the future. Dual left-turn lanes are anticipated to exist on all approaches to the intersection along with northbound and southbound right turn lanes. With this configuration, the intersection is anticipated to operate acceptably, with or without the addition of project traffic in 2035. **Table 4** provides the results of the LOS analysis conducted at this intersection.

Table 4 – State Highway 7 and Sheridan Parkway LOS Results

Scenario	AM Peak Hour		PM Peak Hour	
	Delay (sec/veh)	LOS	Delay (sec/veh)	LOS
2014 Existing	21.9	C	36.2	D
2018 Background	38.5	D	78.1	E
2018 Background Plus Project	94.5	F	99.0	F
2018 Background Plus Project #	31.5	C	40.0	D
2035 Background ##	31.1	C	36.1	D
2035 Background Plus Project ##	35.3	D	52.3	D

Two Through Lanes Eastbound and Westbound

Three Through Lanes EB and WB, Dual Left Turn Lanes all Approaches, NB & SB Right Turn Lanes

<u>Sheridan Parkway and Ridge View Drive</u>

The existing intersection of Sheridan Parkway and Ridge View Drive operates with stop control on the eastbound approach. The intersection currently has all movements operating at a LOS B or better during the morning and afternoon peak hours. With the addition of project traffic through the build out 2018 horizon, this intersection is anticipated to continue to operate acceptably during the morning and afternoon peak hours with its existing configuration and control. By 2035, Sheridan Parkway is expected to have at least two through lanes of travel in each direction (possibly three through lanes in each direction as identified). With or without the addition of project traffic through the 2035 horizon, this intersection is anticipated to operate acceptably during the morning and afternoon peak hours with stop control on the eastbound approach. Therefore, no improvements are anticipated to be needed at this intersection specific to this project. **Table 5** provides the results of the LOS analysis conducted at this intersection.

Table 5 – Sheridan Parkway and Ridge View Drive LOS Results

Scenario	AM Peak Hour Delay (sec/veh)	AM Peak Hour LOS	PM Peak Hour Delay (sec/veh)	PM Peak Hour LOS
2014 Existing				
Northbound Left	7.7	A	7.5	A
Eastbound Left	11.2	B	11.3	B
Eastbound Right	9.5	A	9.1	A
2018 Background				
Northbound Left	8.0	A	7.9	A
Eastbound Left	13.0	B	16.1	C
Eastbound Right	10.2	B	9.8	A
2018 Background Plus Project				
Northbound Left	8.1	A	8.1	A
Eastbound Left	10.6	B	20.0	C
Eastbound Right	10.5	B	10.0	B
2035 Background #				
Northbound Left	8.7	A	8.3	A
Eastbound Left	19.0	C	23.0	C
Eastbound Right	10.7	B	10.0	B
2035 Background Plus Project #				
Northbound Left	8.9	A	8.5	A
Eastbound Left	23.6	C	49.5	E
Eastbound Right	12.8	B	11.4	B

Two Through Lanes NB and SB

<u>Mountain View Boulevard and Ridge View Drive</u>

The existing intersection of Mountain View Drive and Ridge View Drive operates with stop control on the eastbound and westbound approaches of Fairway Point Drive/Ridge View Drive. The westbound approach of Ridge View has a designated left turn lane and a shared through/ right-turn lane. Although not striped, the eastbound approach is wide enough to accommodate a left turn lane and a shared through and right turn lane. With this configuration, all movements operate with acceptable level of service today. In 2018 with or without the addition of project traffic, all movements are anticipated to continue operating acceptably. Therefore, no improvements are anticipated to be needed at this intersection to accommodate project traffic. By 2035, the westbound left turn movement may operate with long delays and a LOS F during the peak hours if the projected future traffic volumes are realized. Alternate control (traffic signal or roundabout) could be considered for the intersection if desired. It is recommended that the Town of Erie continue to monitor traffic volumes at this intersection in the future to determine if and when improvements are needed. Otherwise, it is believed that traffic will divert and reroute on the street network if these long delays are realized for the westbound left turn movement. **Table 6** provides the LOS analysis conducted at this intersection.

Table 6 – Mountain View Boulevard and Ridge View Drive LOS Results

Scenario	AM Peak Hour Delay (sec/veh)	AM Peak Hour LOS	PM Peak Hour Delay (sec/veh)	PM Peak Hour LOS
2015 Existing				
Northbound Left	7.8	A	7.7	A
Southbound Left	7.6	A	7.7	A
Eastbound Left	14.9	B	14.6	B
Eastbound Thru/Right	10.4	B	9.3	A
Westbound Left	16.5	C	15.2	C
Westbound Thru/Right	9.2	A	10.2	B
2018 Background				
Northbound Left	8.0	A	7.8	A
Southbound Left	7.7	A	7.8	A
Eastbound Left	16.9	C	16.3	C
Eastbound Thru/Right	10.9	B	9.5	A
Westbound Left	22.7	C	18.5	C
Westbound Thru/Right	9.4	A	10.5	B
2018 Background Plus Project				
Northbound Left	8.0	A	7.8	A
Southbound Left	7.7	A	7.9	A
Eastbound Left	17.5	C	17.1	C
Eastbound Thru/Right	11.0	B	9.6	A
Westbound Left	25.5	D	20.9	C
Westbound Thru/Right	9.4	A	10.6	B
2035 Background				
Northbound Left	8.7	A	8.4	A
Southbound Left	8.2	A	8.5	A
Eastbound Left	39.9	E	37.4	E
Eastbound Thru/Right	15.3	C	10.9	B
Westbound Left	288.8	F	99.7	F
Westbound Thru/Right	10.5	B	14.1	B
2035 Background Plus Project				
Northbound Left	8.7	A	8.4	A
Southbound Left	8.3	A	8.5	A
Eastbound Left	42.5	E	39.6	E
Eastbound Thru/Right	15.5	C	11.0	B
Westbound Left	379.3	F	159.3	F
Westbound Thru/Right	10.6	B	14.4	B

Mountain View Boulevard and Village Vista Drive

The existing intersection of Mountain View Boulevard and Village Vista Drive operates with stop control on the eastbound approach. The eastbound approach of Village Vista Drive includes an exclusive left-turn lane and a shared through and right-turn lane. Although the westbound approach leads to vacant land, it has been partially constructed and includes a shared through and left-turn lane and a right-turn lane. With the intersection's existing configuration, all movements operate acceptably today. The intersection is anticipated to continue to operate acceptably in 2018 prior to the addition of Vista Ridge Commercial project traffic. With the addition of the project, the east leg of the intersection will provide access to the development. All movements are anticipated to operate acceptably with the exception of the westbound left turn during the afternoon peak hour. It isn't anticipated that the intersection will warrant signalization under this condition. Some traffic may reroute to the SH-7 three-quarter movement access if destined west on SH-7 or to Sheridan Boulevard if heading east on SH-7.

In 2035 the eastbound left turn movement is anticipated to operate with significantly long delays prior to the construction of the project due to the increased north-south through traffic volume growth along Mountain View Boulevard. Therefore, it is believed that this intersection will warrant and require signalization. Signalized and with the addition of project traffic, the intersection is anticipated to operate at LOS B during both peak hours. With signalization a westbound left turn lane is recommended to be designated. **Table 7** provides the LOS analysis conducted at this intersection.

Table 7 – Mountain View Boulevard and Village Vista Drive LOS Results

Scenario	AM Peak Hour Delay (sec/veh)	AM Peak Hour LOS	PM Peak Hour Delay (sec/veh)	PM Peak Hour LOS
2015 Existing				
Northbound Left	8.2	A	8.1	A
Eastbound Left	15.7	C	18.0	C
Eastbound Thru/Right	9.7	A	9.5	A
2018 Background				
Northbound Left	8.5	A	8.2	A
Southbound Left	7.7	A	7.9	A
Eastbound Left	19.8	C	23.6	C
Eastbound Thru/Right	10.0	B	9.7	A
Westbound Thru/Left	19.6	C	21.7	C
Westbound Right	0.0	A	0.0	A
2018 Background Plus Project				
Northbound Left	8.3	A	8.2	A
Southbound Left	8.0	A	8.1	A
Eastbound Left	22.7	C	28.0	D
Eastbound Thru/Right	9.9	A	9.7	A
Westbound Thru/Left	63.2	F	134.6	F
Westbound Right	9.2	A	9.5	A
2035 Background				
Northbound Left	10.1	B	9.6	A
Southbound Left	8.2	A	8.4	A
Eastbound Left	94.1	F	386.5	F
Eastbound Thru/Right	12.2	B	11.5	B
Westbound Thru/Left	65.1	F	89.5	F
Westbound Right	0.0	A	0.0	A
2035 Background Plus Project #	17.8	B	21.5	C

Signalized, WB Left Turn Lane

6.3 Project Access Operational Analysis

An operational analysis was performed for the driveways proposed with this Vista Ridge Commercial project, as well as those that will be used by this project that are shared with Erie King Soopers #129 project to the east. The shared driveways that will allow direct access to the Vista Ridge Commercial project site include the three-quarter driveway along SH-7, the eastern full movement Ridge View Drive access, and the full movement access along Sheridan Parkway. The three-quarter movement driveway (southbound left turn egress restricted) is proposed to be located approximately 1,050 feet west of the SH-7/Sheridan Parkway intersection at the eastern edge of this project. New accesses proposed for Vista Ridge Commercial include a right-in/right-out driveway along Mountain View Boulevard, a full movement driveway to align with the Mountain View Boulevard and Village Vista Drive intersection, and an additional Ridge View Drive access. The right-in/right-out driveway is located approximately 315 feet north of the SH-7 and Mountain View Boulevard intersection.

The operational analysis at the proposed driveways determines the lane and control improvements needed at each access. Of note, the proposed three-quarter movement access along SH-7 will use the existing westbound auxiliary lane for deceleration and acceleration for movements to and from the driveway. **Table 8** provides a summary of the operational analysis at the proposed project accesses in 2018 and 2035. Detailed results of the operational analysis are also provided in **Appendix D**.

Table 8 – Project Access Driveway Intersection LOS Results

Access and Movement	2018 Total Traffic				2035 Total Traffic			
	AM Peak Hour		PM Peak Hour		AM Peak Hour		PM Peak Hour	
	Delay (sec/veh)	LOS	Delay (sec/veh)	LOS	Delay (sec/veh)	LOS	Delay (sec/veh)	LOS
SH-7 Three-Quarter								
EB Left	13.1	B	14.9	B	21.1	C	34.9	D
SB Right	0.0 *	A *	0.0 *	A *	0.0*	A *	0.0	A *
Sheridan Pkwy Full								
NB Left	8.0	A	8.1	A	8.5	A	8.5	A
EB Left	12.7	B	17.0	C	16.7	C	25.6	D
EB Right	10.4	B	12.2	B	12.1	B	14.7	B
Eastern Ridge View Dr.								
Northbound Left	10.2	B	10.6	B	10.7	B	11.1	B
Northbound Right	9.0	A	9.1	A	9.2	A	9.4	A
Westbound Left	7.5	A	7.6	A	7.6	A	7.7	A
Western Ridge View Dr.								
Northbound Approach	9.3	A	9.4	A	9.7	A	9.7	A
Westbound Left	7.4	A	7.4	A	7.5	A	7.5	A
Mountain View RIRO								
Westbound Right	9.8	A	10.5	B	10.7	B	12.4	B

* Free southbound right turn movement with acceleration lane

Recommendations from Access Operational Analysis

It is recommended that the proposed three-quarter movement access along SH-7 have an eastbound left-turn deceleration lane, a westbound right-turn deceleration lane and southbound free right-turn lane with a receiving acceleration lane along westbound SH-7 in 2018 and when the highway is improved to three through lanes in each direction in 2035. It is anticipated that the eastbound left turn movement at this intersection will operate acceptably during the 2018 horizon. In 2035, the eastbound left turn movement may operate with long delays as reported in the HCM 2010 analysis procedure; however it is anticipated to operate better than predicted by the effect of traffic metering from the upstream SH-7/Sheridan Boulevard signalized intersection. This was observed in the simulation and the HCM 2000 procedure shows acceptable level of service is attainable. The existing westbound auxiliary lane will be used for the deceleration and acceleration lane at this access. It is recommended that a R3-2 No Left Turn sign be installed for the southbound approach exiting the development at this access.

It is recommended that the proposed full movement access on Sheridan Parkway be designated with stop control with a R1-1 “STOP” sign installed on the eastbound approach out of the development. The northbound approach is recommended to have a designated left-turn lane. The southbound approach is recommended to have a shared through/right turn lane. With this configuration, the intersection is anticipated to have movements operate acceptably through the 2035 horizon.

The northbound approach at both Ridge View Drive accesses are recommended to be designated with stop control with R1-1 “STOP” signs installed. The eastern Ridge View Drive driveway shared with the Erie King Soopers #129 project is anticipated to receive the most traffic and is recommended to have separate northbound left turn and right turn lanes exiting the development. It is recommended that the driveway along Ridge View Drive be coordinated with Montex North and South developments as well as Vista Ridge Academy. It is also recommended that the existing striped median within Ridge View Drive be restriped as a two-way left-turn lane to accommodate left turn movements for the proposed access points and the existing access points on the north side of the street. With this proposed configuration, both proposed Ridge View Drive accesses are anticipated to operate acceptably throughout the 2035 horizon.

The proposed right-in/right-out access along Mountain View Boulevard is anticipated to operate with acceptable level of service. The westbound right turn movement exiting the development is recommended to operate with stop control with the installation of a R1-1 “STOP” sign. In addition, a R3-2 No Left Turn sign shall be installed underneath the STOP sign.

6.4 Turn Bay Length Analysis

It is recommended that auxiliary lanes along SH-7 adjacent to the project be constructed or designated in accordance with the current CDOT State Highway Access Code (SHAC). The State Highway Access Category Schedule categorizes the segment of State Highway 7 through the project study area as NR-A (Non-Rural Principal Highway). According to the SHAC, the following thresholds apply for category NR-A roadways:

- A left turn deceleration lane and taper with storage length is required for any access with a projected peak hour ingress turning volume greater than 10 vehicles per hour (vph). The taper length will be included within the required deceleration length.
- A right turn deceleration lane and taper is required for any access with a projected peak hour ingress turning volume greater than 25 vph. The taper length will be included within the required deceleration length.

- A right turn acceleration lane and taper is required for any access with a projected peak hour right turning volume greater than 50 vph when the posted speed on the highway is greater than 40 mph. The taper length will be included within the required deceleration length.

Based on traffic projections and the above thresholds, auxiliary turn lane requirements were calculated for the proposed three-quarter movement access along SH-7. Immediately adjacent to the site, SH-7 provides primarily a single through lane of travel in each direction with a 55 mile per hour posted speed limit. A continuous auxiliary lane exists along westbound SH-7 adjacent to the site which is used as an acceleration lane from Sheridan Parkway and a deceleration lane for Mountain View Boulevard. Eastbound there are two through lanes to receive the southbound dual lefts from Mountain View Boulevard, with the outside lane being a forced drop right turn lane at Sheridan Boulevard. As such, turn lane requirements at the proposed site access along SH-7 are as follows:

<u>Proposed Three-quarter Movement Unsignalized Access</u>

- A westbound right turn deceleration lane is warranted with the build out of the project. It is recommended that the existing westbound auxiliary lane be used for this right turn lane. Per SHAC standards, this right turn lane should include a length of 600 feet which includes a 222-foot taper (18.5 to 1), assuming a 12-foot wide turn lane. An acceleration/deceleration lane already exists today along State Highway 7 adjacent to the site. It is recommended that the existing combination acceleration/deceleration lane remain between this three-quarter movement access and Sheridan Parkway, which are separated by approximately 1,050 feet. The combination acceleration (960 feet) and deceleration (600 feet) would include an auxiliary combination lane length of approximately 1,120 feet without the tapers. Therefore, it is believed this distance will be adequate for acceleration, deceleration, and weaving maneuvers along westbound SH-7 between the proposed three-quarter movement access and Mountain View Boulevard. Further, it is recommended that this continuous auxiliary lane exists in 2035 after the highway is widened to be three lanes in each direction to allow for acceptable traffic operations.
- An eastbound left turn deceleration lane is warranted with the build out of the project. The length of the left turn deceleration lane will include deceleration length plus storage. The maximum projected peak hour ingress turning volume is 276 vehicles per hour

which equates to a storage length of 275 feet. It is recommended that the deceleration length be 380 feet plus a 220-foot taper (18.5 to 1), assuming a 12-foot wide lane. The overall left turn deceleration lane length is 655 feet plus a 220-foot taper.

- A westbound acceleration lane along State Highway 7 is warranted. The acceleration lane length needed is 960 feet which includes a 222-foot taper (18.5 to 1), assuming a 12-foot wide lane. It is recommended that the existing auxiliary lane be used as a combination acceleration/deceleration lane between the three-quarter movement access and Mountain View Boulevard. The combination acceleration (960 feet) and deceleration (600 feet) lane lengths would include an auxiliary combination lane length of approximately 1,120 feet without the two 220-foot tapers. There is approximately 1,550 feet between the proposed three-quarter movement access and Mountain View Boulevard which is greater than required. It is believed this distance will be adequate for acceleration, deceleration, and weaving maneuvers along westbound SH-7 between the proposed three-quarter movement access and Mountain View Boulevard. A continuous auxiliary lane for acceleration and deceleration movements should exist in 2035 when the highway is improved as well.

In addition to CDOT SHAC turn lane requirements along SH-7, a queuing analysis was conducted for the SH-7 and Ridge View Drive intersections with Mountain View Boulevard and Sheridan Parkway as well as the proposed accesses. Turn lanes are recommended to be constructed providing the recommended storage length based on the queuing analysis. Results were obtained from the 95th percentile queue lengths obtained from the Synchro analysis. Results are shown in the following **Table 9** with calculations provided within the level of service operational sheets of **Appendix D** for the unsignalized intersections and **Appendix E** for signalized intersections.

Table 9 – Left-turn lane Length Analysis Results

Intersection Turn Lane	Existing Turn Lane Length (feet)	2018 Total Queue Length (feet)	2018 Recommended Turn Lane Length (feet)	2035 Total Queue Length (feet)	2035 Recommended Turn Lane Length (feet)
SH-7 & Mountain View Blvd					
Eastbound Left	750’	436’	750’	570’	750’
Southbound Left	325’ DL	162’ DL	325’ DL	262’ DL	325’ DL
SH-7 & Sheridan Pkwy #					
Eastbound Left	700’	33’	700’	52’	**450’ DL**
Westbound Left	875’	230’	875’	193’	**650’ DL**
Northbound Left	275’	140’	275’	125’	**200’ DL**
Southbound Left	425’	230’	**325’**	228’	**200’ DL**
Ridge View & Sheridan					
Northbound Left	350’	25’	**150’**	25’	**150’**
Eastbound Left	200’	38’	200’	200’	200’
Ridge View & Mountain View					
Northbound Left	75’	25’	75’	25’	75’
Southbound Left	75’	25’	75’	25’	75’
Eastbound Left	125’	25’	125’	25’	125’
Westbound Left	200’	50’	200’	**320’**	**TWLTL**
Village Vista & Mountain View					
Northbound Left	200’	25’	200’	102’	200’
Southbound Left	75’	25’	75’	29’	75’
Eastbound Left	200’	25’	200’	117’	200’
Westbound Left	DNE	165’	**175’**	**240’**	**250’**
Sheridan Pkwy Access					
Northbound Left	DNE	25’	**100’**	25’	**150’**
Eastbound Left	DNE	50’	**50’**	60’	**75’**
Eastern Ridge View Access					
Westbound Left	DNE	25’	**TWLTL**	25’	**TWLTL**
Western Ridge View Access					
Westbound Left	DNE	25’	**TWLTL**	25’	**TWLTL**
Mountain View RIRO Access					
Westbound Right	DNE	25’	25’	25’	25’

DL = Dual Lefts
DNE = Does Not Exist
TWLTL = Two Way Left-turn lane
# Two Through Lanes EB and WB 2018, Three Through Lanes EB and WB 2035

<u>SH-7 and Mountain View Boulevard Signalized Intersection</u>

It is believed that the existing eastbound and southbound left-turn lanes will be sufficient to accommodate future traffic volumes throughout the 2035 horizon. Based on this, no improvements or modifications are anticipated to be needed.

<u>SH-7 and Sheridan Parkway Signalized Intersection</u>

In 2018 all existing left turn storage bays are believed to be adequate to accommodate project traffic. It is recommended that the existing southbound left turn bay be reduced from the existing 425 feet to 325 feet. This will allow for designation of a northbound left-turn lane at the proposed

Sheridan Parkway access. In 2035 it is anticipated that all left-turn lanes will be constructed as dual left-turn lanes. With two lanes for storage, left turn requirements will decrease further.

Ridge View Drive and Sheridan Parkway

The existing northbound and eastbound left-turn lanes are anticipated to be adequate to accommodate projected left turn queues. Based on planned access locations, both left turn bays are recommended to be reduced. The northbound left-turn lane is recommended to be reduced from the existing 350 feet to 150 feet to accommodate the proposed access intersection along Sheridan Parkway so that a southbound left-turn lane could be designated in the future for the development along the east side of Sheridan Parkway.

Ridge View Drive and Mountain View Boulevard

The existing left turn storage bays are sufficient to accommodate 2018 traffic with the addition of the Vista Ridge Commercial project. By 2035, the westbound left turn queue may extend as far as 320 feet if the future traffic volume projections are realized. As recommended, restriping Ridge View Drive with a Two Way Left Turn Lane will help with the queuing.

Village Vista Drive and Mountain View Boulevard

All existing left turn storage bays successfully accommodate existing traffic at the Village Vista Drive and Mountain View Boulevard intersection. With project development, the east leg of the intersection will be created. It is recommended that a westbound left turn lane be constructed and designated with a length of 175 feet for the 2018 horizon and 250 feet for the 2035 horizon. If possible, it would be desirable to construct this left turn lane with a length of 250 feet with project development.

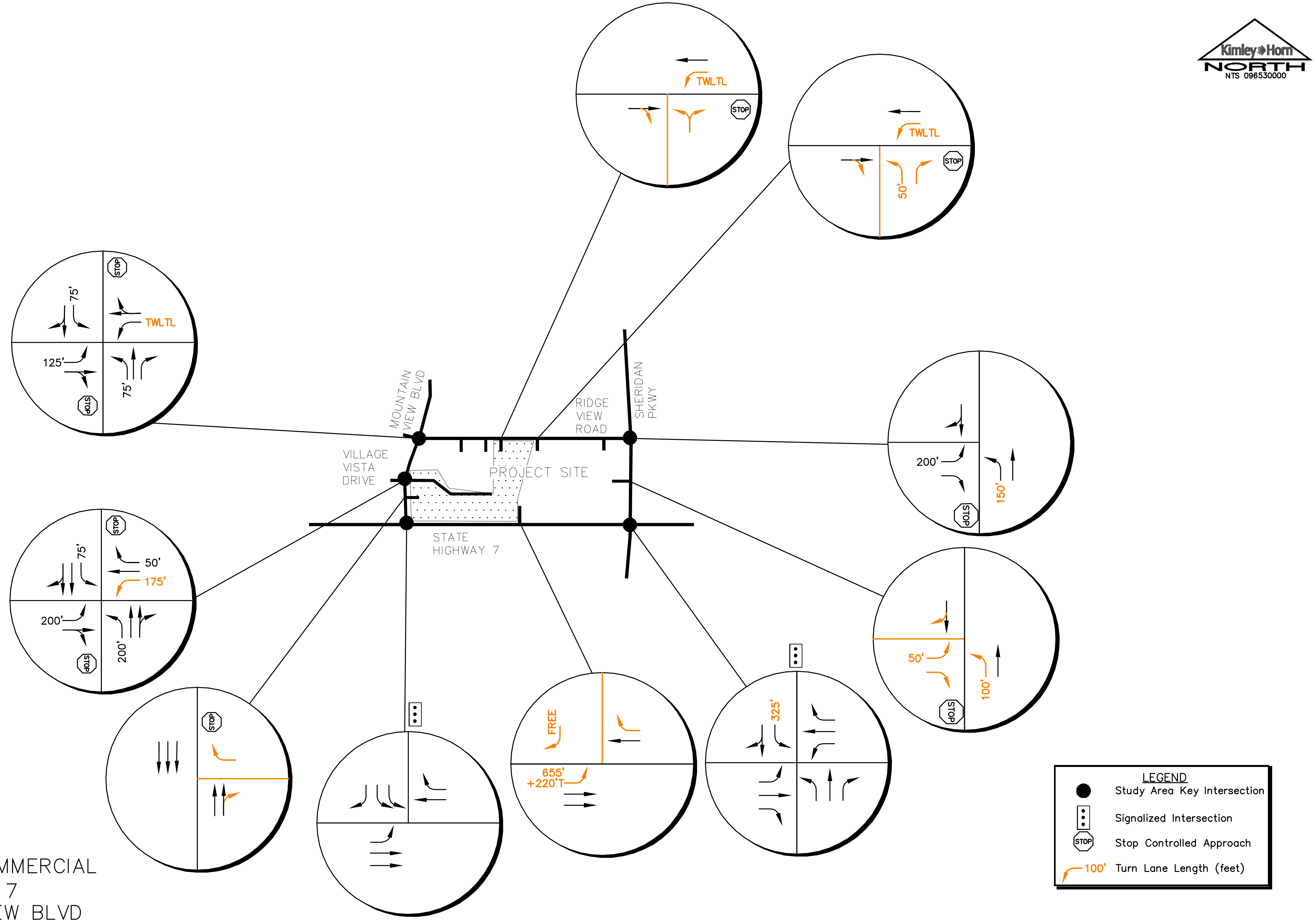
Sheridan Parkway Access

A full movement access is proposed along Sheridan Parkway, approximately halfway between SH-7 and Ridge View Drive. This full movement access will require a northbound left-turn lane. In addition, it is recommended that the eastbound approach exiting the property contain separate left turn and right turn lanes. Since there is approximately 450 feet of back-to-back storage available along Sheridan Parkway for the southbound left turn at SH-7 and northbound left turn at the access, it is recommended that a 100-foot northbound left-turn lane be designated. The southbound left-turn lane at SH-7 is recommended for a length of 325 feet. These lengths can be accommodate with a 25-foot taper between. When dual southbound left-turn lanes are constructed at the SH-7 and Sheridan Parkway intersection, it is recommended the back-to-back storage along Sheridan Parkway includes 200-foot southbound dual left-turn lanes, a 100-foot taper, and a 150-foot northbound left-turn lane at the access. The eastbound left turn out of the property was found to require three vehicles of storage. Therefore it is recommended that 75 feet of storage be provided within the throat of this driveway.

Ridge View Drive Access

It is recommended that a two-way left-turn lane be designated along Ridge View Drive to accommodate the two driveways proposed with the project, as well as the existing driveways for the school on the north side of the roadway. The eastern access is recommended to include separate northbound left turn and right turn lanes exiting the property. One vehicle of storage was found to be needed, so a driveway throat depth of 50 feet should be sufficient.

These improvements are illustrated in **Figure 13** for the 2018 horizon year and **Figure 14** for the 2035 horizon year.



VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
2018 RECOMMENDED LANE
CONFIGURATIONS AND CONTROL

FIGURE 13

Kimley»Horn

Date: May 24, 2016 – 3:09pm / User: matt.farmen
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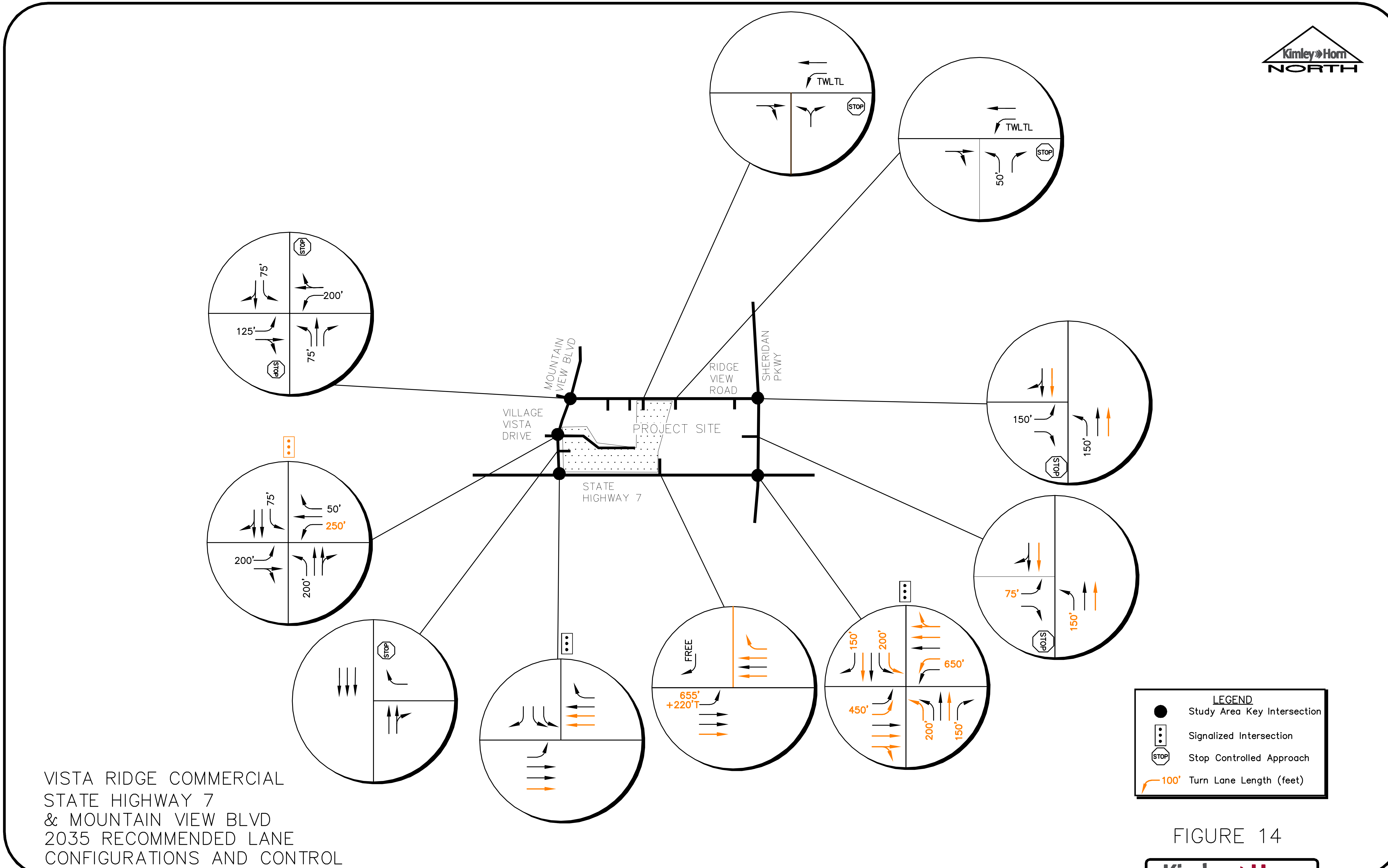

VISTA RIDGE COMMERCIAL
STATE HIGHWAY 7
& MOUNTAIN VIEW BLVD
2035 RECOMMENDED LANE
CONFIGURATIONS AND CONTROL

FIGURE 14

Kimley»Horn

Date: May 24, 2016 – 3:09pm / User: matt.farmen
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7.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the analysis presented in this report, Kimley-Horn believes the proposed Vista Ridge Commercial project will be successfully incorporated into the existing and future roadway network. Analysis of the existing street network, the proposed project development, and expected traffic volumes resulted in the following recommendations:

2018 Year Improvement Recommendations

- It is recommended that the southbound left-turn lane length at the SH-7/Sheridan Parkway intersection be reduced from 425 feet to 325 feet so that back-to-back left turn storage will be available along Sheridan Parkway between SH-7 and the proposed full movement project access. This length is anticipated to be sufficient to accommodate future left turning traffic volumes.
- It is recommended that a 100-foot northbound left-turn lane be designated along Sheridan Parkway for the proposed full movement access. Since there is approximately 450 feet of back-to-back available storage available between SH-7 and the project driveway, it is recommended that the taper between the left-turn lanes be 25 feet to allow for the recommended 325-foot southbound left-turn lane at SH-7.
- It is recommended that the full movement access on Sheridan Parkway be designated with stop control with a R1-1 “STOP” sign installed on the eastbound exiting approach. The eastbound exiting approach is recommended to be constructed with separate left and right turn lanes. The left-turn lane length recommended is the standard driveway throat depth of 75 feet.
- It is recommended that the northbound left-turn lane at the Ridge View Drive and Sheridan Parkway intersection also be reduced due to the proposed project access location along Sheridan Parkway. This left-turn lane is recommended to be reduced from 350 feet to 150 feet. This turn bay length is anticipated to be sufficient to accommodate future left turning traffic volumes.
- With construction of the project, the east leg of the Village Vista Drive and Mountain View Boulevard intersection will be improved. When the project is constructed, it is recommended that the existing striped full lane width median be redesignated with a 175-foot westbound left turn lane. If possible, it is encouraged that this westbound left turn lane be constructed so that the future 250-foot westbound left turn lane can be designated to accommodate 2035 traffic volumes.

- It is recommended that an eastbound left-turn lane be designated within the full width striped median along SH-7 at the proposed three-quarter movement access. It is recommended that this left-turn lane be designated with a length of 655 feet plus a 220-foot taper (875-foot total length).
- A continuous westbound auxiliary acceleration/deceleration lane exists along State Highway 7 between Sheridan Parkway and Mountain View Boulevard. This existing lane will serve as both an acceleration and deceleration lane for the proposed three-quarter SH-7 project access.
- At the proposed SH-7 three quarter movement access, it is recommended that a R3-2 No Left Turn sign be installed for the southbound approach for motorists exiting the development. This sign can be installed under the R1-1 "STOP" or R1-2 "YIELD" sign if desired.
- Both access approaches to Ridge View Drive are recommended to be designated with R1-1 "STOP" signs installed on the northbound approach out of the development. The eastern access is anticipated to receive the most traffic and is therefore recommended to have separate left and right lanes. The western access on Ridge View Drive is believed to operate acceptably with shared northbound left turn/right turn lanes.
- It is recommended that the full lane width median along Ridge View Drive be restriped to include a two-way left-turn lane through the proposed project accesses. It is recommended that this be coordinated with Montex North and South developments to provide a coordinated plan for Ridge View Drive.
- The westbound approach exiting the project at the right-in/right-out access along Mountain View Boulevard is recommended to operate with stop control. Therefore it is recommended that a R1-1 "STOP" sign be installed for this approach. In addition, a R3-2 No Left Turn Sign should be installed underneath the STOP sign to identify the turn movement restriction at this access.

2035 Long Term Twenty Year Planning Horizon Improvement Recommendations

- SH-7 may need to be a six-lane roadway by 2035. It is recommended that the westbound right turn deceleration and acceleration lanes from the three-quarter movement project driveway along SH-7 be reconstructed in addition to the three westbound through lanes. Sheridan Parkway may need to be a four-lane (or six-lane) roadway by 2035 as identified within the Amendment to the SH 7 Access Control Plan.

- The intersection of State Highway 7 with Sheridan Parkway is recommended to have dual left-turn lanes on all approaches and right turn lanes for the northbound and southbound directions.
- Upon construction of the dual southbound left-turn lanes at the SH-7 and Sheridan Parkway intersection, it is believed that the turn lane storage bay length can be reduced to 200 feet. This will allow for a 150-foot northbound left-turn lane at the proposed Sheridan Parkway access with a standard 100-foot taper between the back-to-back left-turn lanes along Sheridan Parkway between the proposed full movement access and SH-7.
- If future traffic volumes are realized along Mountain View Boulevard, the intersection of Village Vista Drive and Mountain View Boulevard will warrant and require signalization. Therefore, the Town of Erie should monitor traffic volumes in the future to determine if and when this improvement is needed.

General Recommendations

- All on-site and off-site roadway improvements should be incorporated into the Civil Drawings, and conform to standards of the Town of Erie, State of Colorado Department of Transportation (CDOT), American Association of State Highway and Transportation Officials (AASHTO) Geometric Design of Highways and Streets, Institute of Transportation Engineers (ITE), and/or the Manual on Traffic Control Devices (MUTCD) – 2009 Edition as appropriate.

APPENDICES

APPENDIX A

Intersection Count Sheets

Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
AM Peak
Ridge View Dr and Mountian View Blvd

File Name : RidgeViewMountainViewAM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 1

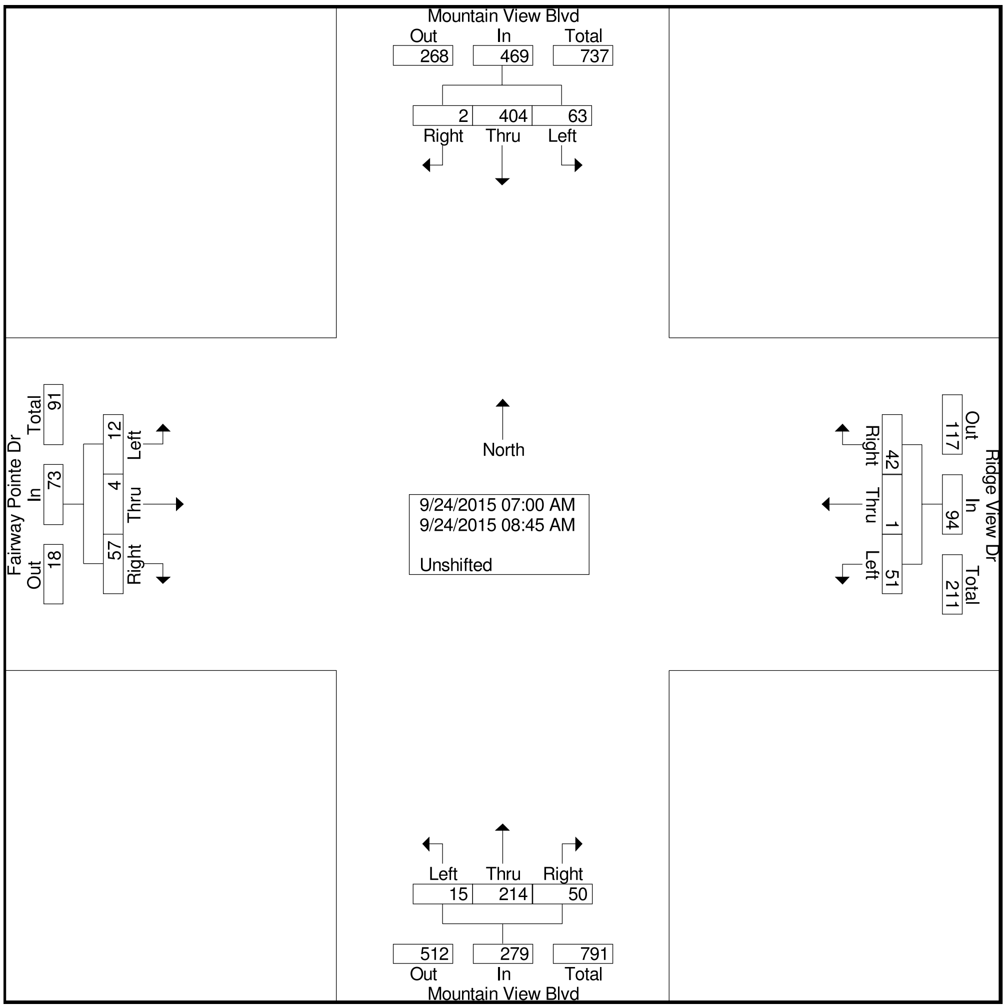
Groups Printed- Unshifted

	Fairway Pointe Dr Eastbound				Ridge View Dr Westbound				Mountain View Blvd Northbound				Mountain View Blvd Southbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
07:00 AM	1	0	17	18	4	0	3	7	1	15	2	18	2	50	1	53	96
07:15 AM	2	0	10	12	3	0	1	4	0	23	3	26	6	49	1	56	98
07:30 AM	1	0	5	6	8	1	2	11	0	18	8	26	6	47	0	53	96
07:45 AM	3	2	1	6	3	0	7	10	2	23	6	31	8	59	0	67	114
Total	7	2	33	42	18	1	13	32	3	79	19	101	22	205	2	229	404
08:00 AM	1	1	7	9	13	0	5	18	1	35	15	51	9	57	0	66	144
08:15 AM	0	0	9	9	7	0	8	15	4	33	6	43	16	44	0	60	127
08:30 AM	2	1	7	10	6	0	9	15	1	29	5	35	7	32	0	39	99
08:45 AM	2	0	1	3	7	0	7	14	6	38	5	49	9	66	0	75	141
Total	5	2	24	31	33	0	29	62	12	135	31	178	41	199	0	240	511
Grand Total	12	4	57	73	51	1	42	94	15	214	50	279	63	404	2	469	915
Apprch %	16.4	5.5	78.1		54.3	1.1	44.7		5.4	76.7	17.9		13.4	86.1	0.4		
Total %	1.3	0.4	6.2	8	5.6	0.1	4.6	10.3	1.6	23.4	5.5	30.5	6.9	44.2	0.2	51.3	

Ridgeview Data Collection
Morrison, CO 80465

Erie, CO
Erie Kentro
AM Peak
Ridge View Dr and Mountian View Blvd

File Name : RidgeViewMountainViewAM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 2



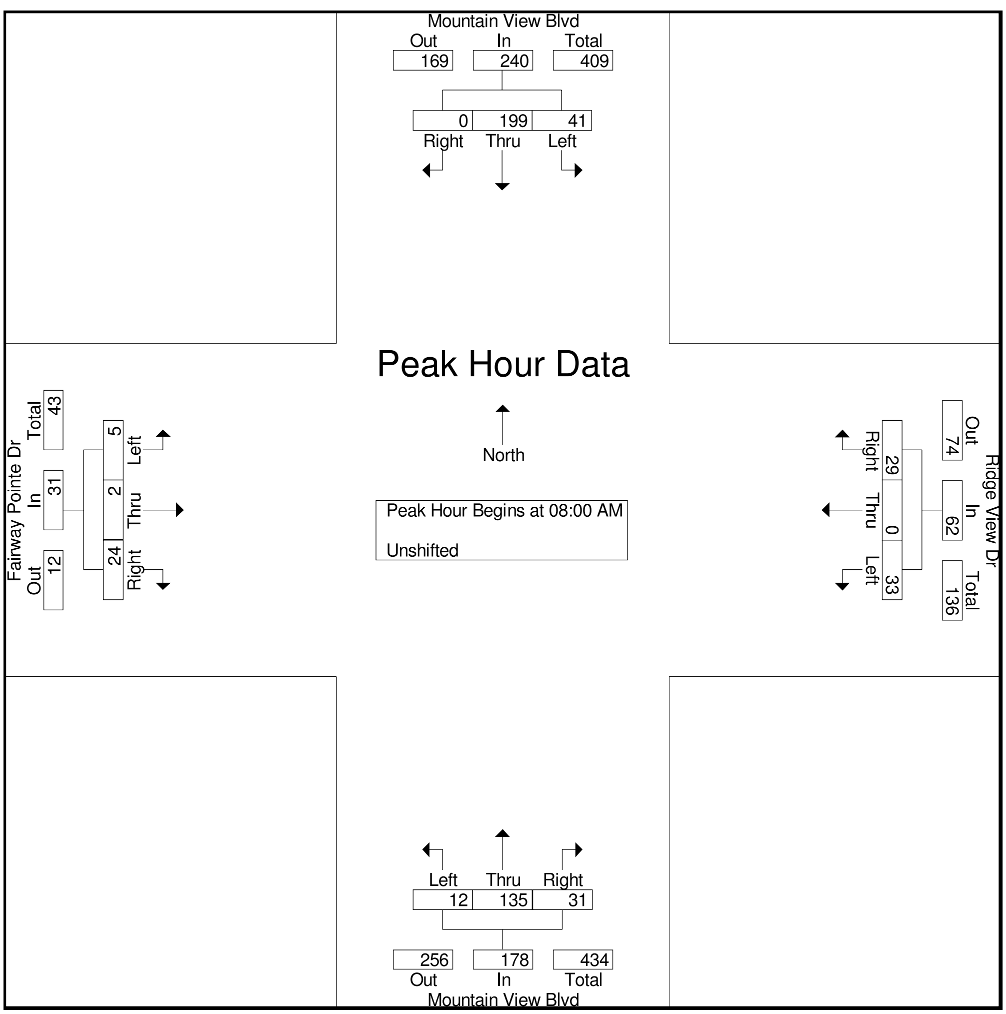
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
AM Peak
Ridge View Dr and Mountian View Blvd

File Name : RidgeViewMountainViewAM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 3

	Fairway Pointe Dr Eastbound				Ridge View Dr Westbound				Mountain View Blvd Northbound				Mountain View Blvd Southbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 07:00 AM to 08:45 AM - Peak 1 of 1																	
Peak Hour for Entire Intersection Begins at 08:00 AM																	
08:00 AM	1	**1**	7	9	**13**	0	5	**18**	1	35	**15**	**51**	9	57	0	66	**144**
08:15 AM	0	0	**9**	9	7	0	8	15	4	33	6	43	**16**	44	0	60	127
08:30 AM	**2**	1	7	**10**	6	0	**9**	15	1	29	5	35	7	32	0	39	99
08:45 AM	2	0	1	3	7	0	7	14	**6**	**38**	5	49	9	**66**	0	**75**	141
Total Volume	5	2	24	31	33	0	29	62	12	135	31	178	41	199	0	240	511
% App. Total	16.1	6.5	77.4		53.2	0	46.8		6.7	75.8	17.4		17.1	82.9	0		
PHF	.625	.500	.667	.775	.635	.000	.806	.861	.500	.888	.517	.873	.641	.754	.000	.800	.887



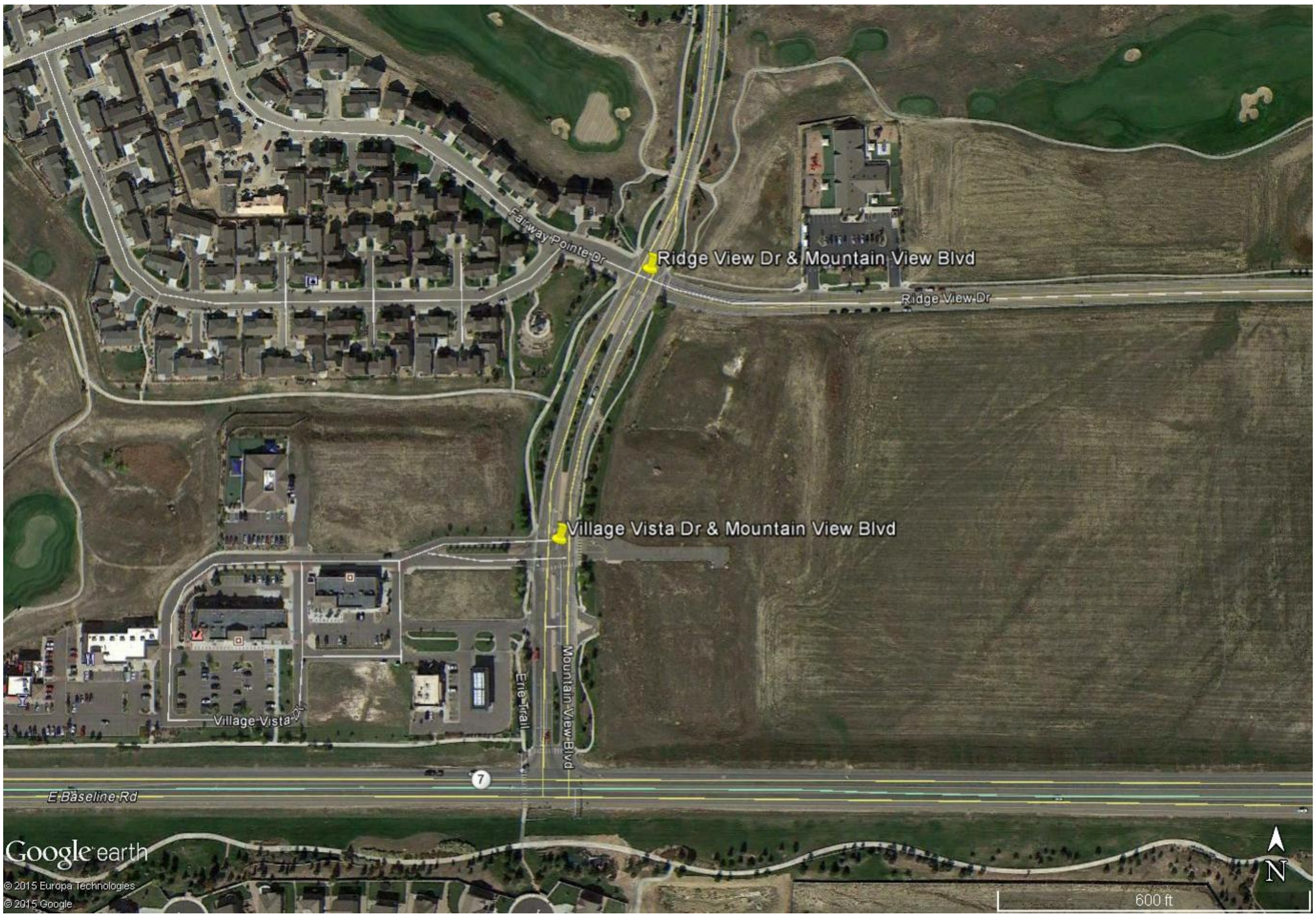
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
AM Peak
Ridge View Dr and Mountian View Blvd

File Name : RidgeViewMountainViewAM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 4

Image 1



RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
PM Peak
Ridge View Dr and Mountain View Blvd

File Name : RidgeViewMountainViewPM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 1

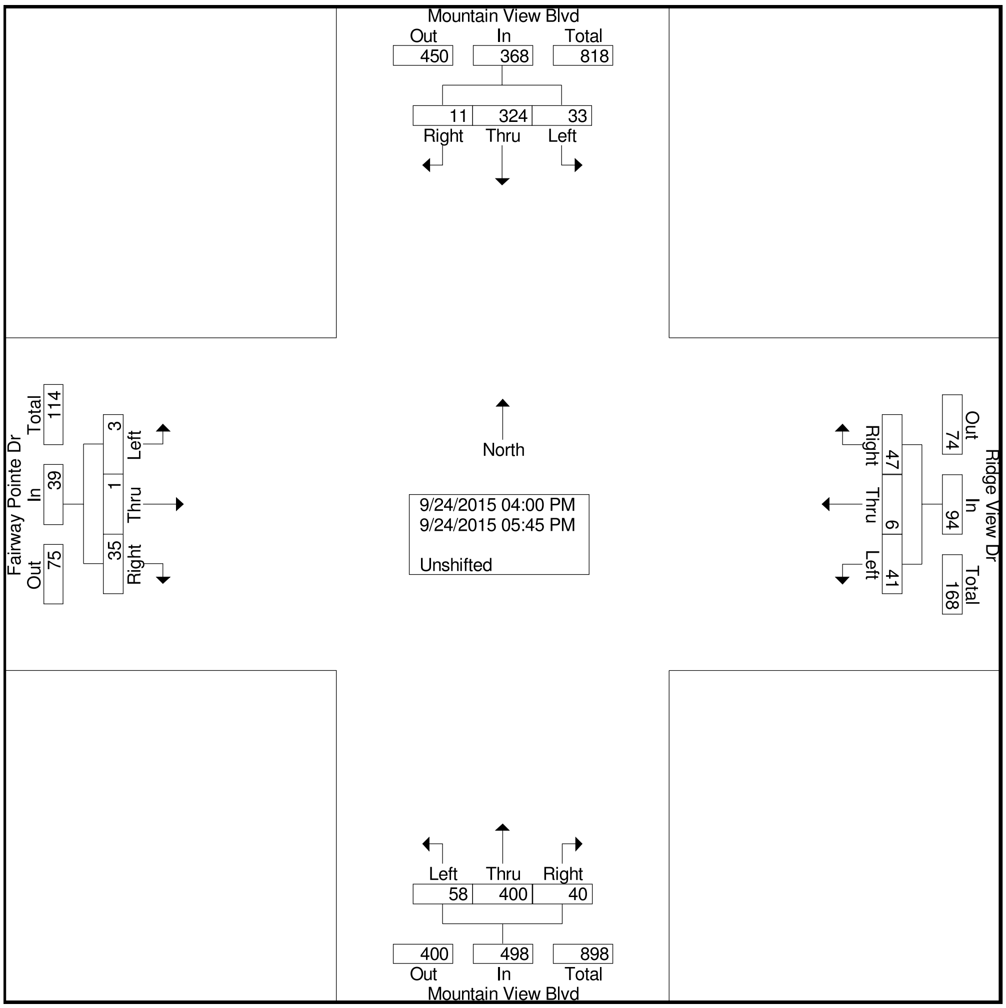
Groups Printed- Unshifted

	Fairway Pointe Dr Eastbound				Ridge View Dr Westbound				Mountain View Blvd Northbound				Mountain View Blvd Southbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
04:00 PM	0	0	7	7	4	0	4	8	6	39	5	50	0	37	0	37	102
04:15 PM	1	1	2	4	3	1	1	5	7	60	0	67	3	45	2	50	126
04:30 PM	0	0	3	3	2	2	4	8	7	37	0	44	3	48	2	53	108
04:45 PM	1	0	4	5	5	0	8	13	5	50	5	60	8	34	3	45	123
Total	2	1	16	19	14	3	17	34	25	186	10	221	14	164	7	185	459
05:00 PM	0	0	4	4	10	0	9	19	8	49	14	71	6	42	3	51	145
05:15 PM	0	0	4	4	11	1	7	19	5	58	6	69	2	45	0	47	139
05:30 PM	0	0	7	7	3	1	3	7	12	52	7	71	6	38	0	44	129
05:45 PM	1	0	4	5	3	1	11	15	8	55	3	66	5	35	1	41	127
Total	1	0	19	20	27	3	30	60	33	214	30	277	19	160	4	183	540
Grand Total	3	1	35	39	41	6	47	94	58	400	40	498	33	324	11	368	999
Apprch %	7.7	2.6	89.7		43.6	6.4	50		11.6	80.3	8		9	88	3		
Total %	0.3	0.1	3.5	3.9	4.1	0.6	4.7	9.4	5.8	40	4	49.8	3.3	32.4	1.1	36.8	

Ridgeview Data Collection

Morrison, CO 80465

Erie, CO	File Name : RidgeViewMountainViewPM
Erie Kentro	Site Code : IPO 129
PM Peak	Start Date : 9/24/2015
Ridge View Dr and Mountain View Blvd	Page No : 2



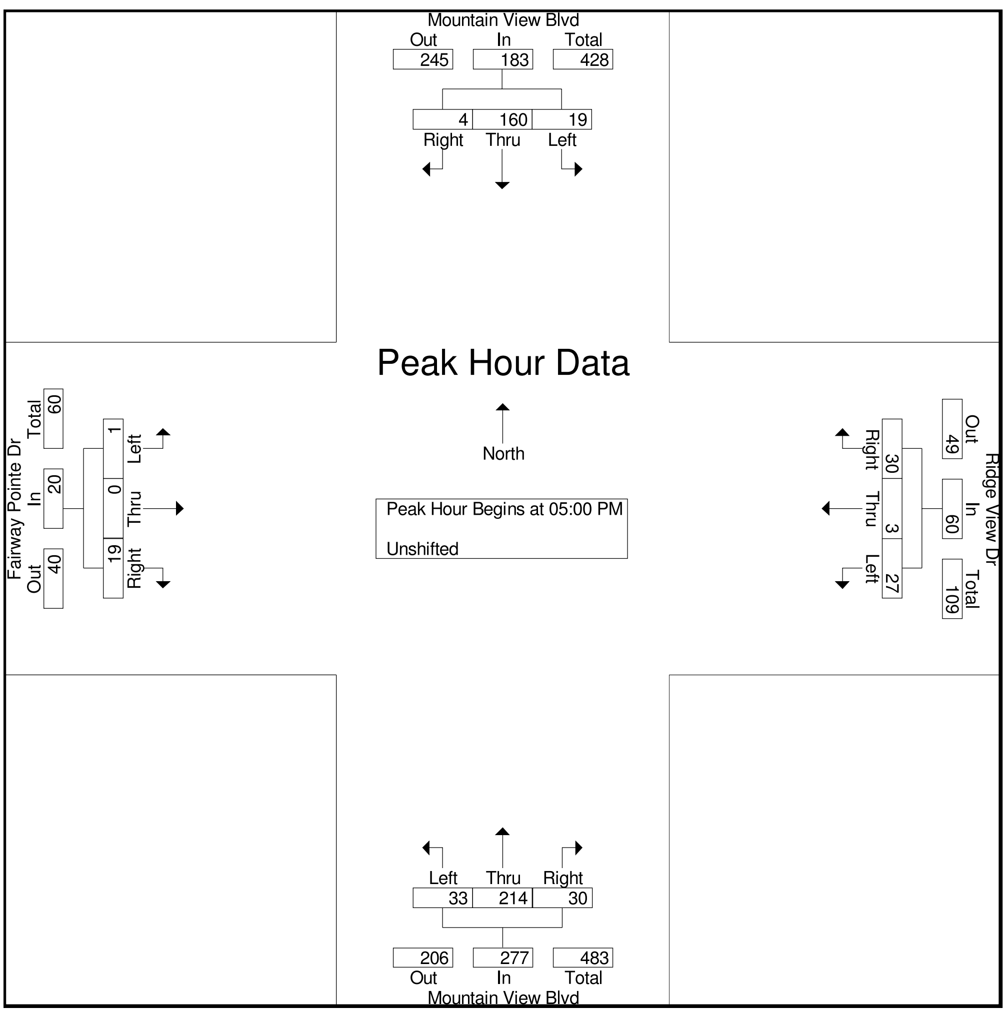
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
PM Peak
Ridge View Dr and Mountain View Blvd

File Name : RidgeViewMountainViewPM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 3

	Fairway Pointe Dr Eastbound				Ridge View Dr Westbound				Mountain View Blvd Northbound				Mountain View Blvd Southbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 04:00 PM to 05:45 PM - Peak 1 of 1																	
Peak Hour for Entire Intersection Begins at 05:00 PM																	
05:00 PM	0	0	4	4	10	0	9	**19**	8	49	**14**	**71**	**6**	42	**3**	**51**	**145**
05:15 PM	0	0	4	4	**11**	**1**	7	19	5	**58**	6	69	2	**45**	0	47	139
05:30 PM	0	0	**7**	**7**	3	1	3	7	**12**	52	7	71	6	38	0	44	129
05:45 PM	**1**	0	4	5	3	1	**11**	15	8	55	3	66	5	35	1	41	127
Total Volume	1	0	19	20	27	3	30	60	33	214	30	277	19	160	4	183	540
% App. Total	5	0	95		45	5	50		11.9	77.3	10.8		10.4	87.4	2.2		
PHF	.250	.000	.679	.714	.614	.750	.682	.789	.688	.922	.536	.975	.792	.889	.333	.897	.931



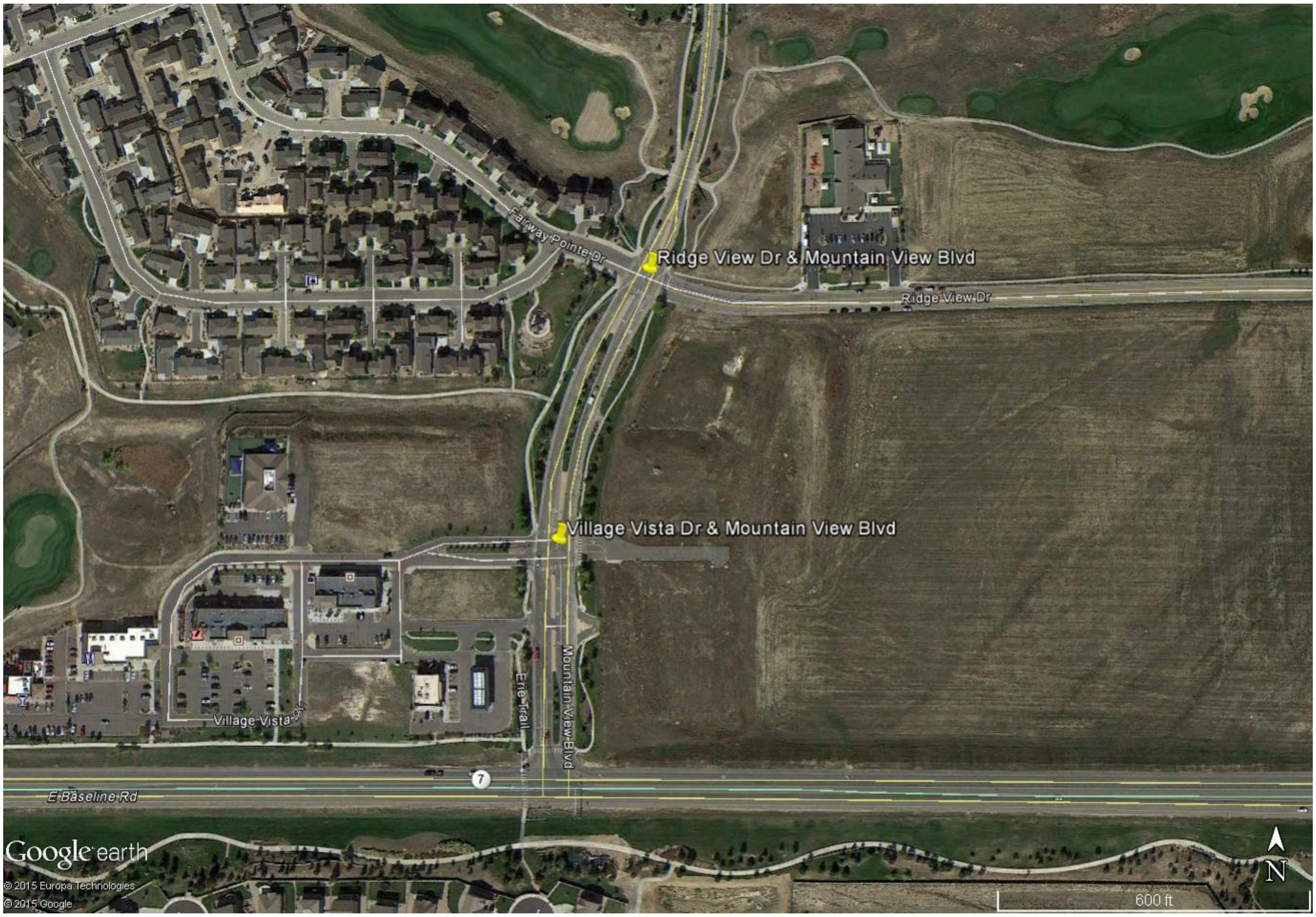
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
PM Peak
Ridge View Dr and Mountain View Blvd

File Name : RidgeViewMountainViewPM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 4

Image 1



Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
AM Peak
Ridge View Dr and Sheridan Pkwy

File Name : RidgeViewSheridanAM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 1

Groups Printed- Unshifted

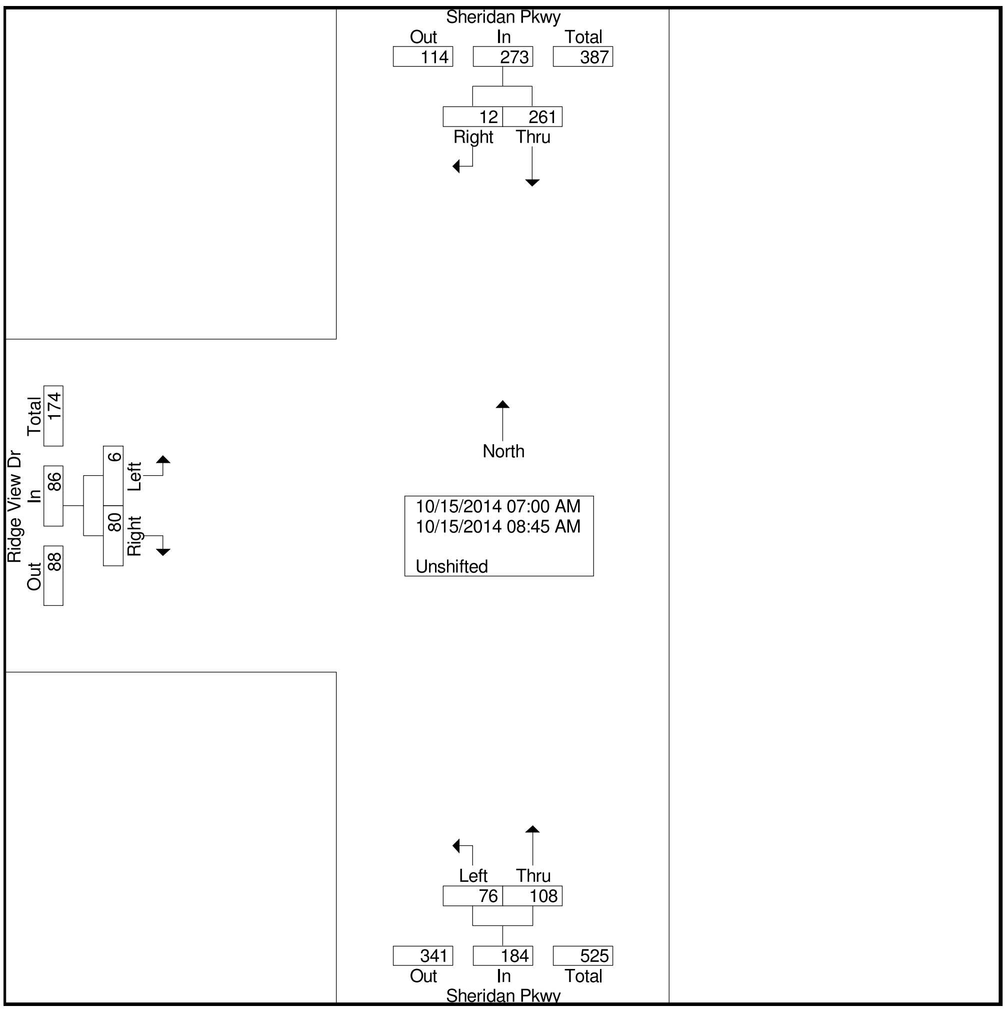
	Ridge View Dr Eastbound			Sheridan Pkwy Northbound			Sheridan Pkwy Southbound			
Start Time	Left	Right	App. Total	Left	Thru	App. Total	Thru	Right	App. Total	Int. Total
07:00 AM	0	8	8	7	15	22	25	1	26	56
07:15 AM	0	8	8	10	12	22	45	3	48	78
07:30 AM	0	14	14	8	8	16	35	1	36	66
07:45 AM	1	13	14	16	20	36	38	1	39	89
Total	1	43	44	41	55	96	143	6	149	289
08:00 AM	1	12	13	16	11	27	29	2	31	71
08:15 AM	3	11	14	12	11	23	34	3	37	74
08:30 AM	1	11	12	5	16	21	27	1	28	61
08:45 AM	0	3	3	2	15	17	28	0	28	48
Total	5	37	42	35	53	88	118	6	124	254
Grand Total	6	80	86	76	108	184	261	12	273	543
Apprch %	7	93		41.3	58.7		95.6	4.4		
Total %	1.1	14.7	15.8	14	19.9	33.9	48.1	2.2	50.3	

RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
AM Peak
Ridge View Dr and Sheridan Pkwy

File Name : RidgeViewSheridanAM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 2



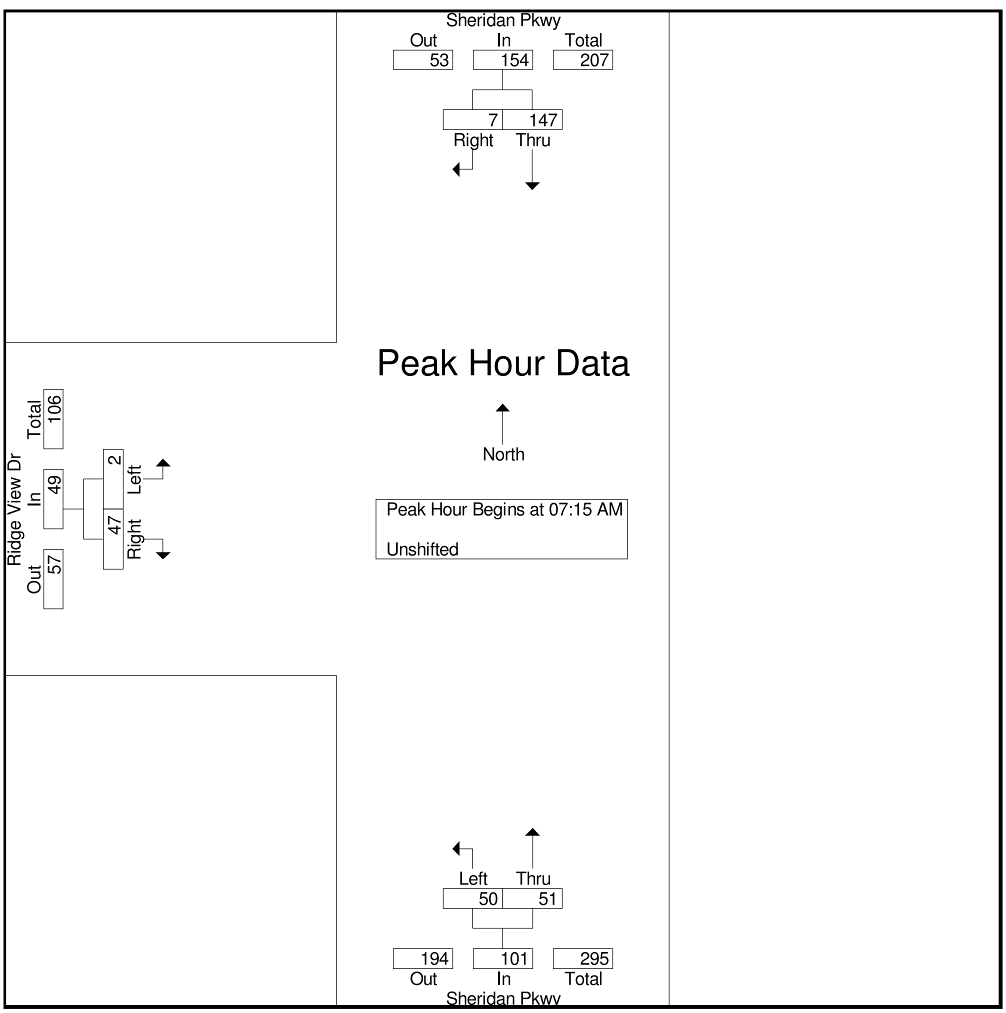
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
AM Peak
Ridge View Dr and Sheridan Pkwy

File Name : RidgeViewSheridanAM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 3

	Ridge View Dr Eastbound			Sheridan Pkwy Northbound			Sheridan Pkwy Southbound			
Start Time	Left	Right	App. Total	Left	Thru	App. Total	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 07:00 AM to 08:45 AM - Peak 1 of 1										
Peak Hour for Entire Intersection Begins at 07:15 AM										
07:15 AM	0	8	8	10	12	22	**45**	**3**	**48**	78
07:30 AM	0	**14**	**14**	8	8	16	35	1	36	66
07:45 AM	**1**	13	14	**16**	**20**	**36**	38	1	39	**89**
08:00 AM	1	12	13	16	11	27	29	2	31	71
Total Volume	2	47	49	50	51	101	147	7	154	304
% App. Total	4.1	95.9		49.5	50.5		95.5	4.5		
PHF	.500	.839	.875	.781	.638	.701	.817	.583	.802	.854



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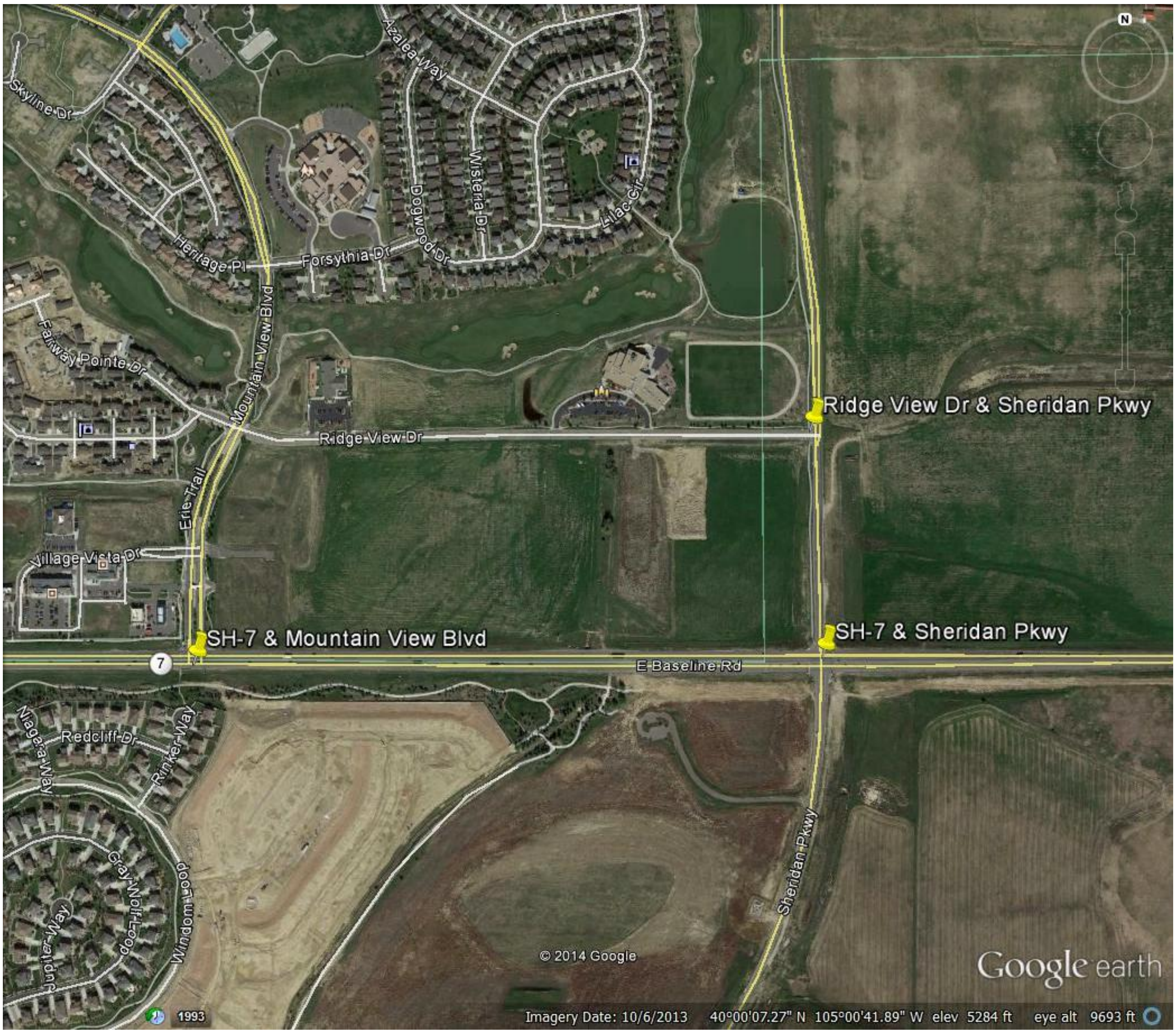
Morrison, CO 80465

Erie, CO
Erie King Soopers #129
AM Peak
Ridge View Dr and Sheridan Pkwy

File Name : RidgeViewSheridanAM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 4

Image 1



Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
PM Peak
Ridge View Dr and Sheridan Pkwy

File Name : RidgeViewSheridanPM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 1

Groups Printed- Unshifted

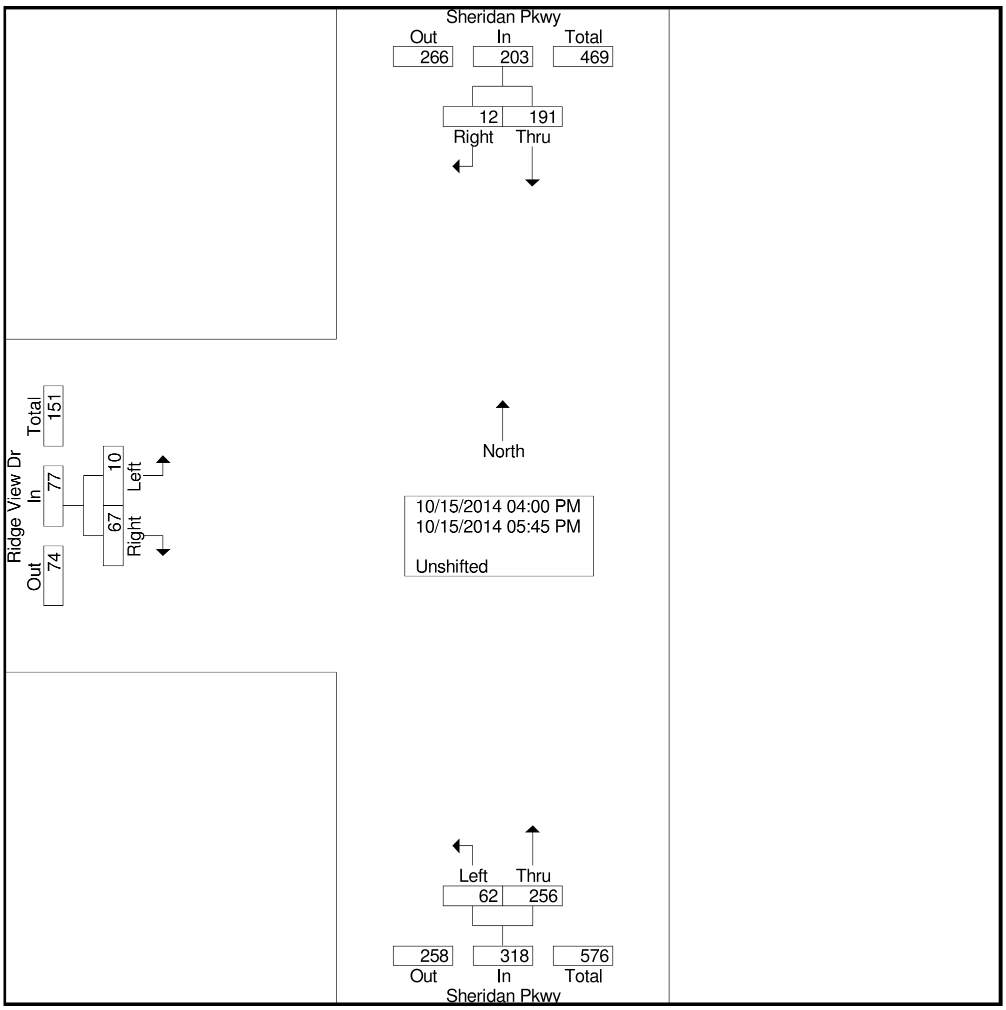
	Ridge View Dr Eastbound			Sheridan Pkwy Northbound			Sheridan Pkwy Southbound			
Start Time	Left	Right	App. Total	Left	Thru	App. Total	Thru	Right	App. Total	Int. Total
04:00 PM	0	8	8	6	26	32	16	2	18	58
04:15 PM	1	6	7	9	22	31	20	1	21	59
04:30 PM	0	9	9	5	41	46	29	2	31	86
04:45 PM	1	8	9	8	26	34	18	1	19	62
Total	2	31	33	28	115	143	83	6	89	265
05:00 PM	1	11	12	9	24	33	28	2	30	75
05:15 PM	4	6	10	12	40	52	29	1	30	92
05:30 PM	3	12	15	8	45	53	22	1	23	91
05:45 PM	0	7	7	5	32	37	29	2	31	75
Total	8	36	44	34	141	175	108	6	114	333
Grand Total	10	67	77	62	256	318	191	12	203	598
Apprch %	13	87		19.5	80.5		94.1	5.9		
Total %	1.7	11.2	12.9	10.4	42.8	53.2	31.9	2	33.9	

RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
PM Peak
Ridge View Dr and Sheridan Pkwy

File Name : RidgeViewSheridanPM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 2



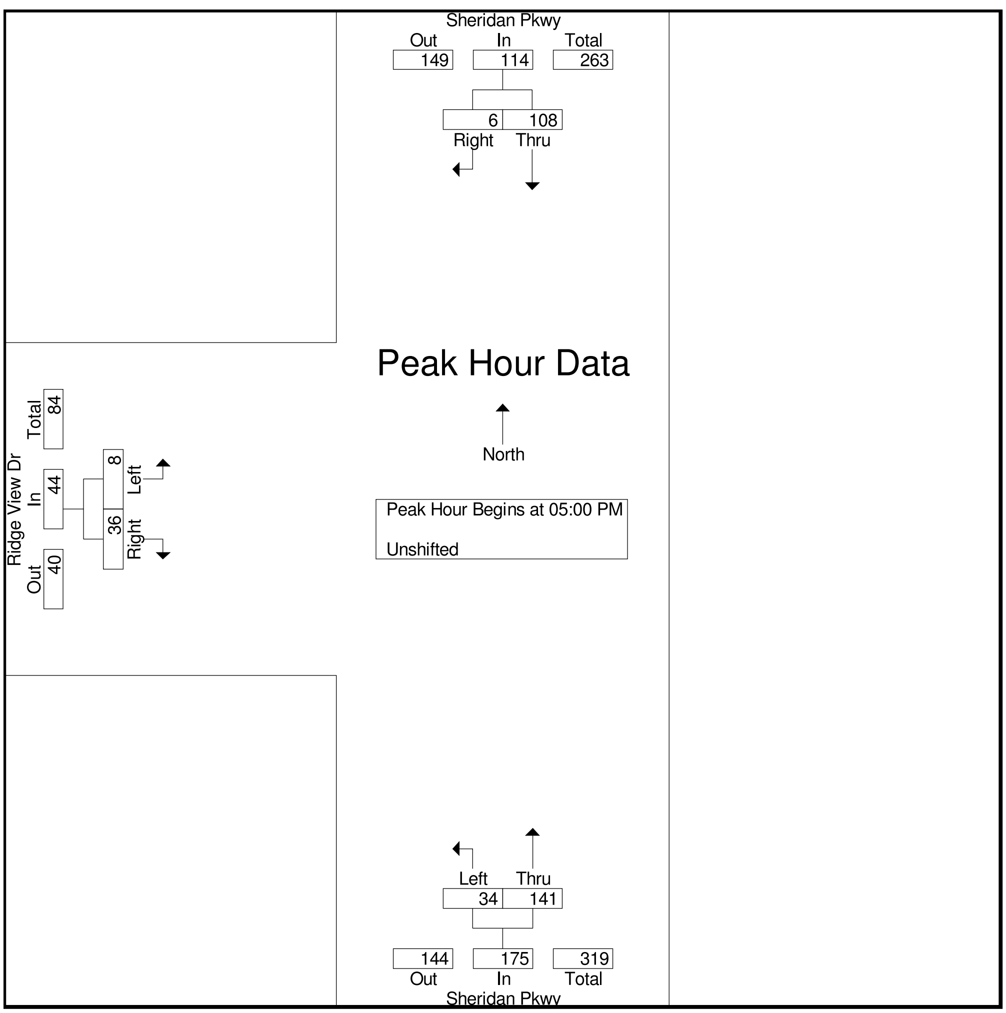
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
PM Peak
Ridge View Dr and Sheridan Pkwy

File Name : RidgeViewSheridanPM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 3

	Ridge View Dr Eastbound			Sheridan Pkwy Northbound			Sheridan Pkwy Southbound			
Start Time	Left	Right	App. Total	Left	Thru	App. Total	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 04:00 PM to 05:45 PM - Peak 1 of 1										
Peak Hour for Entire Intersection Begins at 05:00 PM										
05:00 PM	1	11	12	9	24	33	28	**2**	30	75
05:15 PM	**4**	6	10	**12**	40	52	**29**	1	30	**92**
05:30 PM	3	**12**	**15**	8	**45**	**53**	22	1	23	91
05:45 PM	0	7	7	5	32	37	29	2	**31**	75
Total Volume	8	36	44	34	141	175	108	6	114	333
% App. Total	18.2	81.8		19.4	80.6		94.7	5.3		
PHF	.500	.750	.733	.708	.783	.825	.931	.750	.919	.905



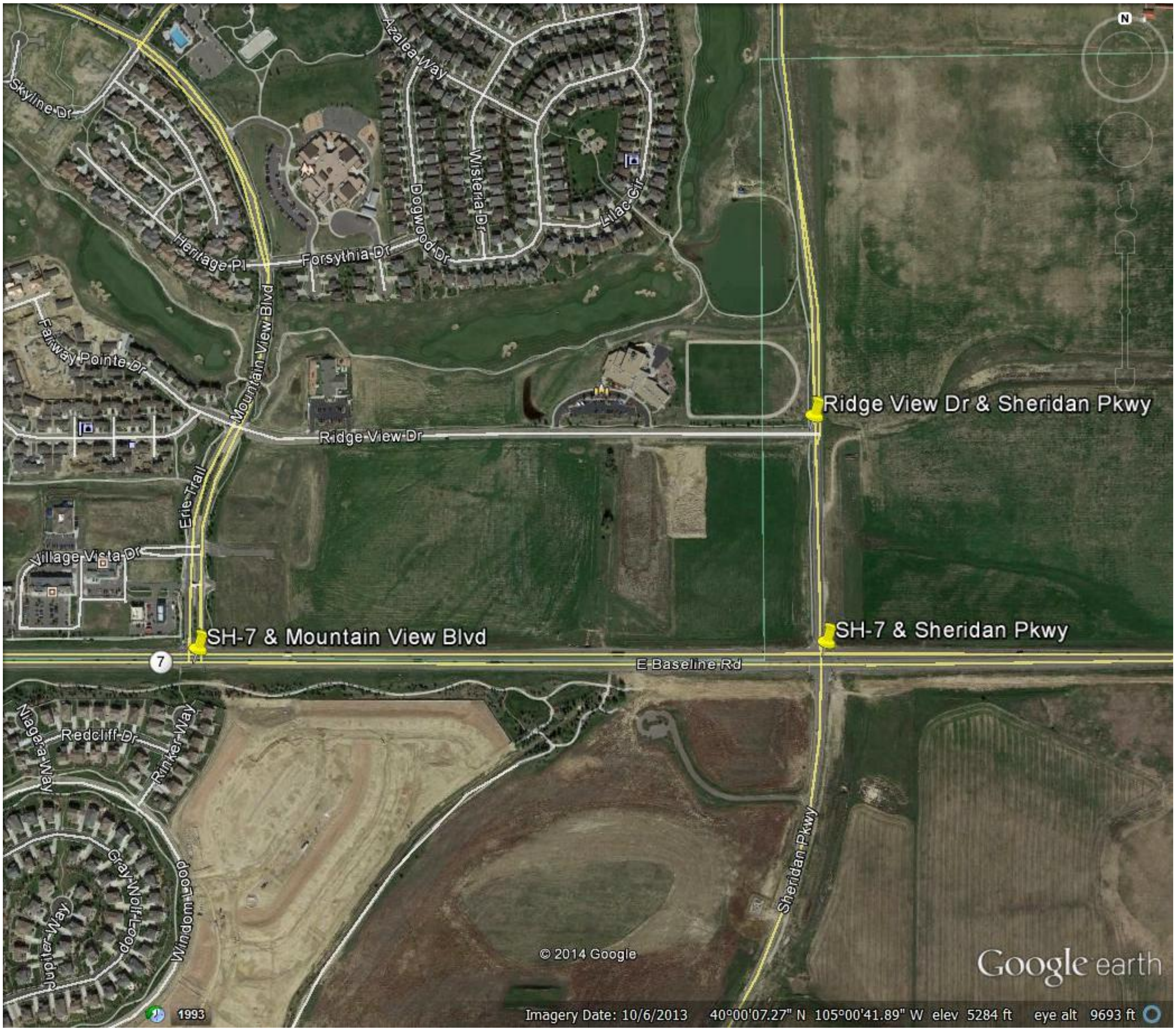
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
PM Peak
Ridge View Dr and Sheridan Pkwy

File Name : RidgeViewSheridanPM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 4

Image 1



Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
AM Peak
SH - 7 and Mountain View Blvd

File Name : SH7MountianViewAM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 1

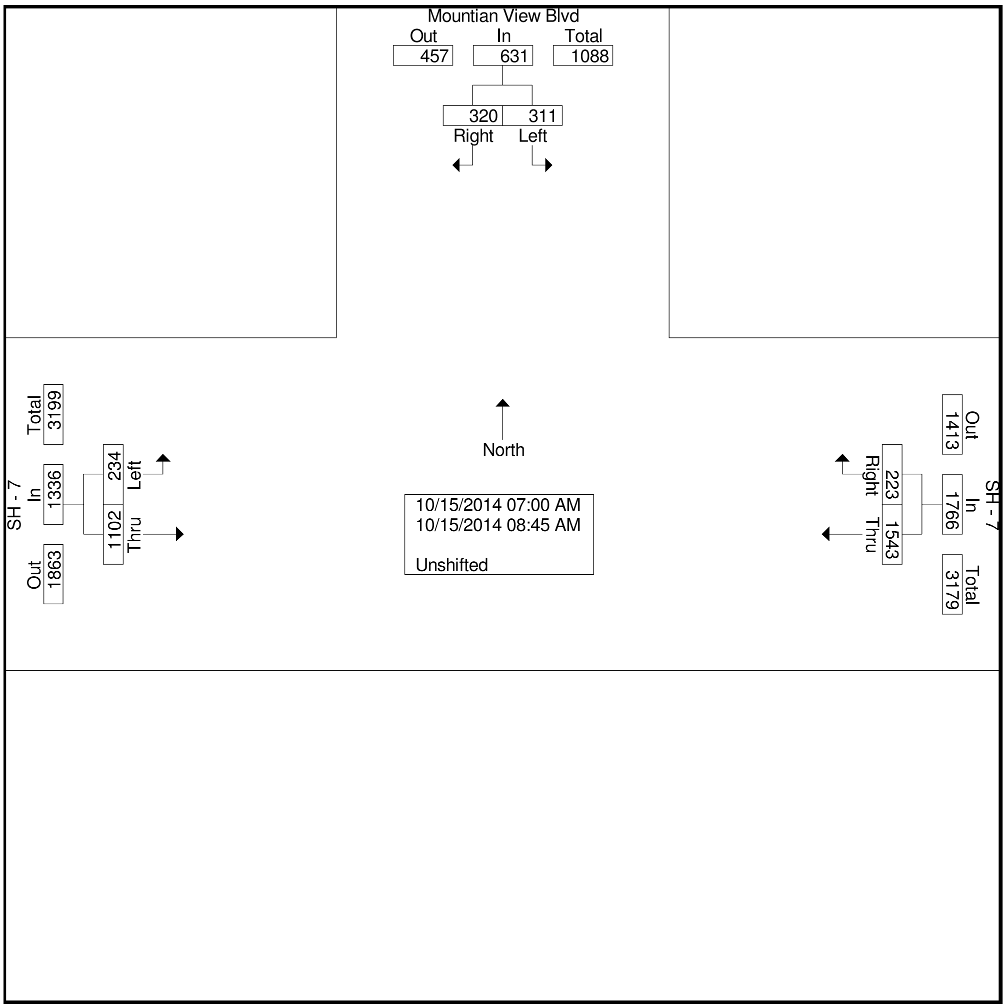
Groups Printed- Unshifted

	SH - 7 Eastbound			SH - 7 Westbound			Mountian View Blvd Southbound			
Start Time	Left	Thru	App. Total	Thru	Right	App. Total	Left	Right	App. Total	Int. Total
07:00 AM	22	129	151	209	21	230	27	47	74	455
07:15 AM	17	153	170	234	23	257	39	42	81	508
07:30 AM	20	149	169	202	28	230	52	46	98	497
07:45 AM	36	150	186	202	34	236	35	26	61	483
Total	95	581	676	847	106	953	153	161	314	1943
08:00 AM	41	164	205	185	39	224	45	39	84	513
08:15 AM	27	112	139	172	25	197	35	41	76	412
08:30 AM	27	137	164	200	17	217	39	33	72	453
08:45 AM	44	108	152	139	36	175	39	46	85	412
Total	139	521	660	696	117	813	158	159	317	1790
Grand Total	234	1102	1336	1543	223	1766	311	320	631	3733
Apprch %	17.5	82.5		87.4	12.6		49.3	50.7		
Total %	6.3	29.5	35.8	41.3	6	47.3	8.3	8.6	16.9	

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
AM Peak
SH - 7 and Mountain View Blvd

File Name : SH7MountianViewAM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 2



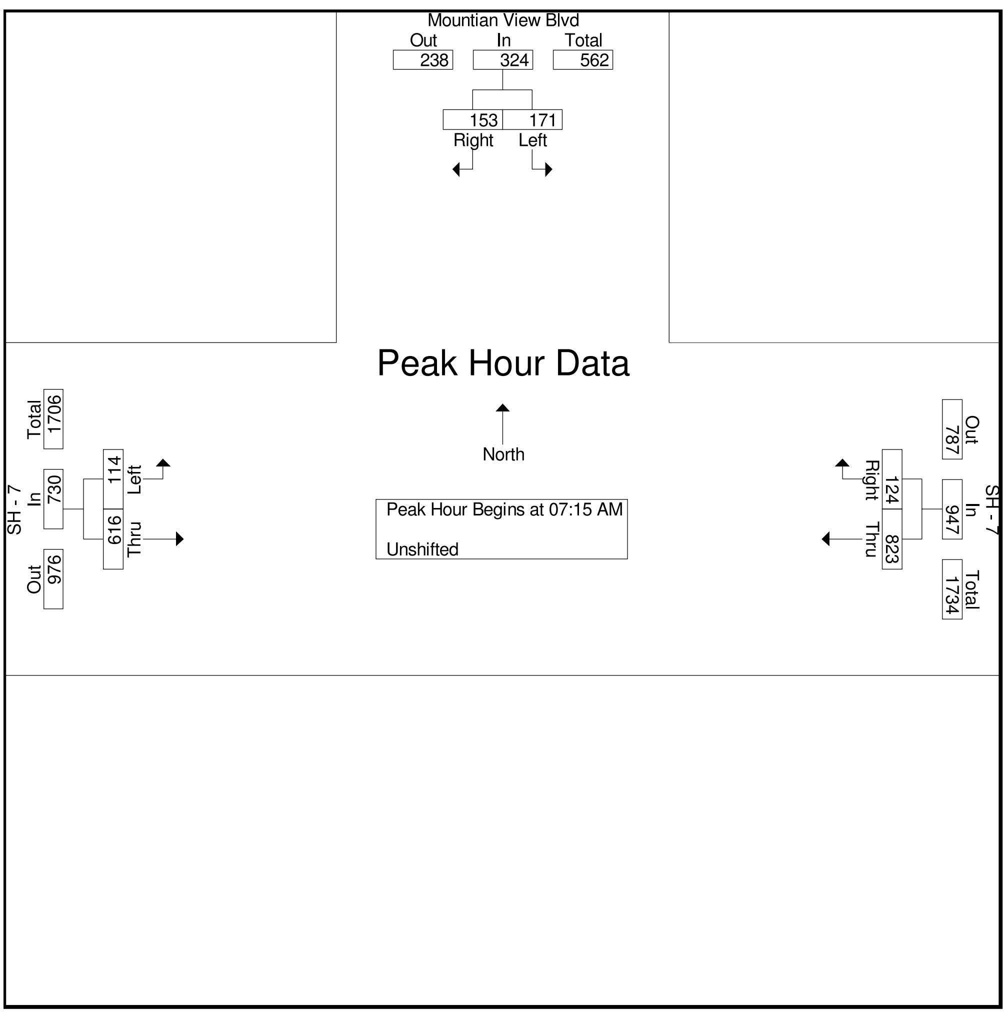
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
AM Peak
SH - 7 and Mountain View Blvd

File Name : SH7MountianViewAM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 3

	SH - 7 Eastbound			SH - 7 Westbound			Mountian View Blvd Southbound			
Start Time	Left	Thru	App. Total	Thru	Right	App. Total	Left	Right	App. Total	Int. Total
Peak Hour Analysis From 07:00 AM to 08:45 AM - Peak 1 of 1										
Peak Hour for Entire Intersection Begins at 07:15 AM										
07:15 AM	17	153	170	**234**	23	**257**	39	42	81	508
07:30 AM	20	149	169	202	28	230	**52**	**46**	**98**	497
07:45 AM	36	150	186	202	34	236	35	26	61	483
08:00 AM	**41**	**164**	**205**	185	**39**	224	45	39	84	**513**
Total Volume	114	616	730	823	124	947	171	153	324	2001
% App. Total	15.6	84.4		86.9	13.1		52.8	47.2		
PHF	.695	.939	.890	.879	.795	.921	.822	.832	.827	.975



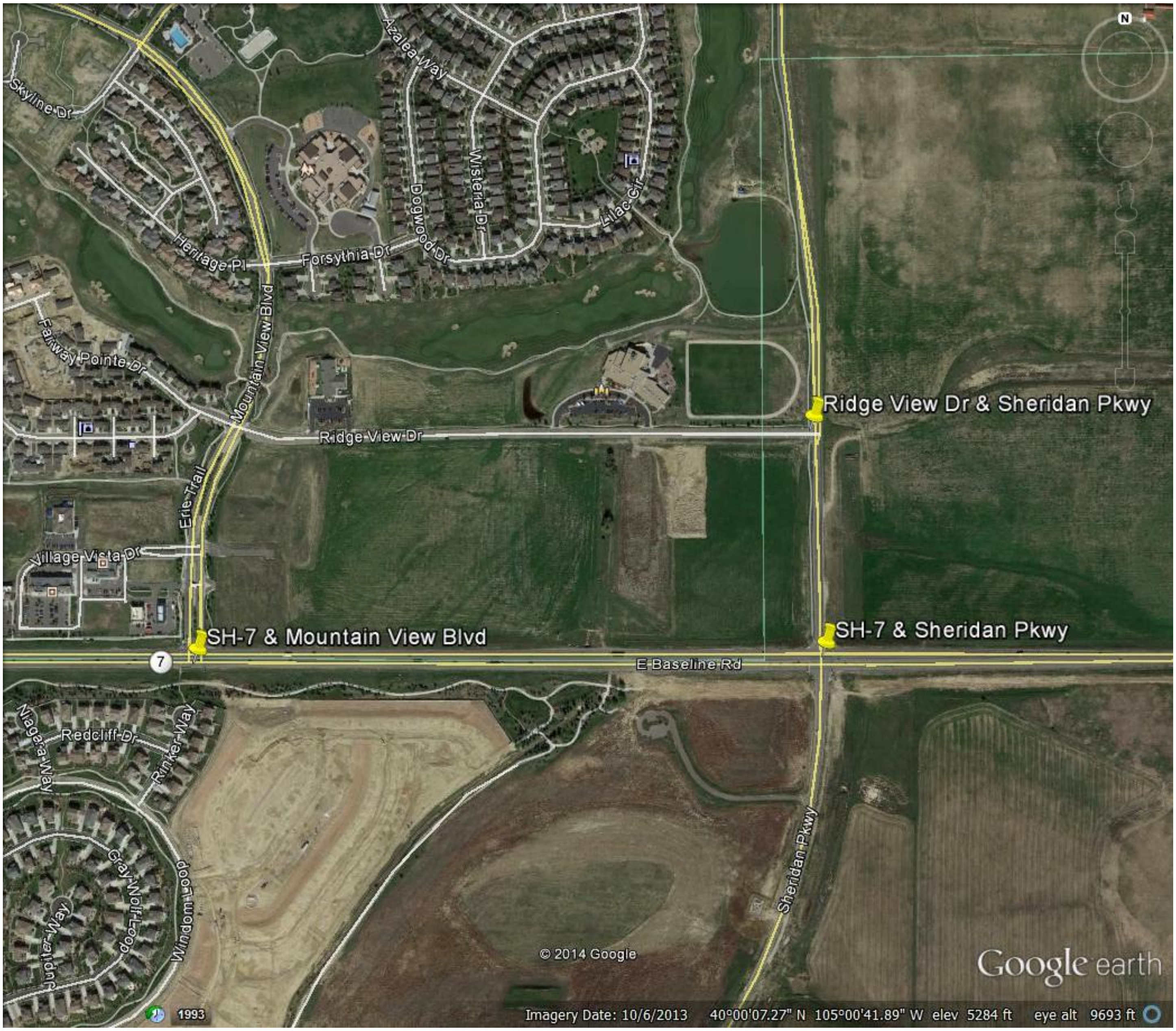
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
AM Peak
SH - 7 and Mountain View Blvd

File Name : SH7MountianViewAM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 4

Image 1



RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
PM Peak
SH - 7 and Mountain View Blvd

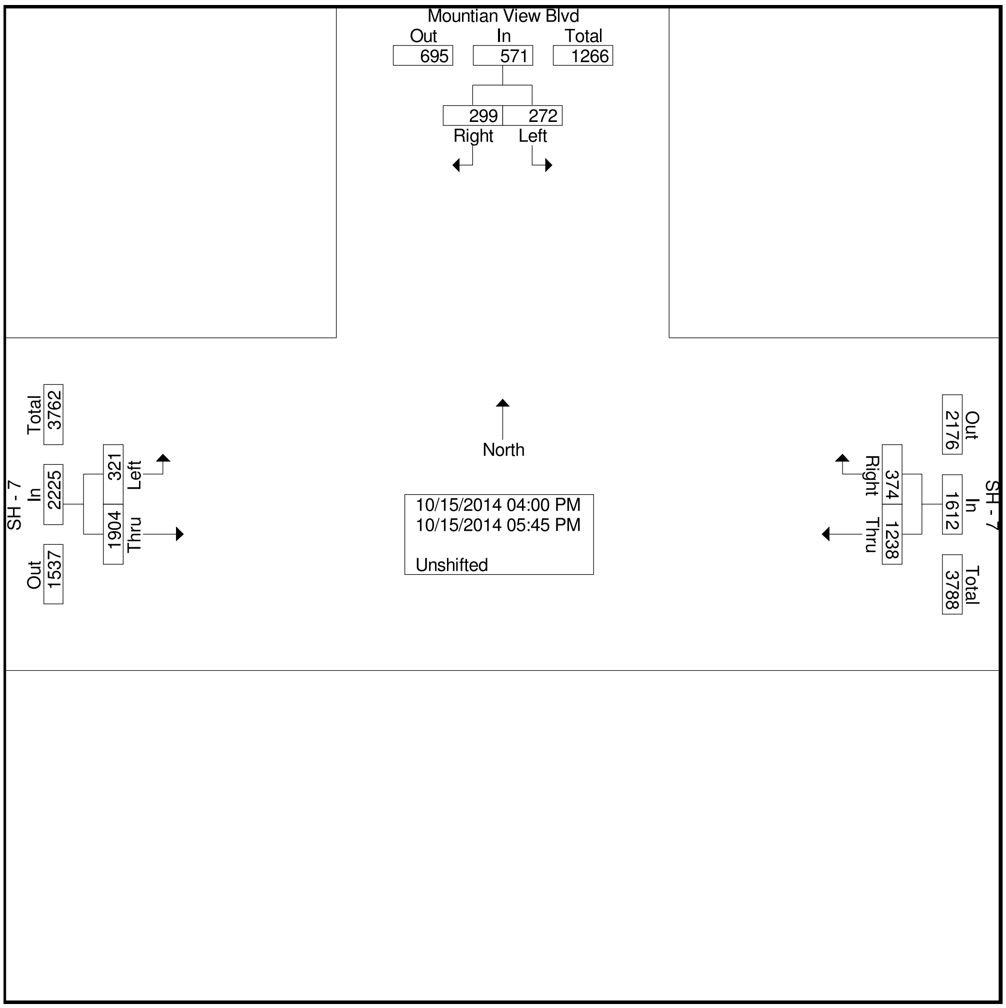
File Name : SH7MountianViewPM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 1

Groups Printed- Unshifted

	SH - 7 Eastbound			SH - 7 Westbound			Mountian View Blvd Southbound			
Start Time	Left	Thru	App. Total	Thru	Right	App. Total	Left	Right	App. Total	Int. Total
04:00 PM	38	211	249	144	25	169	24	42	66	484
04:15 PM	34	251	285	150	44	194	24	41	65	544
04:30 PM	38	250	288	133	51	184	45	21	66	538
04:45 PM	30	259	289	154	41	195	24	41	65	549
Total	140	971	1111	581	161	742	117	145	262	2115
05:00 PM	48	250	298	180	53	233	41	27	68	599
05:15 PM	53	248	301	166	63	229	42	39	81	611
05:30 PM	46	219	265	165	51	216	38	44	82	563
05:45 PM	34	216	250	146	46	192	34	44	78	520
Total	181	933	1114	657	213	870	155	154	309	2293
Grand Total	321	1904	2225	1238	374	1612	272	299	571	4408
Apprch %	14.4	85.6		76.8	23.2		47.6	52.4		
Total %	7.3	43.2	50.5	28.1	8.5	36.6	6.2	6.8	13	

Erie, CO
Erie King Soopers #129
PM Peak
SH - 7 and Mountain View Blvd

File Name : SH7MountianViewPM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 2



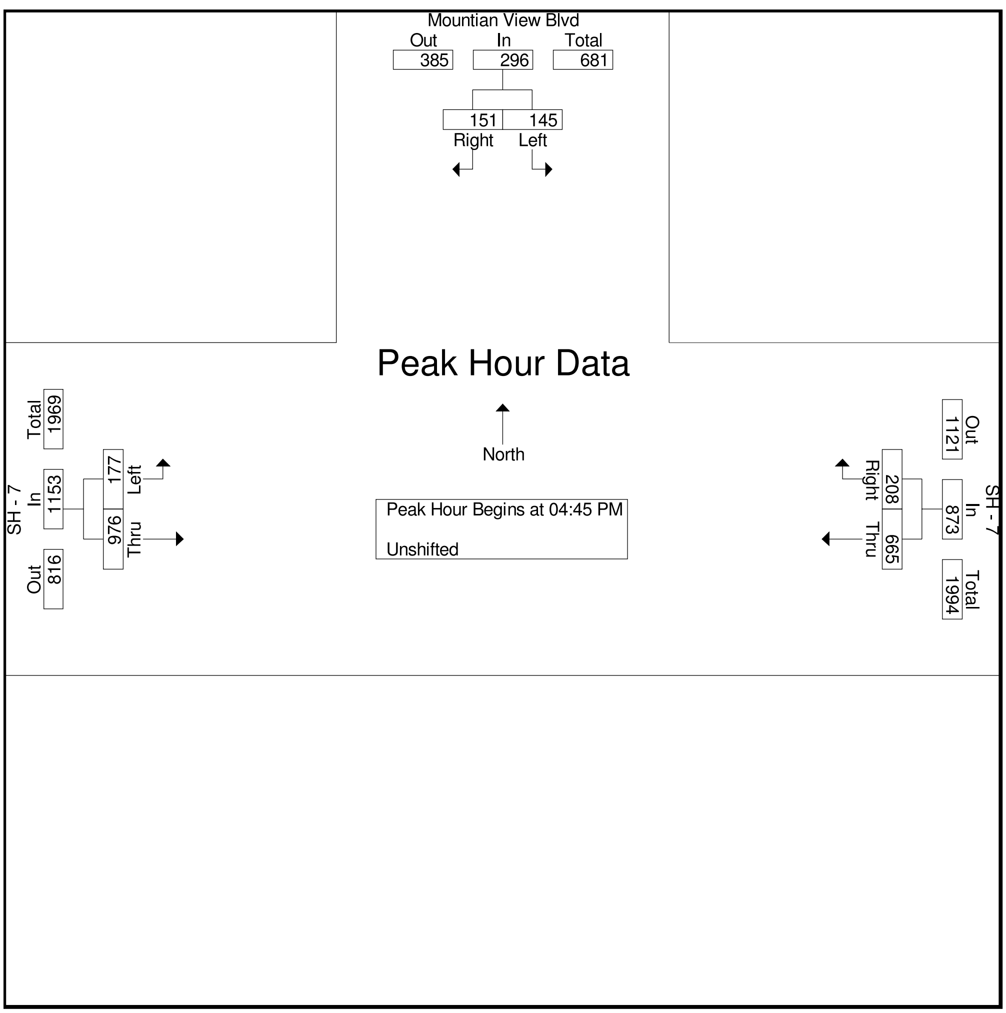
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
PM Peak
SH - 7 and Mountain View Blvd

File Name : SH7MountianViewPM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 3

	SH - 7 Eastbound			SH - 7 Westbound			Mountian View Blvd Southbound			
Start Time	Left	Thru	App. Total	Thru	Right	App. Total	Left	Right	App. Total	Int. Total
Peak Hour Analysis From 04:00 PM to 05:45 PM - Peak 1 of 1										
Peak Hour for Entire Intersection Begins at 04:45 PM										
04:45 PM	30	**259**	289	154	41	195	24	41	65	549
05:00 PM	48	250	298	**180**	53	**233**	41	27	68	599
05:15 PM	**53**	248	**301**	166	**63**	229	**42**	39	81	**611**
05:30 PM	46	219	265	165	51	216	38	**44**	**82**	563
Total Volume	177	976	1153	665	208	873	145	151	296	2322
% App. Total	15.4	84.6		76.2	23.8		49	51		
PHF	.835	.942	.958	.924	.825	.937	.863	.858	.902	.950



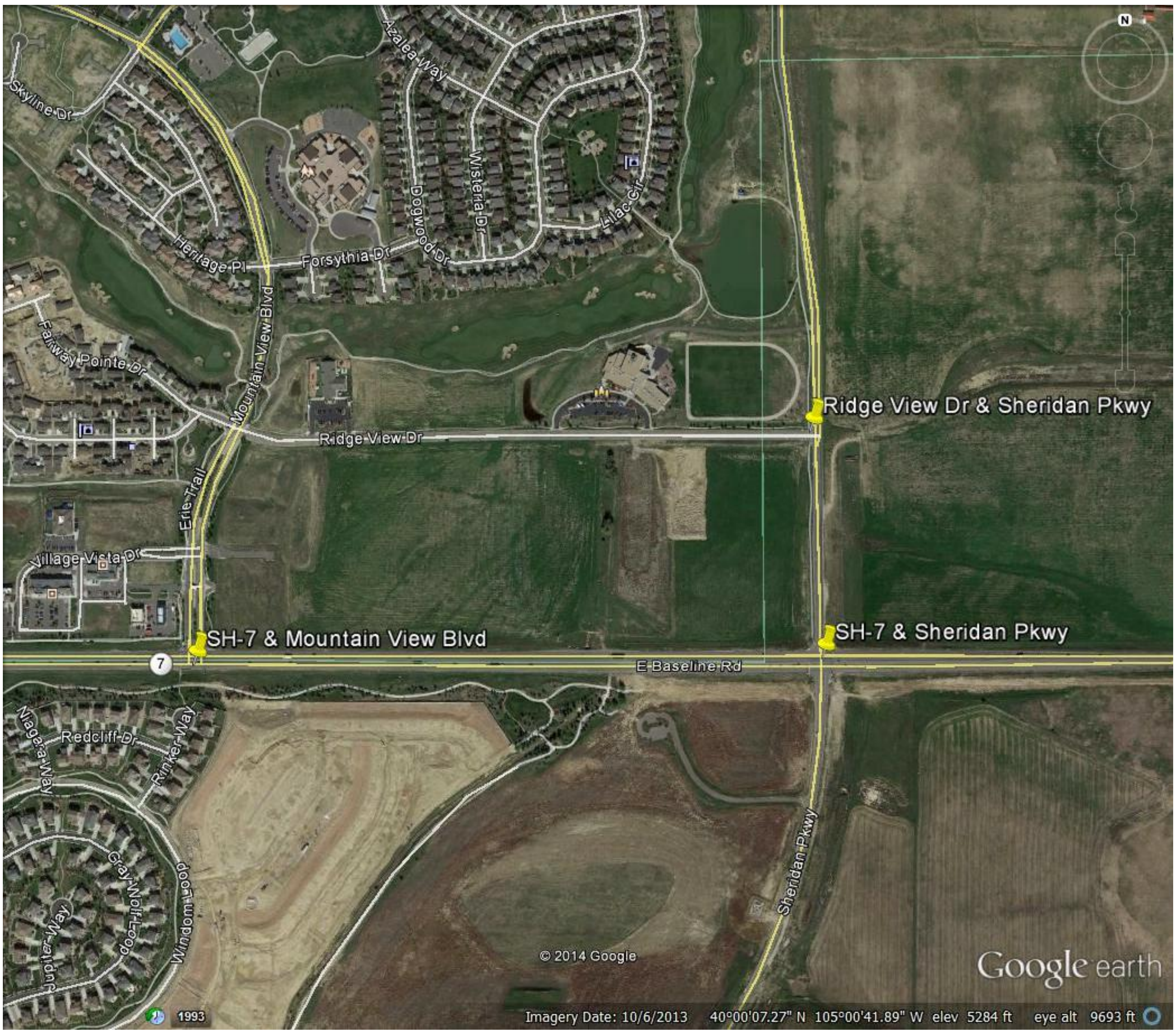
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
PM Peak
SH - 7 and Mountain View Blvd

File Name : SH7MountianViewPM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 4

Image 1



RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
AM Peak
SH - 7 and Sheridan Pkwy

File Name : SH7SheridanAM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 1

Groups Printed- Unshifted

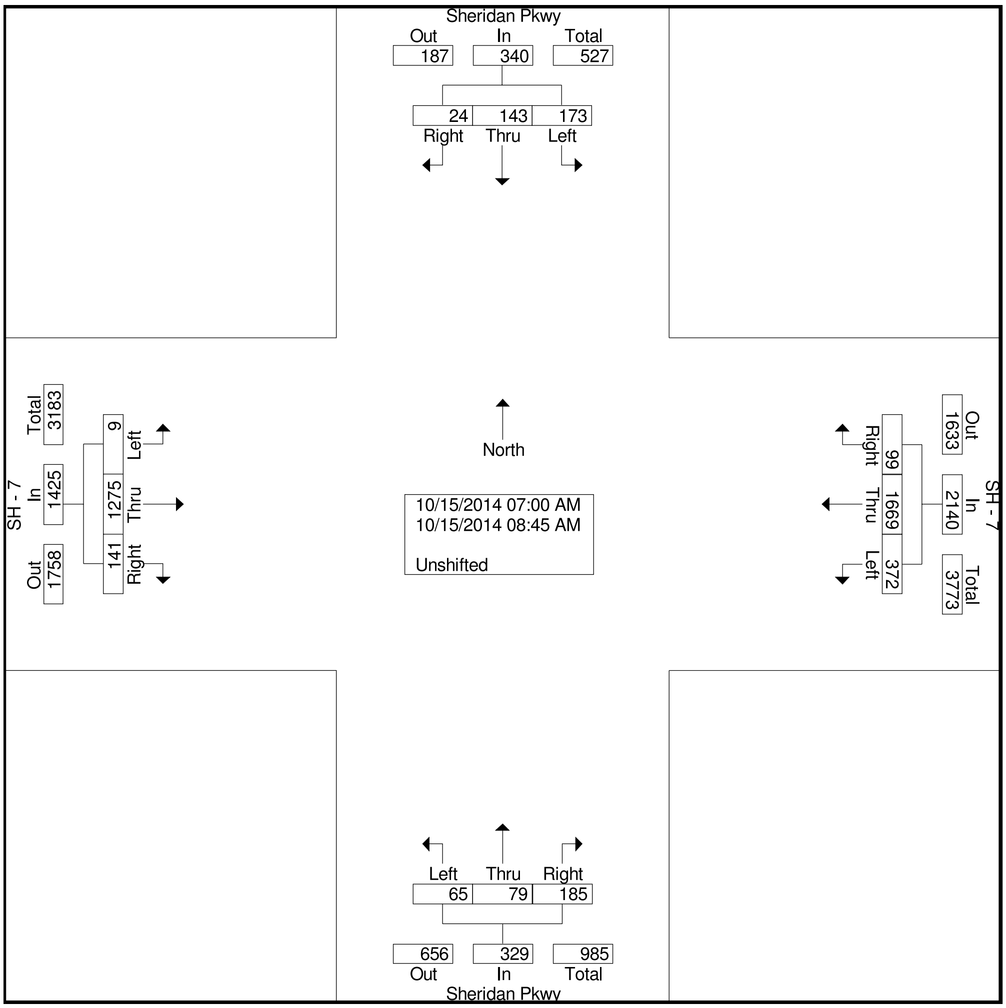
	SH - 7 Eastbound				SH - 7 Westbound				Sheridan Pkwy Northbound				Sheridan Pkwy Southbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
07:00 AM	1	144	3	148	37	217	13	267	5	10	17	32	19	13	4	36	483
07:15 AM	1	174	10	185	50	257	15	322	4	10	22	36	28	18	3	49	592
07:30 AM	1	174	34	209	83	208	7	298	7	5	22	34	19	27	3	49	590
07:45 AM	0	174	34	208	73	211	19	303	20	17	42	79	17	24	4	45	635
Total	3	666	81	750	243	893	54	1190	36	42	103	181	83	82	14	179	2300
08:00 AM	0	185	18	203	46	223	11	280	8	12	27	47	21	21	2	44	574
08:15 AM	1	136	11	148	28	185	13	226	10	11	18	39	33	16	3	52	465
08:30 AM	1	142	18	161	31	205	10	246	3	12	19	34	18	16	2	36	477
08:45 AM	4	146	13	163	24	163	11	198	8	2	18	28	18	8	3	29	418
Total	6	609	60	675	129	776	45	950	29	37	82	148	90	61	10	161	1934
Grand Total	9	1275	141	1425	372	1669	99	2140	65	79	185	329	173	143	24	340	4234
Apprch %	0.6	89.5	9.9		17.4	78	4.6		19.8	24	56.2		50.9	42.1	7.1		
Total %	0.2	30.1	3.3	33.7	8.8	39.4	2.3	50.5	1.5	1.9	4.4	7.8	4.1	3.4	0.6	8	

RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
AM Peak
SH - 7 and Sheridan Pkwy

File Name : SH7SheridanAM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 2



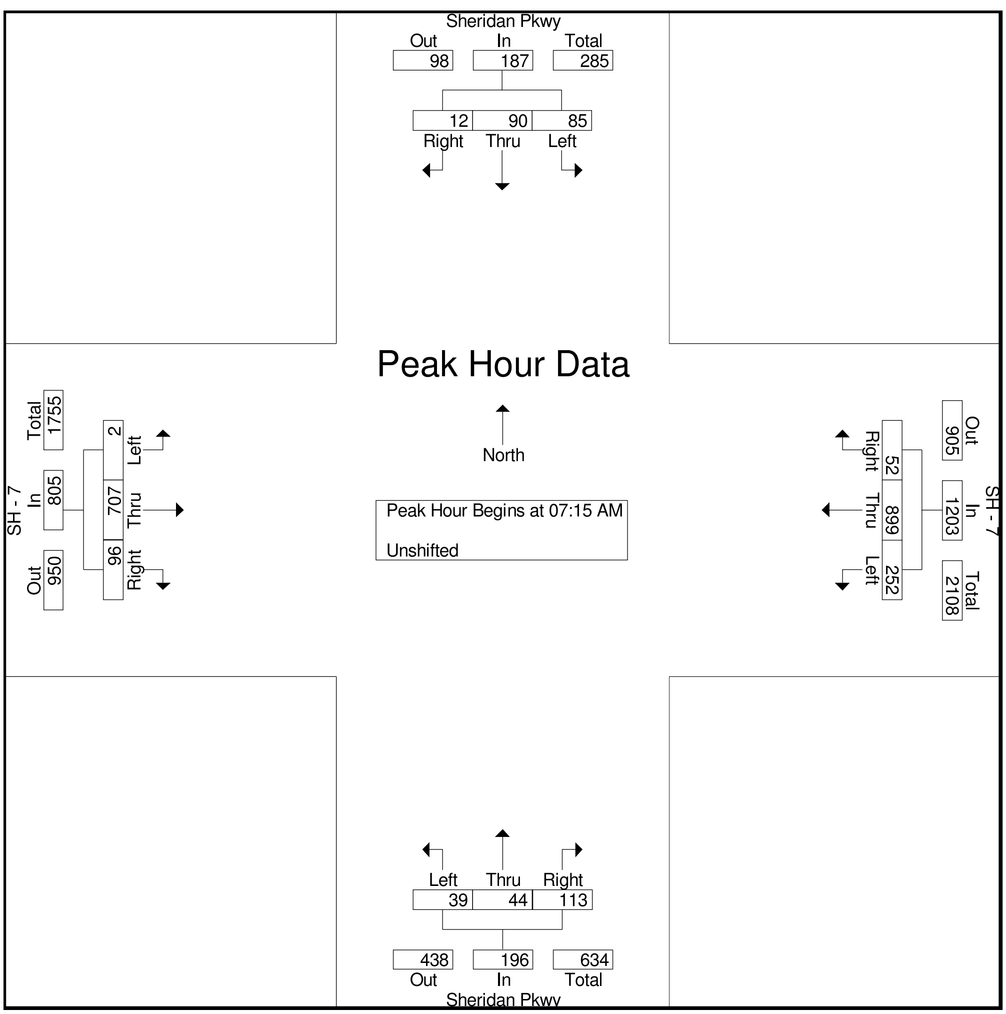
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
AM Peak
SH - 7 and Sheridan Pkwy

File Name : SH7SheridanAM
Site Code : IPO 60
Start Date : 10/15/2014

	SH - 7 Eastbound				SH - 7 Westbound				Sheridan Pkwy Northbound				Sheridan Pkwy Southbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 07:00 AM to 08:45 AM - Peak 1 of 1																	
Peak Hour for Entire Intersection Begins at 07:15 AM																	
07:15 AM	**1**	174	10	185	50	**257**	15	**322**	4	10	22	36	**28**	18	3	**49**	592
07:30 AM	1	174	**34**	**209**	**83**	208	7	298	7	5	22	34	19	**27**	3	49	590
07:45 AM	0	174	34	208	73	211	**19**	303	**20**	**17**	**42**	**79**	17	24	**4**	45	**635**
08:00 AM	0	**185**	18	203	46	223	11	280	8	12	27	47	21	21	2	44	574
Total Volume	2	707	96	805	252	899	52	1203	39	44	113	196	85	90	12	187	2391
% App. Total	0.2	87.8	11.9		20.9	74.7	4.3		19.9	22.4	57.7		45.5	48.1	6.4		
PHF	.500	.955	.706	.963	.759	.875	.684	.934	.488	.647	.673	.620	.759	.833	.750	.954	.941

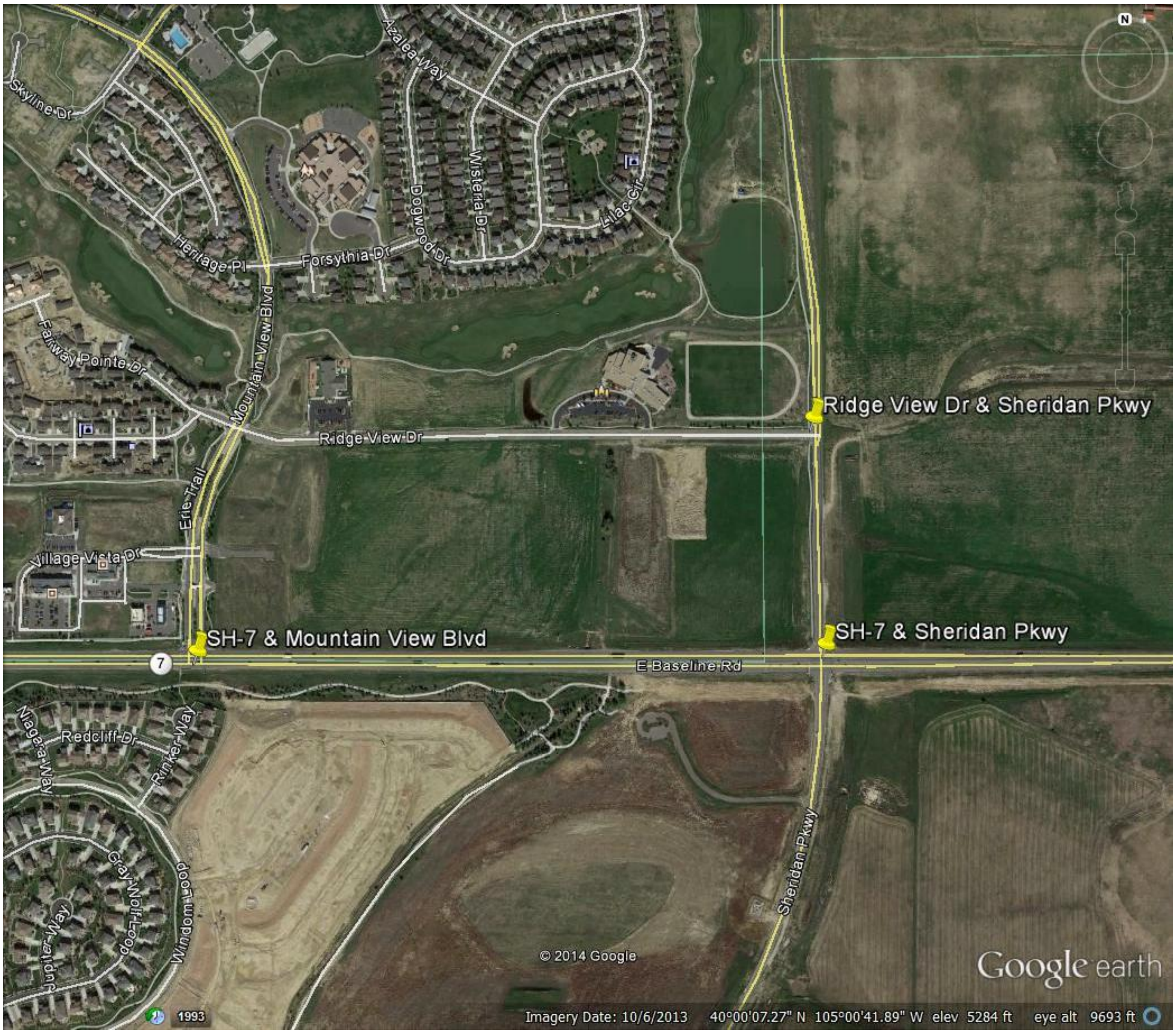


RDC
Ridgeview Data
Collection

Morrison, CO 80465

Erie, CO	File Name : SH7SheridanAM
Erie King Soopers #129	Site Code : IPO 60
AM Peak	Start Date : 10/15/2014
SH - 7 and Sheridan Pkwy	Page No : 4

Image 1



Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
PM Peak
SH - 7 and Sheridan Pkwy

File Name : SH7SheridanPM
Site Code : IPO 60
Start Date : 10/15/2014

Groups Printed- Unshifted

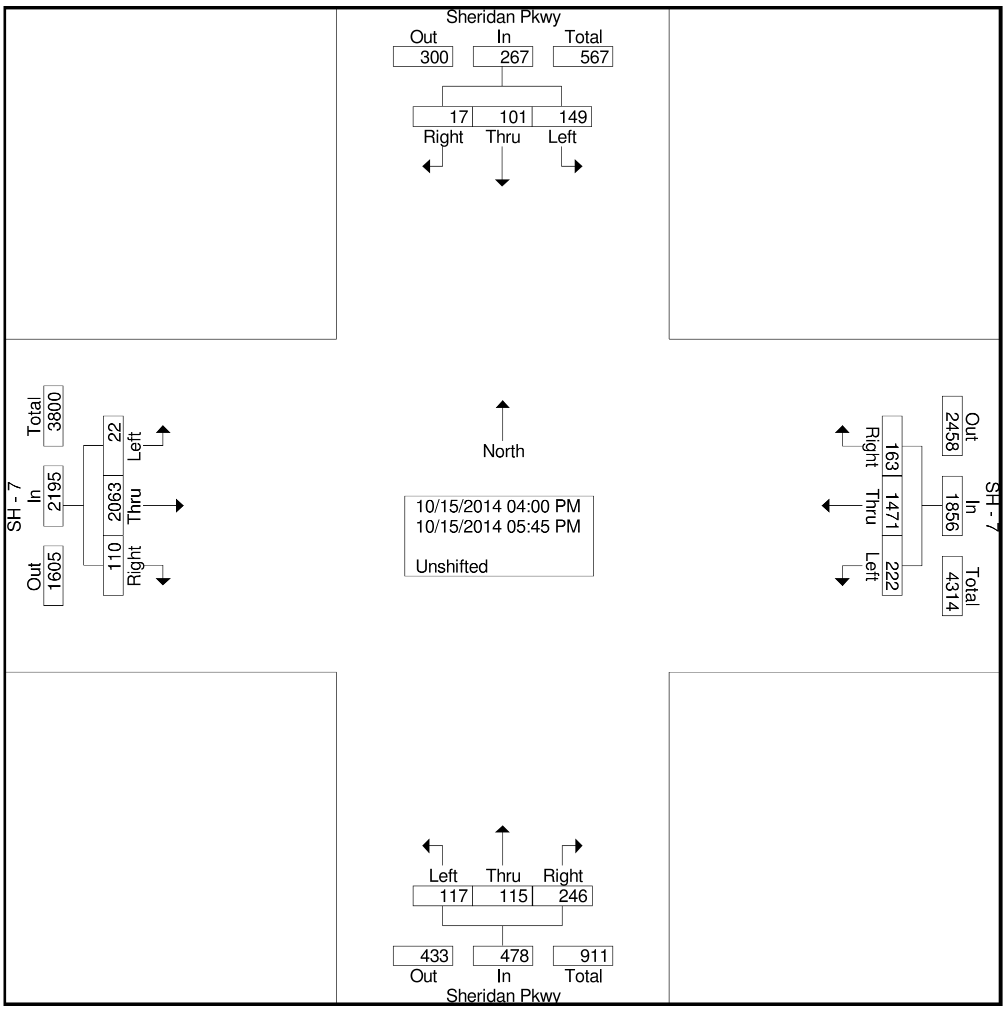
	SH - 7 Eastbound				SH - 7 Westbound				Sheridan Pkwy Northbound				Sheridan Pkwy Southbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
04:00 PM	3	232	6	241	30	167	16	213	6	13	16	35	16	6	3	25	514
04:15 PM	2	255	8	265	28	178	18	224	17	7	37	61	17	9	3	29	579
04:30 PM	5	272	8	285	31	159	20	210	21	16	32	69	19	15	1	35	599
04:45 PM	2	279	16	297	26	178	16	220	13	16	28	57	13	14	0	27	601
Total	12	1038	38	1088	115	682	70	867	57	52	113	222	65	44	7	116	2293
05:00 PM	2	286	18	306	26	203	18	247	22	13	32	67	22	9	5	36	656
05:15 PM	2	259	30	291	30	212	26	268	14	21	42	77	22	18	3	43	679
05:30 PM	5	244	13	262	28	198	28	254	15	17	30	62	19	17	0	36	614
05:45 PM	1	236	11	248	23	176	21	220	9	12	29	50	21	13	2	36	554
Total	10	1025	72	1107	107	789	93	989	60	63	133	256	84	57	10	151	2503
Grand Total	22	2063	110	2195	222	1471	163	1856	117	115	246	478	149	101	17	267	4796
Apprch %	1	94	5		12	79.3	8.8		24.5	24.1	51.5		55.8	37.8	6.4		
Total %	0.5	43	2.3	45.8	4.6	30.7	3.4	38.7	2.4	2.4	5.1	10	3.1	2.1	0.4	5.6	

Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
PM Peak
SH - 7 and Sheridan Pkwy

File Name : SH7SheridanPM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 2



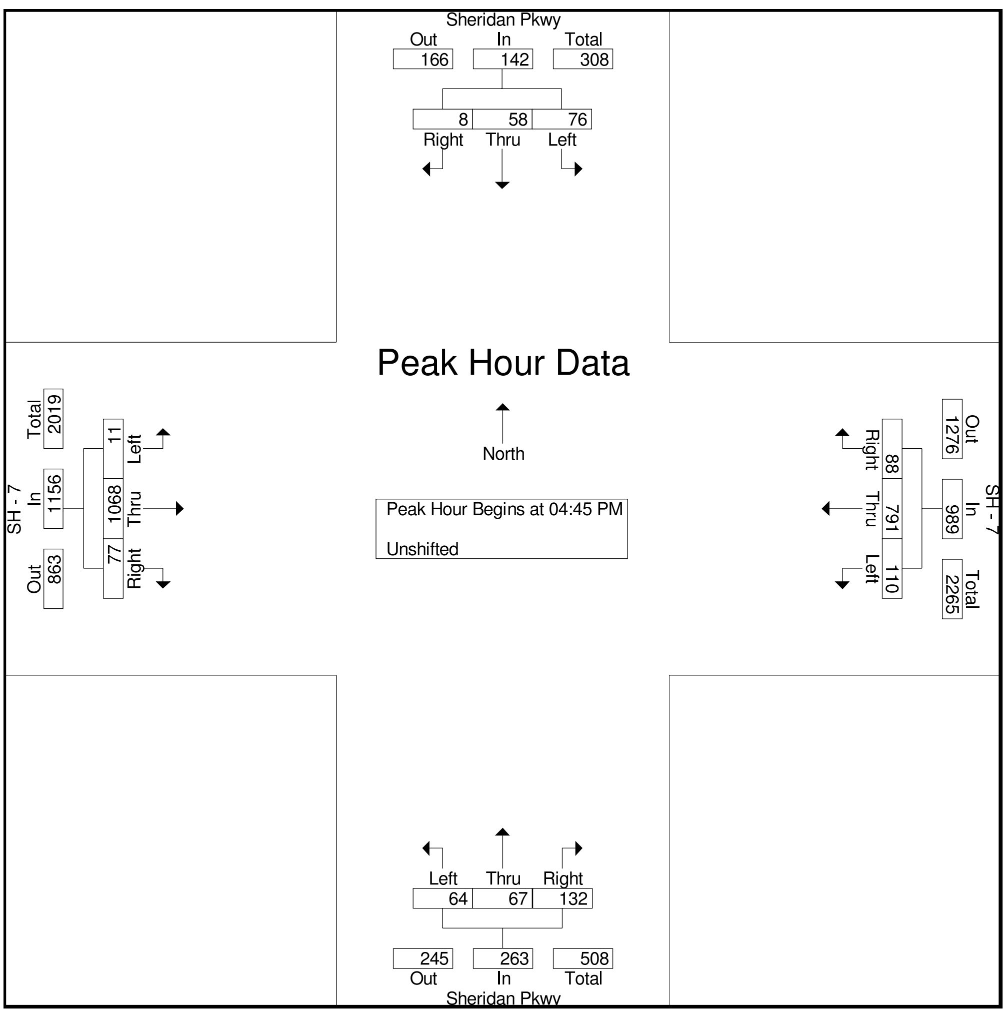
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
PM Peak
SH - 7 and Sheridan Pkwy

File Name : SH7SheridanPM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 3

	SH - 7 Eastbound				SH - 7 Westbound				Sheridan Pkwy Northbound				Sheridan Pkwy Southbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 04:00 PM to 05:45 PM - Peak 1 of 1																	
Peak Hour for Entire Intersection Begins at 04:45 PM																	
04:45 PM	2	279	16	297	26	178	16	220	13	16	28	57	13	14	0	27	601
05:00 PM	2	**286**	18	**306**	26	203	18	247	**22**	13	32	67	**22**	9	**5**	36	656
05:15 PM	2	259	**30**	291	**30**	**212**	26	**268**	14	**21**	**42**	**77**	22	**18**	3	**43**	**679**
05:30 PM	**5**	244	13	262	28	198	**28**	254	15	17	30	62	19	17	0	36	614
Total Volume	11	1068	77	1156	110	791	88	989	64	67	132	263	76	58	8	142	2550
% App. Total	1	92.4	6.7		11.1	80	8.9		24.3	25.5	50.2		53.5	40.8	5.6		
PHF	.550	.934	.642	.944	.917	.933	.786	.923	.727	.798	.786	.854	.864	.806	.400	.826	.939



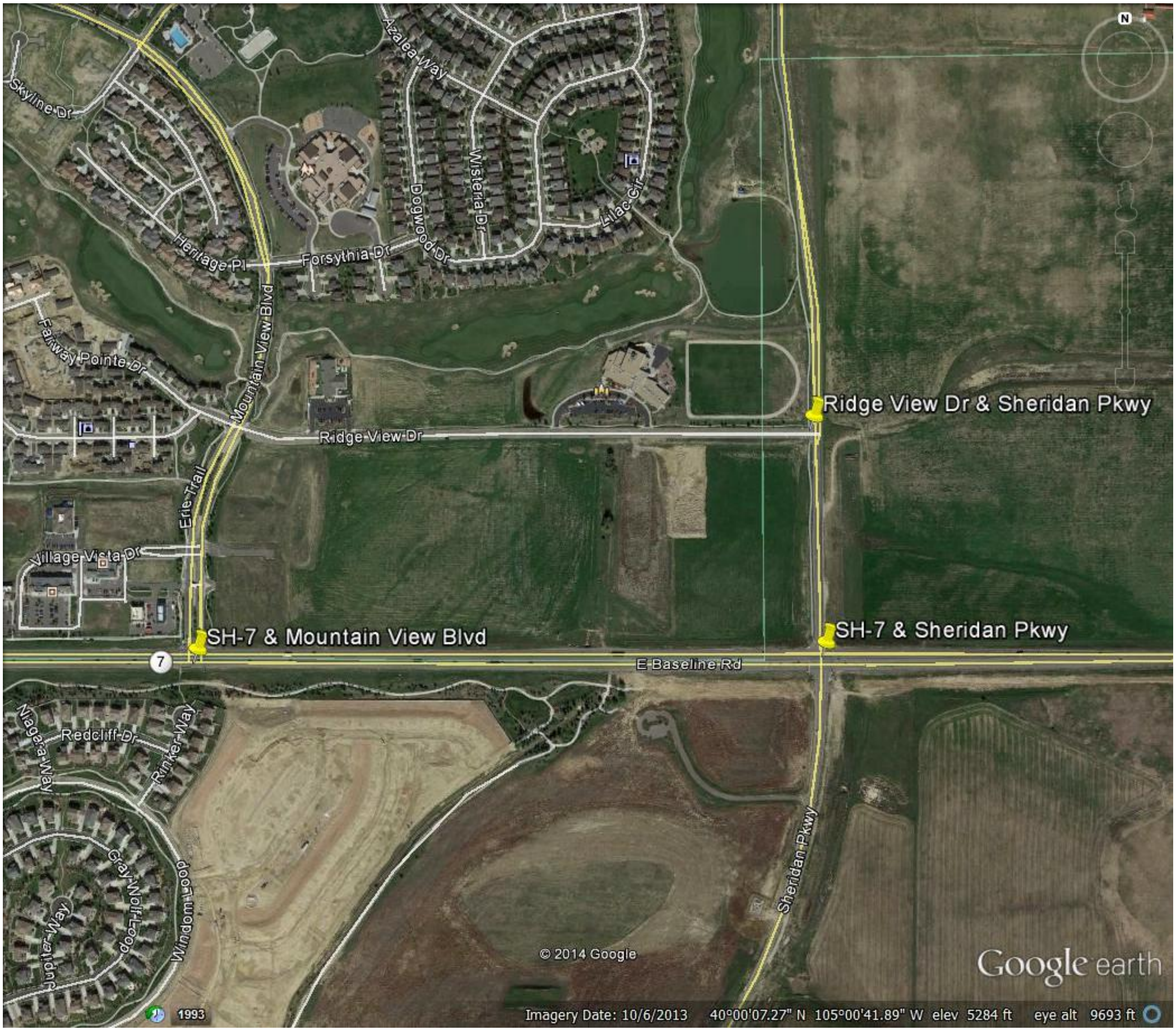
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie King Soopers #129
PM Peak
SH - 7 and Sheridan Pkwy

File Name : SH7SheridanPM
Site Code : IPO 60
Start Date : 10/15/2014
Page No : 4

Image 1



Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
AM Peak
Village Vista Dr and Mountian View Blvd

File Name : VillageVistaMountainViewAM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 1

Groups Printed- Unshifted

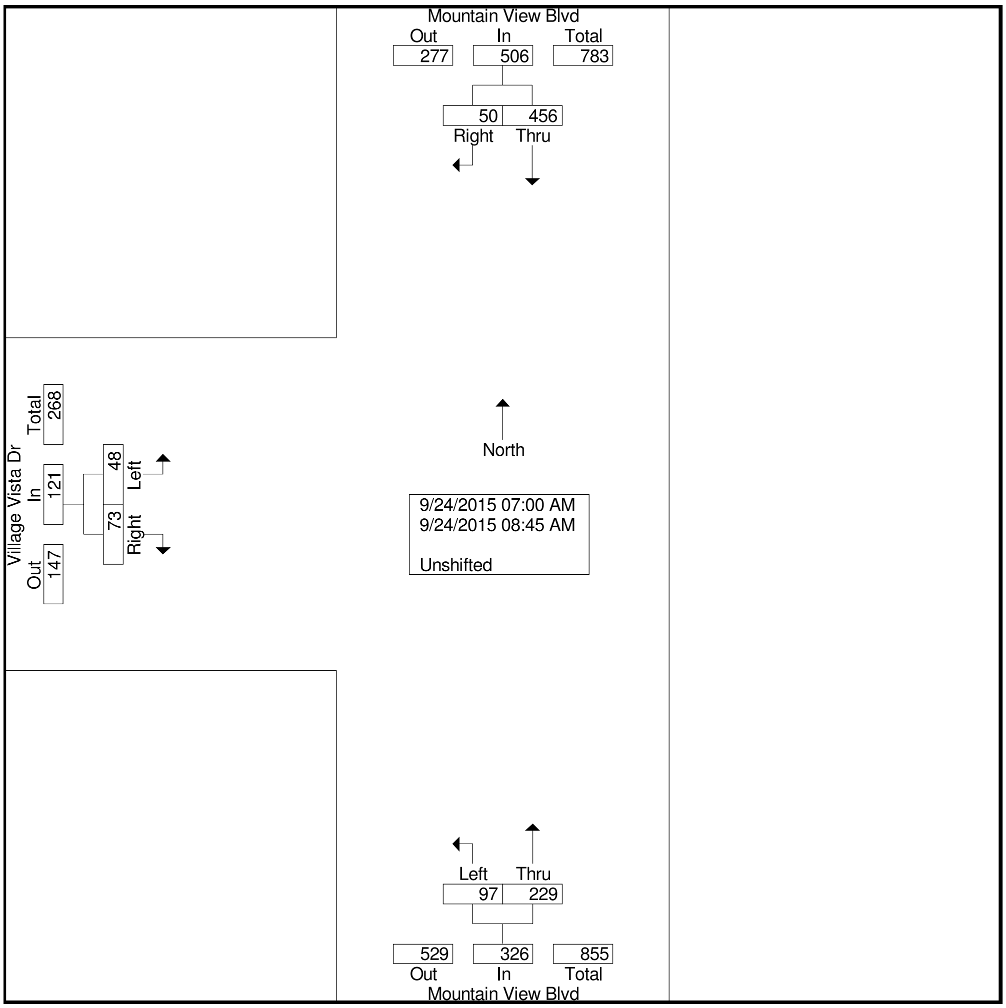
	Village Vista Dr Eastbound			Mountain View Blvd Northbound			Mountain View Blvd Southbound			
Start Time	Left	Right	App. Total	Left	Thru	App. Total	Thru	Right	App. Total	Int. Total
07:00 AM	2	4	6	10	17	27	61	8	69	102
07:15 AM	10	9	19	8	15	23	57	3	60	102
07:30 AM	4	8	12	13	22	35	55	5	60	107
07:45 AM	7	7	14	13	23	36	58	4	62	112
Total	23	28	51	44	77	121	231	20	251	423
08:00 AM	4	10	14	18	47	65	71	9	80	159
08:15 AM	10	12	22	14	33	47	49	11	60	129
08:30 AM	5	15	20	11	29	40	44	2	46	106
08:45 AM	6	8	14	10	43	53	61	8	69	136
Total	25	45	70	53	152	205	225	30	255	530
Grand Total	48	73	121	97	229	326	456	50	506	953
Apprch %	39.7	60.3		29.8	70.2		90.1	9.9		
Total %	5	7.7	12.7	10.2	24	34.2	47.8	5.2	53.1	

RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
AM Peak
Village Vista Dr and Mountian View Blvd

File Name : VillageVistaMountainViewAM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 2



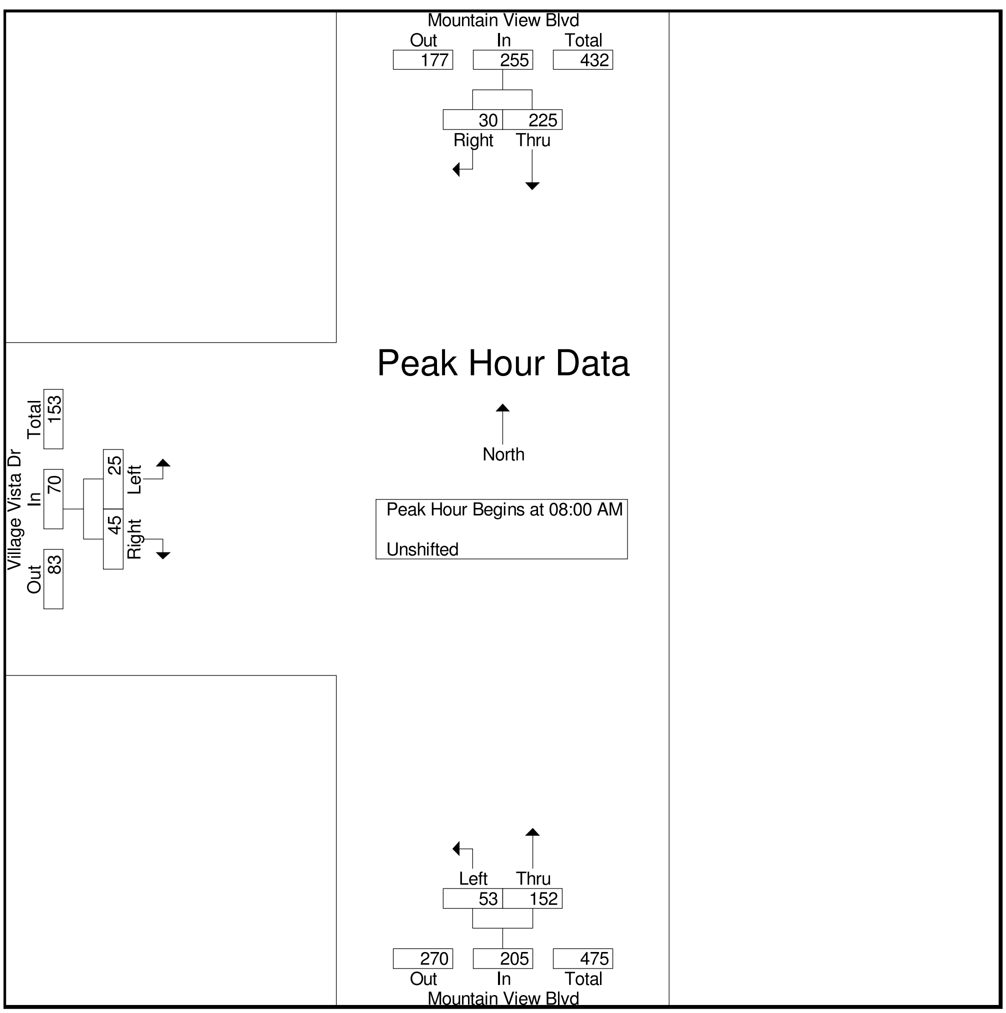
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
AM Peak
Village Vista Dr and Mountian View Blvd

File Name : VillageVistaMountainViewAM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 3

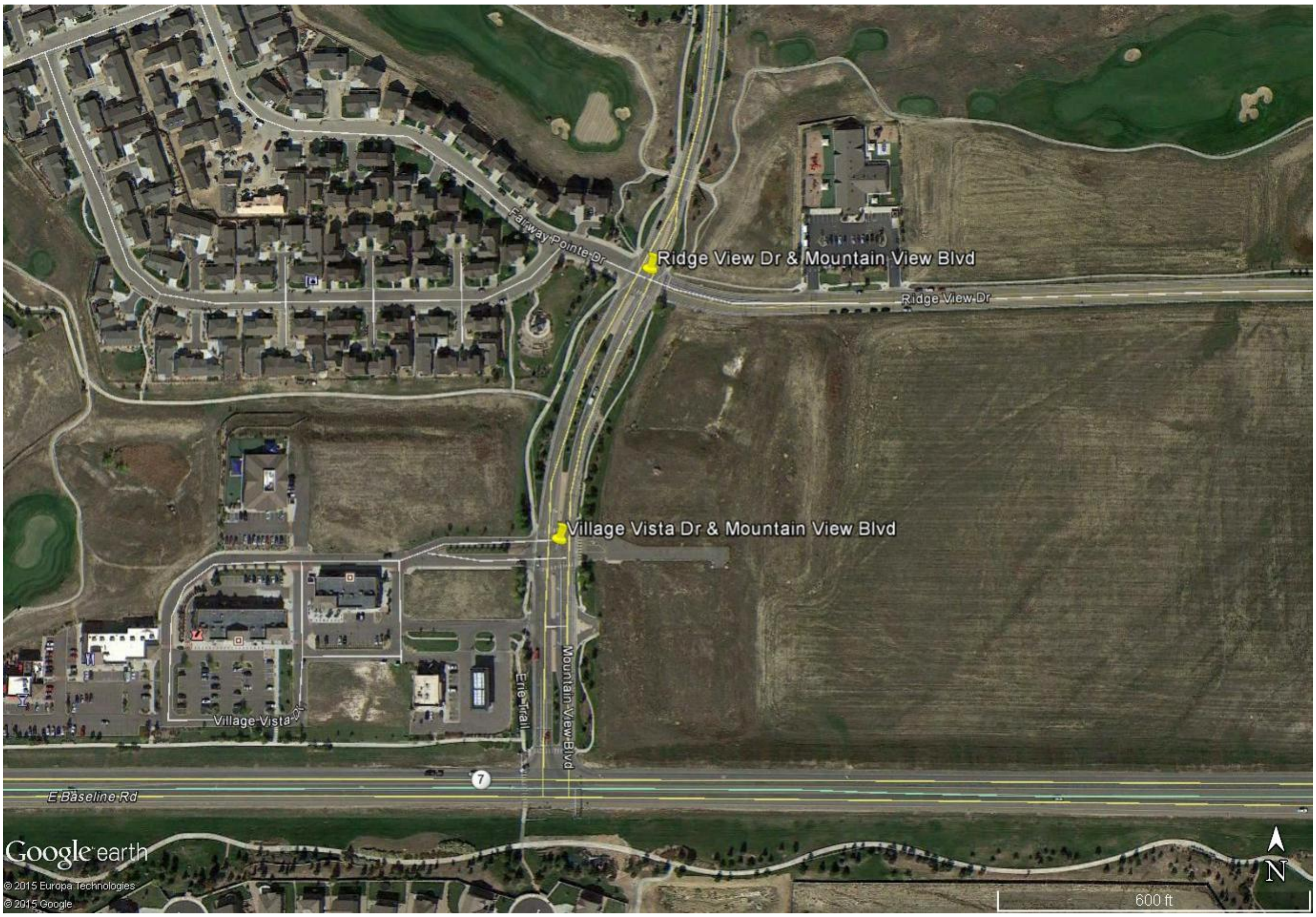
	Village Vista Dr Eastbound			Mountain View Blvd Northbound			Mountain View Blvd Southbound			
Start Time	Left	Right	App. Total	Left	Thru	App. Total	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 07:00 AM to 08:45 AM - Peak 1 of 1										
Peak Hour for Entire Intersection Begins at 08:00 AM										
08:00 AM	4	10	14	**18**	**47**	**65**	**71**	9	**80**	**159**
08:15 AM	**10**	12	**22**	14	33	47	49	**11**	60	129
08:30 AM	5	**15**	20	11	29	40	44	2	46	106
08:45 AM	6	8	14	10	43	53	61	8	69	136
Total Volume	25	45	70	53	152	205	225	30	255	530
% App. Total	35.7	64.3		25.9	74.1		88.2	11.8		
PHF	.625	.750	.795	.736	.809	.788	.792	.682	.797	.833



Erie, CO
Erie Kentro
AM Peak
Village Vista Dr and Mountian View Blvd

File Name : VillageVistaMountainViewAM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 4

Image 1



Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
PM Peak
Village Vista Dr and Mountain View Blvd

File Name : VillageVistaMountainViewPM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 1

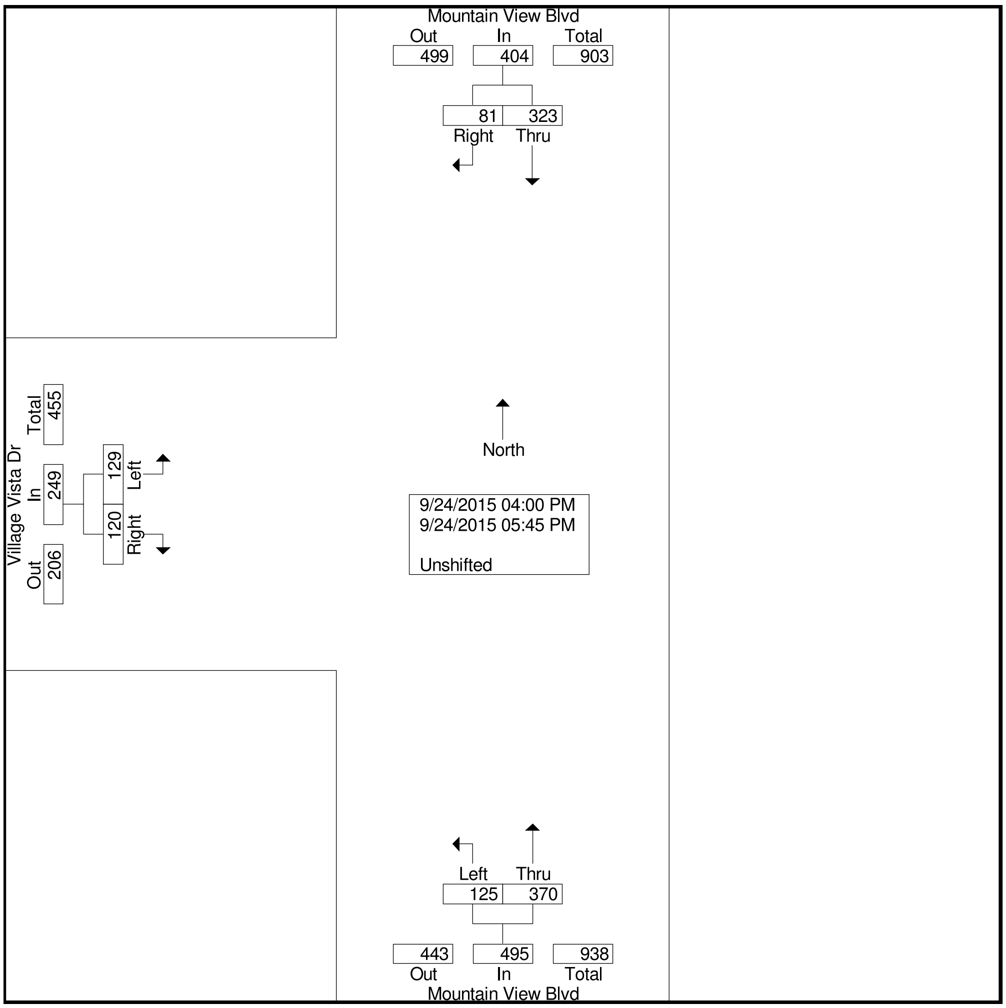
Groups Printed- Unshifted

	Village Vista Dr Eastbound			Mountain View Blvd Northbound			Mountain View Blvd Southbound			
Start Time	Left	Right	App. Total	Left	Thru	App. Total	Thru	Right	App. Total	Int. Total
04:00 PM	14	12	26	12	36	48	40	9	49	123
04:15 PM	20	8	28	9	47	56	44	5	49	133
04:30 PM	11	11	22	15	33	48	45	8	53	123
04:45 PM	17	16	33	17	44	61	35	9	44	138
Total	62	47	109	53	160	213	164	31	195	517
05:00 PM	23	15	38	28	49	77	43	12	55	170
05:15 PM	19	19	38	22	50	72	47	17	64	174
05:30 PM	14	18	32	12	56	68	37	11	48	148
05:45 PM	11	21	32	10	55	65	32	10	42	139
Total	67	73	140	72	210	282	159	50	209	631
Grand Total	129	120	249	125	370	495	323	81	404	1148
Apprch %	51.8	48.2		25.3	74.7		80	20		
Total %	11.2	10.5	21.7	10.9	32.2	43.1	28.1	7.1	35.2	

Ridgeview Data Collection
Morrison, CO 80465

Erie, CO
Erie Kentro
PM Peak
Village Vista Dr and Mountain View Blvd

File Name : VillageVistaMountainViewPM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 2



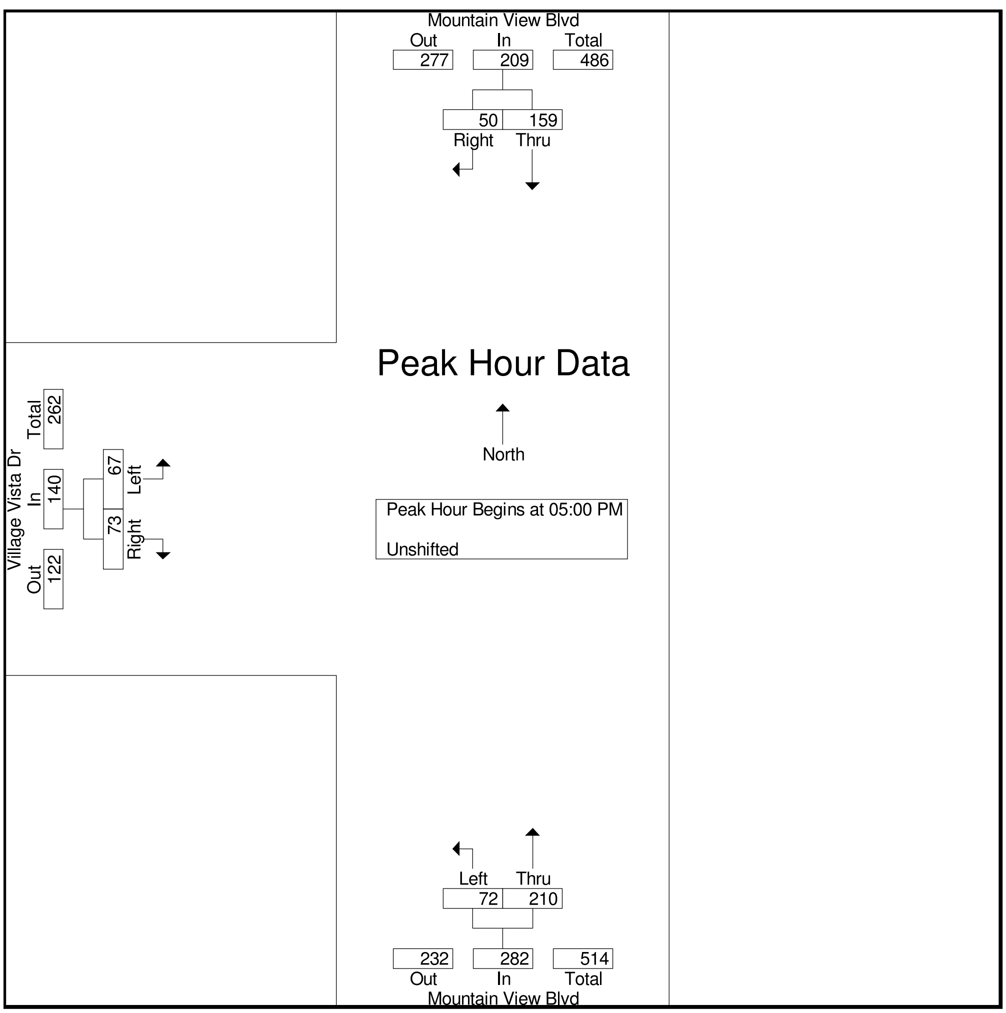
RDC
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
PM Peak
Village Vista Dr and Mountain View Blvd

File Name : VillageVistaMountainViewPM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 3

	Village Vista Dr Eastbound			Mountain View Blvd Northbound			Mountain View Blvd Southbound			
Start Time	Left	Right	App. Total	Left	Thru	App. Total	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 04:00 PM to 05:45 PM - Peak 1 of 1										
Peak Hour for Entire Intersection Begins at 05:00 PM										
05:00 PM	**23**	15	**38**	**28**	49	**77**	43	12	55	170
05:15 PM	19	19	38	22	50	72	**47**	**17**	**64**	**174**
05:30 PM	14	18	32	12	**56**	68	37	11	48	148
05:45 PM	11	**21**	32	10	55	65	32	10	42	139
Total Volume	67	73	140	72	210	282	159	50	209	631
% App. Total	47.9	52.1		25.5	74.5		76.1	23.9		
PHF	.728	.869	.921	.643	.938	.916	.846	.735	.816	.907



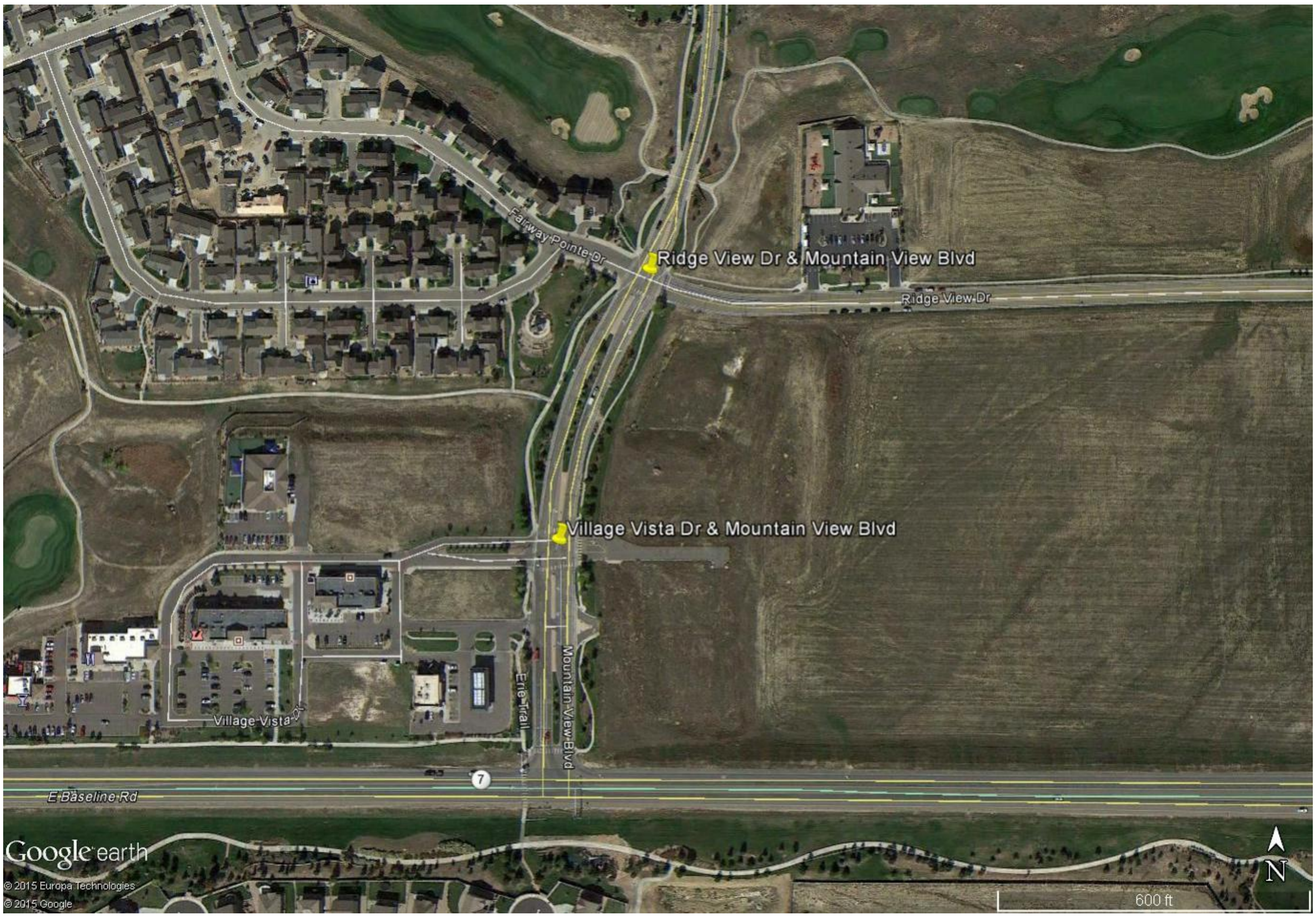
Ridgeview Data Collection

Morrison, CO 80465

Erie, CO
Erie Kentro
PM Peak
Village Vista Dr and Mountain View Blvd

File Name : VillageVistaMountainViewPM
Site Code : IPO 129
Start Date : 9/24/2015
Page No : 4

Image 1



APPENDIX B

CDOT SH-7 Traffic Information

ROUTE	REFPT	ENDREFPT	AADT	YR20FACTOR	LOCATION
007D	64.144	67.488	20000	1.97	ON SH 7 BASELINE RD E/O E COUNTY LINE RD CR 901 LAFAYETTE

APPENDIX C

Trip Generation Worksheets

Trip Generation Planner (ITE 9th Edition) - Summary Report

Kimley»Horn

Weekday Trip Generation
Trips Based on Average Rates/Equations

Project Name **Vista Ridge Commercial**
Project Number **096530000**

ITE Code	Internal Capture Land Use	Land Use Description	Independent Variable	No. of Units	Avg Rate or Eq	Rates			Total Trips							Net Trips after Internal Capture							Net Trips after Internal Capture & Pass-By						
						Daily Rate	AM Rate	PM Rate	Daily Trips	AM Trips	PM Trips	AM Trips In	AM Trips Out	PM Trips In	PM Trips Out	Daily Trips	AM Trips	PM Trips	AM Trips In	AM Trips Out	PM Trips In	PM Trips Out	Daily Trips	AM Trips	PM Trips	AM Trips In	AM Trips Out	PM Trips In	PM Trips Out
820	Retail	Shopping Center	1,000 Sq Ft GLA	76	Eq	N/A	N/A	N/A	5682	132	498	82	50	239	259	3528	52	360	40	12	173	187	2330	38	238	30	9	114	124
912	Retail	Drive-In Bank	1,000 Sq Ft	7	Avg	148.15	12.08	24.30	1038	85	170	48	37	85	85	230	19	91	14	5	47	44	122	10	48	7	3	25	23
934	Retail	Fast-Food Restaurant w/ D.T.	1,000 Sq Ft	14.2	Avg	496.12	45.42	32.65	7046	645	464	329	316	241	223	4892	565	326	295	270	172	154	2446	288	163	151	138	86	77
932	Retail	High-Turnover (Sit-Down) Restaurant	1,000 Sq Ft	5.6	Avg	127.15	10.81	9.85	714	61	55	34	27	33	22	94	7	22	4	3	12	10	54	4	13	2	2	7	6
								Totals	14480	923	1187	493	430	598	589	8744	643	799	353	290	404	395	4952	341	462	190	151	232	230

Notes:
(1) AM and/or PM rates correspond to peak hour of generator

A Trip Generation data from ITE *Trip Generation, 9th Edition*
B AM/PM rates correspond to peak of adjacent street traffic (if data available)
C Includes weekday rates only
D Total trips include pass-by trips w/ no internal capture
E Pass-by rates from ITE *Trip Generation Handbook, 2nd Edition*
F Internal capture rates from ITE *Trip Generation Handbook, 2nd Edition*
G Worksheet is intended as a planning tool. Verify results w/ ITE *Trip Generation 9th Edition*

Kimley»Horn

Project	Vista Ridge Commercial				
Subject	Trip Generation for Apartment				
Designed by	Matt Farmen	*Date*	May 23, 2016	*Job No.*	096530000
Checked by	Curtis Rowe	*Date*	May 24, 2016	*Sheet No.*	1 *of* 1

TRIP GENERATION MANUAL TECHNIQUES

ITE Trip Generation Manual 9th Edition, Fitted Curve Equations

Land Use Code - Apartment, (220)

Independant Variable - Dwelling Units (X)

X = 144
T = Average Vehicle Trip Ends

Peak Hour of Adjacent Street Traffic, One Hour Between 7 and 9 a.m. (page 334)

Daily Weekday
T = 0.49 (X) + 3.73
T = 0.49 * 144.0 + 3.79

Directional Distribution: 20% ent. 80% exit.
T = 74 Average Vehicle Trip Ends
15 entering 59 exiting

15 + 59 = 74

Peak Hour of Adjacent Street Traffic, One Hour Between 4 and 6 p.m. (page 335)

Daily Weekday
T = 0.55 (X) + 17.65
T = 0.55 * 144.0 + 17.65

Directional Distribution: 65% ent. 35% exit.
T = 97 Average Vehicle Trip Ends
63 entering 34 exiting

63 + 34 = 97

Weekday (page 333)

Daily Weekday
T = 6.06 (X) + 123.56
T = 6.06 * 144.0 + 123.56

Directional Distribution: 50% entering, 50% exiting
T = 996 Average Vehicle Trip Ends
498 entering 498 exiting

498 + 498 = 996

Kimley»Horn

Project	Erie Kentro				
Subject	Trip Generation for Medical-Dental Office				
Designed by	Matt Farmen	*Date*	October 27, 2015	*Job No.*	096530000
Checked by	Curtis Rowe	*Date*	October 27, 2015	*Sheet No.*	1 *of* 1

TRIP GENERATION MANUAL TECHNIQUES

ITE Trip Generation Manual 9th Edition - Fitted Curve and Average Rate Equations
Land Use Code - Medical-Dental Office Building (720)

Independent Variable - 1000 Sq Feet Gross Floor Area
SF = 7,000
X = 7.000
T = Average Vehicle Trip Ends

Peak Hour of Adjacent Street Traffic, One Hour Between 7 and 9 a.m. (Page 1295)

Average Rate (R) = 2.39
T = R * X
T = 2.39 * 7.000

Directional Distribution: 79% ent. 21% exit.
T = 17 Average Vehicle Trip Ends
13 entering 4 exiting

13 + 4 = 17

Peak Hour of Adjacent Street Traffic, One Hour Between 4 and 6 p.m. (Page 1296)

Ln(T) = 0.90 Ln(X) + 1.53
T = 0.900 Ln(7.000) + 1.53

Directional Distribution: 28% ent. 72% exit.
T = 27 Average Vehicle Trip Ends
8 entering 19 exiting

8 + 19 = 27

Weekday (page 1294)

Average Weekday

T = 40.89(X) - 214.97
T = 40.89 * 7.000 - 214.97

Directional Distribution: 50% entering, 50% exiting
T = 72 Average Vehicle Trip Ends
36 entering 36 exiting

36 + 36 = 72

Kimley»Horn

Project	Vista Ridge Commercial				
Subject	Trip Generation for Shopping Center				
Designed by	Matt Farmen	*Date*	May 23, 2016	*Job No.*	096530000
Checked by	Curtis Rowe	*Date*	May 24, 2016	*Sheet No.*	1 *of* 1

TRIP GENERATION MANUAL TECHNIQUES

ITE Trip Generation Manual 9th Edition, Fitted Curve Equations
Land Use Code - Shopping Center (820)
Independant Variable - 1000 Square Feet Gross Leasable Area (X)
Gross Leasable Area = 76,000 Square Feet
X = 76.000
T = Average Vehicle Trip Ends

Peak Hour of Adjacent Street Traffic, One Hour Between 7 and 9 a.m. (Page 1562)

Directional Distribution: 62% ent. 38% exit.
Ln(T) = 0.61 Ln(X) + 2.24 — T = 132 Average Vehicle Trip Ends
Ln(T) = 0.61 * Ln(76) + 2.24 — 82 entering 50 exiting

Peak Hour of Adjacent Street Traffic, One Hour Between 4 and 6 p.m. (page 1563)

Directional Distribution: 48% ent. 52% exit.
Ln(T) = 0.67 Ln(X) + 3.31 — T = 498 Average Vehicle Trip Ends
Ln(T) = 0.67 * Ln(76) + 3.31 — 239 entering 259 exiting

Weekday (page 1561)

Daily Weekday — Directional Distribution: 50% entering, 50% exiting
Ln(T) = 0.65 Ln(X) + 5.83 — T = 5682 Average Vehicle Trip Ends
Ln(T) = 0.65 * Ln(76) + 5.83 — 2841 entering 2841 exiting

Saturday Peak Hour of Generator

Average Saturday — Directional Distribution: 52% ent. 48% exit.
Ln(T) = 0.65 Ln(X) + 3.78 — T = 731 Average Vehicle Trip Ends
Ln(T) = 0.65 * Ln(76) + 3.78 — 380 entering 351 exiting

Non Pass-By Trip Volumes (Per ITE Trip Generation Handbook, June 2004)

PM Peak Hour = 34% Pass-by — Saturday Peak Hour = 26% Pass-by

	IN	Out	Total	
AM Peak	60	37	97	*uses lesser of PM and Saturday pass-by rates (26%)
PM Peak	158	171	329	
Daily	1875	1875	3750	*uses PM peak hour pass-by rate
Saturday Peak	281	260	541	

Kimley»Horn

Project	Vista Ridge Commercial						
Subject	Trip Generation for Drive-In Bank						
Designed by	Matt Farmen	*Date*	May 23, 2016	*Job No.*	096530000		
Checked by	Curtis Rowe	*Date*	May 24, 2016	*Sheet No.*	1	*of*	1

TRIP GENERATION MANUAL TECHNIQUES

ITE Trip Generation Manual 9th Edition, Average Rate Equations

Land Use Code - Drive-in Bank (912)

Independant Variable - 1000 Square Feet Gross Floor Area (X)
Gross Floor Area = 7,000 Square Feet
X = 7.000
T = Average Vehicle Trip Ends

Peak Hour of Adjacent Street Traffic, One Hour Between 7 and 9 a.m. (Page 1843)

Average Weekday	Directional Distribution: 57% ent. 43% exit.
T = 12.08 (X)	T = 85 Average Vehicle Trip Ends
T = 12.08 * 7.000	48 entering 37 exiting
	48 + 37 (*) = 85

Peak Hour of Adjacent Street Traffic, One Hour Between 4 and 6 p.m. (page 1844)

Average Weekday	Directional Distribution: 50% ent. 50% exit.
T =24.30 (X)	T = 170 Average Vehicle Trip Ends
T = 24.30 * 7.000	85 entering 85 exiting
	85 + 85 = 170

Weekday (page 1753)

Average Weekday	Directional Distribution: 50% entering, 50% exiting
T = 148.15 (X)	T = 1038 Average Vehicle Trip Ends
T = 148.15 * 7.000	519 entering 519 exiting
	519 + 519 (*) = 1038

Saturday Peak Hour of Generator (page 1848)

Average Saturday	Directional Distribution: 51% ent. 49% exit.
T = 26.31 (X)	T = 184 Average Vehicle Trip Ends
T = 26.31 * 7.000	94 entering 90 exiting
	94 + 90 = 184

Kimley»Horn

Project	Vista Ridge Commercial						
Subject	Trip Generation for High-Turnover (Sit-Down) Restaurant						
Designed by	Matt Farmen	*Date*	May 23, 2016	*Job No.*	096530000		
Checked by	Curtis Rowe	*Date*	May 24, 2016	*Sheet No.*	1	*of*	1

<u>TRIP GENERATION MANUAL TECHNIQUES</u>

ITE <u>Trip Generation Manual</u> 9th Edition, Average Rate Equations
Land Use Code - High Turnover Sit-Down Restaurant (932)
Independant Variable - 1000 Square Feet Gross Floor Area (X)
Gross Floor Area = 5,600 Square Feet
X = 5.600
T = Average Vehicle Trip Ends

<u>Peak Hour of Adjacent Street Traffic, One Hour Between 7 and 9 a.m. (Page 1886)</u>

Average Weekday	Directional Distribution:		55%	ent.	45%	exit.
T = 10.81 (X)	T =	61	Average Vehicle Trip Ends			
T = 10.81 * 5.600	34	entering	27	exiting		

<u>Peak Hour of Adjacent Street Traffic, One Hour Between 4 and 6 p.m. (page 1887)</u>

Average Weekday	Directional Distribution:		60%	ent.	40%	exit.
T = 9.85 (X)	T =	55	Average Vehicle Trip Ends			
T = 9.85 * 5.600	33	entering	22	exiting		

<u>Weekday (page 1885)</u>

Average Weekday	Directional Distribution: 50% entering, 50% exiting			
T = 127.15 (X)	T =	714	Average Vehicle Trip Ends	
T = 127.15 * 5.600	357	entering	357	exiting

<u>P.M. Peak Hour of Generator (page 1889)</u>

Average Weekday	Directional Distribution:		54%	ent.	46%	exit.
T = 18.49 (X)	T =	104	Average Vehicle Trip Ends			
T = 18.49 * 5.600	56	entering	48	exiting		

<u>Saturday Peak Hour of Generator (page 1891)</u>

Average Saturday	Directional Distribution:		53%	ent.	47%	exit.
T = 14.07 (X)	T =	80	Average Vehicle Trip Ends			
T = 14.07 * 5.600	42	entering	38	exiting		

<u>Non-Pass-by Trip Volumes (page 48 Trip Generation Handbook, June 2004)</u>

PM Peak Hour = 57% Non-Pass By

	IN	Out	Total	
AM Peak	19	15	35	
PM Peak	19	13	31	
Daily	203	203	406	PM Peak Hour Rate Applied to All Other Time Periods

<u>Pass-by Trip Volumes (page 48 Trip Generation Handbook, June 2004</u>

PM Peak Hour = 43% Pass By

	IN	Out	Total	
AM Peak	15	12	26	
PM Peak	14	9	24	
Daily	154	154	308	PM Peak Hour Rate Applied to All Other Time Periods

Kimley»Horn

Project	Vista Ridge Commercial						
Subject	Trip Generation for Fast-Food Restaurant with Drive-Through Window						
Designed by	Matt Farmen	*Date*	May 23, 2016	*Job No.*	096530000		
Checked by	Curtis Rowe	*Date*	May 24, 2016	*Sheet No.*	1	*of*	1

TRIP GENERATION MANUAL TECHNIQUES

ITE Trip Generation Manual 9th Edition, Average Rate Equations

Land Use Code - Fast Food Restaurant With Drive-Through Window (934)

Independant Variable - 1000 Square Feet Gross Floor Area (X)
Gross Floor Area = 14,200 Square Feet
X = 14.200
T = Average Vehicle Trip Ends

Peak Hour of Adjacent Street Traffic, One Hour Between 7 and 9 a.m. (Page 1913)

Average Weekday
T = 45.42 (X)
T = 45.42 * 14.200

Directional Distribution: 51% ent. 49% exit.
T = 645 Average Vehicle Trip Ends
329 entering 316 exiting

329 + 316 = 645

Peak Hour of Adjacent Street Traffic, One Hour Between 4 and 6 p.m. (page 1914)

Average Weekday
T = 32.65 (X)
T = 32.65 * 14.200

Directional Distribution: 52% ent. 48% exit.
T = 464 Average Vehicle Trip Ends
241 entering 223 exiting

241 + 223 = 464

Weekday (page 1912)

Average Weekday
T = 496.12 (X)
T = 496.12 * 14.200

Directional Distribution: 50% entering, 50% exiting
T = 7046 Average Vehicle Trip Ends
3523 entering 3523 exiting

3523 + 3523 = 7046

Saturday Peak Hour of Generator (Page 1918)

T = 59.00 (X)
T = 59.00 * 14.200

Directional Distribution: 51% ent. 49% exit.
T = 838 Average Vehicle Trip Ends
427 entering 411 exiting

427 + 411 = 838

Non-Pass-by Trip Volumes (pages 68 and 70, Trip Generation Handbook, June 2004)

AM Peak Hour = 51% Non-Pass By   PM Peak Hour = 50% Non-Pass By

	IN	Out	Total	
AM Peak	168	161	329	
PM Peak	121	112	232	
Daily	1762	1762	3524	PM Peak Hour Rate Applied to Daily

Pass-by Trip Volumes (pages 68 and 70, Trip Generation Handbook, June 2004

AM Peak Hour = 49% Pass By   PM Peak Hour = 50% Pass By

	IN	Out	Total	
AM Peak	161	155	316	
PM Peak	121	112	232	
Daily	1761	1761	3522	PM Peak Hour Rate Applied to Daily

ITE MULTI-USE PROJECT INTERNAL CAPTURE WORKSHEET
(Source: Chapter 7, ITE *Trip Generation Handbook*, June 2004)

Project Number: 096530000
Project Name: Vista Ridge Commercial
Scenario: AM
Analysis Period: AM Peak
Analyst: Kimley-Horn and Associates, Inc

Land Use A: Retail (Shopping Center)
ITE Land Use Code 820
Size: 76

	Total	Internal	External
Enter	82	42	40
Exit	50	38	12
Total	132	80	52
%	100%	**60.7%**	39.3%

Enter from External: 40
Exit to External: 12

Land Use B: Retail (Drive-In Bank)
ITE Land Use Code 912
Size: 7

	Total	Internal	External
Enter	48	34	14
Exit	37	32	5
Total	85	66	19
%	100%	**77.6%**	22.4%

Enter from External: 14
Exit to External: 5

Land Use C: Retail (Fast-Food Restaurant w/ D.T.)
ITE Land Use Code 934
Size: 14.2

	Total	Internal	External
Enter	329	34	295
Exit	316	46	270
Total	645	80	565
%	100%	**12.4%**	87.6%

Enter from External: 295
Exit to External: 270

Land Use D: Retail (High-Turnover (Sit-Do
ITE Land Use Code 932
Size: 5.6

	Total	Internal	External
Enter	34	30	4
Exit	27	24	3
Total	61	54	7
%	100%	**88.5%**	11.5%

Enter from External: 4
Exit to External: 3

A – B:
Demand 30.0% 15
Balanced 13
Demand 28.0% 13

B – A:
28.0% 23 Demand
11 Balanced
30.0% 11 Demand

A – C:
Demand 30.0% 15
Balanced 15
Demand 28.0% 92

A – D:
28.0% 23 Demand
8 Balanced
30.0% 8 Demand

D – A:
Demand 30.0% 15
Balanced 10
Demand 28.0% 10

B – D:
Demand 28.0% 10
10 Balanced
30.0% 11 Demand

D – B:
Demand 28.0% 13
Balanced 8
Demand 30.0% 8

B – C:
28.0% 13 Demand
13 Balanced
30.0% 95 Demand

C – B:
Demand 30.0% 11
Balanced 11
Demand 28.0% 92

C – A:
Demand 28.0% 23
Balanced 23
Demand 30.0% 95

D – C:
Demand 30.0% 8
Balanced 8
Demand 28.0% 92

C – D:
28.0% 10 Demand
10 Balanced
30.0% 95 Demand

NET EXTERNAL TRIPS FOR MULTI-USE DEVELOPMENT					
	Land Use				
Category	A	B	C	D	Total
Enter	40	14	295	4	353
Exit	12	5	270	3	290
Total	52	19	565	7	**643**
Single Use Trip Gen Estimate	132	85	645	61	**923**

Overall Internal Capture = 30.34%

ITE MULTI-USE PROJECT INTERNAL CAPTURE WORKSHEET
(Source: Chapter 7, ITE *Trip Generation Handbook*, June 2004)

Project Number: 096530000
Project Name: Vista Ridge Commercial
Scenario: Daily
Analysis Period: Daily
Analyst: Kimley-Horn and Associates, Inc

Land Use A: Retail (Shopping Center)
ITE Land Use Code 820
Size: 76

	Total	Internal	External
Enter	2,841	1,058	1,783
Exit	2,841	1,097	1,744
Total	5,682	2,155	3,527
%	100%	**37.9%**	62.1%

Enter from External: 1,783
Exit to External: 1,744

Land Use B: Retail (Drive-In Bank)
ITE Land Use Code 912
Size: 7

	Total	Internal	External
Enter	519	397	122
Exit	519	412	107
Total	1038	809	229
%	100%	**77.9%**	22.1%

Enter from External: 122
Exit to External: 107

Land Use C: Retail (Fast-Food Restaurant w/ D.T.)
ITE Land Use Code 934
Size: 14.2

	Total	Internal	External
Enter	3523	1115	2408
Exit	3523	1040	2483
Total	7046	2155	4891
%	100%	**30.6%**	69.4%

Enter from External: 2,408
Exit to External: 2,483

Land Use D: Retail (High-Turnover (Sit-Do
ITE Land Use Code 932
Size: 5.6

	Total	Internal	External
Enter	357	300	57
Exit	357	321	36
Total	714	621	93
%	100%	**87.0%**	13.0%

Enter from External: 57
Exit to External: 36

A–B:

Demand	30.0%	852
Balanced	145	
Demand	28.0%	145

28.0%	795	Demand
	156	Balanced
30.0%	156	Demand

A–C:

Demand	30.0%	852
Balanced	852	
Demand	28.0%	986

A–D:

28.0%	795	Demand
	107	Balanced
30.0%	107	Demand

Demand	30.0%	852
Balanced	100	
Demand	28.0%	100

B–D:

Demand	28.0%	100
	100	Balanced
30.0%	156	Demand

Demand	28.0%	145
Balanced	107	
Demand	30.0%	107

B–C:

28.0%	145	Demand
	145	Balanced
30.0%	1057	Demand

Demand	30.0%	156
Balanced	156	
Demand	28.0%	986

C–D:

Demand	28.0%	795
Balanced	795	
Demand	30.0%	1057

Demand	30.0%	107
Balanced	107	
Demand	28.0%	986

28.0%	100	Demand
	100	Balanced
30.0%	1057	Demand

NET EXTERNAL TRIPS FOR MULTI-USE DEVELOPMENT

Category	Land Use A	Land Use B	Land Use C	Land Use D	Total
Enter	1,783	122	2,408	57	4,370
Exit	1,744	107	2,483	36	4,370
Total	3,527	229	4,891	93	**8,740**
Single Use Trip Gen Estimate	5,682	1,038	7,046	714	**14,480**

Overall Internal Capture = 39.64%

ITE MULTI-USE PROJECT INTERNAL CAPTURE WORKSHEET
(Source: Chapter 7, ITE *Trip Generation Handbook*, June 2004)

Project Number: 096530000
Project Name: Vista Ridge Commercial
Scenario: PM
Analysis Period: PM Peak
Analyst: Kimley-Horn and Associates, Inc

Land Use A: Retail (Shopping Center)
ITE Land Use Code 820
Size: 76

	Total	Internal	External
Enter	239	66	173
Exit	259	72	187
Total	498	138	360
%	100%	**27.7%**	72.3%

Enter from External: 173
Exit to External: 187

Demand 20.0% 52
Balanced 17
Demand 20.0% 17

20.0% 48 Demand
17 Balanced
20.0% 17 Demand

Demand 20.0% 52
Balanced 48
Demand 20.0% 48

20.0% 48 Demand
4 Balanced
20.0% 4 Demand

Demand 20.0% 52
Balanced 7
Demand 20.0% 7

Land Use B: Retail (Drive-In Bank)
ITE Land Use Code 912
Size: 7

	Total	Internal	External
Enter	85	38	47
Exit	85	41	44
Total	170	79	91
%	100%	**46.5%**	53.5%

Enter from External: 47
Exit to External: 44

Demand 20.0% 7
7 Balanced
20.0% 17 Demand

Demand 20.0% 17
Balanced 4
Demand 20.0% 4

Land Use D: Retail (High-Turnover (Sit-Do
ITE Land Use Code 932
Size: 5.6

	Total	Internal	External
Enter	33	21	12
Exit	22	12	10
Total	55	33	22
%	100%	**60.0%**	40.0%

Enter from External: 12
Exit to External: 10

20.0% 17 Demand
17 Balanced
20.0% 45 Demand

Demand 20.0% 48
Balanced 45
Demand 20.0% 45

Demand 20.0% 4
Balanced 4
Demand 20.0% 48

Demand 20.0% 17
Balanced 17
Demand 20.0% 48

20.0% 7 Demand
7 Balanced
20.0% 45 Demand

Land Use C: Retail (Fast-Food Restaurant w/ D.T.)
ITE Land Use Code 934
Size: 14.2

	Total	Internal	External
Enter	241	69	172
Exit	223	69	154
Total	464	138	326
%	100%	**29.7%**	70.3%

Enter from External: 172
Exit to External: 154

NET EXTERNAL TRIPS FOR MULTI-USE DEVELOPMENT

Category	Land Use A	B	C	D	Total
Enter	173	47	172	12	404
Exit	187	44	154	10	395
Total	360	91	326	22	**799**
Single Use Trip Gen Estimate	498	170	464	55	**1,187**

Overall Internal Capture = 32.67%

APPENDIX D

Intersection Analysis Worksheets

Movement	EBL	EBT	WBT	WBR	SBL	SBR		
Lane Configurations								
Traffic Volume (veh/h)	114	616	823	124	171	153		
Future Volume (veh/h)	114	616	823	124	171	153		
Number	7	4	8	18	1	16		
Initial Q (Qb), veh	0	0	0	0	0	0		
Ped-Bike Adj(A_pbT)	1.00			1.00	1.00	1.00		
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00		
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863		
Adj Flow Rate, veh/h	165	655	935	155	209	184		
Adj No. of Lanes	1	2	1	1	2	1		
Peak Hour Factor	0.69	0.94	0.88	0.80	0.82	0.83		
Percent Heavy Veh, %	2	2	2	2	2	2		
Cap, veh/h	241	2096	907	771	1153	531		
Arrive On Green	0.07	0.59	0.97	0.97	0.34	0.34		
Sat Flow, veh/h	1774	3632	1863	1583	3442	1583		
Grp Volume(v), veh/h	165	655	935	155	209	184		
Grp Sat Flow(s),veh/h/ln	1774	1770	1863	1583	1721	1583		
Q Serve(g_s), s	4.8	10.2	53.6	0.3	4.7	9.6		
Cycle Q Clear(g_c), s	4.8	10.2	53.6	0.3	4.7	9.6		
Prop In Lane	1.00			1.00	1.00	1.00		
Lane Grp Cap(c), veh/h	241	2096	907	771	1153	531		
V/C Ratio(X)	0.69	0.31	1.03	0.20	0.18	0.35		
Avail Cap(c_a), veh/h	280	2767	1219	1036	1153	531		
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00		
Upstream Filter(I)	1.00	1.00	0.44	0.44	1.00	1.00		
Uniform Delay (d), s/veh	23.2	11.2	1.4	0.7	25.9	27.5		
Incr Delay (d2), s/veh	5.6	0.1	24.3	0.1	0.3	1.8		
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0		
%ile BackOfQ(50%),veh/ln	2.9	4.9	14.7	0.1	2.3	9.6		
LnGrp Delay(d),s/veh	28.8	11.3	25.7	0.8	26.2	29.3		
LnGrp LOS	C	B	F	A	C	C		
Approach Vol, veh/h		820	1090		393			
Approach Delay, s/veh		14.8	22.2		27.7			
Approach LOS		B	C		C			
Timer	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Assigned Phs				4		6	7	8
Phs Duration (G+Y+Rc), s				74.6		35.4	11.2	63.4
Change Period (Y+Rc), s				4.0		4.0	4.0	4.0
Max Green Setting (Gmax), s				86.0		16.0	10.0	72.0
Max Q Clear Time (g_c+I1), s				12.2		11.6	6.8	55.6
Green Ext Time (p_c), s				16.6		0.6	0.1	9.6
Intersection Summary								
HCM 2010 Ctrl Delay			20.5					
HCM 2010 LOS			C					

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	177	976	665	208	145	151
Future Volume (veh/h)	177	976	665	208	145	151
Number	7	4	8	18	1	16
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00			1.00	1.00	1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1727	1727	1863	1863	1863
Adj Flow Rate, veh/h	213	1038	723	254	169	176
Adj No. of Lanes	1	2	1	1	2	1
Peak Hour Factor	0.83	0.94	0.92	0.82	0.86	0.86
Percent Heavy Veh, %	2	10	10	2	2	2
Cap, veh/h	282	1963	826	757	1133	521
Arrive On Green	0.08	0.60	0.96	0.96	0.33	0.33
Sat Flow, veh/h	1774	3368	1727	1583	3442	1583
Grp Volume(v), veh/h	213	1038	723	254	169	176
Grp Sat Flow(s),veh/h/ln	1774	1641	1727	1583	1721	1583
Q Serve(g_s), s	6.3	20.5	12.3	1.1	3.8	9.2
Cycle Q Clear(g_c), s	6.3	20.5	12.3	1.1	3.8	9.2
Prop In Lane	1.00			1.00	1.00	1.00
Lane Grp Cap(c), veh/h	282	1963	826	757	1133	521
V/C Ratio(X)	0.75	0.53	0.88	0.34	0.15	0.34
Avail Cap(c_a), veh/h	328	2476	1052	964	1133	521
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	0.66	0.66	1.00	1.00
Uniform Delay (d), s/veh	22.5	13.0	1.5	1.3	26.0	27.8
Incr Delay (d2), s/veh	8.2	0.2	4.8	0.2	0.3	1.8
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.8	9.3	4.7	0.5	1.9	9.2
LnGrp Delay(d),s/veh	30.7	13.2	6.3	1.4	26.3	29.6
LnGrp LOS	C	B	A	A	C	C
Approach Vol, veh/h		1251	977		345	
Approach Delay, s/veh		16.2	5.0		28.0	
Approach LOS		B	A		C	

Timer	1	2	3	4	5	6	7	8
Assigned Phs				4		6	7	8
Phs Duration (G+Y+Rc), s				59.6		50.4	12.8	46.8
Change Period (Y+Rc), s				4.0		4.0	4.0	4.0
Max Green Setting (Gmax), s				83.0		19.0	12.0	67.0
Max Q Clear Time (g_c+I1), s				22.5		11.2	8.3	14.3
Green Ext Time (p_c), s				19.2		0.7	0.2	18.6

Intersection Summary	
HCM 2010 Ctrl Delay	13.5
HCM 2010 LOS	B

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	167	781	978	174	216	218
Future Volume (veh/h)	167	781	978	174	216	218
Number	7	4	8	18	1	16
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00			1.00	1.00	1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	242	831	1111	218	263	263
Adj No. of Lanes	1	2	1	1	2	1
Peak Hour Factor	0.69	0.94	0.88	0.80	0.82	0.83
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	297	2221	927	788	1031	474
Arrive On Green	0.09	0.63	1.00	1.00	0.30	0.30
Sat Flow, veh/h	1774	3632	1863	1583	3442	1583
Grp Volume(v), veh/h	242	831	1111	218	263	263
Grp Sat Flow(s),veh/h/ln	1774	1770	1863	1583	1721	1583
Q Serve(g_s), s	6.8	12.6	54.8	0.1	6.4	15.3
Cycle Q Clear(g_c), s	6.8	12.6	54.8	0.1	6.4	15.3
Prop In Lane	1.00			1.00	1.00	1.00
Lane Grp Cap(c), veh/h	297	2221	927	788	1031	474
V/C Ratio(X)	0.81	0.37	1.20	0.28	0.26	0.55
Avail Cap(c_a), veh/h	309	2735	1185	1008	1031	474
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	0.09	0.09	1.00	1.00
Uniform Delay (d), s/veh	21.9	10.0	0.2	0.1	29.2	32.3
Incr Delay (d2), s/veh	14.9	0.1	90.0	0.0	0.6	4.6
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	5.0	6.1	23.3	0.0	3.1	14.2
LnGrp Delay(d),s/veh	36.8	10.1	90.2	0.1	29.8	37.0
LnGrp LOS	D	B	F	A	C	D
Approach Vol, veh/h		1073	1329		526	
Approach Delay, s/veh		16.1	75.4		33.4	
Approach LOS		B	E		C	

Timer	1	2	3	4	5	6	7	8
Assigned Phs				4		6	7	8
Phs Duration (G+Y+Rc), s				78.4		31.6	13.6	64.8
Change Period (Y+Rc), s				4.0		4.0	4.0	4.0
Max Green Setting (Gmax), s				85.0		17.0	11.0	70.0
Max Q Clear Time (g_c+I1), s				14.6		17.3	8.8	56.8
Green Ext Time (p_c), s				26.5		0.0	0.1	10.0

Intersection Summary	
HCM 2010 Ctrl Delay	46.1
HCM 2010 LOS	D

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	255	1276	920	266	182	204
Future Volume (veh/h)	255	1276	920	266	182	204
Number	7	4	8	18	1	16
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00			1.00	1.00	1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1727	1727	1863	1863	1863
Adj Flow Rate, veh/h	307	1357	1000	324	212	237
Adj No. of Lanes	1	2	1	1	2	1
Peak Hour Factor	0.83	0.94	0.92	0.82	0.86	0.86
Percent Heavy Veh, %	2	10	10	2	2	2
Cap, veh/h	383	2071	826	757	1020	469
Arrive On Green	0.12	0.63	0.96	0.96	0.30	0.30
Sat Flow, veh/h	1774	3368	1727	1583	3442	1583
Grp Volume(v), veh/h	307	1357	1000	324	212	237
Grp Sat Flow(s),veh/h/ln	1774	1641	1727	1583	1721	1583
Q Serve(g_s), s	8.9	28.6	52.6	1.7	5.1	13.6
Cycle Q Clear(g_c), s	8.9	28.6	52.6	1.7	5.1	13.6
Prop In Lane	1.00			1.00	1.00	1.00
Lane Grp Cap(c), veh/h	383	2071	826	757	1020	469
V/C Ratio(X)	0.80	0.66	1.21	0.43	0.21	0.51
Avail Cap(c_a), veh/h	386	2536	1068	979	1020	469
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	0.10	0.10	1.00	1.00
Uniform Delay (d), s/veh	18.1	12.8	2.4	1.3	29.0	32.0
Incr Delay (d2), s/veh	11.4	0.4	96.0	0.0	0.5	3.8
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	5.8	12.9	37.2	0.5	2.5	12.8
LnGrp Delay(d),s/veh	29.5	13.2	98.4	1.3	29.5	35.9
LnGrp LOS	C	B	F	A	C	D
Approach Vol, veh/h		1664	1324		449	
Approach Delay, s/veh		16.2	74.6		32.9	
Approach LOS		B	E		C	

Timer	1	2	3	4	5	6	7	8
Assigned Phs				4		6	7	8
Phs Duration (G+Y+Rc), s				79.2		30.8	16.0	63.3
Change Period (Y+Rc), s				4.0		4.0	4.0	4.0
Max Green Setting (Gmax), s				85.0		17.0	13.0	68.0
Max Q Clear Time (g_c+I1), s				30.6		15.6	10.9	54.6
Green Ext Time (p_c), s				32.3		0.3	0.2	11.3

Intersection Summary		
HCM 2010 Ctrl Delay		40.9
HCM 2010 LOS		D

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	252	787	1009	168	288	257
Future Volume (veh/h)	252	787	1009	168	288	257
Number	7	4	8	18	1	16
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00			1.00	1.00	1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	365	837	1147	210	351	310
Adj No. of Lanes	1	2	1	1	2	1
Peak Hour Factor	0.69	0.94	0.88	0.80	0.82	0.83
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	340	2767	1101	936	501	230
Arrive On Green	0.15	0.78	1.00	1.00	0.15	0.15
Sat Flow, veh/h	1774	3632	1863	1583	3442	1583
Grp Volume(v), veh/h	365	837	1147	210	351	310
Grp Sat Flow(s),veh/h/ln	1774	1770	1863	1583	1721	1583
Q Serve(g_s), s	17.0	7.4	65.0	0.0	10.7	16.0
Cycle Q Clear(g_c), s	17.0	7.4	65.0	0.0	10.7	16.0
Prop In Lane	1.00			1.00	1.00	1.00
Lane Grp Cap(c), veh/h	340	2767	1101	936	501	230
V/C Ratio(X)	1.07	0.30	1.04	0.22	0.70	1.35
Avail Cap(c_a), veh/h	340	2767	1101	936	501	230
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	39.9	3.4	0.0	0.0	44.7	47.0
Incr Delay (d2), s/veh	70.2	0.1	38.7	0.1	8.0	181.8
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	16.9	3.6	11.8	0.0	5.6	24.6
LnGrp Delay(d),s/veh	110.0	3.5	38.7	0.1	52.7	228.8
LnGrp LOS	F	A	F	A	D	F
Approach Vol, veh/h		1202	1357		661	
Approach Delay, s/veh		35.8	32.8		135.3	
Approach LOS		D	C		F	

Timer	1	2	3	4	5	6	7	8
Assigned Phs				4		6	7	8
Phs Duration (G+Y+Rc), s				90.0		20.0	21.0	69.0
Change Period (Y+Rc), s				4.0		4.0	4.0	4.0
Max Green Setting (Gmax), s				86.0		16.0	17.0	65.0
Max Q Clear Time (g_c+I1), s				9.4		18.0	19.0	67.0
Green Ext Time (p_c), s				28.7		0.0	0.0	0.0

Intersection Summary	
HCM 2010 Ctrl Delay	54.9
HCM 2010 LOS	D

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	361	1278	970	251	298	266
Future Volume (veh/h)	361	1278	970	251	298	266
Number	7	4	8	18	1	16
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00			1.00	1.00	1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1727	1727	1863	1863	1863
Adj Flow Rate, veh/h	435	1360	1054	306	347	309
Adj No. of Lanes	1	2	1	1	2	1
Peak Hour Factor	0.83	0.94	0.92	0.82	0.86	0.86
Percent Heavy Veh, %	2	10	10	2	2	2
Cap, veh/h	388	2566	974	892	501	230
Arrive On Green	0.18	0.78	1.00	1.00	0.15	0.15
Sat Flow, veh/h	1774	3368	1727	1583	3442	1583
Grp Volume(v), veh/h	435	1360	1054	306	347	309
Grp Sat Flow(s),veh/h/ln	1774	1641	1727	1583	1721	1583
Q Serve(g_s), s	20.0	17.0	62.0	0.0	10.5	16.0
Cycle Q Clear(g_c), s	20.0	17.0	62.0	0.0	10.5	16.0
Prop In Lane	1.00			1.00	1.00	1.00
Lane Grp Cap(c), veh/h	388	2566	974	892	501	230
V/C Ratio(X)	1.12	0.53	1.08	0.34	0.69	1.34
Avail Cap(c_a), veh/h	388	2566	974	892	501	230
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	38.9	4.5	0.0	0.0	44.7	47.0
Incr Delay (d2), s/veh	82.8	0.2	53.9	0.2	7.7	180.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	20.7	7.6	14.6	0.1	5.5	24.5
LnGrp Delay(d),s/veh	121.7	4.7	53.9	0.2	52.4	227.0
LnGrp LOS	F	A	F	A	D	F
Approach Vol, veh/h		1795	1360		656	
Approach Delay, s/veh		33.0	41.8		134.6	
Approach LOS		C	D		F	

Timer	1	2	3	4	5	6	7	8
Assigned Phs				4		6	7	8
Phs Duration (G+Y+Rc), s				90.0		20.0	24.0	66.0
Change Period (Y+Rc), s				4.0		4.0	4.0	4.0
Max Green Setting (Gmax), s				86.0		16.0	20.0	62.0
Max Q Clear Time (g_c+I1), s				19.0		18.0	22.0	64.0
Green Ext Time (p_c), s				37.7		0.0	0.0	0.0

Intersection Summary	
HCM 2010 Ctrl Delay	53.7
HCM 2010 LOS	D

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	271	1343	1729	287	372	357
Future Volume (veh/h)	271	1343	1729	287	372	357
Number	7	4	8	18	1	16
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00			1.00	1.00	1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1900	1863	1863
Adj Flow Rate, veh/h	393	1429	1965	359	454	430
Adj No. of Lanes	1	3	3	0	2	1
Peak Hour Factor	0.69	0.94	0.88	0.80	0.82	0.83
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	426	3557	2072	371	784	654
Arrive On Green	0.19	0.70	0.96	0.96	0.23	0.23
Sat Flow, veh/h	1774	5253	4507	777	3442	1583
Grp Volume(v), veh/h	393	1429	1526	798	454	430
Grp Sat Flow(s),veh/h/ln	1774	1695	1695	1726	1721	1583
Q Serve(g_s), s	17.8	12.9	22.3	30.2	12.9	24.1
Cycle Q Clear(g_c), s	17.8	12.9	22.3	30.2	12.9	24.1
Prop In Lane	1.00			0.45	1.00	1.00
Lane Grp Cap(c), veh/h	426	3557	1619	824	784	654
V/C Ratio(X)	0.92	0.40	0.94	0.97	0.58	0.66
Avail Cap(c_a), veh/h	516	3883	1664	847	784	654
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	0.54	0.54	1.00	1.00
Uniform Delay (d), s/veh	32.0	6.9	1.8	2.0	37.8	26.0
Incr Delay (d2), s/veh	19.9	0.1	6.8	15.6	3.1	5.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	14.0	6.0	7.9	12.4	6.5	21.7
LnGrp Delay(d),s/veh	51.9	7.0	8.6	17.6	40.9	31.1
LnGrp LOS	D	A	A	B	D	C
Approach Vol, veh/h		1822	2324		884	
Approach Delay, s/veh		16.7	11.7		36.1	
Approach LOS		B	B		D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs				4		6	7	8
Phs Duration (G+Y+Rc), s				80.9		29.1	24.4	56.5
Change Period (Y+Rc), s				4.0		4.0	4.0	4.0
Max Green Setting (Gmax), s				84.0		18.0	26.0	54.0
Max Q Clear Time (g_c+I1), s				14.9		26.1	19.8	32.2
Green Ext Time (p_c), s				56.7		0.0	0.6	20.4

Intersection Summary	
HCM 2010 Ctrl Delay	17.8
HCM 2010 LOS	B

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	417	2166	1527	455	315	342
Future Volume (veh/h)	417	2166	1527	455	315	342
Number	7	4	8	18	1	16
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00			1.00	1.00	1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1727	1759	1900	1863	1863
Adj Flow Rate, veh/h	502	2304	1660	555	366	398
Adj No. of Lanes	1	3	3	0	2	1
Peak Hour Factor	0.83	0.94	0.92	0.82	0.86	0.86
Percent Heavy Veh, %	2	10	10	10	2	2
Cap, veh/h	529	3623	1726	561	547	649
Arrive On Green	0.25	0.77	0.96	0.96	0.16	0.16
Sat Flow, veh/h	1774	4871	3747	1166	3442	1583
Grp Volume(v), veh/h	502	2304	1476	739	366	398
Grp Sat Flow(s),veh/h/ln	1774	1572	1601	1553	1721	1583
Q Serve(g_s), s	25.3	24.4	24.8	41.5	11.0	17.5
Cycle Q Clear(g_c), s	25.3	24.4	24.8	41.5	11.0	17.5
Prop In Lane	1.00			0.75	1.00	1.00
Lane Grp Cap(c), veh/h	529	3623	1540	747	547	649
V/C Ratio(X)	0.95	0.64	0.96	0.99	0.67	0.61
Avail Cap(c_a), veh/h	551	3687	1543	749	547	649
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	0.50	0.50	1.00	1.00
Uniform Delay (d), s/veh	33.4	5.8	1.6	1.9	43.5	25.6
Incr Delay (d2), s/veh	25.7	0.4	8.6	20.7	6.4	4.3
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	18.7	10.4	8.0	14.1	5.7	20.0
LnGrp Delay(d),s/veh	59.2	6.1	10.2	22.6	49.9	29.9
LnGrp LOS	E	A	B	C	D	C
Approach Vol, veh/h		2806	2215		764	
Approach Delay, s/veh		15.6	14.3		39.5	
Approach LOS		B	B		D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs				4		6	7	8
Phs Duration (G+Y+Rc), s				88.5		21.5	31.6	56.9
Change Period (Y+Rc), s				4.0		4.0	4.0	4.0
Max Green Setting (Gmax), s				86.0		16.0	29.0	53.0
Max Q Clear Time (g_c+I1), s				26.4		19.5	27.3	43.5
Green Ext Time (p_c), s				56.2		0.0	0.3	9.4

Intersection Summary	
HCM 2010 Ctrl Delay	18.3
HCM 2010 LOS	B

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	356	1349	1760	281	444	396
Future Volume (veh/h)	356	1349	1760	281	444	396
Number	7	4	8	18	1	16
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00			1.00	1.00	1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1900	1863	1863
Adj Flow Rate, veh/h	516	1435	2000	351	541	477
Adj No. of Lanes	1	3	3	0	2	1
Peak Hour Factor	0.69	0.94	0.88	0.80	0.82	0.83
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	533	3883	2026	348	563	677
Arrive On Green	0.26	0.76	0.46	0.46	0.16	0.16
Sat Flow, veh/h	1774	5253	4537	751	3442	1583
Grp Volume(v), veh/h	516	1435	1542	809	541	477
Grp Sat Flow(s),veh/h/ln	1774	1695	1695	1730	1721	1583
Q Serve(g_s), s	27.5	10.2	49.2	51.0	17.2	18.0
Cycle Q Clear(g_c), s	27.5	10.2	49.2	51.0	17.2	18.0
Prop In Lane	1.00			0.43	1.00	1.00
Lane Grp Cap(c), veh/h	533	3883	1572	802	563	677
V/C Ratio(X)	0.97	0.37	0.98	1.01	0.96	0.71
Avail Cap(c_a), veh/h	533	3883	1572	802	563	677
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	34.7	4.3	29.0	29.5	45.6	25.8
Incr Delay (d2), s/veh	30.8	0.1	18.3	33.8	29.4	6.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	20.0	4.7	26.9	31.8	10.4	23.9
LnGrp Delay(d),s/veh	65.5	4.3	47.4	63.3	75.0	31.9
LnGrp LOS	E	A	D	F	E	C
Approach Vol, veh/h		1951	2351		1018	
Approach Delay, s/veh		20.5	52.8		54.8	
Approach LOS		C	D		D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs				4		6	7	8
Phs Duration (G+Y+Rc), s				88.0		22.0	33.0	55.0
Change Period (Y+Rc), s				4.0		4.0	4.0	4.0
Max Green Setting (Gmax), s				84.0		18.0	29.0	51.0
Max Q Clear Time (g_c+I1), s				12.2		20.0	29.5	53.0
Green Ext Time (p_c), s				59.0		0.0	0.0	0.0

Intersection Summary	
HCM 2010 Ctrl Delay	41.4
HCM 2010 LOS	D

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	523	2168	1577	440	431	404
Future Volume (veh/h)	523	2168	1577	440	431	404
Number	7	4	8	18	1	16
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00			1.00	1.00	1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1727	1758	1900	1863	1863
Adj Flow Rate, veh/h	630	2306	1714	537	501	470
Adj No. of Lanes	1	3	3	0	2	1
Peak Hour Factor	0.83	0.94	0.92	0.82	0.86	0.86
Percent Heavy Veh, %	2	10	10	10	2	2
Cap, veh/h	598	3687	1626	495	501	705
Arrive On Green	0.30	0.78	0.45	0.45	0.15	0.15
Sat Flow, veh/h	1774	4871	3808	1111	3442	1583
Grp Volume(v), veh/h	630	2306	1496	755	501	470
Grp Sat Flow(s),veh/h/ln	1774	1572	1600	1562	1721	1583
Q Serve(g_s), s	33.0	23.0	49.0	49.0	16.0	16.0
Cycle Q Clear(g_c), s	33.0	23.0	49.0	49.0	16.0	16.0
Prop In Lane	1.00			0.71	1.00	1.00
Lane Grp Cap(c), veh/h	598	3687	1425	696	501	705
V/C Ratio(X)	1.05	0.63	1.05	1.09	1.00	0.67
Avail Cap(c_a), veh/h	598	3687	1425	696	501	705
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	33.8	5.1	30.5	30.5	47.0	24.1
Incr Delay (d2), s/veh	51.9	0.3	38.1	59.6	40.4	4.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	26.7	9.9	29.1	32.6	10.4	23.2
LnGrp Delay(d),s/veh	85.7	5.5	68.6	90.1	87.4	29.0
LnGrp LOS	F	A	F	F	F	C
Approach Vol, veh/h		2936	2251		971	
Approach Delay, s/veh		22.7	75.8		59.1	
Approach LOS		C	E		E	

Timer	1	2	3	4	5	6	7	8
Assigned Phs				4		6	7	8
Phs Duration (G+Y+Rc), s				90.0		20.0	37.0	53.0
Change Period (Y+Rc), s				4.0		4.0	4.0	4.0
Max Green Setting (Gmax), s				86.0		16.0	33.0	49.0
Max Q Clear Time (g_c+I1), s				25.0		18.0	35.0	51.0
Green Ext Time (p_c), s				57.6		0.0	0.0	0.0

Intersection Summary	
HCM 2010 Ctrl Delay	47.8
HCM 2010 LOS	D

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	2	707	96	252	899	52	39	44	113	85	90	12
Future Volume (veh/h)	2	707	96	252	899	52	39	44	113	85	90	12
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863	1863	1863	1863	1863	1863	1900
Adj Flow Rate, veh/h	4	744	135	332	1022	76	80	68	169	112	108	16
Adj No. of Lanes	1	1	1	1	1	1	1	1	1	1	1	0
Peak Hour Factor	0.50	0.95	0.71	0.76	0.88	0.68	0.49	0.65	0.67	0.76	0.83	0.75
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	124	908	772	537	1106	940	336	410	349	347	349	52
Arrive On Green	0.01	0.98	0.98	0.11	0.59	0.59	0.04	0.22	0.22	0.04	0.22	0.22
Sat Flow, veh/h	1774	1863	1583	1774	1863	1583	1774	1863	1583	1774	1586	235
Grp Volume(v), veh/h	4	744	135	332	1022	76	80	68	169	112	0	124
Grp Sat Flow(s),veh/h/ln	1774	1863	1583	1774	1863	1583	1774	1863	1583	1774	0	1821
Q Serve(g_s), s	0.1	5.4	0.3	9.7	54.3	2.3	3.9	3.2	10.2	4.0	0.0	6.3
Cycle Q Clear(g_c), s	0.1	5.4	0.3	9.7	54.3	2.3	3.9	3.2	10.2	4.0	0.0	6.3
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		0.13
Lane Grp Cap(c), veh/h	124	908	772	537	1106	940	336	410	349	347	0	401
V/C Ratio(X)	0.03	0.82	0.17	0.62	0.92	0.08	0.24	0.17	0.48	0.32	0.00	0.31
Avail Cap(c_a), veh/h	181	965	820	599	1168	993	336	410	349	347	0	401
HCM Platoon Ratio	2.00	2.00	2.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.97	0.97	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	22.5	0.8	0.7	10.5	20.1	9.5	32.0	34.7	37.4	33.3	0.0	35.9
Incr Delay (d2), s/veh	0.1	5.2	0.1	1.6	11.8	0.0	0.4	0.9	4.8	0.5	0.0	2.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.1	2.1	0.1	4.9	31.2	1.0	1.9	1.8	5.0	0.9	0.0	3.4
LnGrp Delay(d),s/veh	22.6	6.0	0.8	12.1	32.0	9.6	32.4	35.6	42.2	33.9	0.0	37.9
LnGrp LOS	C	A	A	B	C	A	C	D	D	C		D
Approach Vol, veh/h		883			1430			317			236	
Approach Delay, s/veh		5.3			26.2			38.3			36.0	
Approach LOS		A			C			D			D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	8.0	28.2	16.1	57.6	8.0	28.2	4.5	69.3
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	4.0	17.0	16.0	57.0	4.0	17.0	4.0	69.0
Max Q Clear Time (g_c+I1), s	6.0	12.2	11.7	7.4	5.9	8.3	2.1	56.3
Green Ext Time (p_c), s	0.0	0.7	0.4	19.7	0.0	1.0	0.0	9.0

Intersection Summary	
HCM 2010 Ctrl Delay	21.9
HCM 2010 LOS	C

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	11	1068	77	110	791	88	64	67	132	76	58	8
Future Volume (veh/h)	11	1068	77	110	791	88	64	67	132	76	58	8
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1727	1863	1863	1727	1863	1863	1863	1863	1863	1863	1900
Adj Flow Rate, veh/h	20	1148	120	120	851	111	88	84	167	88	72	20
Adj No. of Lanes	1	1	1	1	1	1	1	1	1	1	1	0
Peak Hour Factor	0.55	0.93	0.64	0.92	0.93	0.79	0.73	0.80	0.79	0.86	0.81	0.40
Percent Heavy Veh, %	2	10	2	2	10	2	2	2	2	2	2	2
Cap, veh/h	282	1099	1008	130	1133	1039	259	271	230	248	204	57
Arrive On Green	0.02	0.85	0.85	0.04	0.66	0.66	0.04	0.15	0.15	0.04	0.15	0.15
Sat Flow, veh/h	1774	1727	1583	1774	1727	1583	1774	1863	1583	1774	1404	390
Grp Volume(v), veh/h	20	1148	120	120	851	111	88	84	167	88	0	92
Grp Sat Flow(s),veh/h/ln	1774	1727	1583	1774	1727	1583	1774	1863	1583	1774	0	1794
Q Serve(g_s), s	0.4	70.0	1.4	3.3	36.7	2.9	4.0	4.4	11.1	4.0	0.0	5.1
Cycle Q Clear(g_c), s	0.4	70.0	1.4	3.3	36.7	2.9	4.0	4.4	11.1	4.0	0.0	5.1
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		0.22
Lane Grp Cap(c), veh/h	282	1099	1008	130	1133	1039	259	271	230	248	0	261
V/C Ratio(X)	0.07	1.04	0.12	0.92	0.75	0.11	0.34	0.31	0.73	0.35	0.00	0.35
Avail Cap(c_a), veh/h	317	1099	1008	130	1133	1039	259	271	230	248	0	261
HCM Platoon Ratio	1.33	1.33	1.33	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.90	0.90	0.90	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	12.3	8.4	3.2	32.2	12.8	7.0	39.5	42.1	44.9	39.4	0.0	42.3
Incr Delay (d2), s/veh	0.1	38.2	0.0	56.0	2.8	0.0	0.8	3.0	18.0	0.9	0.0	3.7
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	42.8	0.6	5.7	18.2	1.2	0.5	2.5	6.0	2.3	0.0	2.8
LnGrp Delay(d),s/veh	12.3	46.6	3.2	88.2	15.7	7.0	40.3	45.0	62.9	40.3	0.0	46.0
LnGrp LOS	B	F	A	F	B	A	D	D	E	D		D
Approach Vol, veh/h		1288			1082			339			180	
Approach Delay, s/veh		42.1			22.8			52.6			43.2	
Approach LOS		D			C			D			D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	8.0	20.0	8.0	74.0	8.0	20.0	5.8	76.2
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	4.0	16.0	4.0	70.0	4.0	16.0	4.0	70.0
Max Q Clear Time (g_c+I1), s	6.0	13.1	5.3	72.0	6.0	7.1	2.4	38.7
Green Ext Time (p_c), s	0.0	0.4	0.0	0.0	0.0	0.9	0.0	19.9

Intersection Summary			
HCM 2010 Ctrl Delay			36.2
HCM 2010 LOS			D

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	30	786	112	276	1081	74	79	76	125	159	144	19
Future Volume (veh/h)	30	786	112	276	1081	74	79	76	125	159	144	19
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863	1863	1863	1863	1863	1863	1900
Adj Flow Rate, veh/h	60	827	158	363	1228	109	161	117	187	209	173	25
Adj No. of Lanes	1	1	1	1	1	1	1	1	1	1	1	0
Peak Hour Factor	0.50	0.95	0.71	0.76	0.88	0.68	0.49	0.65	0.67	0.76	0.83	0.75
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	120	1026	872	569	1168	993	202	282	240	247	241	35
Arrive On Green	0.06	1.00	1.00	0.11	0.63	0.63	0.05	0.15	0.15	0.05	0.15	0.15
Sat Flow, veh/h	1774	1863	1583	1774	1863	1583	1774	1863	1583	1774	1592	230
Grp Volume(v), veh/h	60	827	158	363	1228	109	161	117	187	209	0	198
Grp Sat Flow(s),veh/h/ln	1774	1863	1583	1774	1863	1583	1774	1863	1583	1774	0	1822
Q Serve(g_s), s	1.6	0.0	0.0	9.2	69.0	3.0	5.0	6.3	12.5	5.0	0.0	11.4
Cycle Q Clear(g_c), s	1.6	0.0	0.0	9.2	69.0	3.0	5.0	6.3	12.5	5.0	0.0	11.4
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		0.13
Lane Grp Cap(c), veh/h	120	1026	872	569	1168	993	202	282	240	247	0	276
V/C Ratio(X)	0.50	0.81	0.18	0.64	1.05	0.11	0.80	0.42	0.78	0.85	0.00	0.72
Avail Cap(c_a), veh/h	130	1026	872	669	1168	993	202	282	240	247	0	276
HCM Platoon Ratio	2.00	2.00	2.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.95	0.95	0.95	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	25.8	0.0	0.0	7.3	20.5	8.2	44.5	42.3	44.9	45.1	0.0	44.4
Incr Delay (d2), s/veh	3.1	4.6	0.1	1.6	40.8	0.0	19.4	4.5	21.9	22.8	0.0	14.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.1	1.3	0.0	4.7	48.3	1.3	3.5	3.6	6.9	6.9	0.0	6.9
LnGrp Delay(d),s/veh	28.9	4.6	0.1	8.9	61.3	8.3	63.8	46.7	66.8	67.9	0.0	59.4
LnGrp LOS	C	A	A	A	F	A	E	D	E	E		E
Approach Vol, veh/h		1045			1700			465			407	
Approach Delay, s/veh		5.3			46.7			60.7			63.7	
Approach LOS		A			D			E			E	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	9.0	20.6	15.8	64.6	9.0	20.6	7.4	73.0
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	5.0	16.0	18.0	55.0	5.0	16.0	4.0	69.0
Max Q Clear Time (g_c+I1), s	7.0	14.5	11.2	2.0	7.0	13.4	3.6	71.0
Green Ext Time (p_c), s	0.0	0.4	0.6	28.9	0.0	0.6	0.0	0.0

Intersection Summary	
HCM 2010 Ctrl Delay	38.5
HCM 2010 LOS	D

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	68	1147	89	118	970	138	121	131	143	248	160	27
Future Volume (veh/h)	68	1147	89	118	970	138	121	131	143	248	160	27
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1727	1863	1863	1727	1863	1863	1863	1863	1863	1863	1900
Adj Flow Rate, veh/h	124	1233	139	128	1043	175	166	164	181	288	198	68
Adj No. of Lanes	1	1	1	1	1	1	1	1	1	1	1	0
Peak Hour Factor	0.55	0.93	0.64	0.92	0.93	0.79	0.73	0.80	0.79	0.86	0.81	0.40
Percent Heavy Veh, %	2	10	2	2	10	2	2	2	2	2	2	2
Cap, veh/h	136	1052	964	130	1052	964	169	288	245	237	205	70
Arrive On Green	0.05	0.81	0.81	0.04	0.61	0.61	0.05	0.15	0.15	0.05	0.15	0.15
Sat Flow, veh/h	1774	1727	1583	1774	1727	1583	1774	1863	1583	1774	1327	456
Grp Volume(v), veh/h	124	1233	139	128	1043	175	166	164	181	288	0	266
Grp Sat Flow(s),veh/h/ln	1774	1727	1583	1774	1727	1583	1774	1863	1583	1774	0	1782
Q Serve(g_s), s	3.2	67.0	2.1	3.9	65.5	5.3	6.0	9.0	12.0	6.0	0.0	16.3
Cycle Q Clear(g_c), s	3.2	67.0	2.1	3.9	65.5	5.3	6.0	9.0	12.0	6.0	0.0	16.3
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		0.26
Lane Grp Cap(c), veh/h	136	1052	964	130	1052	964	169	288	245	237	0	275
V/C Ratio(X)	0.91	1.17	0.14	0.98	0.99	0.18	0.98	0.57	0.74	1.21	0.00	0.97
Avail Cap(c_a), veh/h	136	1052	964	130	1052	964	169	288	245	237	0	275
HCM Platoon Ratio	1.33	1.33	1.33	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.84	0.84	0.84	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	28.5	10.4	4.3	32.0	21.2	9.4	43.3	43.1	44.4	45.6	0.0	46.2
Incr Delay (d2), s/veh	46.0	86.2	0.1	74.1	25.6	0.1	63.5	8.0	18.1	128.1	0.0	46.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	5.5	55.4	0.9	6.6	38.5	2.4	5.1	5.2	6.5	18.8	0.0	11.6
LnGrp Delay(d),s/veh	74.5	96.7	4.3	106.1	46.8	9.5	106.8	51.1	62.5	173.7	0.0	92.3
LnGrp LOS	E	F	A	F	D	A	F	D	E	F		F
Approach Vol, veh/h		1496			1346			511			554	
Approach Delay, s/veh		86.2			47.6			73.2			134.6	
Approach LOS		F			D			E			F	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	10.0	21.0	8.0	71.0	10.0	21.0	8.0	71.0
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	6.0	17.0	4.0	67.0	6.0	17.0	4.0	67.0
Max Q Clear Time (g_c+I1), s	8.0	14.0	5.9	69.0	8.0	18.3	5.2	67.5
Green Ext Time (p_c), s	0.0	0.9	0.0	0.0	0.0	0.0	0.0	0.0

Intersection Summary	
HCM 2010 Ctrl Delay	78.1
HCM 2010 LOS	E

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	30	802	128	276	1109	76	116	80	125	167	160	19
Future Volume (veh/h)	30	802	128	276	1109	76	116	80	125	167	160	19
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863	1863	1863	1863	1863	1863	1900
Adj Flow Rate, veh/h	60	844	180	363	1260	112	237	123	187	220	193	25
Adj No. of Lanes	1	1	1	1	1	1	1	1	1	1	1	0
Peak Hour Factor	0.50	0.95	0.71	0.76	0.88	0.68	0.49	0.65	0.67	0.76	0.83	0.75
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	120	1022	869	226	1033	878	309	333	283	355	274	35
Arrive On Green	0.03	0.55	0.55	0.04	0.55	0.55	0.10	0.18	0.18	0.09	0.17	0.17
Sat Flow, veh/h	1774	1863	1583	1774	1863	1583	1774	1863	1583	1774	1616	209
Grp Volume(v), veh/h	60	844	180	363	1260	112	237	123	187	220	0	218
Grp Sat Flow(s),veh/h/ln	1774	1863	1583	1774	1863	1583	1774	1863	1583	1774	0	1826
Q Serve(g_s), s	1.6	41.1	6.4	4.0	61.0	3.7	11.0	6.4	12.1	10.0	0.0	12.4
Cycle Q Clear(g_c), s	1.6	41.1	6.4	4.0	61.0	3.7	11.0	6.4	12.1	10.0	0.0	12.4
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		0.11
Lane Grp Cap(c), veh/h	120	1022	869	226	1033	878	309	333	283	355	0	309
V/C Ratio(X)	0.50	0.83	0.21	1.61	1.22	0.13	0.77	0.37	0.66	0.62	0.00	0.70
Avail Cap(c_a), veh/h	130	1033	878	226	1033	878	309	333	283	355	0	309
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	26.6	20.5	12.6	33.1	24.5	11.7	36.1	39.7	42.1	35.2	0.0	43.1
Incr Delay (d2), s/veh	3.2	5.6	0.1	292.9	107.8	0.1	11.1	3.1	11.6	3.3	0.0	12.7
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.1	22.7	2.8	23.3	61.9	1.6	2.7	3.6	6.2	5.8	0.0	7.3
LnGrp Delay(d),s/veh	29.9	26.0	12.8	326.0	132.3	11.8	47.2	42.9	53.6	38.5	0.0	55.8
LnGrp LOS	C	C	B	F	F	B	D	D	D	D		E
Approach Vol, veh/h		1084			1735			547			438	
Approach Delay, s/veh		24.0			165.0			48.4			47.1	
Approach LOS		C			F			D			D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	14.0	23.6	8.0	64.4	15.0	22.6	7.4	65.0
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	10.0	19.0	4.0	61.0	11.0	18.0	4.0	61.0
Max Q Clear Time (g_c+I1), s	12.0	14.1	6.0	43.1	13.0	14.4	3.6	63.0
Green Ext Time (p_c), s	0.0	1.1	0.0	14.2	0.0	0.9	0.0	0.0

Intersection Summary	
HCM 2010 Ctrl Delay	94.5
HCM 2010 LOS	F

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	68	1172	114	118	1004	140	164	136	143	260	185	27
Future Volume (veh/h)	68	1172	114	118	1004	140	164	136	143	260	185	27
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1727	1863	1863	1727	1863	1863	1863	1863	1863	1863	1900
Adj Flow Rate, veh/h	124	1260	178	128	1080	177	225	170	181	302	228	68
Adj No. of Lanes	1	1	1	1	1	1	1	1	1	1	1	0
Peak Hour Factor	0.55	0.93	0.64	0.92	0.93	0.79	0.73	0.80	0.79	0.86	0.81	0.40
Percent Heavy Veh, %	2	10	2	2	10	2	2	2	2	2	2	2
Cap, veh/h	130	1036	950	130	1036	950	178	288	245	250	213	64
Arrive On Green	0.04	0.60	0.60	0.04	0.60	0.60	0.06	0.15	0.15	0.06	0.15	0.15
Sat Flow, veh/h	1774	1727	1583	1774	1727	1583	1774	1863	1583	1774	1379	411
Grp Volume(v), veh/h	124	1260	178	128	1080	177	225	170	181	302	0	296
Grp Sat Flow(s),veh/h/ln	1774	1727	1583	1774	1727	1583	1774	1863	1583	1774	0	1790
Q Serve(g_s), s	3.6	66.0	5.6	3.9	66.0	5.5	7.0	9.3	12.0	7.0	0.0	17.0
Cycle Q Clear(g_c), s	3.6	66.0	5.6	3.9	66.0	5.5	7.0	9.3	12.0	7.0	0.0	17.0
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		0.23
Lane Grp Cap(c), veh/h	130	1036	950	130	1036	950	178	288	245	250	0	277
V/C Ratio(X)	0.95	1.22	0.19	0.98	1.04	0.19	1.26	0.59	0.74	1.21	0.00	1.07
Avail Cap(c_a), veh/h	130	1036	950	130	1036	950	178	288	245	250	0	277
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	30.7	22.0	9.9	31.5	22.0	9.9	42.0	43.3	44.4	44.6	0.0	46.5
Incr Delay (d2), s/veh	64.7	106.1	0.1	74.1	39.5	0.1	154.7	8.6	18.1	125.3	0.0	73.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	6.1	61.6	2.4	6.6	42.5	2.4	9.7	5.5	6.5	8.7	0.0	14.1
LnGrp Delay(d),s/veh	95.4	128.1	10.0	105.6	61.5	10.0	196.8	51.9	62.5	170.0	0.0	120.4
LnGrp LOS	F	F	B	F	F	B	F	D	E	F		F
Approach Vol, veh/h		1562			1385			576			598	
Approach Delay, s/veh		112.0			59.0			111.8			145.4	
Approach LOS		F			E			F			F	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	11.0	21.0	8.0	70.0	11.0	21.0	8.0	70.0
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	7.0	17.0	4.0	66.0	7.0	17.0	4.0	66.0
Max Q Clear Time (g_c+I1), s	9.0	14.0	5.9	68.0	9.0	19.0	5.6	68.0
Green Ext Time (p_c), s	0.0	1.0	0.0	0.0	0.0	0.0	0.0	0.0

Intersection Summary	
HCM 2010 Ctrl Delay	99.0
HCM 2010 LOS	F

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	30	802	128	276	1109	76	116	80	125	167	160	19
Future Volume (veh/h)	30	802	128	276	1109	76	116	80	125	167	160	19
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1900	1863	1863	1863	1863	1863	1900
Adj Flow Rate, veh/h	60	844	180	363	1260	112	237	123	187	220	193	25
Adj No. of Lanes	1	2	0	1	2	0	1	1	1	1	1	0
Peak Hour Factor	0.50	0.95	0.71	0.76	0.88	0.68	0.49	0.65	0.67	0.76	0.83	0.75
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	173	993	212	399	1508	134	435	472	401	472	409	53
Arrive On Green	0.07	0.68	0.68	0.15	0.46	0.46	0.11	0.25	0.25	0.11	0.25	0.25
Sat Flow, veh/h	1774	2904	619	1774	3289	292	1774	1863	1583	1774	1616	209
Grp Volume(v), veh/h	60	514	510	363	676	696	237	123	187	220	0	218
Grp Sat Flow(s),veh/h/ln	1774	1770	1753	1774	1770	1811	1774	1863	1583	1774	0	1826
Q Serve(g_s), s	2.4	24.1	24.1	13.9	36.9	37.1	10.8	5.8	11.0	9.9	0.0	11.1
Cycle Q Clear(g_c), s	2.4	24.1	24.1	13.9	36.9	37.1	10.8	5.8	11.0	9.9	0.0	11.1
Prop In Lane	1.00		0.35	1.00		0.16	1.00		1.00	1.00		0.11
Lane Grp Cap(c), veh/h	173	605	600	399	812	831	435	472	401	472	0	462
V/C Ratio(X)	0.35	0.85	0.85	0.91	0.83	0.84	0.54	0.26	0.47	0.47	0.00	0.47
Avail Cap(c_a), veh/h	210	605	600	520	885	906	435	472	401	489	0	462
HCM Platoon Ratio	2.00	2.00	2.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	24.4	15.2	15.2	22.5	26.1	26.2	26.5	32.8	34.8	25.7	0.0	34.8
Incr Delay (d2), s/veh	1.2	11.1	11.2	16.8	6.4	6.5	1.4	1.3	3.9	0.7	0.0	3.4
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.2	13.3	13.2	8.6	19.3	20.1	5.4	3.1	5.2	4.9	0.0	6.1
LnGrp Delay(d),s/veh	25.6	26.3	26.4	39.3	32.5	32.7	27.9	34.2	38.6	26.5	0.0	38.3
LnGrp LOS	C	C	C	D	C	C	C	C	D	C		D
Approach Vol, veh/h		1084			1735			547			438	
Approach Delay, s/veh		26.3			34.0			33.0			32.3	
Approach LOS		C			C			C			C	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	16.0	31.9	20.5	41.6	16.0	31.8	7.7	54.4
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	13.0	20.0	24.0	37.0	12.0	21.0	6.0	55.0
Max Q Clear Time (g_c+I1), s	11.9	13.0	15.9	26.1	12.8	13.1	4.4	39.1
Green Ext Time (p_c), s	0.1	1.4	0.7	8.6	0.0	1.5	0.0	11.3

Intersection Summary	
HCM 2010 Ctrl Delay	31.5
HCM 2010 LOS	C

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	68	1172	114	118	1004	140	164	136	143	260	185	27
Future Volume (veh/h)	68	1172	114	118	1004	140	164	136	143	260	185	27
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1743	1900	1863	1745	1900	1863	1863	1863	1863	1863	1900
Adj Flow Rate, veh/h	124	1260	178	128	1080	177	225	170	181	302	228	68
Adj No. of Lanes	1	2	0	1	2	0	1	1	1	1	1	0
Peak Hour Factor	0.55	0.93	0.64	0.92	0.93	0.79	0.73	0.80	0.79	0.86	0.81	0.40
Percent Heavy Veh, %	2	10	10	2	10	10	2	2	2	2	2	2
Cap, veh/h	220	1345	189	180	1320	216	342	359	306	438	316	94
Arrive On Green	0.05	0.46	0.46	0.05	0.46	0.46	0.11	0.19	0.19	0.15	0.23	0.23
Sat Flow, veh/h	1774	2917	410	1774	2854	467	1774	1863	1583	1774	1379	411
Grp Volume(v), veh/h	124	712	726	128	626	631	225	170	181	302	0	296
Grp Sat Flow(s),veh/h/ln	1774	1656	1671	1774	1658	1663	1774	1863	1583	1774	0	1790
Q Serve(g_s), s	4.0	44.8	45.5	4.1	35.9	36.1	11.2	8.9	11.5	14.5	0.0	16.8
Cycle Q Clear(g_c), s	4.0	44.8	45.5	4.1	35.9	36.1	11.2	8.9	11.5	14.5	0.0	16.8
Prop In Lane	1.00		0.25	1.00		0.28	1.00		1.00	1.00		0.23
Lane Grp Cap(c), veh/h	220	764	771	180	767	769	342	359	306	438	0	411
V/C Ratio(X)	0.56	0.93	0.94	0.71	0.82	0.82	0.66	0.47	0.59	0.69	0.00	0.72
Avail Cap(c_a), veh/h	237	768	775	196	769	771	342	359	306	438	0	411
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	22.1	28.0	28.2	25.2	25.5	25.6	31.6	39.4	40.4	28.2	0.0	39.1
Incr Delay (d2), s/veh	2.6	18.1	19.5	10.3	6.9	7.0	4.6	4.4	8.2	4.5	0.0	10.5
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.1	24.2	25.1	2.5	17.8	18.0	5.8	5.0	5.7	7.6	0.0	9.5
LnGrp Delay(d),s/veh	24.8	46.2	47.7	35.5	32.4	32.6	36.2	43.8	48.6	32.7	0.0	49.6
LnGrp LOS	C	D	D	D	C	C	D	D	D	C		D
Approach Vol, veh/h		1562			1385			576			598	
Approach Delay, s/veh		45.2			32.8			42.4			41.1	
Approach LOS		D			C			D			D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	20.0	25.2	10.0	54.7	16.0	29.2	9.9	54.9
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	16.0	20.0	7.0	51.0	12.0	24.0	7.0	51.0
Max Q Clear Time (g_c+I1), s	16.5	13.5	6.1	47.5	13.2	18.8	6.0	38.1
Green Ext Time (p_c), s	0.0	1.8	0.0	3.2	0.0	1.5	0.0	10.8

Intersection Summary			
HCM 2010 Ctrl Delay			40.0
HCM 2010 LOS			D

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	32	1431	200	506	1900	121	114	117	228	236	226	30
Future Volume (veh/h)	32	1431	200	506	1900	121	114	117	228	236	226	30
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	64	1506	282	666	2159	178	233	180	340	311	272	40
Adj No. of Lanes	2	3	0	2	3	0	2	2	1	2	2	1
Peak Hour Factor	0.50	0.95	0.71	0.76	0.88	0.68	0.49	0.65	0.67	0.76	0.83	0.75
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	288	1790	334	749	2615	214	634	822	368	613	822	368
Arrive On Green	0.04	0.55	0.55	0.16	0.55	0.55	0.05	0.23	0.23	0.05	0.23	0.23
Sat Flow, veh/h	3442	4307	804	3442	4793	391	3442	3539	1583	3442	3539	1583
Grp Volume(v), veh/h	64	1184	604	666	1521	816	233	180	340	311	272	40
Grp Sat Flow(s),veh/h/ln	1721	1695	1721	1721	1695	1794	1721	1770	1583	1721	1770	1583
Q Serve(g_s), s	1.1	32.1	32.3	14.5	40.7	41.7	5.0	4.5	23.1	5.0	7.0	2.2
Cycle Q Clear(g_c), s	1.1	32.1	32.3	14.5	40.7	41.7	5.0	4.5	23.1	5.0	7.0	2.2
Prop In Lane	1.00		0.47	1.00		0.22	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	288	1409	715	749	1850	979	634	822	368	613	822	368
V/C Ratio(X)	0.22	0.84	0.84	0.89	0.82	0.83	0.37	0.22	0.93	0.51	0.33	0.11
Avail Cap(c_a), veh/h	306	1409	715	881	1911	1011	634	822	368	613	822	368
HCM Platoon Ratio	1.33	1.33	1.33	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.88	0.88	0.88	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	21.2	21.6	21.6	29.1	20.6	20.8	31.8	34.2	41.3	33.6	35.1	33.3
Incr Delay (d2), s/veh	0.3	4.2	8.1	10.0	3.0	5.9	0.4	0.6	31.3	0.7	1.1	0.6
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.6	15.6	16.7	10.8	19.6	22.0	0.7	2.3	13.3	1.8	3.6	1.0
LnGrp Delay(d),s/veh	21.6	25.8	29.7	39.1	23.5	26.8	32.2	34.8	72.6	34.3	36.2	33.9
LnGrp LOS	C	C	C	D	C	C	C	C	E	C	D	C
Approach Vol, veh/h		1852			3003			753			623	
Approach Delay, s/veh		26.9			27.9			51.1			35.1	
Approach LOS		C			C			D			D	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	9.0	29.5	21.8	49.7	9.0	29.5	7.4	64.0				
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0				
Max Green Setting (Gmax), s	5.0	23.0	22.0	44.0	5.0	23.0	4.0	62.0				
Max Q Clear Time (g_c+I1), s	7.0	25.1	16.5	34.3	7.0	9.0	3.1	43.7				
Green Ext Time (p_c), s	0.0	0.0	1.3	9.4	0.0	3.4	0.0	16.3				
Intersection Summary												
HCM 2010 Ctrl Delay			31.1									
HCM 2010 LOS			C									

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	78	2120	160	219	1691	218	180	192	264	318	212	34
Future Volume (veh/h)	78	2120	160	219	1691	218	180	192	264	318	212	34
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1740	1900	1863	1744	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	142	2280	250	238	1818	276	247	240	334	370	262	85
Adj No. of Lanes	1	3	0	2	3	0	2	2	1	2	2	1
Peak Hour Factor	0.55	0.93	0.64	0.92	0.93	0.79	0.73	0.80	0.79	0.86	0.81	0.40
Percent Heavy Veh, %	2	10	10	2	10	10	2	2	2	2	2	2
Cap, veh/h	195	2366	255	293	2234	336	615	780	349	553	748	335
Arrive On Green	0.05	0.54	0.54	0.05	0.53	0.53	0.05	0.22	0.22	0.05	0.21	0.21
Sat Flow, veh/h	1774	4355	469	3442	4179	629	3442	3539	1583	3442	3539	1583
Grp Volume(v), veh/h	142	1646	884	238	1377	717	247	240	334	370	262	85
Grp Sat Flow(s),veh/h/ln	1774	1583	1657	1721	1587	1633	1721	1770	1583	1721	1770	1583
Q Serve(g_s), s	3.9	54.4	57.5	3.4	39.2	40.1	6.0	6.2	22.9	5.0	6.9	4.9
Cycle Q Clear(g_c), s	3.9	54.4	57.5	3.4	39.2	40.1	6.0	6.2	22.9	5.0	6.9	4.9
Prop In Lane	1.00		0.28	1.00		0.38	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	195	1720	900	293	1697	873	615	780	349	553	748	335
V/C Ratio(X)	0.73	0.96	0.98	0.81	0.81	0.82	0.40	0.31	0.96	0.67	0.35	0.25
Avail Cap(c_a), veh/h	244	1727	904	293	1697	873	615	780	349	553	748	335
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.72	0.72	0.72	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	23.6	23.9	24.6	26.1	21.0	21.2	32.4	35.9	42.4	38.0	36.9	36.2
Incr Delay (d2), s/veh	5.9	10.2	21.1	16.0	3.1	6.3	0.4	1.0	38.5	3.1	1.3	1.8
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.6	26.0	31.3	2.6	17.8	19.5	0.4	3.1	13.7	4.6	3.5	2.3
LnGrp Delay(d),s/veh	29.6	34.1	45.7	42.1	24.1	27.6	32.8	36.9	80.8	41.1	38.2	38.0
LnGrp LOS	C	C	D	D	C	C	C	D	F	D	D	D
Approach Vol, veh/h		2672			2332			821			717	
Approach Delay, s/veh		37.7			27.0			53.5			39.7	
Approach LOS		D			C			D			D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	9.0	28.2	9.0	63.8	10.0	27.2	9.9	62.8
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	5.0	24.0	5.0	60.0	6.0	23.0	9.0	56.0
Max Q Clear Time (g_c+I1), s	7.0	24.9	5.4	59.5	8.0	8.9	5.9	42.1
Green Ext Time (p_c), s	0.0	0.0	0.0	0.3	0.0	3.8	0.1	13.7

Intersection Summary	
HCM 2010 Ctrl Delay	36.1
HCM 2010 LOS	D

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	32	1447	216	506	1928	123	151	121	228	244	242	30
Future Volume (veh/h)	32	1447	216	506	1928	123	151	121	228	244	242	30
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	64	1523	304	666	2191	181	308	186	340	321	292	40
Adj No. of Lanes	2	3	0	2	3	0	2	2	1	2	2	1
Peak Hour Factor	0.50	0.95	0.71	0.76	0.88	0.68	0.49	0.65	0.67	0.76	0.83	0.75
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	282	1740	346	740	2605	213	667	829	371	612	701	313
Arrive On Green	0.03	0.41	0.41	0.17	0.54	0.54	0.08	0.23	0.23	0.05	0.20	0.20
Sat Flow, veh/h	3442	4257	846	3442	4792	392	3442	3539	1583	3442	3539	1583
Grp Volume(v), veh/h	64	1212	615	666	1543	829	308	186	340	321	292	40
Grp Sat Flow(s),veh/h/ln	1721	1695	1713	1721	1695	1794	1721	1770	1583	1721	1770	1583
Q Serve(g_s), s	1.2	36.2	36.5	15.3	42.0	43.1	7.6	4.7	23.0	5.0	7.9	2.3
Cycle Q Clear(g_c), s	1.2	36.2	36.5	15.3	42.0	43.1	7.6	4.7	23.0	5.0	7.9	2.3
Prop In Lane	1.00		0.49	1.00		0.22	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	282	1386	700	740	1843	975	667	829	371	612	701	313
V/C Ratio(X)	0.23	0.87	0.88	0.90	0.84	0.85	0.46	0.22	0.92	0.52	0.42	0.13
Avail Cap(c_a), veh/h	300	1386	700	826	1880	995	667	829	371	612	701	313
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	22.3	29.9	30.0	31.1	21.0	21.3	30.3	34.0	41.1	35.7	38.6	36.3
Incr Delay (d2), s/veh	0.4	6.5	12.3	12.0	3.5	7.0	0.5	0.6	29.7	0.8	1.8	0.8
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.6	18.1	19.7	11.0	20.4	23.1	3.6	2.3	13.1	2.0	4.1	1.1
LnGrp Delay(d),s/veh	22.7	36.4	42.3	43.1	24.5	28.3	30.8	34.7	70.8	36.5	40.4	37.1
LnGrp LOS	C	D	D	D	C	C	C	C	E	D	D	D
Approach Vol, veh/h		1891			3038			834			653	
Approach Delay, s/veh		37.9			29.6			48.0			38.3	
Approach LOS		D			C			D			D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	9.0	29.8	22.3	49.0	13.0	25.8	7.4	63.8
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	5.0	24.0	21.0	44.0	9.0	20.0	4.0	61.0
Max Q Clear Time (g_c+I1), s	7.0	25.0	17.3	38.5	9.6	9.9	3.2	45.1
Green Ext Time (p_c), s	0.0	0.0	1.0	5.5	0.0	3.0	0.0	14.7

Intersection Summary	
HCM 2010 Ctrl Delay	35.3
HCM 2010 LOS	D

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	78	2145	185	219	1725	220	223	197	264	330	237	34
Future Volume (veh/h)	78	2145	185	219	1725	220	223	197	264	330	237	34
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1741	1900	1863	1744	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	142	2306	289	238	1855	278	305	246	334	384	293	85
Adj No. of Lanes	2	3	0	2	3	0	2	2	1	2	2	1
Peak Hour Factor	0.55	0.93	0.64	0.92	0.93	0.79	0.73	0.80	0.79	0.86	0.81	0.40
Percent Heavy Veh, %	2	10	10	2	10	10	2	2	2	2	2	2
Cap, veh/h	188	2224	272	282	2284	339	363	515	360	375	527	236
Arrive On Green	0.05	0.52	0.52	0.08	0.55	0.55	0.11	0.15	0.15	0.11	0.15	0.15
Sat Flow, veh/h	3442	4293	525	3442	4187	621	3442	3539	1583	3442	3539	1583
Grp Volume(v), veh/h	142	1688	907	238	1402	731	305	246	334	384	293	85
Grp Sat Flow(s),veh/h/ln	1721	1585	1649	1721	1587	1634	1721	1770	1583	1721	1770	1583
Q Serve(g_s), s	4.5	57.0	57.0	7.5	39.5	40.5	9.6	7.0	16.0	12.0	8.4	5.3
Cycle Q Clear(g_c), s	4.5	57.0	57.0	7.5	39.5	40.5	9.6	7.0	16.0	12.0	8.4	5.3
Prop In Lane	1.00		0.32	1.00		0.38	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	188	1642	854	282	1731	891	363	515	360	375	527	236
V/C Ratio(X)	0.76	1.03	1.06	0.85	0.81	0.82	0.84	0.48	0.93	1.02	0.56	0.36
Avail Cap(c_a), veh/h	188	1642	854	282	1731	891	375	515	360	375	527	236
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	51.3	26.5	26.5	49.8	20.4	20.6	48.3	43.2	41.6	49.0	43.4	42.1
Incr Delay (d2), s/veh	16.1	29.6	48.6	20.4	3.0	6.2	15.1	3.2	32.3	52.4	4.2	4.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.5	31.4	37.2	4.4	17.9	19.6	5.3	3.6	13.2	8.4	4.5	2.6
LnGrp Delay(d),s/veh	67.3	56.1	75.1	70.2	23.3	26.7	63.4	46.3	73.9	101.4	47.6	46.3
LnGrp LOS	E	F	F	E	C	C	E	D	E	F	D	D
Approach Vol, veh/h		2737			2371			885			762	
Approach Delay, s/veh		63.0			29.1			62.6			74.6	
Approach LOS		E			C			E			E	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	16.0	20.0	13.0	61.0	15.6	20.4	10.0	64.0
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	12.0	16.0	9.0	57.0	12.0	16.0	6.0	60.0
Max Q Clear Time (g_c+I1), s	14.0	18.0	9.5	59.0	11.6	10.4	6.5	42.5
Green Ext Time (p_c), s	0.0	0.0	0.0	0.0	0.1	2.3	0.0	17.2

Intersection Summary	
HCM 2010 Ctrl Delay	52.3
HCM 2010 LOS	D

Notes

Intersection	
Int Delay, s/veh	2.7

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	2	47	50	51	147	7
Future Vol, veh/h	2	47	50	51	147	7
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	200	0	350	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	50	84	78	64	82	58
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	4	56	64	80	179	12

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	393	185	191	0	-	0
Stage 1	185	-	-	-	-	-
Stage 2	208	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	611	857	1383	-	-	-
Stage 1	847	-	-	-	-	-
Stage 2	827	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	583	857	1383	-	-	-
Mov Cap-2 Maneuver	583	-	-	-	-	-
Stage 1	847	-	-	-	-	-
Stage 2	789	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	9.6	3.4	0
HCM LOS	A		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1383	-	583	857	-	-
HCM Lane V/C Ratio	0.046	-	0.007	0.065	-	-
HCM Control Delay (s)	7.7	-	11.2	9.5	-	-
HCM Lane LOS	A	-	B	A	-	-
HCM 95th %tile Q(veh)	0.1	-	0	0.2	-	-

Intersection	
Int Delay, s/veh	2.4

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	8	36	34	141	108	6
Future Vol, veh/h	8	36	34	141	108	6
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	200	0	350	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	50	75	71	78	93	75
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	16	48	48	181	116	8

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	397	120	124	0	-	0
Stage 1	120	-	-	-	-	-
Stage 2	277	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	608	931	1463	-	-	-
Stage 1	905	-	-	-	-	-
Stage 2	770	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	588	931	1463	-	-	-
Mov Cap-2 Maneuver	588	-	-	-	-	-
Stage 1	905	-	-	-	-	-
Stage 2	745	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	9.7	1.6	0
HCM LOS	A		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1463	-	588	931	-	-
HCM Lane V/C Ratio	0.033	-	0.027	0.052	-	-
HCM Control Delay (s)	7.5	-	11.3	9.1	-	-
HCM Lane LOS	A	-	B	A	-	-
HCM 95th %tile Q(veh)	0.1	-	0.1	0.2	-	-

Intersection	
Int Delay, s/veh	3.4

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	7	86	73	70	188	17
Future Vol, veh/h	7	86	73	70	188	17
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	200	0	350	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	50	84	78	64	82	58
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	14	102	94	109	229	29

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	541	244	259	0	-	0
Stage 1	244	-	-	-	-	-
Stage 2	297	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	502	795	1306	-	-	-
Stage 1	797	-	-	-	-	-
Stage 2	754	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	466	795	1306	-	-	-
Mov Cap-2 Maneuver	466	-	-	-	-	-
Stage 1	797	-	-	-	-	-
Stage 2	700	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	10.5	3.7	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1306	-	466	795	-	-
HCM Lane V/C Ratio	0.072	-	0.03	0.129	-	-
HCM Control Delay (s)	8	-	13	10.2	-	-
HCM Lane LOS	A	-	B	B	-	-
HCM 95th %tile Q(veh)	0.2	-	0.1	0.4	-	-

Intersection	
Int Delay, s/veh	3.8

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	27	74	85	199	161	25
Future Vol, veh/h	27	74	85	199	161	25
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	200	0	350	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	50	75	71	78	93	75
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	54	99	120	255	173	33

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	685	190	206	0	-	0
Stage 1	190	-	-	-	-	-
Stage 2	495	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	414	852	1365	-	-	-
Stage 1	842	-	-	-	-	-
Stage 2	613	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	378	852	1365	-	-	-
Mov Cap-2 Maneuver	378	-	-	-	-	-
Stage 1	842	-	-	-	-	-
Stage 2	559	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	12	2.5	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1365	-	378	852	-	-
HCM Lane V/C Ratio	0.088	-	0.143	0.116	-	-
HCM Control Delay (s)	7.9	-	16.1	9.8	-	-
HCM Lane LOS	A	-	C	A	-	-
HCM 95th %tile Q(veh)	0.3	-	0.5	0.4	-	-

Intersection	
Int Delay, s/veh	3.7

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	29	91	73	72	190	45
Future Vol, veh/h	29	91	73	72	190	45
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	200	0	350	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	50	84	78	64	82	58
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	58	108	94	113	232	78

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	571	271	309	0	-	0
Stage 1	271	-	-	-	-	-
Stage 2	300	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	482	768	1252	-	-	-
Stage 1	775	-	-	-	-	-
Stage 2	752	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	446	768	1252	-	-	-
Mov Cap-2 Maneuver	696	-	-	-	-	-
Stage 1	775	-	-	-	-	-
Stage 2	696	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	10.5	3.7	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1252	-	696	768	-	-
HCM Lane V/C Ratio	0.075	-	0.083	0.141	-	-
HCM Control Delay (s)	8.1	-	10.6	10.5	-	-
HCM Lane LOS	A	-	B	B	-	-
HCM 95th %tile Q(veh)	0.2	-	0.3	0.5	-	-

Intersection	
Int Delay, s/veh	5.2

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	62	81	85	201	163	59
Future Vol, veh/h	62	81	85	201	163	59
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	200	0	350	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	50	75	71	78	93	75
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	124	108	120	258	175	79

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	712	215	254	0	-	0
Stage 1	215	-	-	-	-	-
Stage 2	497	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	399	825	1311	-	-	-
Stage 1	821	-	-	-	-	-
Stage 2	611	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	362	825	1311	-	-	-
Mov Cap-2 Maneuver	362	-	-	-	-	-
Stage 1	821	-	-	-	-	-
Stage 2	555	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	15.3	2.5	0
HCM LOS	C		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1311	-	362	825	-	-
HCM Lane V/C Ratio	0.091	-	0.343	0.131	-	-
HCM Control Delay (s)	8	-	20	10	-	-
HCM Lane LOS	A	-	C	B	-	-
HCM 95th %tile Q(veh)	0.3	-	1.5	0.5	-	-

Intersection	
Int Delay, s/veh	3.5

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	9	129	119	116	322	23
Future Vol, veh/h	9	129	119	116	322	23
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	200	0	350	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	50	84	78	64	82	58
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	18	154	153	181	393	40

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	809	216	432	0	-	0
Stage 1	413	-	-	-	-	-
Stage 2	396	-	-	-	-	-
Critical Hdwy	6.84	6.94	4.14	-	-	-
Critical Hdwy Stg 1	5.84	-	-	-	-	-
Critical Hdwy Stg 2	5.84	-	-	-	-	-
Follow-up Hdwy	3.52	3.32	2.22	-	-	-
Pot Cap-1 Maneuver	318	789	1124	-	-	-
Stage 1	636	-	-	-	-	-
Stage 2	649	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	275	789	1124	-	-	-
Mov Cap-2 Maneuver	275	-	-	-	-	-
Stage 1	636	-	-	-	-	-
Stage 2	561	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	11.6	4	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1124	-	275	789	-	-
HCM Lane V/C Ratio	0.136	-	0.065	0.195	-	-
HCM Control Delay (s)	8.7	-	19	10.7	-	-
HCM Lane LOS	A	-	C	B	-	-
HCM 95th %tile Q(veh)	0.5	-	0.2	0.7	-	-

Intersection	
Int Delay, s/veh	3.9

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	34	107	116	327	259	30
Future Vol, veh/h	34	107	116	327	259	30
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	200	0	350	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	50	75	71	78	93	75
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	68	143	163	419	278	40

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	834	159	318	0	-	0
Stage 1	298	-	-	-	-	-
Stage 2	536	-	-	-	-	-
Critical Hdwy	6.84	6.94	4.14	-	-	-
Critical Hdwy Stg 1	5.84	-	-	-	-	-
Critical Hdwy Stg 2	5.84	-	-	-	-	-
Follow-up Hdwy	3.52	3.32	2.22	-	-	-
Pot Cap-1 Maneuver	307	858	1239	-	-	-
Stage 1	727	-	-	-	-	-
Stage 2	551	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	267	858	1239	-	-	-
Mov Cap-2 Maneuver	267	-	-	-	-	-
Stage 1	727	-	-	-	-	-
Stage 2	479	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	14.2	2.3	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1239	-	267	858	-	-
HCM Lane V/C Ratio	0.132	-	0.255	0.166	-	-
HCM Control Delay (s)	8.3	-	23	10	-	-
HCM Lane LOS	A	-	C	B	-	-
HCM 95th %tile Q(veh)	0.5	-	1	0.6	-	-

Intersection	
Int Delay, s/veh	4.7

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	31	134	119	118	324	51
Future Vol, veh/h	31	134	119	118	324	51
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	200	0	350	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	50	84	78	64	82	58
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	62	160	153	184	395	88

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	929	439	483	0	-	0
Stage 1	439	-	-	-	-	-
Stage 2	490	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	297	618	1080	-	-	-
Stage 1	650	-	-	-	-	-
Stage 2	616	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	255	618	1080	-	-	-
Mov Cap-2 Maneuver	255	-	-	-	-	-
Stage 1	650	-	-	-	-	-
Stage 2	529	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	15.8	4	0
HCM LOS	C		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1080	-	255	618	-	-
HCM Lane V/C Ratio	0.141	-	0.243	0.258	-	-
HCM Control Delay (s)	8.9	-	23.6	12.8	-	-
HCM Lane LOS	A	-	C	B	-	-
HCM 95th %tile Q(veh)	0.5	-	0.9	1	-	-

Intersection	
Int Delay, s/veh	8

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	69	114	116	329	261	64
Future Vol, veh/h	69	114	116	329	261	64
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	200	0	350	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	50	75	71	78	93	75
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	138	152	163	422	281	85

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	1072	323	366	0	-	0
Stage 1	323	-	-	-	-	-
Stage 2	749	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	244	718	1193	-	-	-
Stage 1	734	-	-	-	-	-
Stage 2	467	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	211	718	1193	-	-	-
Mov Cap-2 Maneuver	211	-	-	-	-	-
Stage 1	734	-	-	-	-	-
Stage 2	403	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	29.5	2.4	0
HCM LOS	D		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1193	-	211	718	-	-
HCM Lane V/C Ratio	0.137	-	0.654	0.212	-	-
HCM Control Delay (s)	8.5	-	49.5	11.4	-	-
HCM Lane LOS	A	-	E	B	-	-
HCM 95th %tile Q(veh)	0.5	-	4	0.8	-	-

Intersection	
Int Delay, s/veh	3.4

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	5	2	24	33	0	29	12	135	31	41	199	0
Future Vol, veh/h	5	2	24	33	0	29	12	135	31	41	199	0
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	125	-	-	200	-	-	75	-	0	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	62	50	67	64	25	81	50	89	52	64	75	25
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	8	4	36	52	0	36	24	152	60	64	265	0

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	611	593	265	613	593	152	265	0	0	152	0	0
Stage 1	393	393	-	200	200	-	-	-	-	-	-	-
Stage 2	218	200	-	413	393	-	-	-	-	-	-	-
Critical Hdwy	7.12	6.52	6.22	7.12	6.52	6.22	4.12	-	-	4.12	-	-
Critical Hdwy Stg 1	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Follow-up Hdwy	3.518	4.018	3.318	3.518	4.018	3.318	2.218	-	-	2.218	-	-
Pot Cap-1 Maneuver	406	418	774	405	418	894	1299	-	-	1429	-	-
Stage 1	632	606	-	802	736	-	-	-	-	-	-	-
Stage 2	784	736	-	616	606	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	371	392	774	365	392	894	1299	-	-	1429	-	-
Mov Cap-2 Maneuver	371	392	-	365	392	-	-	-	-	-	-	-
Stage 1	620	579	-	787	722	-	-	-	-	-	-	-
Stage 2	739	722	-	557	579	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	11.2	13.5	0.8	1.5
HCM LOS	B	B		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1299	-	-	371	705	365	894	1429	-	-
HCM Lane V/C Ratio	0.018	-	-	0.022	0.056	0.141	0.04	0.045	-	-
HCM Control Delay (s)	7.8	-	-	14.9	10.4	16.5	9.2	7.6	-	-
HCM Lane LOS	A	-	-	B	B	C	A	A	-	-
HCM 95th %tile Q(veh)	0.1	-	-	0.1	0.2	0.5	0.1	0.1	-	-

Intersection	
Int Delay, s/veh	3

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	1	0	19	27	3	30	33	214	30	19	160	4
Future Vol, veh/h	1	0	19	27	3	30	33	214	30	19	160	4
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	125	-	-	200	-	-	75	-	0	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	25	25	68	61	75	68	69	92	54	79	89	33
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	4	0	28	44	4	44	48	233	56	24	180	12

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	586	562	186	576	568	233	192	0	0	233	0	0
Stage 1	234	234	-	328	328	-	-	-	-	-	-	-
Stage 2	352	328	-	248	240	-	-	-	-	-	-	-
Critical Hdwy	7.12	6.52	6.22	7.12	6.52	6.22	4.12	-	-	4.12	-	-
Critical Hdwy Stg 1	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Follow-up Hdwy	3.518	4.018	3.318	3.518	4.018	3.318	2.218	-	-	2.218	-	-
Pot Cap-1 Maneuver	422	436	856	428	432	806	1381	-	-	1335	-	-
Stage 1	769	711	-	685	647	-	-	-	-	-	-	-
Stage 2	665	647	-	756	707	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	380	413	856	398	409	806	1381	-	-	1335	-	-
Mov Cap-2 Maneuver	380	413	-	398	409	-	-	-	-	-	-	-
Stage 1	742	698	-	661	625	-	-	-	-	-	-	-
Stage 2	603	625	-	718	694	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	10	12.6	1.1	0.9
HCM LOS	B	B		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1381	-	-	380	856	398	746	1335	-	-
HCM Lane V/C Ratio	0.035	-	-	0.011	0.033	0.111	0.065	0.018	-	-
HCM Control Delay (s)	7.7	-	-	14.6	9.3	15.2	10.2	7.7	-	-
HCM Lane LOS	A	-	-	B	A	C	B	A	-	-
HCM 95th %tile Q(veh)	0.1	-	-	0	0.1	0.4	0.2	0.1	-	-

Intersection	
Int Delay, s/veh	4.8

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	6	2	27	67	0	35	13	158	42	46	237	0
Future Vol, veh/h	6	2	27	67	0	35	13	158	42	46	237	0
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	125	-	-	200	-	-	75	-	0	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	62	50	67	64	25	81	50	89	52	64	75	25
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	10	4	40	105	0	43	26	178	81	72	316	0

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	711	690	316	712	690	178	316	0	0	178	0	0
Stage 1	460	460	-	230	230	-	-	-	-	-	-	-
Stage 2	251	230	-	482	460	-	-	-	-	-	-	-
Critical Hdwy	7.12	6.52	6.22	7.12	6.52	6.22	4.12	-	-	4.12	-	-
Critical Hdwy Stg 1	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Follow-up Hdwy	3.518	4.018	3.318	3.518	4.018	3.318	2.218	-	-	2.218	-	-
Pot Cap-1 Maneuver	348	368	724	347	368	865	1244	-	-	1398	-	-
Stage 1	581	566	-	773	714	-	-	-	-	-	-	-
Stage 2	753	714	-	565	566	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	313	342	724	307	342	865	1244	-	-	1398	-	-
Mov Cap-2 Maneuver	313	342	-	307	342	-	-	-	-	-	-	-
Stage 1	569	537	-	757	699	-	-	-	-	-	-	-
Stage 2	700	699	-	502	537	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	12	18.8	0.7	1.4
HCM LOS	B	C		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1244	-	-	313	658	307	865	1398	-	-
HCM Lane V/C Ratio	0.021	-	-	0.031	0.067	0.341	0.05	0.051	-	-
HCM Control Delay (s)	8	-	-	16.9	10.9	22.7	9.4	7.7	-	-
HCM Lane LOS	A	-	-	C	B	C	A	A	-	-
HCM 95th %tile Q(veh)	0.1	-	-	0.1	0.2	1.5	0.2	0.2	-	-

Intersection	
Int Delay, s/veh	3.5

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	1	0	21	47	3	35	37	244	65	24	188	4
Future Vol, veh/h	1	0	21	47	3	35	37	244	65	24	188	4
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	125	-	-	200	-	-	75	-	0	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	25	25	68	61	75	68	69	92	54	79	89	33
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	4	0	31	77	4	51	54	265	120	30	211	12

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	678	650	217	665	656	265	223	0	0	265	0	0
Stage 1	278	278	-	372	372	-	-	-	-	-	-	-
Stage 2	400	372	-	293	284	-	-	-	-	-	-	-
Critical Hdwy	7.12	6.52	6.22	7.12	6.52	6.22	4.12	-	-	4.12	-	-
Critical Hdwy Stg 1	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Follow-up Hdwy	3.518	4.018	3.318	3.518	4.018	3.318	2.218	-	-	2.218	-	-
Pot Cap-1 Maneuver	366	388	823	374	385	774	1346	-	-	1299	-	-
Stage 1	728	680	-	648	619	-	-	-	-	-	-	-
Stage 2	626	619	-	715	676	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	323	364	823	343	361	774	1346	-	-	1299	-	-
Mov Cap-2 Maneuver	323	364	-	343	361	-	-	-	-	-	-	-
Stage 1	699	664	-	622	594	-	-	-	-	-	-	-
Stage 2	557	594	-	672	660	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	10.3	15.2	1	0.9
HCM LOS	B	C		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1346	-	-	323	823	343	715	1299	-	-
HCM Lane V/C Ratio	0.04	-	-	0.012	0.038	0.225	0.078	0.023	-	-
HCM Control Delay (s)	7.8	-	-	16.3	9.5	18.5	10.5	7.8	-	-
HCM Lane LOS	A	-	-	C	A	C	B	A	-	-
HCM 95th %tile Q(veh)	0.1	-	-	0	0.1	0.8	0.3	0.1	-	-

Intersection	
Int Delay, s/veh	5.4

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	6	2	27	77	0	38	13	163	44	50	243	0
Future Vol, veh/h	6	2	27	77	0	38	13	163	44	50	243	0
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	125	-	-	200	-	-	75	-	0	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	62	50	67	64	25	81	50	89	52	64	75	25
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	10	4	40	120	0	47	26	183	85	78	324	0

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	739	715	324	737	715	183	324	0	0	183	0	0
Stage 1	480	480	-	235	235	-	-	-	-	-	-	-
Stage 2	259	235	-	502	480	-	-	-	-	-	-	-
Critical Hdwy	7.12	6.52	6.22	7.12	6.52	6.22	4.12	-	-	4.12	-	-
Critical Hdwy Stg 1	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Follow-up Hdwy	3.518	4.018	3.318	3.518	4.018	3.318	2.218	-	-	2.218	-	-
Pot Cap-1 Maneuver	333	356	717	334	356	859	1236	-	-	1392	-	-
Stage 1	567	554	-	768	710	-	-	-	-	-	-	-
Stage 2	746	710	-	552	554	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	297	329	717	294	329	859	1236	-	-	1392	-	-
Mov Cap-2 Maneuver	297	329	-	294	329	-	-	-	-	-	-	-
Stage 1	555	523	-	752	695	-	-	-	-	-	-	-
Stage 2	690	695	-	488	523	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	12.2	21	0.7	1.5
HCM LOS	B	C		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1236	-	-	297	648	294	859	1392	-	-
HCM Lane V/C Ratio	0.021	-	-	0.033	0.068	0.409	0.055	0.056	-	-
HCM Control Delay (s)	8	-	-	17.5	11	25.5	9.4	7.7	-	-
HCM Lane LOS	A	-	-	C	B	D	A	A	-	-
HCM 95th %tile Q(veh)	0.1	-	-	0.1	0.2	1.9	0.2	0.2	-	-

Intersection	
Int Delay, s/veh	4.2

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	1	0	21	61	3	40	37	251	67	29	195	4
Future Vol, veh/h	1	0	21	61	3	40	37	251	67	29	195	4
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	125	-	-	200	-	-	75	-	0	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	25	25	68	61	75	68	69	92	54	79	89	33
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	4	0	31	100	4	59	54	273	124	37	219	12

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	710	679	225	694	685	273	231	0	0	273	0	0
Stage 1	299	299	-	380	380	-	-	-	-	-	-	-
Stage 2	411	380	-	314	305	-	-	-	-	-	-	-
Critical Hdwy	7.12	6.52	6.22	7.12	6.52	6.22	4.12	-	-	4.12	-	-
Critical Hdwy Stg 1	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Follow-up Hdwy	3.518	4.018	3.318	3.518	4.018	3.318	2.218	-	-	2.218	-	-
Pot Cap-1 Maneuver	348	374	814	357	371	766	1337	-	-	1290	-	-
Stage 1	710	666	-	642	614	-	-	-	-	-	-	-
Stage 2	618	614	-	697	662	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	302	349	814	326	346	766	1337	-	-	1290	-	-
Mov Cap-2 Maneuver	302	349	-	326	346	-	-	-	-	-	-	-
Stage 1	681	647	-	616	589	-	-	-	-	-	-	-
Stage 2	544	589	-	651	643	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	10.5	16.9	0.9	1.1
HCM LOS	B	C		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1337	-	-	302	814	326	711	1290	-	-
HCM Lane V/C Ratio	0.04	-	-	0.013	0.038	0.307	0.088	0.028	-	-
HCM Control Delay (s)	7.8	-	-	17.1	9.6	20.9	10.6	7.9	-	-
HCM Lane LOS	A	-	-	C	A	C	B	A	-	-
HCM 95th %tile Q(veh)	0.1	-	-	0	0.1	1.3	0.3	0.1	-	-

Intersection	
Int Delay, s/veh	31.8

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	10	4	48	96	0	61	24	277	70	83	412	0
Future Vol, veh/h	10	4	48	96	0	61	24	277	70	83	412	0
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	125	-	-	200	-	-	75	-	0	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	62	50	67	64	25	81	50	89	52	64	75	25
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	16	8	72	150	0	75	48	311	135	130	549	0

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	1254	1216	549	1256	1216	311	549	0	0	311	0	0
Stage 1	809	809	-	407	407	-	-	-	-	-	-	-
Stage 2	445	407	-	849	809	-	-	-	-	-	-	-
Critical Hdwy	7.12	6.52	6.22	7.12	6.52	6.22	4.12	-	-	4.12	-	-
Critical Hdwy Stg 1	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Follow-up Hdwy	3.518	4.018	3.318	3.518	4.018	3.318	2.218	-	-	2.218	-	-
Pot Cap-1 Maneuver	149	181	535	~ 148	181	729	1021	-	-	1249	-	-
Stage 1	374	394	-	621	597	-	-	-	-	-	-	-
Stage 2	592	597	-	356	394	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	119	155	535	~ 109	155	729	1021	-	-	1249	-	-
Mov Cap-2 Maneuver	119	155	-	~ 109	155	-	-	-	-	-	-	-
Stage 1	356	353	-	592	569	-	-	-	-	-	-	-
Stage 2	506	569	-	270	353	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	19.4	195.8	0.8	1.6
HCM LOS	C	F		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1021	-	-	119	429	109	729	1249	-	-
HCM Lane V/C Ratio	0.047	-	-	0.136	0.186	1.376	0.103	0.104	-	-
HCM Control Delay (s)	8.7	-	-	39.9	15.3	288.8	10.5	8.2	-	-
HCM Lane LOS	A	-	-	E	C	F	B	A	-	-
HCM 95th %tile Q(veh)	0.1	-	-	0.5	0.7	10.5	0.3	0.3	-	-

Notes			
~: Volume exceeds capacity	$: Delay exceeds 300s	+: Computation Not Defined	*: All major volume in platoon

Intersection	
Int Delay, s/veh	10.4

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	2	0	38	71	6	62	66	433	92	41	329	8
Future Vol, veh/h	2	0	38	71	6	62	66	433	92	41	329	8
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	125	-	-	200	-	-	75	-	0	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	25	25	68	61	75	68	69	92	54	79	89	33
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	8	0	56	116	8	91	96	471	170	52	370	24

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	1198	1148	382	1176	1160	471	394	0	0	471	0	0
Stage 1	486	486	-	662	662	-	-	-	-	-	-	-
Stage 2	712	662	-	514	498	-	-	-	-	-	-	-
Critical Hdwy	7.12	6.52	6.22	7.12	6.52	6.22	4.12	-	-	4.12	-	-
Critical Hdwy Stg 1	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Follow-up Hdwy	3.518	4.018	3.318	3.518	4.018	3.318	2.218	-	-	2.218	-	-
Pot Cap-1 Maneuver	162	199	665	168	195	593	1165	-	-	1091	-	-
Stage 1	563	551	-	451	459	-	-	-	-	-	-	-
Stage 2	423	459	-	543	544	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	119	174	665	139	170	593	1165	-	-	1091	-	-
Mov Cap-2 Maneuver	119	174	-	139	170	-	-	-	-	-	-	-
Stage 1	517	525	-	414	421	-	-	-	-	-	-	-
Stage 2	322	421	-	474	518	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	14.2	60.3	1.1	1
HCM LOS	B	F		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1165	-	-	119	665	139	494	1091	-	-
HCM Lane V/C Ratio	0.082	-	-	0.067	0.084	0.837	0.201	0.048	-	-
HCM Control Delay (s)	8.4	-	-	37.4	10.9	99.7	14.1	8.5	-	-
HCM Lane LOS	A	-	-	E	B	F	B	A	-	-
HCM 95th %tile Q(veh)	0.3	-	-	0.2	0.3	5.3	0.7	0.1	-	-

Intersection	
Int Delay, s/veh	43.6

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	10	4	48	106	0	64	24	282	72	87	418	0
Future Vol, veh/h	10	4	48	106	0	64	24	282	72	87	418	0
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	125	-	-	200	-	-	75	-	0	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	62	50	67	64	25	81	50	89	52	64	75	25
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	16	8	72	166	0	79	48	317	138	136	557	0

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	1281	1242	557	1282	1242	317	557	0	0	317	0	0
Stage 1	829	829	-	413	413	-	-	-	-	-	-	-
Stage 2	452	413	-	869	829	-	-	-	-	-	-	-
Critical Hdwy	7.12	6.52	6.22	7.12	6.52	6.22	4.12	-	-	4.12	-	-
Critical Hdwy Stg 1	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Follow-up Hdwy	3.518	4.018	3.318	3.518	4.018	3.318	2.218	-	-	2.218	-	-
Pot Cap-1 Maneuver	142	175	530	~ 142	175	724	1014	-	-	1243	-	-
Stage 1	365	385	-	616	594	-	-	-	-	-	-	-
Stage 2	587	594	-	347	385	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	112	148	530	~ 104	148	724	1014	-	-	1243	-	-
Mov Cap-2 Maneuver	112	148	-	~ 104	148	-	-	-	-	-	-	-
Stage 1	348	343	-	587	566	-	-	-	-	-	-	-
Stage 2	498	566	-	261	343	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	20	260.2	0.8	1.6
HCM LOS	C	F		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1014	-	-	112	421	104	724	1243	-	-
HCM Lane V/C Ratio	0.047	-	-	0.144	0.189	1.593	0.109	0.109	-	-
HCM Control Delay (s)	8.7	-	-	42.5	15.5	$ 379.3	10.6	8.3	-	-
HCM Lane LOS	A	-	-	E	C	F	B	A	-	-
HCM 95th %tile Q(veh)	0.1	-	-	0.5	0.7	12.6	0.4	0.4	-	-

Notes			
~: Volume exceeds capacity	$: Delay exceeds 300s	+: Computation Not Defined	*: All major volume in platoon

Intersection	
Int Delay, s/veh	17.1

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	2	0	38	85	6	67	66	440	94	46	336	8
Future Vol, veh/h	2	0	38	85	6	67	66	440	94	46	336	8
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	125	-	-	200	-	-	75	-	0	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	25	25	68	61	75	68	69	92	54	79	89	33
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	8	0	56	139	8	99	96	478	174	58	378	24

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	1229	1176	390	1204	1188	478	402	0	0	478	0	0
Stage 1	506	506	-	670	670	-	-	-	-	-	-	-
Stage 2	723	670	-	534	518	-	-	-	-	-	-	-
Critical Hdwy	7.12	6.52	6.22	7.12	6.52	6.22	4.12	-	-	4.12	-	-
Critical Hdwy Stg 1	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.12	5.52	-	6.12	5.52	-	-	-	-	-	-	-
Follow-up Hdwy	3.518	4.018	3.318	3.518	4.018	3.318	2.218	-	-	2.218	-	-
Pot Cap-1 Maneuver	155	191	658	161	188	587	1157	-	-	1084	-	-
Stage 1	549	540	-	446	455	-	-	-	-	-	-	-
Stage 2	417	455	-	530	533	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	112	166	658	~ 132	163	587	1157	-	-	1084	-	-
Mov Cap-2 Maneuver	112	166	-	~ 132	163	-	-	-	-	-	-	-
Stage 1	503	511	-	409	417	-	-	-	-	-	-	-
Stage 2	312	417	-	459	504	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	14.6	96.5	1.1	1.1
HCM LOS	B	F		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1157	-	-	112	658	132	491	1084	-	-
HCM Lane V/C Ratio	0.083	-	-	0.071	0.085	1.056	0.217	0.054	-	-
HCM Control Delay (s)	8.4	-	-	39.6	11	159.3	14.4	8.5	-	-
HCM Lane LOS	A	-	-	E	B	F	B	A	-	-
HCM 95th %tile Q(veh)	0.3	-	-	0.2	0.3	7.7	0.8	0.2	-	-

Notes			
~: Volume exceeds capacity	$: Delay exceeds 300s	+: Computation Not Defined	*: All major volume in platoon

Intersection	
Int Delay, s/veh	2.5

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	25	0	45	0	0	0	53	152	0	0	225	30
Future Vol, veh/h	25	0	45	0	0	0	53	152	0	0	225	30
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	200	-	-	-	-	50	200	-	-	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	62	25	75	92	25	92	74	81	92	79	68	92
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	40	0	60	0	0	0	72	188	0	0	331	33

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	584	678	182	496	694	94	363	0	0	188	0	0
Stage 1	347	347	-	331	331	-	-	-	-	-	-	-
Stage 2	237	331	-	165	363	-	-	-	-	-	-	-
Critical Hdwy	7.54	6.54	6.94	7.54	6.54	6.94	4.14	-	-	4.14	-	-
Critical Hdwy Stg 1	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Follow-up Hdwy	3.52	4.02	3.32	3.52	4.02	3.32	2.22	-	-	2.22	-	-
Pot Cap-1 Maneuver	395	373	829	457	365	944	1192	-	-	1384	-	-
Stage 1	642	633	-	656	644	-	-	-	-	-	-	-
Stage 2	745	644	-	821	623	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	377	350	829	404	343	944	1192	-	-	1384	-	-
Mov Cap-2 Maneuver	377	350	-	404	343	-	-	-	-	-	-	-
Stage 1	603	633	-	616	605	-	-	-	-	-	-	-
Stage 2	700	605	-	762	623	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	12.1	0	2.3	0
HCM LOS	B	A		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1192	-	-	377	829	-	-	1384	-	-
HCM Lane V/C Ratio	0.06	-	-	0.107	0.072	-	-	-	-	-
HCM Control Delay (s)	8.2	-	-	15.7	9.7	0	0	0	-	-
HCM Lane LOS	A	-	-	C	A	A	A	A	-	-
HCM 95th %tile Q(veh)	0.2	-	-	0.4	0.2	-	-	0	-	-

Intersection	
Int Delay, s/veh	4.3

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	67	0	73	0	0	0	72	210	0	0	159	50
Future Vol, veh/h	67	0	73	0	0	0	72	210	0	0	159	50
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	200	-	-	-	-	50	200	-	-	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	73	25	87	92	25	92	64	94	92	85	73	92
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	92	0	84	0	0	0	113	223	0	0	218	54

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	582	693	136	557	720	112	272	0	0	223	0	0
Stage 1	245	245	-	448	448	-	-	-	-	-	-	-
Stage 2	337	448	-	109	272	-	-	-	-	-	-	-
Critical Hdwy	7.54	6.54	6.94	7.54	6.54	6.94	4.14	-	-	4.14	-	-
Critical Hdwy Stg 1	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Follow-up Hdwy	3.52	4.02	3.32	3.52	4.02	3.32	2.22	-	-	2.22	-	-
Pot Cap-1 Maneuver	396	365	888	413	352	920	1288	-	-	1343	-	-
Stage 1	737	702	-	560	571	-	-	-	-	-	-	-
Stage 2	651	571	-	885	683	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	369	333	888	349	321	920	1288	-	-	1343	-	-
Mov Cap-2 Maneuver	369	333	-	349	321	-	-	-	-	-	-	-
Stage 1	672	702	-	511	521	-	-	-	-	-	-	-
Stage 2	594	521	-	801	683	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	13.9	0	2.7	0
HCM LOS	B	A		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1288	-	-	369	888	-	-	1343	-	-
HCM Lane V/C Ratio	0.087	-	-	0.249	0.094	-	-	-	-	-
HCM Control Delay (s)	8.1	-	-	18	9.5	0	0	0	-	-
HCM Lane LOS	A	-	-	C	A	A	A	A	-	-
HCM 95th %tile Q(veh)	0.3	-	-	1	0.3	-	-	0	-	-

Intersection	
Int Delay, s/veh	3.9

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	28	0	50	62	0	0	59	185	0	16	272	33
Future Vol, veh/h	28	0	50	62	0	0	59	185	0	16	272	33
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	200	-	-	-	-	50	200	-	-	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	62	25	75	92	25	92	74	81	92	79	68	92
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	45	0	67	67	0	0	80	228	0	20	400	36

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	732	846	218	629	864	114	436	0	0	228	0	0
Stage 1	458	458	-	388	388	-	-	-	-	-	-	-
Stage 2	274	388	-	241	476	-	-	-	-	-	-	-
Critical Hdwy	7.54	6.54	6.94	7.54	6.54	6.94	4.14	-	-	4.14	-	-
Critical Hdwy Stg 1	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Follow-up Hdwy	3.52	4.02	3.32	3.52	4.02	3.32	2.22	-	-	2.22	-	-
Pot Cap-1 Maneuver	309	298	786	367	291	917	1120	-	-	1337	-	-
Stage 1	552	565	-	607	607	-	-	-	-	-	-	-
Stage 2	709	607	-	741	555	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	289	273	786	314	266	917	1120	-	-	1337	-	-
Mov Cap-2 Maneuver	289	273	-	314	266	-	-	-	-	-	-	-
Stage 1	513	557	-	564	564	-	-	-	-	-	-	-
Stage 2	658	564	-	668	547	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	14	19.6	2.2	0.3
HCM LOS	B	C		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1120	-	-	289	786	314	-	1337	-	-
HCM Lane V/C Ratio	0.071	-	-	0.156	0.085	0.215	-	0.015	-	-
HCM Control Delay (s)	8.5	-	-	19.8	10	19.6	0	7.7	-	-
HCM Lane LOS	A	-	-	C	B	C	A	A	-	-
HCM 95th %tile Q(veh)	0.2	-	-	0.5	0.3	0.8	-	0	-	-

Intersection	
Int Delay, s/veh	5.6

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	74	0	81	49	0	0	80	272	0	11	190	55
Future Vol, veh/h	74	0	81	49	0	0	80	272	0	11	190	55
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	200	-	-	-	-	50	200	-	-	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	73	25	87	92	25	92	64	94	92	85	73	92
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	101	0	93	53	0	0	125	289	0	13	260	60

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	711	855	160	695	885	145	320	0	0	289	0	0
Stage 1	316	316	-	539	539	-	-	-	-	-	-	-
Stage 2	395	539	-	156	346	-	-	-	-	-	-	-
Critical Hdwy	7.54	6.54	6.94	7.54	6.54	6.94	4.14	-	-	4.14	-	-
Critical Hdwy Stg 1	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Follow-up Hdwy	3.52	4.02	3.32	3.52	4.02	3.32	2.22	-	-	2.22	-	-
Pot Cap-1 Maneuver	320	294	857	329	282	876	1237	-	-	1270	-	-
Stage 1	670	654	-	494	520	-	-	-	-	-	-	-
Stage 2	602	520	-	831	634	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	293	262	857	268	251	876	1237	-	-	1270	-	-
Mov Cap-2 Maneuver	293	262	-	268	251	-	-	-	-	-	-	-
Stage 1	602	647	-	444	467	-	-	-	-	-	-	-
Stage 2	541	467	-	733	628	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	16.9	21.7	2.5	0.3
HCM LOS	C	C		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1237	-	-	293	857	268	-	1270	-	-
HCM Lane V/C Ratio	0.101	-	-	0.346	0.109	0.199	-	0.01	-	-
HCM Control Delay (s)	8.2	-	-	23.6	9.7	21.7	0	7.9	-	-
HCM Lane LOS	A	-	-	C	A	C	A	A	-	-
HCM 95th %tile Q(veh)	0.3	-	-	1.5	0.4	0.7	-	0	-	-

Intersection	
Int Delay, s/veh	13.8

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	28	0	50	193	0	12	59	179	65	57	247	33
Future Vol, veh/h	28	0	50	193	0	12	59	179	65	57	247	33
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	200	-	-	-	-	50	200	-	-	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	62	25	75	92	25	92	74	81	92	79	68	92
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	45	0	67	210	0	13	80	221	71	72	363	36

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	795	976	200	742	959	146	399	0	0	292	0	0
Stage 1	525	525	-	416	416	-	-	-	-	-	-	-
Stage 2	270	451	-	326	543	-	-	-	-	-	-	-
Critical Hdwy	7.54	6.54	6.94	7.54	6.54	6.94	4.14	-	-	4.14	-	-
Critical Hdwy Stg 1	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Follow-up Hdwy	3.52	4.02	3.32	3.52	4.02	3.32	2.22	-	-	2.22	-	-
Pot Cap-1 Maneuver	278	250	808	304	256	875	1156	-	-	1267	-	-
Stage 1	504	528	-	585	590	-	-	-	-	-	-	-
Stage 2	713	569	-	661	518	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	248	219	808	253	225	875	1156	-	-	1267	-	-
Mov Cap-2 Maneuver	248	219	-	253	225	-	-	-	-	-	-	-
Stage 1	469	498	-	545	549	-	-	-	-	-	-	-
Stage 2	654	530	-	572	489	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	15.1	60	1.8	1.2
HCM LOS	C	F		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1156	-	-	248	808	253	875	1267	-	-
HCM Lane V/C Ratio	0.069	-	-	0.182	0.083	0.829	0.015	0.057	-	-
HCM Control Delay (s)	8.3	-	-	22.7	9.9	63.2	9.2	8	-	-
HCM Lane LOS	A	-	-	C	A	F	A	A	-	-
HCM 95th %tile Q(veh)	0.2	-	-	0.7	0.3	6.6	0	0.2	-	-

Intersection	
Int Delay, s/veh	29.7

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	74	0	81	232	0	32	80	248	91	38	185	55
Future Vol, veh/h	74	0	81	232	0	32	80	248	91	38	185	55
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	200	-	-	-	-	50	200	-	-	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	73	25	87	92	25	92	64	94	92	85	73	92
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	101	0	93	252	0	35	125	264	99	45	253	60

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	755	986	157	779	966	181	313	0	0	363	0	0
Stage 1	373	373	-	563	563	-	-	-	-	-	-	-
Stage 2	382	613	-	216	403	-	-	-	-	-	-	-
Critical Hdwy	7.54	6.54	6.94	7.54	6.54	6.94	4.14	-	-	4.14	-	-
Critical Hdwy Stg 1	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Follow-up Hdwy	3.52	4.02	3.32	3.52	4.02	3.32	2.22	-	-	2.22	-	-
Pot Cap-1 Maneuver	298	246	861	286	253	831	1244	-	-	1192	-	-
Stage 1	620	617	-	478	507	-	-	-	-	-	-	-
Stage 2	612	481	-	766	598	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	256	213	861	~ 229	219	831	1244	-	-	1192	-	-
Mov Cap-2 Maneuver	256	213	-	~ 229	219	-	-	-	-	-	-	-
Stage 1	558	594	-	430	456	-	-	-	-	-	-	-
Stage 2	527	433	-	657	575	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	19.2	119.4	2.1	1
HCM LOS	C	F		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1244	-	-	256	861	229	831	1192	-	-
HCM Lane V/C Ratio	0.1	-	-	0.396	0.108	1.101	0.042	0.038	-	-
HCM Control Delay (s)	8.2	-	-	28	9.7	134.6	9.5	8.1	-	-
HCM Lane LOS	A	-	-	D	A	F	A	A	-	-
HCM 95th %tile Q(veh)	0.3	-	-	1.8	0.4	11.3	0.1	0.1	-	-

Notes			
~: Volume exceeds capacity	$: Delay exceeds 300s	+: Computation Not Defined	*: All major volume in platoon

Intersection	
Int Delay, s/veh	9.5

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	50	0	90	62	0	0	105	318	0	16	471	60
Future Vol, veh/h	50	0	90	62	0	0	105	318	0	16	471	60
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	200	-	-	-	-	50	200	-	-	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	62	25	75	92	25	92	74	81	92	79	68	92
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	81	0	120	67	0	0	142	393	0	20	693	65

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	1246	1442	379	1063	1474	196	758	0	0	393	0	0
Stage 1	766	766	-	676	676	-	-	-	-	-	-	-
Stage 2	480	676	-	387	798	-	-	-	-	-	-	-
Critical Hdwy	7.54	6.54	6.94	7.54	6.54	6.94	4.14	-	-	4.14	-	-
Critical Hdwy Stg 1	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Follow-up Hdwy	3.52	4.02	3.32	3.52	4.02	3.32	2.22	-	-	2.22	-	-
Pot Cap-1 Maneuver	130	131	619	177	125	812	849	-	-	1162	-	-
Stage 1	361	410	-	409	451	-	-	-	-	-	-	-
Stage 2	536	451	-	608	396	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	112	107	619	123	102	812	849	-	-	1162	-	-
Mov Cap-2 Maneuver	112	107	-	123	102	-	-	-	-	-	-	-
Stage 1	301	403	-	341	376	-	-	-	-	-	-	-
Stage 2	446	376	-	482	389	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	45.1	65.1	2.7	0.2
HCM LOS	E	F		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	849	-	-	112	619	123	-	1162	-	-
HCM Lane V/C Ratio	0.167	-	-	0.72	0.194	0.548	-	0.017	-	-
HCM Control Delay (s)	10.1	-	-	94.1	12.2	65.1	0	8.2	-	-
HCM Lane LOS	B	-	-	F	B	F	A	A	-	-
HCM 95th %tile Q(veh)	0.6	-	-	3.9	0.7	2.6	-	0.1	-	-

Intersection	
Int Delay, s/veh	47.1

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Vol, veh/h	133	0	145	49	0	0	143	457	0	11	330	99
Future Vol, veh/h	133	0	145	49	0	0	143	457	0	11	330	99
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop	Free	Free	Free	Free	Free	Free
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	200	-	-	-	-	50	200	-	-	75	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	73	25	87	92	25	92	64	94	92	85	73	92
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	182	0	167	53	0	0	223	486	0	13	452	108

Major/Minor	Minor2			Minor1			Major1			Major2		
Conflicting Flow All	1222	1465	280	1185	1519	243	560	0	0	486	0	0
Stage 1	532	532	-	933	933	-	-	-	-	-	-	-
Stage 2	690	933	-	252	586	-	-	-	-	-	-	-
Critical Hdwy	7.54	6.54	6.94	7.54	6.54	6.94	4.14	-	-	4.14	-	-
Critical Hdwy Stg 1	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Critical Hdwy Stg 2	6.54	5.54	-	6.54	5.54	-	-	-	-	-	-	-
Follow-up Hdwy	3.52	4.02	3.32	3.52	4.02	3.32	2.22	-	-	2.22	-	-
Pot Cap-1 Maneuver	~ 136	127	717	144	118	758	1007	-	-	1073	-	-
Stage 1	499	524	-	286	343	-	-	-	-	-	-	-
Stage 2	401	343	-	730	495	-	-	-	-	-	-	-
Platoon blocked, %								-	-		-	-
Mov Cap-1 Maneuver	~ 112	98	717	91	91	758	1007	-	-	1073	-	-
Mov Cap-2 Maneuver	~ 112	98	-	91	91	-	-	-	-	-	-	-
Stage 1	388	518	-	223	267	-	-	-	-	-	-	-
Stage 2	312	267	-	554	489	-	-	-	-	-	-	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	207.3	89.5	3	0.2
HCM LOS	F	F		

Minor Lane/Major Mvmt	NBL	NBT	NBR	EBLn1	EBLn2	WBLn1	WBLn2	SBL	SBT	SBR
Capacity (veh/h)	1007	-	-	112	717	91	-	1073	-	-
HCM Lane V/C Ratio	0.222	-	-	1.627	0.232	0.585	-	0.012	-	-
HCM Control Delay (s)	9.6	-		$ 386.5	11.5	89.5	0	8.4	-	-
HCM Lane LOS	A	-	-	F	B	F	A	A	-	-
HCM 95th %tile Q(veh)	0.8	-	-	13.7	0.9	2.7	-	0	-	-

Notes			
~: Volume exceeds capacity	$: Delay exceeds 300s	+: Computation Not Defined	*: All major volume in platoon

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	50	0	90	193	0	12	105	312	65	57	446	60
Future Volume (veh/h)	50	0	90	193	0	12	105	312	65	57	446	60
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1900	1863	1863	1900
Adj Flow Rate, veh/h	81	0	120	210	0	13	142	385	71	72	656	65
Adj No. of Lanes	1	1	0	1	1	1	1	2	0	1	2	0
Peak Hour Factor	0.62	0.25	0.75	0.92	0.25	0.92	0.74	0.81	0.92	0.79	0.68	0.92
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	419	0	401	309	472	401	507	1809	331	632	1908	189
Arrive On Green	0.25	0.00	0.25	0.25	0.00	0.25	0.05	0.61	0.61	0.03	0.59	0.59
Sat Flow, veh/h	1395	0	1583	1266	1863	1583	1774	2989	547	1774	3254	322
Grp Volume(v), veh/h	81	0	120	210	0	13	142	227	229	72	356	365
Grp Sat Flow(s),veh/h/ln	1395	0	1583	1266	1863	1583	1774	1770	1766	1774	1770	1806
Q Serve(g_s), s	5.1	0.0	6.7	17.7	0.0	0.7	3.5	6.4	6.5	1.8	11.5	11.5
Cycle Q Clear(g_c), s	5.1	0.0	6.7	24.4	0.0	0.7	3.5	6.4	6.5	1.8	11.5	11.5
Prop In Lane	1.00		1.00	1.00		1.00	1.00		0.31	1.00		0.18
Lane Grp Cap(c), veh/h	419	0	401	309	472	401	507	1071	1069	632	1038	1059
V/C Ratio(X)	0.19	0.00	0.30	0.68	0.00	0.03	0.28	0.21	0.21	0.11	0.34	0.34
Avail Cap(c_a), veh/h	573	0	576	448	677	576	659	1071	1069	687	1038	1059
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	32.6	0.0	33.2	43.0	0.0	30.9	8.6	9.8	9.9	8.4	11.8	11.8
Incr Delay (d2), s/veh	0.2	0.0	0.4	2.6	0.0	0.0	0.3	0.5	0.5	0.1	0.9	0.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.0	0.0	3.0	6.4	0.0	0.3	1.7	3.2	3.3	0.9	5.8	5.9
LnGrp Delay(d),s/veh	32.8	0.0	33.6	45.7	0.0	30.9	8.9	10.3	10.3	8.5	12.7	12.7
LnGrp LOS	C		C	D		C	A	B	B	A	B	B
Approach Vol, veh/h		201			223			598			793	
Approach Delay, s/veh		33.3			44.8			10.0			12.3	
Approach LOS		C			D			A			B	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	7.6	70.6		31.9	9.6	68.5		31.9
Change Period (Y+Rc), s	4.0	4.0		4.0	4.0	4.0		4.0
Max Green Setting (Gmax), s	7.0	51.0		40.0	15.0	43.0		40.0
Max Q Clear Time (g_c+I1), s	3.8	8.5		8.7	5.5	13.5		26.4
Green Ext Time (p_c), s	0.0	8.8		1.8	0.2	8.2		1.5

Intersection Summary			
HCM 2010 Ctrl Delay			17.8
HCM 2010 LOS			B

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	133	0	145	232	0	32	143	433	91	38	325	99
Future Volume (veh/h)	133	0	145	232	0	32	143	433	91	38	325	99
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1900	1863	1863	1900
Adj Flow Rate, veh/h	182	0	167	252	0	35	223	461	99	45	445	108
Adj No. of Lanes	1	1	0	1	1	1	1	2	0	1	2	0
Peak Hour Factor	0.73	0.25	0.87	0.92	0.25	0.92	0.64	0.94	0.92	0.85	0.73	0.92
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	502	0	505	355	594	505	546	1582	338	501	1388	334
Arrive On Green	0.32	0.00	0.32	0.32	0.00	0.32	0.08	0.54	0.54	0.03	0.49	0.49
Sat Flow, veh/h	1368	0	1583	1214	1863	1583	1774	2904	620	1774	2830	682
Grp Volume(v), veh/h	182	0	167	252	0	35	223	280	280	45	277	276
Grp Sat Flow(s),veh/h/ln	1368	0	1583	1214	1863	1583	1774	1770	1753	1774	1770	1742
Q Serve(g_s), s	11.5	0.0	8.8	22.0	0.0	1.7	6.5	9.4	9.5	1.4	10.4	10.5
Cycle Q Clear(g_c), s	11.5	0.0	8.8	30.8	0.0	1.7	6.5	9.4	9.5	1.4	10.4	10.5
Prop In Lane	1.00		1.00	1.00		1.00	1.00		0.35	1.00		0.39
Lane Grp Cap(c), veh/h	502	0	505	355	594	505	546	964	955	501	868	855
V/C Ratio(X)	0.36	0.00	0.33	0.71	0.00	0.07	0.41	0.29	0.29	0.09	0.32	0.32
Avail Cap(c_a), veh/h	637	0	662	475	779	662	740	964	955	518	868	855
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	29.4	0.0	28.5	40.2	0.0	26.1	11.5	13.5	13.6	13.2	16.9	17.0
Incr Delay (d2), s/veh	0.4	0.0	0.4	3.2	0.0	0.1	0.5	0.8	0.8	0.1	1.0	1.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	4.4	0.0	3.9	7.7	0.0	0.7	3.2	4.8	4.8	0.7	5.3	5.3
LnGrp Delay(d),s/veh	29.9	0.0	28.9	43.4	0.0	26.1	12.0	14.3	14.3	13.3	17.9	18.0
LnGrp LOS	C		C	D		C	B	B	B	B	B	B
Approach Vol, veh/h		349			287			783			598	
Approach Delay, s/veh		29.4			41.3			13.7			17.6	
Approach LOS		C			D			B			B	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	7.0	63.9		39.1	13.0	58.0		39.1
Change Period (Y+Rc), s	4.0	4.0		4.0	4.0	4.0		4.0
Max Green Setting (Gmax), s	4.0	48.0		46.0	21.0	31.0		46.0
Max Q Clear Time (g_c+I1), s	3.4	11.5		13.5	8.5	12.5		32.8
Green Ext Time (p_c), s	0.0	8.0		2.8	0.5	6.5		2.3

Intersection Summary	
HCM 2010 Ctrl Delay	21.5
HCM 2010 LOS	C

Intersection	
Int Delay, s/veh	0.8

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Traffic Vol, veh/h	153	1003	1056	236	0	160
Future Vol, veh/h	153	1003	1056	236	0	160
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	Free
Storage Length	100	-	-	0	-	0
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	166	1090	1148	257	0	174

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	1148	0	-	0	-	-
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-
Critical Hdwy	4.12	-	-	-	-	-
Critical Hdwy Stg 1	-	-	-	-	-	-
Critical Hdwy Stg 2	-	-	-	-	-	-
Follow-up Hdwy	2.218	-	-	-	-	-
Pot Cap-1 Maneuver	609	-	-	-	0	0
Stage 1	-	-	-	-	0	0
Stage 2	-	-	-	-	0	0
Platoon blocked, %		-	-	-	-	
Mov Cap-1 Maneuver	609	-	-	-	-	-
Mov Cap-2 Maneuver	-	-	-	-	-	-
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-

Approach	EB	WB	SB
HCM Control Delay, s	1.7	0	0
HCM LOS			A

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	609	-	-	-	-
HCM Lane V/C Ratio	0.273	-	-	-	-
HCM Control Delay (s)	13.1	-	-	-	0
HCM Lane LOS	B	-	-	-	A
HCM 95th %tile Q(veh)	1.1	-	-	-	-

Intersection	
Int Delay, s/veh	1.4

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Traffic Vol, veh/h	276	1448	969	298	0	319
Future Vol, veh/h	276	1448	969	298	0	319
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	Free
Storage Length	100	-	-	0	-	0
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	300	1574	1053	324	0	347

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	1053	0	-	0	-	-
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-
Critical Hdwy	4.12	-	-	-	-	-
Critical Hdwy Stg 1	-	-	-	-	-	-
Critical Hdwy Stg 2	-	-	-	-	-	-
Follow-up Hdwy	2.218	-	-	-	-	-
Pot Cap-1 Maneuver	661	-	-	-	0	0
Stage 1	-	-	-	-	0	0
Stage 2	-	-	-	-	0	0
Platoon blocked, %		-	-	-	-	
Mov Cap-1 Maneuver	661	-	-	-	-	-
Mov Cap-2 Maneuver	-	-	-	-	-	-
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-

Approach	EB	WB	SB
HCM Control Delay, s	2.4	0	0
HCM LOS			A

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	661	-	-	-	-
HCM Lane V/C Ratio	0.454	-	-	-	-
HCM Control Delay (s)	14.9	-	-	-	0
HCM Lane LOS	B	-	-	-	A
HCM 95th %tile Q(veh)	2.4	-	-	-	-

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	153	1808	1922	236	0	160
Future Volume (Veh/h)	153	1808	1922	236	0	160
Sign Control		Free	Free		Stop	
Grade		0%	0%		0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	166	1965	2089	257	0	174
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type		None	None			
Median storage veh)						
Upstream signal (ft)			1072			
pX, platoon unblocked	0.64				0.64	0.64
vC, conflicting volume	2346				3076	696
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol	1152				2287	0
tC, single (s)	4.1				6.8	6.9
tC, 2 stage (s)						
tF (s)	2.2				3.5	3.3
p0 queue free %	57				100	75
cM capacity (veh/h)	387				12	698

Direction, Lane #	EB 1	EB 2	EB 3	EB 4	WB 1	WB 2	WB 3	WB 4	SB 1
Volume Total	166	655	655	655	696	696	696	257	174
Volume Left	166	0	0	0	0	0	0	0	0
Volume Right	0	0	0	0	0	0	0	257	174
cSH	387	1700	1700	1700	1700	1700	1700	1700	698
Volume to Capacity	0.43	0.39	0.39	0.39	0.41	0.41	0.41	0.15	0.25
Queue Length 95th (ft)	52	0	0	0	0	0	0	0	25
Control Delay (s)	21.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	11.9
Lane LOS	C								B
Approach Delay (s)	1.6				0.0				11.9
Approach LOS									B

Intersection Summary			
Average Delay	1.2		
Intersection Capacity Utilization	53.7%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	276	2447	1756	298	0	319
Future Volume (Veh/h)	276	2447	1756	298	0	319
Sign Control		Free	Free		Stop	
Grade		0%	0%		0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	300	2660	1909	324	0	347
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type		None	None			
Median storage veh)						
Upstream signal (ft)			1032			
pX, platoon unblocked	0.68				0.68	0.68
vC, conflicting volume	2233				3396	636
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol	1156				2871	0
tC, single (s)	4.1				6.8	6.9
tC, 2 stage (s)						
tF (s)	2.2				3.5	3.3
p0 queue free %	26				100	53
cM capacity (veh/h)	407				2	735

Direction, Lane #	EB 1	EB 2	EB 3	EB 4	WB 1	WB 2	WB 3	WB 4	SB 1
Volume Total	300	887	887	887	636	636	636	324	347
Volume Left	300	0	0	0	0	0	0	0	0
Volume Right	0	0	0	0	0	0	0	324	347
cSH	407	1700	1700	1700	1700	1700	1700	1700	735
Volume to Capacity	0.74	0.52	0.52	0.52	0.37	0.37	0.37	0.19	0.47
Queue Length 95th (ft)	146	0	0	0	0	0	0	0	64
Control Delay (s)	34.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.2
Lane LOS	D								B
Approach Delay (s)	3.5				0.0				14.2
Approach LOS									B

Intersection Summary			
Average Delay	2.8		
Intersection Capacity Utilization	60.3%	ICU Level of Service	B
Analysis Period (min)	15		

Intersection	
Int Delay, s/veh	2.4

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Traffic Vol, veh/h	98	8	35	87	11	29
Future Vol, veh/h	98	8	35	87	11	29
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	100	-	0	0
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	107	9	38	95	12	32

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	115	0	282	111
Stage 1	-	-	-	-	111	-
Stage 2	-	-	-	-	171	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1474	-	708	942
Stage 1	-	-	-	-	914	-
Stage 2	-	-	-	-	859	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1474	-	690	942
Mov Cap-2 Maneuver	-	-	-	-	711	-
Stage 1	-	-	-	-	914	-
Stage 2	-	-	-	-	837	-

Approach	EB	WB	NB
HCM Control Delay, s	0	2.2	9.3
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	NBLn2	EBT	EBR	WBL	WBT
Capacity (veh/h)	711	942	-	-	1474	-
HCM Lane V/C Ratio	0.017	0.033	-	-	0.026	-
HCM Control Delay (s)	10.2	9	-	-	7.5	-
HCM Lane LOS	B	A	-	-	A	-
HCM 95th %tile Q(veh)	0.1	0.1	-	-	0.1	-

Intersection	
Int Delay, s/veh	3.3

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Traffic Vol, veh/h	100	13	51	109	23	59
Future Vol, veh/h	100	13	51	109	23	59
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	100	-	0	0
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	109	14	55	118	25	64

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	123	0	345	116
Stage 1	-	-	-	-	116	-
Stage 2	-	-	-	-	229	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1464	-	652	936
Stage 1	-	-	-	-	909	-
Stage 2	-	-	-	-	809	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1464	-	628	936
Mov Cap-2 Maneuver	-	-	-	-	663	-
Stage 1	-	-	-	-	909	-
Stage 2	-	-	-	-	779	-

Approach	EB	WB	NB
HCM Control Delay, s	0	2.4	9.5
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	NBLn2	EBT	EBR	WBL	WBT
Capacity (veh/h)	663	936	-	-	1464	-
HCM Lane V/C Ratio	0.038	0.069	-	-	0.038	-
HCM Control Delay (s)	10.6	9.1	-	-	7.6	-
HCM Lane LOS	B	A	-	-	A	-
HCM 95th %tile Q(veh)	0.1	0.2	-	-	0.1	-

Intersection	
Int Delay, s/veh	1.8

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Traffic Vol, veh/h	143	8	35	139	11	29
Future Vol, veh/h	143	8	35	139	11	29
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	100	-	0	0
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	155	9	38	151	12	32

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	164	0	387	160
Stage 1	-	-	-	-	160	-
Stage 2	-	-	-	-	227	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1414	-	616	885
Stage 1	-	-	-	-	869	-
Stage 2	-	-	-	-	811	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1414	-	599	885
Mov Cap-2 Maneuver	-	-	-	-	649	-
Stage 1	-	-	-	-	869	-
Stage 2	-	-	-	-	789	-

Approach	EB	WB	NB
HCM Control Delay, s	0	1.5	9.6
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	NBLn2	EBT	EBR	WBL	WBT
Capacity (veh/h)	649	885	-	-	1414	-
HCM Lane V/C Ratio	0.018	0.036	-	-	0.027	-
HCM Control Delay (s)	10.7	9.2	-	-	7.6	-
HCM Lane LOS	B	A	-	-	A	-
HCM 95th %tile Q(veh)	0.1	0.1	-	-	0.1	-

Intersection	
Int Delay, s/veh	2.8

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Traffic Vol, veh/h	141	13	51	145	23	59
Future Vol, veh/h	141	13	51	145	23	59
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	100	-	0	0
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	153	14	55	158	25	64

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	167	0	428	160
Stage 1	-	-	-	-	160	-
Stage 2	-	-	-	-	268	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1411	-	584	885
Stage 1	-	-	-	-	869	-
Stage 2	-	-	-	-	777	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1411	-	561	885
Mov Cap-2 Maneuver	-	-	-	-	618	-
Stage 1	-	-	-	-	869	-
Stage 2	-	-	-	-	747	-

Approach	EB	WB	NB
HCM Control Delay, s	0	2	9.9
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	NBLn2	EBT	EBR	WBL	WBT
Capacity (veh/h)	618	885	-	-	1411	-
HCM Lane V/C Ratio	0.04	0.072	-	-	0.039	-
HCM Control Delay (s)	11.1	9.4	-	-	7.7	-
HCM Lane LOS	B	A	-	-	A	-
HCM 95th %tile Q(veh)	0.1	0.2	-	-	0.1	-

Intersection	
Int Delay, s/veh	0.5

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Traffic Vol, veh/h	95	4	4	89	4	3
Future Vol, veh/h	95	4	4	89	4	3
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	100	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	103	4	4	97	4	3

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	108	0	210	105
Stage 1	-	-	-	-	105	-
Stage 2	-	-	-	-	105	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1483	-	778	949
Stage 1	-	-	-	-	919	-
Stage 2	-	-	-	-	919	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1483	-	776	949
Mov Cap-2 Maneuver	-	-	-	-	773	-
Stage 1	-	-	-	-	919	-
Stage 2	-	-	-	-	917	-

Approach	EB	WB	NB
HCM Control Delay, s	0	0.3	9.3
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	840	-	-	1483	-
HCM Lane V/C Ratio	0.009	-	-	0.003	-
HCM Control Delay (s)	9.3	-	-	7.4	-
HCM Lane LOS	A	-	-	A	-
HCM 95th %tile Q(veh)	0	-	-	0	-

Intersection	
Int Delay, s/veh	0.8

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Traffic Vol, veh/h	87	6	6	106	9	6
Future Vol, veh/h	87	6	6	106	9	6
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	100	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	95	7	7	115	10	7

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	101	0	226	98
Stage 1	-	-	-	-	98	-
Stage 2	-	-	-	-	128	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1491	-	762	958
Stage 1	-	-	-	-	926	-
Stage 2	-	-	-	-	898	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1491	-	758	958
Mov Cap-2 Maneuver	-	-	-	-	760	-
Stage 1	-	-	-	-	926	-
Stage 2	-	-	-	-	894	-

Approach	EB	WB	NB
HCM Control Delay, s	0	0.4	9.4
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	828	-	-	1491	-
HCM Lane V/C Ratio	0.02	-	-	0.004	-
HCM Control Delay (s)	9.4	-	-	7.4	-
HCM Lane LOS	A	-	-	A	-
HCM 95th %tile Q(veh)	0.1	-	-	0	-

Intersection	
Int Delay, s/veh	0.3

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Traffic Vol, veh/h	140	4	4	141	4	3
Future Vol, veh/h	140	4	4	141	4	3
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	100	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	152	4	4	153	4	3

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	157	0	316	154
Stage 1	-	-	-	-	154	-
Stage 2	-	-	-	-	162	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1423	-	677	892
Stage 1	-	-	-	-	874	-
Stage 2	-	-	-	-	867	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1423	-	675	892
Mov Cap-2 Maneuver	-	-	-	-	705	-
Stage 1	-	-	-	-	874	-
Stage 2	-	-	-	-	865	-

Approach	EB	WB	NB
HCM Control Delay, s	0	0.2	9.7
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	775	-	-	1423	-
HCM Lane V/C Ratio	0.01	-	-	0.003	-
HCM Control Delay (s)	9.7	-	-	7.5	-
HCM Lane LOS	A	-	-	A	-
HCM 95th %tile Q(veh)	0	-	-	0	-

Intersection	
Int Delay, s/veh	0.6

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Traffic Vol, veh/h	128	6	6	142	9	6
Future Vol, veh/h	128	6	6	142	9	6
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	100	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	139	7	7	154	10	7

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	146	0	309	142
Stage 1	-	-	-	-	142	-
Stage 2	-	-	-	-	167	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1436	-	683	906
Stage 1	-	-	-	-	885	-
Stage 2	-	-	-	-	863	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1436	-	680	906
Mov Cap-2 Maneuver	-	-	-	-	708	-
Stage 1	-	-	-	-	885	-
Stage 2	-	-	-	-	859	-

Approach	EB	WB	NB
HCM Control Delay, s	0	0.3	9.7
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	776	-	-	1436	-
HCM Lane V/C Ratio	0.021	-	-	0.005	-
HCM Control Delay (s)	9.7	-	-	7.5	-
HCM Lane LOS	A	-	-	A	-
HCM 95th %tile Q(veh)	0.1	-	-	0	-

Intersection	
Int Delay, s/veh	0.2

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Traffic Vol, veh/h	0	20	319	93	0	538
Future Vol, veh/h	0	20	319	93	0	538
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	-	0	-	-	-	-
Veh in Median Storage, #	0	-	0	-	-	0
Grade, %	0	-	0	-	-	0
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	0	22	347	101	0	585

Major/Minor	Minor1		Major1		Major2	
Conflicting Flow All	631	224	0	0	448	0
Stage 1	397	-	-	-	-	-
Stage 2	234	-	-	-	-	-
Critical Hdwy	6.29	6.94	-	-	4.14	-
Critical Hdwy Stg 1	5.84	-	-	-	-	-
Critical Hdwy Stg 2	6.04	-	-	-	-	-
Follow-up Hdwy	3.67	3.32	-	-	2.22	-
Pot Cap-1 Maneuver	442	779	-	-	1109	-
Stage 1	626	-	-	-	-	-
Stage 2	745	-	-	-	-	-
Platoon blocked, %			-	-		-
Mov Cap-1 Maneuver	442	779	-	-	1109	-
Mov Cap-2 Maneuver	442	-	-	-	-	-
Stage 1	626	-	-	-	-	-
Stage 2	745	-	-	-	-	-

Approach	WB	NB	SB
HCM Control Delay, s	9.8	0	0
HCM LOS	A		

Minor Lane/Major Mvmt	NBT	NBR	WBLn1	SBL	SBT
Capacity (veh/h)	-	-	779	1109	-
HCM Lane V/C Ratio	-	-	0.028	-	-
HCM Control Delay (s)	-	-	9.8	0	-
HCM Lane LOS	-	-	A	A	-
HCM 95th %tile Q(veh)	-	-	0.1	0	-

Intersection	
Int Delay, s/veh	0.2

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Traffic Vol, veh/h	0	22	494	88	0	565
Future Vol, veh/h	0	22	494	88	0	565
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	-	0	-	-	-	-
Veh in Median Storage, #	0	-	0	-	-	0
Grade, %	0	-	0	-	-	0
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	0	24	537	96	0	614

Major/Minor	Minor1		Major1		Major2	
Conflicting Flow All	831	316	0	0	633	0
Stage 1	585	-	-	-	-	-
Stage 2	246	-	-	-	-	-
Critical Hdwy	6.29	6.94	-	-	4.14	-
Critical Hdwy Stg 1	5.84	-	-	-	-	-
Critical Hdwy Stg 2	6.04	-	-	-	-	-
Follow-up Hdwy	3.67	3.32	-	-	2.22	-
Pot Cap-1 Maneuver	340	680	-	-	946	-
Stage 1	504	-	-	-	-	-
Stage 2	734	-	-	-	-	-
Platoon blocked, %			-	-		-
Mov Cap-1 Maneuver	340	680	-	-	946	-
Mov Cap-2 Maneuver	340	-	-	-	-	-
Stage 1	504	-	-	-	-	-
Stage 2	734	-	-	-	-	-

Approach	WB	NB	SB
HCM Control Delay, s	10.5	0	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBT	NBR	WBLn1	SBL	SBT
Capacity (veh/h)	-	-	680	946	-
HCM Lane V/C Ratio	-	-	0.035	-	-
HCM Control Delay (s)	-	-	10.5	0	-
HCM Lane LOS	-	-	B	A	-
HCM 95th %tile Q(veh)	-	-	0.1	0	-

Intersection	
Int Delay, s/veh	0.1

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Traffic Vol, veh/h	0	20	536	93	0	833
Future Vol, veh/h	0	20	536	93	0	833
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	-	0	-	-	-	-
Veh in Median Storage, #	0	-	0	-	-	0
Grade, %	0	-	0	-	-	0
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	0	22	583	101	0	905

Major/Minor	Minor1		Major1		Major2	
Conflicting Flow All	995	342	0	0	684	0
Stage 1	633	-	-	-	-	-
Stage 2	362	-	-	-	-	-
Critical Hdwy	6.29	6.94	-	-	4.14	-
Critical Hdwy Stg 1	5.84	-	-	-	-	-
Critical Hdwy Stg 2	6.04	-	-	-	-	-
Follow-up Hdwy	3.67	3.32	-	-	2.22	-
Pot Cap-1 Maneuver	274	654	-	-	905	-
Stage 1	477	-	-	-	-	-
Stage 2	639	-	-	-	-	-
Platoon blocked, %			-	-		-
Mov Cap-1 Maneuver	274	654	-	-	905	-
Mov Cap-2 Maneuver	274	-	-	-	-	-
Stage 1	477	-	-	-	-	-
Stage 2	639	-	-	-	-	-

Approach	WB	NB	SB
HCM Control Delay, s	10.7	0	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBT	NBR	WBLn1	SBL	SBT
Capacity (veh/h)	-	-	654	905	-
HCM Lane V/C Ratio	-	-	0.033	-	-
HCM Control Delay (s)	-	-	10.7	0	-
HCM Lane LOS	-	-	B	A	-
HCM 95th %tile Q(veh)	-	-	0.1	0	-

Intersection	
Int Delay, s/veh	0.2

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Traffic Vol, veh/h	0	22	845	88	0	835
Future Vol, veh/h	0	22	845	88	0	835
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	-	0	-	-	-	-
Veh in Median Storage, #	0	-	0	-	-	0
Grade, %	0	-	0	-	-	0
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	0	24	918	96	0	908

Major/Minor	Minor1		Major1		Major2	
Conflicting Flow All	1329	507	0	0	1014	0
Stage 1	966	-	-	-	-	-
Stage 2	363	-	-	-	-	-
Critical Hdwy	6.29	6.94	-	-	4.14	-
Critical Hdwy Stg 1	5.84	-	-	-	-	-
Critical Hdwy Stg 2	6.04	-	-	-	-	-
Follow-up Hdwy	3.67	3.32	-	-	2.22	-
Pot Cap-1 Maneuver	176	511	-	-	680	-
Stage 1	322	-	-	-	-	-
Stage 2	638	-	-	-	-	-
Platoon blocked, %			-	-		-
Mov Cap-1 Maneuver	176	511	-	-	680	-
Mov Cap-2 Maneuver	176	-	-	-	-	-
Stage 1	322	-	-	-	-	-
Stage 2	638	-	-	-	-	-

Approach	WB	NB	SB
HCM Control Delay, s	12.4	0	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBT	NBR	WBLn1	SBL	SBT
Capacity (veh/h)	-	-	511	680	-
HCM Lane V/C Ratio	-	-	0.047	-	-
HCM Control Delay (s)	-	-	12.4	0	-
HCM Lane LOS	-	-	B	A	-
HCM 95th %tile Q(veh)	-	-	0.1	0	-

Intersection	
Int Delay, s/veh	3.1

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	16	100	63	121	225	31
Future Vol, veh/h	16	100	63	121	225	31
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	0	0	100	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	17	109	68	132	245	34

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	529	261	278	0	-	0
Stage 1	261	-	-	-	-	-
Stage 2	268	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	510	778	1285	-	-	-
Stage 1	783	-	-	-	-	-
Stage 2	777	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	483	778	1285	-	-	-
Mov Cap-2 Maneuver	483	-	-	-	-	-
Stage 1	783	-	-	-	-	-
Stage 2	736	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	10.7	2.7	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1285	-	483	778	-	-
HCM Lane V/C Ratio	0.053	-	0.036	0.14	-	-
HCM Control Delay (s)	8	-	12.7	10.4	-	-
HCM Lane LOS	A	-	B	B	-	-
HCM 95th %tile Q(veh)	0.2	-	0.1	0.5	-	-

Intersection	
Int Delay, s/veh	5.9

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	52	281	120	208	188	52
Future Vol, veh/h	52	281	120	208	188	52
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	0	0	100	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	57	305	130	226	204	57

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	720	233	261	0	-	0
Stage 1	233	-	-	-	-	-
Stage 2	487	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	395	806	1303	-	-	-
Stage 1	806	-	-	-	-	-
Stage 2	618	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	356	806	1303	-	-	-
Mov Cap-2 Maneuver	356	-	-	-	-	-
Stage 1	806	-	-	-	-	-
Stage 2	556	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	12.9	3	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1303	-	356	806	-	-
HCM Lane V/C Ratio	0.1	-	0.159	0.379	-	-
HCM Control Delay (s)	8.1	-	17	12.2	-	-
HCM Lane LOS	A	-	C	B	-	-
HCM 95th %tile Q(veh)	0.3	-	0.6	1.8	-	-

Intersection	
Int Delay, s/veh	2.5

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	16	100	63	211	395	31
Future Vol, veh/h	16	100	63	211	395	31
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	0	0	100	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	17	109	68	229	429	34

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	812	446	463	0	-	0
Stage 1	446	-	-	-	-	-
Stage 2	366	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	348	612	1098	-	-	-
Stage 1	645	-	-	-	-	-
Stage 2	702	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	326	612	1098	-	-	-
Mov Cap-2 Maneuver	326	-	-	-	-	-
Stage 1	645	-	-	-	-	-
Stage 2	659	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	12.7	2	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1098	-	326	612	-	-
HCM Lane V/C Ratio	0.062	-	0.053	0.178	-	-
HCM Control Delay (s)	8.5	-	16.7	12.1	-	-
HCM Lane LOS	A	-	C	B	-	-
HCM 95th %tile Q(veh)	0.2	-	0.2	0.6	-	-

Intersection	
Int Delay, s/veh	5.5

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Traffic Vol, veh/h	52	281	120	360	317	52
Future Vol, veh/h	52	281	120	360	317	52
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	0	0	100	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	57	305	130	391	345	57

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	1025	373	401	0	-	0
Stage 1	373	-	-	-	-	-
Stage 2	652	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	260	673	1158	-	-	-
Stage 1	696	-	-	-	-	-
Stage 2	518	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	231	673	1158	-	-	-
Mov Cap-2 Maneuver	231	-	-	-	-	-
Stage 1	696	-	-	-	-	-
Stage 2	460	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	16.4	2.1	0
HCM LOS	C		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	EBLn2	SBT	SBR
Capacity (veh/h)	1158	-	231	673	-	-
HCM Lane V/C Ratio	0.113	-	0.245	0.454	-	-
HCM Control Delay (s)	8.5	-	25.6	14.7	-	-
HCM Lane LOS	A	-	D	B	-	-
HCM 95th %tile Q(veh)	0.4	-	0.9	2.4	-	-

APPENDIX E

Queuing Analysis Worksheets

Lane Group	EBL	EBT	WBT	WBR	SBL	SBR
Lane Group Flow (vph)	365	837	1147	210	351	310
v/c Ratio	1.07	0.30	1.04	0.21	0.70	0.65
Control Delay	102.0	3.7	47.8	4.4	53.2	13.5
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	102.0	3.7	47.8	4.4	53.2	13.5
Queue Length 50th (ft)	~235	72	~868	13	123	12
Queue Length 95th (ft)	#252	91	m369	m12	155	69
Internal Link Dist (ft)		289	1560		233	
Turn Bay Length (ft)	750					
Base Capacity (vph)	341	2766	1100	1001	499	478
Starvation Cap Reductn	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0
Reduced v/c Ratio	1.07	0.30	1.04	0.21	0.70	0.65

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBT	WBR	SBL	SBR
Lane Group Flow (vph)	435	1360	1054	306	347	309
v/c Ratio	1.12	0.53	1.08	0.31	0.70	0.63
Control Delay	113.4	5.4	71.1	4.5	52.9	11.0
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	113.4	5.4	71.1	4.5	52.9	11.0
Queue Length 50th (ft)	~302	152	~833	17	121	0
Queue Length 95th (ft)	#436	190	m#819	m18	162	65
Internal Link Dist (ft)		289	1600		223	
Turn Bay Length (ft)	750					
Base Capacity (vph)	390	2565	973	996	499	494
Starvation Cap Reductn	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0
Reduced v/c Ratio	1.12	0.53	1.08	0.31	0.70	0.63

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBT	SBL	SBR
Lane Group Flow (vph)	516	1435	2351	541	477
v/c Ratio	0.97	0.37	1.01	0.96	0.65
Control Delay	63.5	4.6	39.1	70.8	26.5
Queue Delay	0.0	0.0	0.0	0.0	0.5
Total Delay	63.5	4.6	39.1	70.8	27.0
Queue Length 50th (ft)	306	102	~358	201	279
Queue Length 95th (ft)	294	120	#686	#247	352
Internal Link Dist (ft)		289	799	233	
Turn Bay Length (ft)	750				
Base Capacity (vph)	534	3883	2328	561	735
Starvation Cap Reductn	0	0	0	0	57
Spillback Cap Reductn	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0
Reduced v/c Ratio	0.97	0.37	1.01	0.96	0.70

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBT	SBL	SBR
Lane Group Flow (vph)	630	2306	2251	501	470
v/c Ratio	1.05	0.63	1.07	1.00	0.61
Control Delay	83.0	6.0	59.2	85.2	26.2
Queue Delay	0.0	0.0	0.0	0.0	1.0
Total Delay	83.0	6.0	59.2	85.2	27.2
Queue Length 50th (ft)	~437	206	~620	~189	261
Queue Length 95th (ft)	#570	240	#716	#262	362
Internal Link Dist (ft)		289	740	223	
Turn Bay Length (ft)	750				
Base Capacity (vph)	598	3686	2112	499	765
Starvation Cap Reductn	0	0	0	0	115
Spillback Cap Reductn	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0
Reduced v/c Ratio	1.05	0.63	1.07	1.00	0.72

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	60	1024	363	1372	237	123	187	220	218
v/c Ratio	0.37	0.90	0.87	0.80	0.65	0.30	0.38	0.49	0.53
Control Delay	23.3	41.0	50.4	28.0	37.1	40.5	8.1	30.9	43.9
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	23.3	41.0	50.4	28.0	37.1	40.5	8.1	30.9	43.9
Queue Length 50th (ft)	13	366	188	401	127	77	0	117	139
Queue Length 95th (ft)	14	#468	230	474	97	94	15	151	200
Internal Link Dist (ft)		992		349		838			447
Turn Bay Length (ft)	700		875		275		125	425	
Base Capacity (vph)	163	1178	448	1754	371	411	494	460	411
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.37	0.87	0.81	0.78	0.64	0.30	0.38	0.48	0.53

Intersection Summary

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	124	1438	128	1257	225	170	181	302	296
v/c Ratio	0.66	0.95	0.71	0.83	0.74	0.48	0.42	0.72	0.72
Control Delay	34.0	37.3	40.1	31.5	42.9	45.4	11.1	38.0	49.4
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	34.0	37.3	40.1	31.5	42.9	45.4	11.1	38.0	49.4
Queue Length 50th (ft)	32	516	40	389	114	109	9	162	188
Queue Length 95th (ft)	33	#666	#126	487	140	156	48	230	251
Internal Link Dist (ft)		952		349		838			477
Turn Bay Length (ft)	700		875		275		125	425	
Base Capacity (vph)	188	1516	180	1516	307	354	434	427	411
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.66	0.95	0.71	0.83	0.73	0.48	0.42	0.71	0.72

Intersection Summary

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	64	1827	666	2372	308	186	340	321	292	40
v/c Ratio	0.25	0.91	0.89	0.84	0.52	0.24	0.59	0.52	0.43	0.10
Control Delay	11.6	33.2	44.2	23.6	32.9	36.4	12.0	35.9	42.0	0.5
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	11.6	33.2	44.2	23.6	32.9	36.4	12.0	35.9	42.0	0.5
Queue Length 50th (ft)	6	456	182	486	86	57	28	90	97	0
Queue Length 95th (ft)	m6	m510	193	535	63	63	30	106	128	0
Internal Link Dist (ft)		992		349		838			447	
Turn Bay Length (ft)	700		875		275		125	425		100
Base Capacity (vph)	253	2021	781	2824	588	774	573	621	673	405
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.25	0.90	0.85	0.84	0.52	0.24	0.59	0.52	0.43	0.10

Intersection Summary
m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	142	2595	238	2133	305	246	334	384	293	85
v/c Ratio	0.76	1.07	0.85	0.83	0.82	0.48	0.74	1.03	0.57	0.28
Control Delay	74.2	59.3	76.7	24.1	67.0	46.7	42.2	102.2	48.5	10.9
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	74.2	59.3	76.7	24.1	67.0	46.7	42.2	102.2	48.5	10.9
Queue Length 50th (ft)	52	~745	87	430	110	85	185	~149	102	0
Queue Length 95th (ft)	52	m#823	#155	503	125	111	241	#228	132	0
Internal Link Dist (ft)		952		349		838			477	
Turn Bay Length (ft)	700		875		275		125	425		100
Base Capacity (vph)	187	2435	280	2562	374	514	454	374	518	308
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.76	1.07	0.85	0.83	0.82	0.48	0.74	1.03	0.57	0.28

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBR	NBL	NBT	SBL	SBT
Lane Group Flow (vph)	81	120	210	13	142	456	72	721
v/c Ratio	0.26	0.20	0.82	0.02	0.28	0.21	0.11	0.35
Control Delay	35.2	0.8	65.0	0.1	10.8	15.5	6.9	13.3
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	35.2	0.8	65.0	0.1	10.8	15.5	6.9	13.3
Queue Length 50th (ft)	48	0	142	0	42	91	14	126
Queue Length 95th (ft)	54	0	208	0	m55	m117	32	150
Internal Link Dist (ft)		185				109		430
Turn Bay Length (ft)	200			50	200		75	
Base Capacity (vph)	512	777	420	872	579	2167	660	2071
Starvation Cap Reductn	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	3	0	0	0	0	0	33
Storage Cap Reductn	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.16	0.16	0.50	0.01	0.25	0.21	0.11	0.35

Intersection Summary

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBR	NBL	NBT	SBL	SBT
Lane Group Flow (vph)	182	167	252	35	223	560	45	553
v/c Ratio	0.47	0.21	0.88	0.05	0.40	0.28	0.09	0.31
Control Delay	35.7	0.6	66.6	0.2	15.2	21.4	10.1	17.6
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	35.7	0.6	66.6	0.2	15.2	21.4	10.1	17.6
Queue Length 50th (ft)	107	0	170	0	86	142	10	107
Queue Length 95th (ft)	117	0	240	0	m102	m166	29	150
Internal Link Dist (ft)		127				130		414
Turn Bay Length (ft)	200			50	200		75	
Base Capacity (vph)	589	937	437	862	655	1988	528	1773
Starvation Cap Reductn	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	4
Storage Cap Reductn	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.31	0.18	0.58	0.04	0.34	0.28	0.09	0.31

Intersection Summary
m Volume for 95th percentile queue is metered by upstream signal.

APPENDIX F

Conceptual Site Plan

SITE LEGEND

- PROPERTY BOUNDARY LINE
- R.O.W. LINE
- EASEMENT BOUNDARY LINE
- EXISTING TO REMAIN
- EXISTING CURB & GUTTER TO REMAIN
- PROPOSED CURB & GUTTER
- PROPOSED CONCRETE PAVING
- PROPOSED ASPHALT PAVING
- PROPOSED HEAVY DUTY ASPHALT PAVING
- PROPOSED RETAINING WALL
- PROPOSED SIDEWALK
- STREET LIGHT
- EXISTING FIRE HYDRANT
- PROPOSED MANHOLE COVER
- PROPOSED REGULATORY SIGN
- PROPOSED INLET

BENCHMARK

CITY OF AND COUNTY OF BROOMFIELD BM "LUCY" ELEVATION: 5297.00 FEET (NAVD 1988 DATUM)

BASIS OF BEARING

BEARINGS ARE BASED ON THE NORTH RIGHT-OF-WAY LINE OF STATE HIGHWAY NO. 7 BEARING N89°38'37"W AS REFERENCED AND BOUNDED BY THE MONUMENTS SHOWN HEREON.

LEGAL DESCRIPTION

A PARCEL OF LAND ACROSS PARCELS 33 AND 34 OF "VISTA RIDGE MASTER FINAL PLAT" AND LOT 2 AND TRACT A OF "VISTA RIDGE FILING NO. 12", LOCATED IN THE SOUTHWEST QUARTER OF SECTION 33, TOWNSHIP 1 NORTH, RANGE 68 WEST OF THE 6TH P.M., TOWN OF ERIE, COUNTY OF WELD, STATE OF COLORADO. SEE SURVEY FOR DETAILED DESCRIPTION.

CAUTION - NOTICE TO CONTRACTOR

1. ALL UTILITY LOCATIONS SHOWN ARE BASED ON MAPS PROVIDED BY THE APPROPRIATE UTILITY COMPANY AND FIELD SURFACE EVIDENCE AT THE TIME OF SURVEY AND IS TO BE CONSIDERED AN APPROXIMATE LOCATION ONLY. IT IS THE CONTRACTOR'S RESPONSIBILITY TO FIELD VERIFY THE LOCATION OF ALL UTILITIES, PUBLIC OR PRIVATE, WHETHER SHOWN ON THE PLANS OR NOT, PRIOR TO CONSTRUCTION. REPORT ANY DISCREPANCIES TO THE ENGINEER PRIOR TO CONSTRUCTION.
2. WHERE A PROPOSED UTILITY CROSSES AN EXISTING UTILITY, IT IS THE CONTRACTOR'S RESPONSIBILITY TO FIELD VERIFY THE HORIZONTAL AND VERTICAL LOCATION OF SUCH EXISTING UTILITY, EITHER THROUGH POTHOLING OR ALTERNATIVE METHOD. REPORT INFORMATION TO THE ENGINEER PRIOR TO CONSTRUCTION.

811 Know what's **below.** **Call** before you dig.

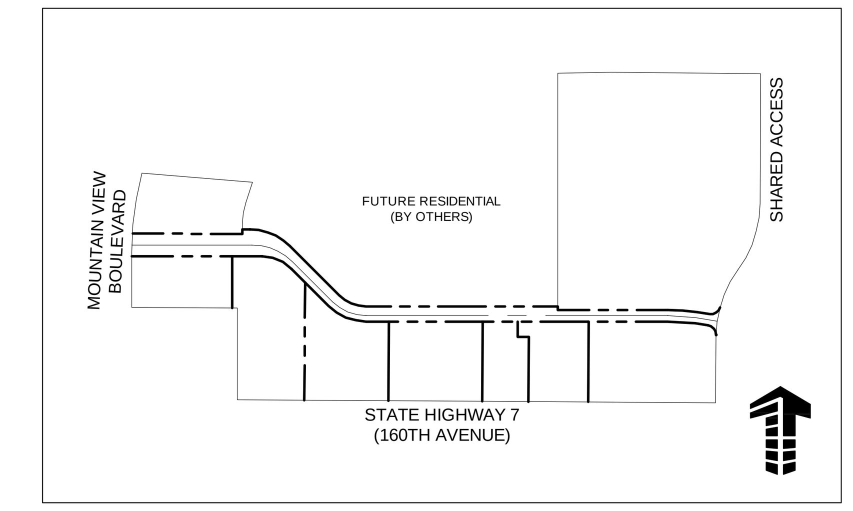
SCHEDULE

1. CURB AND GUTTER TO REMAIN
2. PROPOSED 8' WIDE SIDEWALK TO BE DEVELOPED WITH EACH PADS.
3. PROPOSED RETAINING WALL SEE SHEET C7.1
4. PROPOSED 10' TYPE R INLET
5. PROPOSED DETENTION POND - SEE SHEET C_._
6. EXISTING DETENTION POND - SEE SHEET C_._
7. EXISTING ASPHALT TO REMAIN - SEE SHEET C_._
8. PROPOSED ASPHALT - SEE SHEET C_._
9. PROPOSED CURB & GUTTER - SEE DETAIL ___ , SHEET C_._ (1' PAN 6" VERTICAL)
10. PROPOSED OUTLET STRUCTURE - SEE SHEET C_._
11. PROPOSED R1.1 STOP SIGN (SEE SITE DETAILS SHEET C_._)
12. PROPOSED STOP BAR (SEE SITE DETAILS SHEET C_._)
13. PROPOSED TRAFFIC DIRECTIONAL ARROW (SEE SITE DETAILS SHEET C_._)
14. PROPOSED GUARDRAIL /WALL - SEE SHEET C_._
15. EXISTING CURB & GUTTER TO BE REMOVED

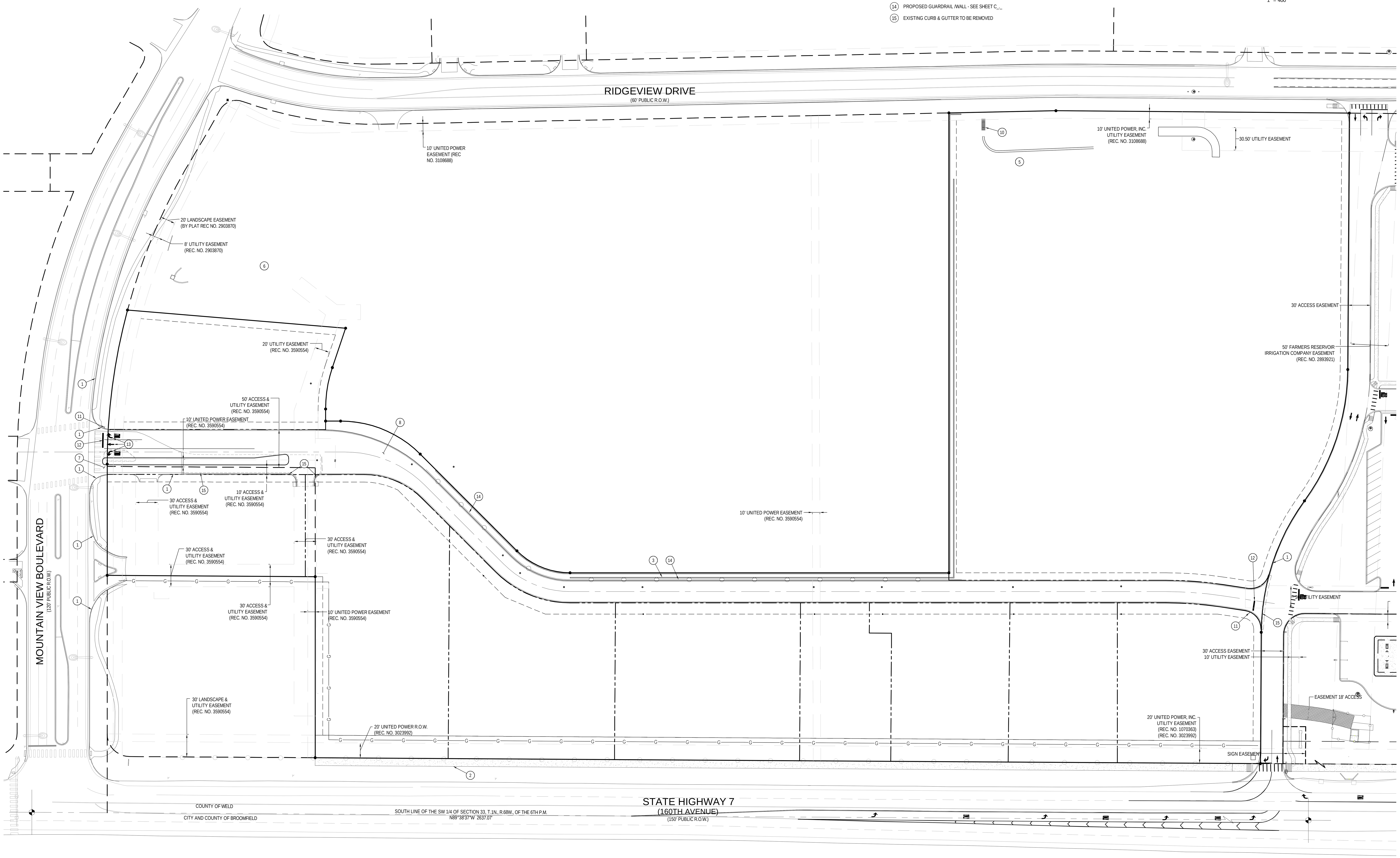
N

0 10 30 60

SCALE: 1"=60'



KEY MAP
1" = 400'



Galloway
Planning. Architecture. Engineering.
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Greenwood Village, CO 80111
303.770.8884 O
www.gallowayUS.com

VISTA RIDGE COMMERCIAL WEST
CIVIL CONSTRUCTION DRAWINGS

ERIE, COLORADO

#	Date	Issue / Description	Init.

Project No:	SH7000001.01
Drawn By:	KRL
Checked By:	PJD
Date:	5/20/16

SITE PLAN

C1.1

Galloway
Planning. Architecture. Engineering.

Phase III Drainage Study
Vista Ridge Commercial West

NEC of Mountain View Boulevard & Hwy 7
Located in the S ½ of Section 33, Township 1 North, Range 68 West, of the 6th Principal Meridian, Town of Erie, County of Weld, State of Colorado

Date: May 26, 2016
Revised July 28, 2016

Prepared for:
State Highway 7 Marketplace
c/o Marathon Land Company
9750 W. Cambridge Place
Littleton, CO 80127
Phone (303) 920-9400
Attn: James Spehalski

Prepared by:
Galloway & Company, Inc.
6162 S. Willow Drive, Suite 320
Greenwood Village, CO 80111
Phone (303) 770-8884
Fax (303) 770-3636
Attn: Brandon S. McCrary, P.E.

Galloway
Planning. Architecture. Engineering.

ENGINEER'S CERTIFICATION

I hereby certify that this Final Drainage Report for the Vista Ridge Commercial West development located at the northeast corner of the intersection of Mountain View Boulevard and Highway 7 in Erie, Colorado, was prepared by me (or under my direct supervision) in accordance the provisions of the Town of Erie Standards and Specifications for Design and Construction of Public Improvements and the Urban Drainage and Flood Control District's Urban Storm Drainage Criteria Manual for the owners thereof.

Brandon S. McCrary, PE
Colorado Registered Professional Engineer

Town Acceptance

This report had been reviewed and found to be in general compliance with the *Town of Erie Standards and Specifications for Design and Construction* and other Town requirements. **THE ACCURACY AND VALIDITY OF THE ENGINEERING DESIGN, DETAILS, DIMENSIONS, QUANTITIES, AND CONCEPTS IN THIS REPORT REMAINS THE SOLE RESPONSIBILITY OF THE PROFESSIONAL ENGINEER WHOSE STAMP AND SIGNATURE APPEAR HEREON.**

Accepted by:______________________________ __________________
Deputy Public Works Director Date

Table of Contents

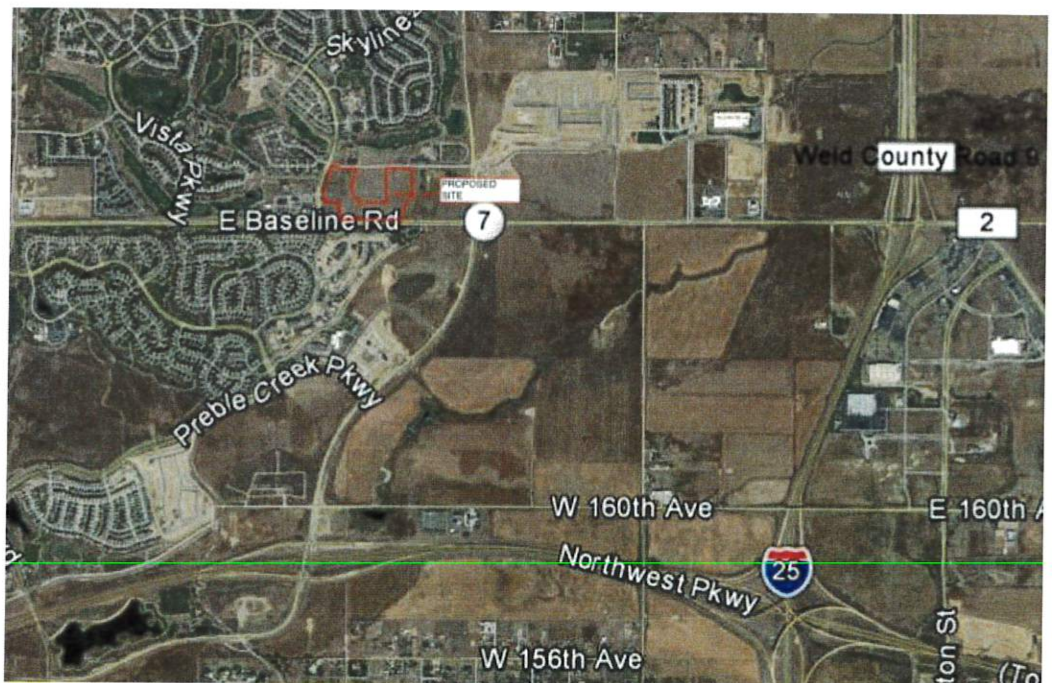
Introduction
This final drainage report has been prepared by Galloway & Company, Inc. for the Vista Ridge Commercial West development in Erie, Colorado. The objective of this report is to define the site's final drainage basins and characteristics for the shopping center and address the general conformance with previous drainage reports related to the property and historic drainage patterns that exist on site. The report analyzes on-site runoff for both the minor 5-year and 10-year frequencies and the major 100-year frequency.

The site will include multiple pad sites along with a junior anchor parcel on the northeastern portion of the site. The site will be bisected by a private roadway running east-west to provide access to all of the proposed pad sites. The total site area is approximately 17.6 acres. The development will be served by two major access points, one along Mountain View Boulevard, and one along the private roadway between the site and the proposed King Soopers (currently under construction).

I. Project Location and Description

A. Location
The King Soopers at Vista Ridge Marketplace is located at the NEC of Mountain View Boulevard & Hwy 7 located in the SW ¼ of Section 33, Township 1 North, Range 68 West, of the 6th Principal Meridian, Town of Erie, County of Weld, State of Colorado. The site is bounded by Ridge View Drive to the north, Mountain View Boulevard to the west, Highway 7 to the south, and a private roadway and King Soopers to the east.

PROJECT LOCATION MAP



NOT TO SCALE

B. Description of Property

The existing property is currently vacant and mainly vegetated with natural grasses and weeds. The ultimate proposed development will include roof and paved areas, along with pockets of landscaping, including mulch, sod, shrubs and trees consistent with commercial development.

The site will include multiple pad sites along with a junior anchor parcel on the northeastern portion of the site. The site will be bisected by a private roadway running east-west to provide access to all of the proposed pad sites. The total site area is approximately 17.6 acres. The development will be served by two major access points, one along Mountain View Boulevard, and one along the private roadway between the site and the proposed King Soopers (currently under construction).

Natural soils on site consist of Ulm clay loam, Midway-Shingle complex, and Renohill clay loam, with average slopes of 1-10%. The soil type for this area is classified as Hydrologic Type C and D soils, as defined by the USDA SCS Soil Survey of Weld County, Colorado. A copy of the Soils Map has been included in Appendix H.

Based on the geotechnical report for the site, prepared by Kumar and Associates, dated October 12, 2015, the subsurface conditions consist of a variable thickness top soil overlying overburden man-placed fills and natural soils. The underlying bedrock consisted generally of claystone with frequent zones of interbedded claystone and sandstone ranging from a few inches to about 18 feet below ground surface.

The imported soils encountered on the site ranged from 5 to 8 feet in depth. The degree of compaction of the existing fill material was not determined at this time. Groundwater was encountered in two borings at depths ranging from 8 to 18 feet. Follow up groundwater measurements were taken 14 days after drilling and no groundwater was encountered.

A review of the Flood Insurance Rate Map (FIRM) by the Federal Emergency Management Agency (FEMA) shows the entire proposed development in Zone C. By definition of Zone C, all lots within the Vista Ridge Commercial West development are designated as areas of minimal flooding and are outside of the 500 year flood plain according to FIRM Map 080266 0970D (See Appendix I).

II. Historic and Overall Basin Characteristics

The subject property of 17.6 acres primarily resides in Vista Ridge Parcels 32 and 33. These two parcels primarily consist of native grasses and the site slopes from east to west towards Mountain View Boulevard. An existing detention pond has been built at the northwest corner of the Mountain View Boulevard and Ridgeview Drive. This pond has been sized to capture the majority of the runoff from the site, with the exception of the northeast portion of the site (proposed junior anchor site). A new detention pond has been designed to handle storm water from this portion of the site. A drainage study was prepared for Parcels 32 and 33 by Hurst & Associates which outlines the historic runoff patterns from this site as well as contemplates this site as commercial use with the requirement to detail and treat flows in the developed condition (See Appendix A for details). A second drainage report has been prepared for the Montex South property adjacent to the proposed site, and is referenced in Appendix B. A third report for the Les Schwab site at the NE corner of Mountain View Blvd. and State Highway 7 has been included as flows from this site drain through the proposed development. This report can be found in Appendix C.

There are no floodplains encumbering the site. (Refer to Appendix I for FIRM Map)

III. Drainage Design Criteria

A. Development Constraints and Criteria

A drainage study was prepared for Parcels 32 and 33 by Hurst & Associates which outlines the historic runoff patterns from this site as well as contemplates this site as commercial use with the requirement to detail and treat flows in the developed condition (See Appendix A for details). This study defines a 5-year and 100-year minor and major storm respectively for the on-site storm sewer design, as well as 10-year and 100-year for detention pond flows/releases. Based on the proposed site layout, a composite imperviousness for the site of 78% was assumed for the pad sites, and a composite imperviousness of 77% for the junior anchor parcel. These values were assigned to evaluate post developed flows and overall detention requirements via the V=KA method. A 40-hr drain time was also outlined for the water quality release duration.

The proposed Vista Ridge Commercial West development will conform to the historic release and detention pond/water quality parameters as outlined in the Hurst & Associates Drainage study.

This final report was prepared using the criteria from the Town of Erie's "Standards and Specifications for Design and Construction of Public Improvements," Section 800, as well as Urban Drainage and Flood Control District's Urban Storm Drainage Criteria Manual.

B. Hydrologic Criteria

Design rainfall intensity-duration-frequency (IDF) data is provided by the Town of Erie. The rational method (Q=CIA) has been used to compute times of concentration and total runoff for the minor (5-year) and major (100-year) storm events. Hydrological calculations are included in the appendix. 5-year and 100-year runoff coefficients were calculated using the Urban Storm Drainage Criteria Manual for the rational method and UDFCD Tables RO-3 and RO-5. (See Appendix D).

10-year and 100-year detention volumes were sized using the V=KA method outlined in the Denver Urban Storm Drainage Criteria Manual (USDCM). For purposes of sizing the basin, the WQCV has been included within the 100-year detention volume. (See Appendix E).

Stormwater quality has also been provided by designing the detention facility as a full spectrum extended detention basin in accordance with the EURV method as defined by Urban Storm Drainage Criteria Manual (USDCM) Volume 3. Full spectrum detention is designed to best replicate predevelopment peak flows for a broad range of storm intensities and durations.

C. Hydraulic Criteria

A detailed analysis of the storm sewer inlets and storm sewer conveyance system has been provided for both the on-site inlet capacities and storm sewer capacity by the following methods:

- Storm sewer capacities were evaluated using AutoDesk Storm and Sanitary Analysis (SSA) utilizing hydrodynamic routing.
- Inlet capacities were evaluated using the UDFCD Street and inlet Hydraulics Workbook version 3.12, November 2012 and the Storm Drainage Design and Technical Criteria Inlet Capacity Chart - Figure 803.

A detailed analysis for determination of all storm sewer sizes, hydraulic and energy grade lines has been provided in Appendix G.

IV. Drainage Facility Design

A. General Concept

On-Site Flows and Concept

On-Site flows will be collected in a series of basins and storm sewer network throughout the development which will convey storm flows to the proposed development detention and water quality ponds located at the northwest corner of the site and along Ridgeview Drive. The on-site basins will account for all entire proposed 17.8 acre development, including the future overlot graded pad sites and junior anchor parcel. The ultimate outfall will be the existing 24" leaving detention pond A1 at the western edge of the site. The 24" pipe runs under Mountain View Boulevard and outfalls into an existing open channel to the west of the roadway. The proposed detention ponds will be owned and maintained by the owner.

Off-Site Flows

Highway 7 acts as a ridge to the south of the property, thus keeping any offsite flows from the south reaching the property. The King Soopers site to the east drains away from the proposed site. Properties to the north and west of Ridgeview Drive and Mountain View Boulevard are lower than the proposed site, and no offsite drainage enters the site. A small amount of runoff from the Highway 7 embankment and the access drive to the east will drain on to the site. The proposed storm sewer system and detention ponds have been sized accordingly to handle this runoff.

B. Specific Details

Proposed Basin Description

The basin and storm sewer network is divided into A and B basins. (Refer to Drainage Plan DP1.2 located in the rear pocket of this report).

Basin A (7.34 acres) collects runoff from the junior anchor parcel on the northeast corner of the site. Runoff is conveyed to the proposed detention pond via overland flow and a proposed storm sewer system which will be constructed when the parcel is developed. Once in the pond, the runoff will outfall into the existing storm sewer system in Ridgeview Drive.

Basin B (10.64 acres) consists of the remaining pad sites and proposed private driveway. Runoff will be conveyed to the existing detention pond at the northwest corner of the site via overland flow and the proposed storm sewer system. The main trunk line storm sewer will be installed as part of this project, and each pad will connect to the trunk line as they are developed. The existing pond has been modified to handle a slight increase in tributary area and to accommodate the existing basins that currently drain to the pond. The pond appears to have been constructed slightly smaller than the design, and this modification will make the adjustment. No modification to the existing outlet structure is proposed at this time, as the water surface elevations for each stage in the pond did not significantly change.

<u>Sub-Basin Description</u>

Basin A-1 (7.34 acres, 77% Impervious) consists of future rooftops, paved and landscaped areas within the junior anchor parcel. Runoff is conveyed via sheet flow, gutter flow and future storm sewer to the northwest towards the proposed detention pond at the northwest corner of this parcel. The 100-year runoff coefficients and flow rate for this basin are 0.72 and 40.7 cfs respectively.

Basin B-1 (0.98 acres, 77% Impervious) consists of future rooftops, paved and landscaped areas within the proposed pad site. Runoff is conveyed via sheet flow, gutter flow and future storm sewer to the northwest towards the proposed storm sewer stub. This parcel will extend the storm sewer system into the site as it develops. The 100-year runoff coefficients and flow rate for this basin are 0.72 and 6.3 cfs respectively.

Basin B-2 (0.70 acres, 79% Impervious) consists of future rooftops, paved and landscaped areas within the proposed pad site. Runoff is conveyed via sheet flow, gutter flow and future storm sewer to the northwest towards the proposed storm sewer stub. This parcel will extend the storm sewer system into the site as it develops. The 100-year runoff coefficients and flow rate for this basin are 0.74 and 4.7 cfs respectively.

Basin B-3 (0.99 acres, 76% Impervious) consists of future rooftops, paved and landscaped areas within the proposed pad site. Runoff is conveyed via sheet flow, gutter flow and future storm sewer to the northwest towards the proposed storm sewer stub. This parcel will extend the storm sewer system into the site as it develops. The 100-year runoff coefficients and flow rate for this basin are 0.72 and 6.5 cfs respectively.

Basin B-4 (1.01 acres, 75% Impervious) consists of future rooftops, paved and landscaped areas within the proposed pad site. Runoff is conveyed via sheet flow, gutter flow and future storm sewer to the northwest towards the proposed storm sewer stub. This parcel will extend the storm sewer system into the site as it develops. The 100-year runoff coefficients and flow rate for this basin are 0.71 and 6.7 cfs respectively.

Basin B-5 (0.64 acres, 77% Impervious) consists of future rooftops, paved and landscaped areas within the proposed pad site. Runoff is conveyed via sheet flow, gutter flow and future storm sewer to the northwest towards the proposed storm sewer stub. This parcel will extend the storm sewer system into the site as it develops. The 100-year runoff coefficients and flow rate for this basin are 0.72 and 4.2 cfs respectively.

Basin B-6 (1.38 acres, 77% Impervious) consists of future rooftops, paved and landscaped areas within the proposed pad site. Runoff is conveyed via sheet flow, gutter flow and future storm sewer to the northwest towards the proposed storm sewer stub. This parcel will extend the storm sewer system into the site as it develops. The 100-year runoff coefficients and flow rate for this basin are 0.72 and 8.6 cfs respectively.

Basin B-7 (1.11 acres, 79% Impervious) consists of future rooftops, paved and landscaped areas within the proposed pad site. Runoff is conveyed via sheet flow, gutter flow and future storm sewer to the northwest towards the existing storm sewer stub at the northwest corner of the site. This parcel will extend the storm sewer system into the site as it develops. The 100-year runoff coefficients and flow rate for this basin are 0.74 and 7.4 cfs respectively.

Basin B-8 (0.28 acres, 95% Impervious) consists of future rooftops, paved and landscaped areas within the proposed pad site. Runoff is conveyed via sheet flow, gutter flow to the northeast towards the existing storm sewer inlet at the northeast corner of the site. The 100-year runoff coefficients and flow rate for this basin are 0.89 and 2.3 cfs respectively.

Basin B-9 (1.40 acres, 96% Impervious) consists of future rooftops, paved and landscaped areas within the proposed pad site along with the southern half of the proposed private roadway. Runoff is conveyed via sheet flow, gutter flow to the northwest towards the existing storm sewer inlet at the western edge of the site. The 100-year runoff coefficients and flow rate for this basin are 0.90 and 9.4 cfs respectively.

Basin B-10 (0.50 acres, 100% Impervious) consists of the northern half of the proposed private roadway. Runoff is conveyed via sheet flow, gutter flow to the west towards a proposed storm sewer inlet at the west end of the basin. The inlet will connect into the existing storm sewer system. The 100-year runoff coefficients and flow rate for this basin are 0.96 and 3.9 cfs respectively.

Basin B-11 (0.47 acres, 95% Impervious) consists of future rooftops, paved and landscaped areas within the proposed pad site. Runoff is conveyed via sheet flow, gutter flow to the west towards an existing storm sewer inlet at the west end of the basin. The inlet connects into the existing storm sewer system. The 100-year runoff coefficients and flow rate for this basin are 0.89 and 3.8 cfs respectively.

Basin B-12 (0.98 acres, 50% Impervious) consists of future rooftops, paved and landscaped areas within the proposed pad site. Runoff is conveyed via sheet flow, gutter flow to the north towards a proposed storm sewer inlet at the north end of the basin. The inlet will connect into the proposed storm sewer system for the pad which will be developed by the pad user at a later date. The 100-year runoff coefficients and flow rate for this basin are 0.60 and 4.4 cfs respectively.

Basin B-13 (0.22 acres, 78% Impervious) consists of the retaining wall area north of the proposed private roadway. Runoff is conveyed via sheet flow north offsite to the adjacent development. This runoff will drain around the proposed buildings and drain into the proposed storm sewer system for the adjacent site. The 100-year runoff coefficients and flow rate for this basin are 0.96 and 1.0 cfs respectively.

Basin OS-1 (1.52 acres, 76% Impervious) consists of the future commercial pad site at the northeast corner of Mountain View Blvd., and State Highway 7. Much of this parcel drains into an on-site storm sewer system which will connect to the existing storm sewer system at Design Point 7. The storm sewer system ultimately drains into the existing detention pond. The 100-year runoff coefficients and flow rate for this basin are 0.76 and 10.9 cfs respectively.

Pond Design

The detention pond (Pond A1A) is designed as extended detention basin with side slopes of 4:1 and the bottoms sloping 2% to a concrete trickle channel which is sloped at 0.68% to the outlet. This flat slope facilitates sedimentation and will promote infiltration within the basins.

Water quality in the pond will be designed utilizing Urban Drainage and Flood Control District's EURV Spreadsheet. A forebay and initial surcharge volume will not be included with this design. The water quality outlet structure will be a Colorado Department of Transportation modified type C inlet with a well screen on the front face and a column of orifices on the inside face, covering an opening in the concrete. The orifice plate allows for release of the water quality capture volume over 72 hours and facilitates sedimentation. A small precast micropool will be included at the face of the inlet. This will help to prevent clogging of the well screen and facilitate biological uptake of nutrients contained in runoff.

The pond 100-year release rate has been determined to be 7.34 CFS based on table 800-4 of "Standards and Specifications for Design and Construction of Public Improvements." An overflow spillway will be constructed at the 100-year water surface elevation directing overflows to Ridgeview Drive and armored with Type H; riprap. The spillway will have the ability to convey two times the peak runoff from the upstream area without overtopping the pond. This will allow runoff in excess of the 100-year event to be conveyed downstream following historic drainage patterns.

The existing detention pond A1 has been modified to handle a slight increase in tributary area and to accommodate the existing basins that currently drain to the pond. The pond appears to have been constructed slightly smaller than the design, and this modification will make the adjustment. No modification to the existing outlet structure is proposed at this time, as the water surface elevations for each stage in the pond did not significantly change.

The ponds will be privately maintained and access can be achieved by small power equipment and personnel from the south side of the pond via a 20% grade maintenance path.

V. Conclusions

This final drainage design is in compliance with applicable regional standards including the Town of Erie's "Standards and Specifications for Design and Construction of Public Improvements, Section 800" and Urban Drainage and Flood Control District Urban Storm Drainage Criteria Manual. This design provides for on-site release of a maximum of 1 CFS per acre, which is well below the approximate 2 CFS per acre historical release rates. It also provides stormwater quality management and 100-year protection to the downstream drainage system.

VI. References

1. Urban Storm Drainage Criteria Manual, Urban Drainage and Flood Control District, March 2006 (with current revisions).

2. Town of Erie, Standards and Specifications for Design and Construction of Public Improvements, latest revision.

3. Brownlee Annexation Drainage Plan #24 – Provided by Town of Erie

Appendix A
Hurst & Associates Drainage Report

DRAINAGE REPORT
VISTA RIDGE PARCELS 32 & 33
ERIE, COLORADO

Prepared For:
Vista Ridge Development Corporation
4950 S. Yosemite Street
F2 #503
Greenwood Village, Colorado 80111

Prepared By:
Hurst and Associates, Inc.
4999 Pearl East Circle
Suite 106
Boulder, Colorado 80301

Job Number 2142-03
June 3, 2008
Revised June 27, 2008
Revised August 13, 2008
Revised September 10, 2008
Revised September 23, 2008

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INTRODUCTION

Vista Ridge Parcels 32 & 33 are located in the Southwest Quarter of Section 33, Township 1 North, Range 68 West of the 6^{th} P.M., Town of Erie, County of Weld, State of Colorado. Vista Ridge is a master planned mixed use community with Parcels 32 & 33 (31.8 acres) zoned for commercial use. The purpose of this report is to establish the overall drainage design concepts for the commercial parcels. Several different types of commercial/retail uses are possible within these parcels. This report establishes the design and locations of conveyance facilities with detention and water quality requirements.

EXISTING CONDITIONS

Parcels 32 & 33 are bordered by Mountain View Boulevard to the west, State Highway 7 to the south, and Ridge View Drive to the north. The parcels generally drain from the southeast to northwest at slopes of 2 to 4 percent. Vegetation is currently native grasses.

The existing soil type is made up primarily of Ulm clay loam with slopes of 0-3 percent and slopes of 3 to 5 percent as determined by the *Soil Survey of Weld Count, Colorado*. The soil is deep and well drained soil that has a slow permeability. It is in the "C" hydrologic group which generalizes soils as having slow infiltration rates.

PROPOSED FACILITIES

Pond A1 is an extended detention water quality pond located in the northeast corner of Vista Ridge Parcel 32. Pond A1 collects storm runoff from Parcel 32 and a portion of Parcel 33. Pond A1A will collect storm runoff from the remainder of Parcel 33 and outfalls to Pond A1.

Pond A1 will utilize retaining walls and vegated slopes at a maximum of 4:1 side slopes. A concrete outlet structure restricts storm runoff to the required release rates for water quality, 10-year and 100-year storm events.

The detention ponds were sized using CUHP and UDSWM. The volumes and release rates are less than the Vista Ridge Master Drainage Plan due to the reduced basin size. Approximately 7.5 acres of the original Basin A1 drains east to a proposed pond on the Brownlee Annexation. The WQCV was calculated using *Urban Storm Drainage Volume 3*.

The release rates for the 10 year and the 100 year events were all calculated using *Town of Erie's Standards and Specification for Design and Construction of Public Improvements Section 814.08*. The WQCV release rate was calculated using *Urban Storm Drainage Volume 3*.

The storage elevations, volume of storage, and release rates for the WQCV, 10 year storm, and 100 year storm for Pond A1 are as follows:

Pond A1

	Elevation	Volume (ac-ft)	Release Rate (cfs)
WQCV	5225.42	0.86	0.43
10-Yr Storm	5227.33	2.40	8.00
100-Yr Storm	5228.94	3.70	27.00

The concrete outlet structure is a 5' x 5' structure with a grate top and manhole steps for access to the structure. The WQCV release is controlled by an orifice half way up the outlet structure at the bottom of the pond. A pipe screen will be placed over the orifice to prevent debris from becoming clogged. The 10 year release is controlled by an orifice that is a quarter of the way down from the top of the outlet structure. The 100 year release is at the top of the structure and an emergency spillway has been provided.

Post development water quality is improved by providing extended detention with a 40 hour drain time. During the construction phase, runoff quality is controlled by the use of silt fences and inlet protection.

APPENDIX A
RUNOFF CALCULATIONS

Runoff Coefficients
Vista Ridge Commercial Parcels 32 & 33
Job Number: 2142-03

Land Use Characteristics

	C_5	C_{10}	C_{100}	Impervious
Commercial	0.87	0.88	0.89	95%
Lawns, Clay Soil	0.15	0.25	0.50	0%
Drives & Walks	0.87	0.88	0.89	96%
Streets, Paved	0.88	0.90	0.93	100%

Basin	Area (acres)	Commercial (acres)	Lawns (acres)	Walk (acres)	Streets (acres)	Runoff C_5	Runoff C_{10}	Runoff C_{100}	% Impervious
A1A	7.67	7.67	0.00	0.00	0.00	**0.87**	**0.88**	**0.89**	95.0%
A1B	12.40	11.64	0.76	0.00	0.00	**0.83**	**0.84**	**0.87**	89.2%
A1C	1.12	1.12	0.00	0.00	0.00	**0.87**	**0.88**	**0.89**	95.0%
A1D	1.51	0.00	0.47	0.19	0.85	**0.65**	**0.70**	**0.79**	68.4%
A1E	1.49	0.00	0.45	0.20	0.84	**0.66**	**0.70**	**0.79**	69.2%
A1F	3.27	3.27	0.00	0.00	0.00	**0.87**	**0.88**	**0.89**	95.0%
A1G	0.53	0.53	0.00	0.00	0.00	**0.87**	**0.88**	**0.89**	95.0%
A1H	5.84	5.84	0.00	0.00	0.00	**0.87**	**0.88**	**0.89**	95.0%
A1J	0.40	0.40	0.00	0.00	0.00	**0.87**	**0.88**	**0.89**	95.0%
A1K	0.83	0.83	0.00	0.00	0.00	**0.87**	**0.88**	**0.89**	95.0%
A1Z	1.85	0.00	0.49	0.13	1.23	**0.69**	**0.73**	**0.81**	73.2%

Time Of Concentration
Vista Ridge Commercial Parcels 32 & 33
Job Number: 2142-03

Sub-basin Data			Overland Flow			Travel Time in Channel					T_c Check		
Basin	C_5	Area (acres)	Length (ft)	Slope (%)	T_i (fig.3-1) (mins)	Length (ft)	Slope	Velocity (ft/sec) (fig.3-2)	T_t (mins)	T_c (mins)	Total Length (ft)	T_c = (L/180)+10 (mins)	Final T_c (min = 5 mins)
A1A	0.87	7.67	125	3.5	3.0	550	5.0%	4.5	2.0	5.1	675	13.8	**5.1**
A1B	0.83	12.40	93	14.0	2.0	950	2.4%	3.0	5.3	7.3	1043	15.8	**7.3**
A1C	0.87	1.12	-	-	-	-	-	-	-	-	-	-	**5.0**
A1D	0.65	1.51	30	2.0	3.5	1565	3.0%	3.4	7.7	11.2	1595	18.9	**11.2**
A1E	0.66	1.49	30	2.0	3.5	1580	3.0%	3.4	7.7	11.2	1610	18.9	**11.2**
A1F	0.87	3.27	120	3.5	3.0	330	5.0%	4.5	1.2	4.2	450	12.5	**5.0**
A1G	0.87	0.53	-	-	-	-	-	-	-	-	-	-	**5.0**
A1H	0.87	5.84	150	3.5	3.3	245	PIPE	5.0	0.8	4.2	395	12.2	**5.0**
A1J	0.87	0.40	-	-	-	-	-	-	-	-	-	-	**5.0**
A1K	0.87	0.83	-	-	-	-	-	-	-	-	-	-	**5.0**
A1Z	0.69	1.85	55	2.0	4.4	764	3.1%	3.4	3.7	8.1	819	14.6	**8.1**

$$T_i = \frac{1.8(1.1 - C_5)\sqrt{L}}{S^{0.33}}$$

Developed Runoff
Vista Ridge Commercial Parcels 32 & 33
Job Number: 2142-03

Basin	Total Area (acres)	C_5	C_{10}	C_{100}	T_c (mins)	Intensity			Design Flow		
						I_5 (in/hr)	I_{10} (in/hr)	I_{100} (in/hr)	Q_5 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
A1A	7.67	0.87	0.88	0.89	5.1	*5.00*	*6.00*	*9.35*	**33.36**	**40.49**	**63.81**
A1B	12.40	0.83	0.84	0.87	7.3	*4.43*	*5.38*	*8.40*	**45.37**	**56.13**	**90.21**
A1C	1.12	0.87	0.88	0.89	5.0	*5.00*	*6.00*	*9.40*	**4.87**	**5.92**	**9.37**
A1D	1.51	0.65	0.70	0.79	11.2	*3.65*	*4.45*	*6.85*	**3.59**	**4.67**	**8.19**
A1E	1.49	0.66	0.70	0.79	11.2	*3.65*	*4.45*	*6.85*	**3.58**	**4.65**	**8.11**
A1F	3.27	0.87	0.88	0.89	5.0	*5.00*	*6.00*	*9.40*	**14.21**	**17.24**	**27.32**
A1G	0.53	0.87	0.88	0.89	5.0	*5.00*	*6.00*	*9.40*	**2.32**	**2.82**	**4.46**
A1H	5.84	0.87	0.88	0.89	5.0	*5.00*	*6.00*	*9.40*	**25.41**	**30.84**	**48.86**
A1J	0.40	0.87	0.88	0.89	5.0	*5.00*	*6.00*	*9.40*	**1.73**	**2.10**	**3.33**
A1K	0.83	0.87	0.88	0.89	5.0	*5.00*	*6.00*	*9.40*	**3.60**	**4.37**	**6.93**
A1Z	1.85	0.69	0.73	0.81	8.1	*4.10*	*5.00*	*7.60*	**5.20**	**6.72**	**11.43**

Existing Conditions
Vista Ridge Commercial Parcels 32 & 33
Job Number: 2142-03

Land Use Characteristics

	C_5	C_{10}	C_{100}	Impervious
Undeveloped (Historic)	0.15	0.25	0.50	2%

Runoff Coefficents

Area (acres)	Undeveloped (acres)	Runoff Coefficients			Impervious
		C_5	C_{10}	C_{100}	
39.68	39.68	0.15	0.25	0.50	2.0%

Time of Concentration

		Overland Flow			Travel Time in Channel					T_c Check		
C_5	Area (acres)	Length (ft)	Slope (%)	T_i (mins)	Length (ft)	Slope	Velocity (ft/sec) (fig.3-2)	T_t (mins)	T_c (mins)	Total Length (ft)	T_c= (L/180)+10 (mins)	Final T_c (min = 5 mins)
0.25	39.68	100	4.0	9.6	1629	3.7%	2.0	13.6	23.2	1729	19.6	19.6

$$T_i = \frac{1.8(1.1 - C_5)\sqrt{L}}{S^{0.33}}$$

Existing Flows

Total Area (acres)	Weighted Runoff Coefficients			T_c (mins)	Intensity			Existing Flow		
	C_5	C_{10}	C_{100}		I_5 (in/hr)	I_{10} (in/hr)	I_{100} (in/hr)	Q_5 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
39.68	0.15	0.25	0.50	19.6	2.8	3.40	5.39	16.7	33.7	106.9

APPENDIX D
DETENTION POND ANALYSIS

Pond A1 Calculations
Vista Ridge
Job Number: 2142-3

% Impervious	
Commercial	95%
Lawns, Clay Soil	0%
Drives & Walks	96%
Streets, Paved	100%

Onsite Contributing Area (acres)	Commercial (acres)	Lawns (acres)	Walk (acres)	Streets (acres)	Impervious
22.96	18.78	2.11	0.39	1.69	0.87

Water Quality Capture Volume
Design Volume = (WQCV / 12) * Area * 1.2
$WQCV = a *(0.91 * i^3 - 1.19 * i^2 + 0.78 * i)$
Using a 40-hour drain time, a = 1.0

% Impervious	WQCV (inches)	Contributing Area (acres)	WQ Volume (acre-feet)	WQ Volume (c.f.)	WQ Release (cfs)
86.7%	0.37	22.96	**0.86**	37,471	**0.43**

From CUHP & UDSWM

10-Year Release Rate =	**8.0 cfs**	
100-Year Release Rate =	**27.0 cfs**	
10-Year Volume =	**2.40 ac-ft**	104,544 c.f.
100-Year Volume =	**3.70 ac-ft**	161,172 c.f.

Pond A1 Stage Storage
Vista Ridge
Job Number: 2142-3

Detention **Pond A1** - Stage - Storage Relationship

Elevation	Area (s.f.)	Incremental Volume (c.f.)	Total Volume (c.f.)	Total Volume (ac-ft)	
5223.0	0	0	0	0.00	Bottom of Pond
5224.0	10,685	5,343	5,343	0.12	
5225.0	29,006	19,846	25,188	0.58	
5225.42	**30,155**	**12,436**	**37,624**	**0.86**	*WQCV
5226.0	31,738	30,372	55,560	1.28	
5227.0	34,542	33,140	88,700	2.04	
5227.46	**34,491**	**15,844**	**104,544**	**2.40**	*10-Year Volume
5228.0	37,458	36,000	124,700	2.86	
5228.94	**40,233**	**36,476**	**161,176**	**3.70**	*100-Year Volume
5229.0	40,413	38,936	163,636	3.76	
5230.0	43,086	41,750	205,385	4.71	

Pond A1 Release Structure
Vista Ridge
Job Number: 2142-3

Pond A1
Water Quality Orifice Release
Orifice Equation: $Q = 0.60 * A * (2gh)^{1/2}$

WQCV Elevation	Pond Bottom	WQCV Release (cfs)	Effective Head (ft)	% Blockage	Orifice Area from Eqn. (in^2)	Orifice Box Height (inch)	Design Orifice Area (in^2)
5225.42	5223.00	0.43	2.23	50%	17.36	**4.5**	**17.3**

**Orifice Box is 3.85" X 4"(W X H)*

Pond A1
10-Year Orifice Release
Orifice Equation: $Q = 0.60 * A * (2gh)^{1/2}$

10-Year WSE	WQCV Elevation	10-Year Allowable Release (cfs)	WQ Orifice Release at 10-Yr WSE (cfs)	Allowable 10-Year Orifice Release (cfs)	Effective Head (ft)	% Blockage	Orifice Area from Eqn. (in^2)	Orifice Box Height (inch)	Design Orifice Area (in^2)
5227.46	5225.42	8.00	1.20	6.80	1.25	50%	364.38	**19**	**364.23**

**Orifice Box is 19.17" X 19" (W X H)*

Pond A1
100-Year Weir Release
Weir Equation: $Q = 2.61 * b * h^{3/2}$

100-Year WSE	10-Year WSE	100-Year Allowable Release (cfs)	WQ Orifice Release at 100-Yr WSE (cfs)	10-Year Orifice Release (cfs)	Allowable 100-Year Weir Release (cfs)	Effective Head (ft)	Max. Weir Length (ft)	Design Length (ft)
5228.94	5227.46	27.00	1.39	20.11	5.50	0.37	**9.36**	**9.30**

Pond A1
100-Year Spillway Release
Weir Equation: $Q = 2.61 * b * h^{3/2}$

100-Year WSE + Additional Head	100-Year WSE	100-Year Allowable Release (cfs)	Head (ft)	Max. Weir Length (ft)	Design Length (ft)
5230.00	5228.94	27.00	0.95	11.17	**11.00**

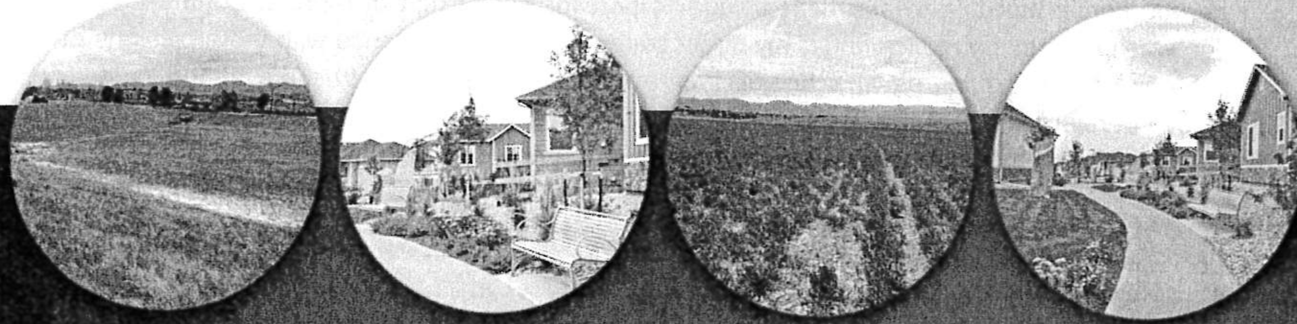
**Emergency Spillway would need to be11 feet long to pass the 100-Yr flow.*

Appendix B

Montex South at Vista Ridge Drainage Study

ENERTIA CONSULTING GROUP

prepared by
Enertia Consulting Group, LLC
1437 Larimer Street
Denver, CO 80202



Phase III Drainage Report

Montex South at Vista Ridge, Erie, Colorado

Revised February 23, 2016

prepared for

Chartered Development Corp
3160 Village Vista Drive
Erie, CO 80516

CHARTERED HOMES OF COLORADO

LAND • ENERGY • INFRASTRUCTURE

ENGINEER'S CERTIFICATION

"I hereby certify that this Phase III Drainage Report for the design of Montex South at Vista Ridge was prepared by me (or under my direct supervision) in accordance with the provisions of the *Town of Erie Standards and Specifications for Design and Construction* for the owners thereof. I understand that the Town of Erie does not and will not assume liability for drainage facilities designed by others, including the designs presented in this report."

____
Shawn C. Merz, PE
State of Colorado Registration No. 41241
For and on Behalf of Enertia Consulting Group

TOWN ACCEPTANCE

This report has been reviewed and found to be in general compliance with the *Town of Erie Standards and Specifications for Design and Construction* and other Town requirements. **THE ACCURACY AND VALIDITY OF THE ENGINEERING DESIGN, DETAILS, DIMENSIONS, QUANTITIES, AND CONCEPTS IN THIS REPORT REMAINS THE SOLE RESPONSIBILITY OF THE PROFESSIONAL ENGINEER WHOSE STAMP AND SIGNATURE APPEAR HEREON.**

Accepted by: ________________________________ Date: ______________________
Deputy Public Works Director

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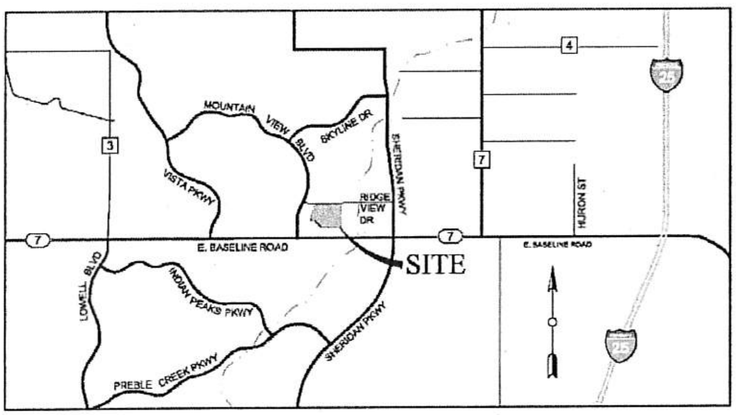
1. GENERAL LOCATION AND DESCRIPTION

A. Site Location

The project site is the 1st Amendment of Vista Ridge Filing No. 12 and is located in the Southwest Quarter of Section 33, Township 1 North, Range 68 West of the 6th Principle Meridian. The project is bounded to the north by Ridge View Drive, future commercial development to the east and south, and Detention Pond A1the west. The adjacent major roadways are Mountain View Boulevard to the west, Sheridan Parkway to the east and East Baseline Road to the south.

Montex at Vista Ridge Vicinity Map



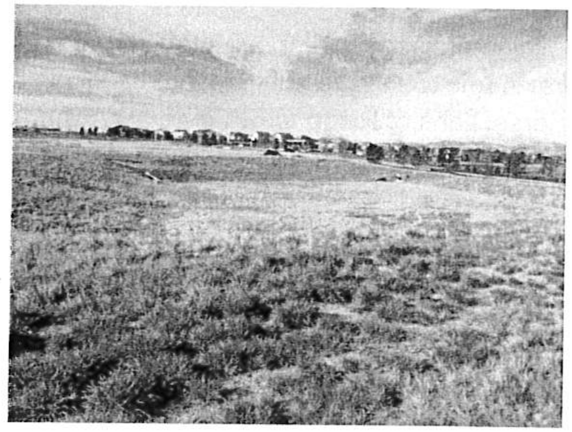
B. Description of Property

The proposed site consists of 10.2 acres. The site is gently sloping from east to west with an existing slope of roughly 3.2%. Existing ground cover consists of natural grasses. There are no wetlands on the proposed site. The developed parcel will consist of 36 four dwelling manor homes and a single-family home which will be used as a rental office. A total of 144 units results in a density of 14.2 dwelling units/acre. There is an existing 10' utility easement along the north property line as well as a 10' United Power Easement running north south through the middle of the property which will be vacated. There is also a pocket utility easement along the west property line provided for the drainage stub to the site. The ALTA Survey for the site has been included in Appendix C.

2. DRAINAGE BASINS

A. Major Basin Description

The project is located within the FEMA Floodplain Panel 08013C0444J. This panel was not printed by FEMA. The FIRM Index notes this panel as having "*NO SPECIAL FLOOD HAZARD AREAS IDENTIFIED". Therefore the project is clear of any floodplain hazards. The project will discharge to a regional detention pond (Pond A1) west of the site and east of Mountain View Boulevard. This detention pond has a 100-year water surface elevation of 5228.94. The lowest elevation of the site is 5237.3 on the west side of the property. The pond is owned and maintained by the Vista Ridge Metro District. The existing site is not irrigated. Currently the proposed site is vacant. This project will develop the infrastructure for 144 multi-family residential homes and a rental office. Water quality for this project is provided in Pond A1.



B. Sub-Basin Descriptions

Basin A1-A9

Basins A1-A9 are located along the southern portion of the property and currently is comprised of native grasses. In the developed condition, these basins will consist of multi-family roof drainage, landscaped open space areas sidewalks, and roadways. Open space drainage will be captured in area drains and roadways will be captured in inlets. 10' and 15' on-grade Type R inlets have been provided at the southwest corner of Ridge View Circle. A 5' Type R sump inlet has also been provided at the low point between Buildings 6 and 7. Additional 12" and 15" Nyloplast area drains will be provided in open space areas. The water will be conveyed in a storm drainage system (identified on Drainage Maps as Storm System 1) which will tie to a storm sewer stub south of Building 7. Basin A9 cannot be captured in the proposed onsite storm drainage system and will drain south to a drainage swale. All of basins A1-A9 will end up in Detention Pond A1. The area, imperviousness, minor event runoff and major event runoff are listed below:

Basin	Area (acres)	Imperviousness (%)	Q_2	Q_{100}
A1	0.41	74	0.73	2.60
A2	0.09	100	0.27	0.78
A3	0.58	74	1.00	3.53
A4	0.13	71	0.22	0.82
A5	0.95	78	1.85	6.31
A6	0.26	76	0.48	1.67
A7	0.29	67	0.46	1.75
A8	0.17	57	0.22	0.95
A9	0.18	0	0.02	0.82

Basin B1-B23

The B1-B-23 basins are located along the northern and eastern portion of the property and currently are comprised of native grasses. In the developed condition, these basins will consist of multi-family roof drainage, landscaped open space areas sidewalks, and roadways. The building drainage will be collected in roof drains. Open space drainage will be captured in area drains and roadways will be captured in inlets. Type R inlets have been provided at the intersections of Ridge View Circle and Ridge View Drive. 5' Type R sump inlets are provided at low points between Buildings 22-26 and 26-29. A Type C inlet is provided at the low point between Buildings 2-3. Additional 12" and 15" Nyloplast area drains will be provided in open space areas. The water will be conveyed in a storm drainage system (identified on Drainage Maps as Storm System 2) which will discharge into the northeast corner of Detention Pond A1. Basins 22 and 23 cannot be captured in the proposed onsite storm drainage system and will drain north to the Ridge View Drive. The water will be conveyed in the gutter pan to the existing inlets in Ridge View Drive and to Pond A1. The area, imperviousness, minor event runoff and major event runoff are listed below:

Basin	**Area (acres)**	**Imperviousness (%)**	**Q_2**	**Q_{100}**
B1	0.21	46	0.31	0.59
B2	0.31	63	0.43	0.65
B3	0.12	76	0.55	0.71
B4	0.31	64	0.44	0.65
B5	0.12	83	0.63	0.77
B6	0.31	63	0.43	0.65
B7	0.19	52	0.35	0.61
B8	0.55	74	0.53	0.70
B9	0.33	77	0.56	0.72
B10	0.13	72	0.51	0.69
B11	0.18	100	0.89	0.96
B12	0.26	72	0.51	0.69
B13	0.56	72	0.51	0.69
B14	0.19	87	0.68	0.80
B15	0.17	43	0.29	0.59
B16	0.14	100	0.89	0.96
B17	1.23	56	0.37	0.62
B18	0.27	80	0.59	0.74
B19	0.11	82	0.62	0.76
B20	0.13	71	0.50	0.68
B21	0.38	84	0.78	0.88
B22	0.09	17	0.15	0.55
B23	0.18	58	0.40	0.63

Basin C1-C2

Basin C1 consists of 0.43 acres and currently is comprised of native grasses. In the developed condition, this basin will consist of multi-family roof drainage landscaped open space. The imperviousness of Basin C1 is anticipated to be 92% which results in Q_2=1.13 cfs and Q_{100}=3.40 cfs. The building drainage will be collected in roof drains. Runoff from the private drive and alley will be captured by a 5' Type R sump inlet located between Buildings 6 and 7. The captured water is conveyed in a proposed storm sewer system (identified on Drainage Maps as Storm System 3) with will be connected to the existing storm sewer system which discharges to the southeast corner of Pond A1.

Basin C2 consists of 0.26 acres and currently is comprised of native grasses. In the developed condition, this basin will consist of landscaped open space. The imperviousness is anticipated to be 30% which results in Q_2=0.20 cfs and Q_{100}=1.34 cfs. This water cannot be capture in the proposed onsite storm drainage system and will drain west directly into Pond A1.

Basin OS1

Basin OS1 consists of 10.46 acres and currently is comprised of native grasses. This basin will ultimately be developed commercial area. In the interim, the imperviousness is anticipated to be 5% which results in Q_2=1.83 cfs and Q_{100}=32.94 cfs. This water will be captured in a swale and conveyed to flared end section which discharges to the storm drainage system in Ridge View Drive. An emergency overflow to Ridge View Drive has been provided.

Basin OS2

Basin OS2 consists of 8.48 acres and currently is comprised of native grasses. This basin will ultimately be developed commercial area. In the interim, the imperviousness is anticipated to be 5% which results in Q_2=1.42 cfs and Q_{100}=25.56 cfs. This water will be captured in a swale and conveyed to an existing Type 13 inlet and conveyed to Pond A1.

Basin OS3

Basin OS3 consists of 0.67 acres and is comprised of Ridge View Drive right-of-way. A 10' on-grade Type R inlet has been added to prevent offsite flow from entering the proposed drainage system. The imperviousness is 56% which results in Q_2=0.78 cfs and Q_{100}=3.4 cfs.

3. DRAINAGE DESIGN CRITERIA

A. Development Criteria Reference and Constraints

The site parcel is identified as Parcel 32 and 33 in the Vista Ridge Master Drainage Report. Since the master drainage report was completed, the property boundaries have been reconfigured to accommodate the Montex South Multi-Family Residential Development. To determine the anticipated allotted runoff from the proposed development, the Montex South boundary was overlaid onto the Parcel 32-33 Master Drainage Plan map. 98.6 percent of Basin A1H is located on the Montex South Site. The remaining 1.4 percent of the basin is located within the detention pond. The calculated runoff from basin A1H is 48.86 cfs. Therefore, 98.6 percent of 48.86 results in 48.17 cfs of anticipated runoff available for the Montex South Development.

Basin A1F is 3.27 acres. 67.6 percent of Basin A1F is within the Montex South Boundary. Using the same methodology as above results in 18.46 cfs of runoff available from Basin A1F for the Montex South Development.

Basin A1B has been reduced in size. 2.43 acres of Basin A1B will now be discharged to Pond A1. Therefore, the size of Pond A1A can be reduced in size. Pond A1A will be designed by the the master developer and is not part of this report.

The resulting anticipated discharge from Basin A1H and A1F for the Montex South Development is 66.63 cfs.

An existing storm sewer stub in Ridge View Drive has been extended east to capture the offsite swale drainage. This stub was originally intended to be used for Pond A1A and will be used for the future pond outlet when Basin OS3 is developed. This pipe was not sized to accommodate developed flow. This 18" RCP stub has been installed at 2.2% which can accommodate 15.6 cfs based on Manning's Formula. Basin OS1 is approximately 10.5 acres. Assuming a release rate of 1.0 cfs per acre and adding the 100-year flow from Basin OS3 (3.4 cfs) the total flow in the stub is anticipated to be 13.9 cfs. In the interim condition if water exceeds the capacity of the pipe or if the pipe plugs, water will be conveyed to Ridge View Drive.

B. Hydrological Criteria

Basin Runoff has been calculated using criteria from the Town of Erie *"STANDARDS AND SPECIFICATIONS FOR DESIGN AND CONSTRUCTION OF PUBLIC IMPROVEMENTS, 2014 Edition"* and Urban Drainage and Flood Control District (UDFCD). The design storm return periods for residential land use are 2-year for the initial storm and 100-year for the major storm. Imperviousness values were selected using Table 800-3. One-hour rainfall depths of 1.01 for the 2-year design storm and 2.70 for the 100-year design storm were used to calculate the intensities using Urban Drainage equation RA-3. Using the rational method, runoffs for each basin were determined. Time of concentrations were calculated using the UDFCD SF2 form. The hydrologic calculations are located in Appendix B.

C. Hydraulic Criteria

Detention and water quality for the site is provided in Pond A1. The maximum allowable runoff from the project may not exceed 66.6 cfs as outlined in section 3A above. During the 2-year event, the depth of flow for local roads may not overtop the curb and may extend to the crown of the road. Residential buildings will be designed to be 12-inches above the 100-year water surface elevation at the lowest building entry or ground line where adjacent stem walls are provided. The water depth may not exceed 18-inches at the gutter flowline. The storm drainage system will be designed to convey the 100-year storm event.

D. Adaptations from Criteria

No adaptions are requested at this time.

4. DRAINAGE FACILITY DESIGN

A. General Concept

The site has been graded with a high point at the southeast corner of the site. The roadway has been graded to allow water to be conveyed to Ridge View Drive or the pond should any inlets become plugged. Failure of inlets in parking areas will spill to swales and open space area drains.

The proposed drainage system is made up of 3 separate storm sewer systems. The first system captures the 100-year runoff from the A designated basins along the southern end of the site. This system will tie to an existing Type 13 inlet which discharges to Pond A1. The existing storm sewer system was sized to allow for 27.32 cfs from Basin A1F in the master drainage report of which 18.46 cfs can be contributed from Montex South.

The second storm sewer system runs along the north portion of the site and captures basins which are designated with a B. This system will capture and convey the 100-year storm event and discharge to the northeast corner of Pond A1.

The third storm sewer system picks up a Type R sump inlet between Building 6 and 7. This system will connect to the stub location provided for the site in the master drainage report near the southwest corner of Pond A1 and can accommodate the 100-year storm event.

The discharge from storm sewer system 1 (A Basins) cannot exceed 18.46 cfs allotted from the basin A1F in the master drainage study. The combined discharge from storm sewer system 2 (B Basins) and storm sewer system 3 (C Basins) will not exceed 48.17 cfs which was allotted from basin A1H in the master drainage study.

All downspouts will be captured in a pipe and swales have been provided between the buildings to convey open space drainage to 12" and 15" Nyloplast area inlets located throughout the site. The buildings have been grading with a minimum grade of 10% away from the building for 10' (5' where 10' could not be achieved). The swales have been graded with a minimum slope of 2%. Should the area drain inlets become plugged, emergency overflow paths have been provided to ensure the buildings are not flooded.

Offsite swales will be provided to divert drainage from Basin OS1 and OS2 around the Montex South Site. A 4 foot bottom, 4:1 side slopes and a water surface depth of 1.0' will accommodate flow from Basin OS1 at a slope of 1.5% (Ridge View Circle eastern roadway slope). A 4 foot bottom, 4:1 side slopes and a water surface depth of 1.0' will accommodate flow from Basin OS2 at a slope of 1.5% (Ridge View Circle southern roadway slope). Calculations are provided in Appendix B.

B. Specific Details

The inlet and street capacities were sized using Urban Drainage and Flood Control District Street Capacity and Inlet Sizing spreadsheet. Types R on-grade and sump inlets are provided to capture runoff and maintain street capacities during the initial and major storm. The inlets have been spaced to meet the Town of Erie Design Criteria. Due to the roadway cross section, the water depth has been limited to 6-inches during a major event. 10' on-grade Type R inlets have been provided at the west intersection of Ridge View Drive and Ridge View Circle. 5' sump Type R inlets have been provided at the east intersection of Ridge View Drive and Ridge View Circle. Additional 10' and 15' on-grade Type R inlets have been provided at the southwest corner of the site near Building 32. 12" and 15" area drain sump inlets with a minimum sump depth of 0.5' have been provided throughout the site to capture additional runoff. According to the Nyloplast 12" Standard Grate Inlet Capacity Chart, the 12" area drain inlet in a sump with a ponding depth of 0.5' has a capacity of 1.4 cfs. A 15" area drain inlet in a sump can capture 2.2 cfs. Several of these area drains are provided in each open space drainage basin which can easily accommodate the major storm basin flow. The UDFCD street capacity and inlet sizing spreadsheet results for the Type R inlets have been provided in Appendix B. Type C inlets were sized with the capacity chart provided in Appendix B. A capacity chart for the 12" and 15" Nyloplast area drain inlets have also been provided. 8" HDPE piping with a minimum slope of 1.5% and 12" HDPE with a minimum slope of 1% was used for the area drain system.

The storm sewer system has been sized for the 100-year storm event using the AutoCAD Civil 3D Hydraflow Storm Sewer Extension which uses the Rational Method and intensity duration frequency curves to calculate flow and design the appropriate pipe sizes and calculate the hydraulic grade lines.

The peak runoff from Storm Sewer System 1 (Basin A) is 15.64 cfs at design point 6. Combining this flow with the runoff from Basin A9 (0.82 cfs), which is not captured in the storm sewer system, results in a peak Basin A runoff of 16.46 cfs. This is less than the 18.46 cfs allowed for Montex South contributing area of master drainage basin A1F.

Storm Sewer System 2 (Basin B) peak discharge at design point 23 is 26.86 cfs. Storm Sewer System 3 (Basin C) peak discharge at design point 24 is 3.76 cfs. Basin B22 (0.47 cfs), Basin B23 (1.03 cfs) and Basin C2 (1.34 cfs) are not captured in the storm sewer system. Adding the design points 23, 24, Basin B22, B23, and C2 results in a total discharge of 33.46 cfs. This discharge is less than the 48.17 cfs which is allowed for the Montex South contributing area master drainage basin A1H. Therefore, the site discharge is in line with the runoff allowed for the Pond A1 design in the master drainage report.

The storm sewer pipes have been sized for the 100-year event using Hydraflow Express Extension for AutoCAD Civil 3D 2013 with a manning's roughness coefficient of 0.13 for RCP pipe. Flow and HGEL calculation sheets are provided in Appendix B.

Emergency Spillway overflow was analyzed for between Buildings 2-3, 6-7, 22-26, 26-29 and the OS3 overflow to Ridge View Drive. The street capacities and spillways between Buildings 6-7 were analyzed assuming a plugged storm sewer system upstream which yields 14.9 cfs of runoff from Basins A4, A5, A6, A7 and A8.

Blanket drainage easements have been provided on the site for drainage conveyance.

5. SUMMARY

The proposed Montex South Multi-Family Residential Development will fall within the drainage guidelines outlined in the Town of Erie Standards and Specification as well as the previous drainage studies. The proposed development will construct a storm sewer system to convey the 100-year runoff to the existing Pond A1 detention facility. No adverse impacts are anticipated to the existing detention facilities.

6. REFERENCES

STANDARDS AND SPECIFICATIONS FOR DESIGN AND CONSTRUCTION OF PUBLIC IMPROVEMENTS, Town of Erie, Colorado, 2014 Edition.

URBAN STORM DRAINAGE CRITERIA MANUAL, VOLUME 1, 2 & 3., UDFCD, Denver, Colorado, Revised April 2008.

DRAINAGE REPORT, VISTA RIDGE PARCELS 32 & 33, Hurst and Associates, Inc, Revised September 23, 2008.

APPENDIX A

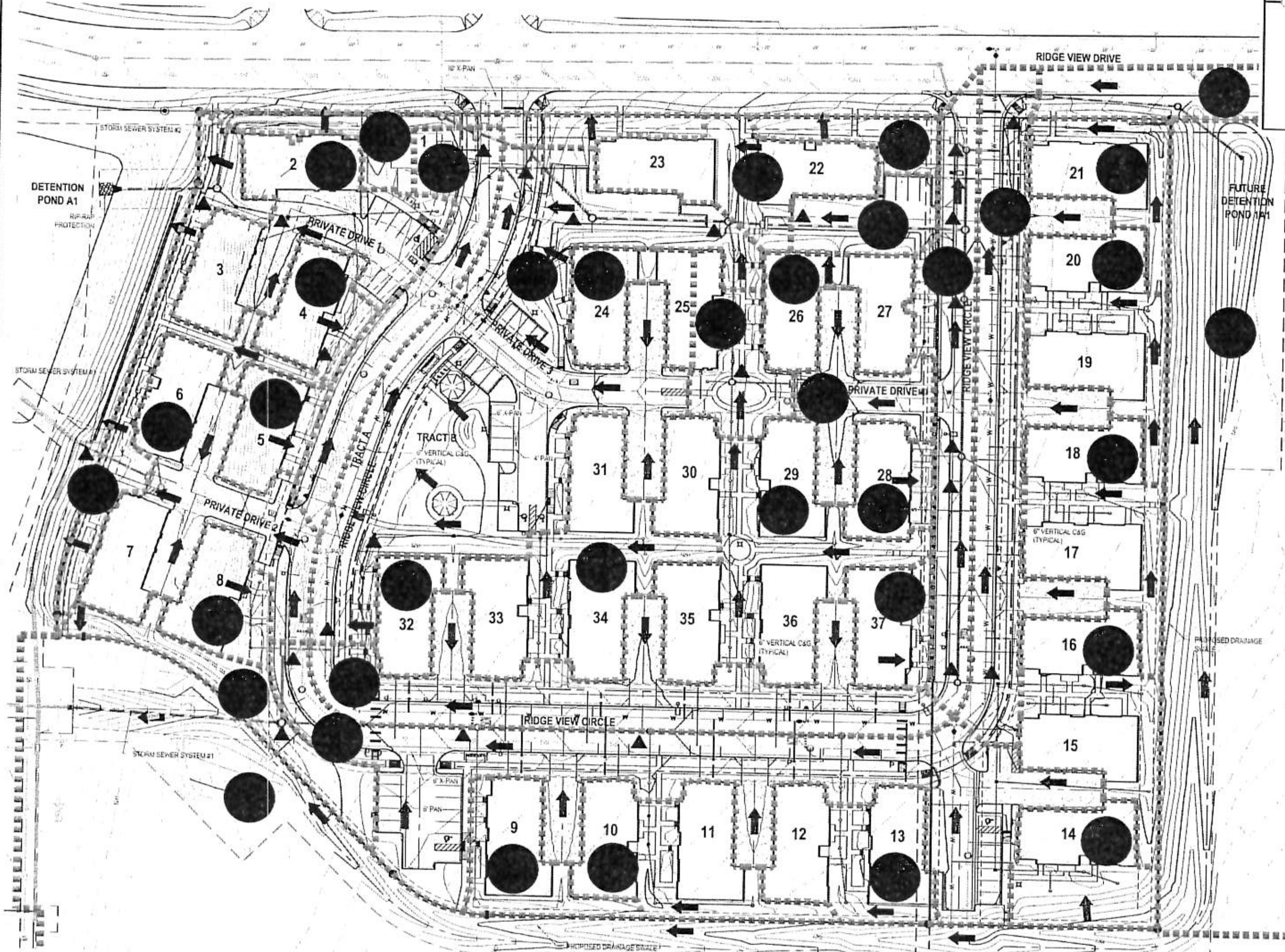
Drainage Map

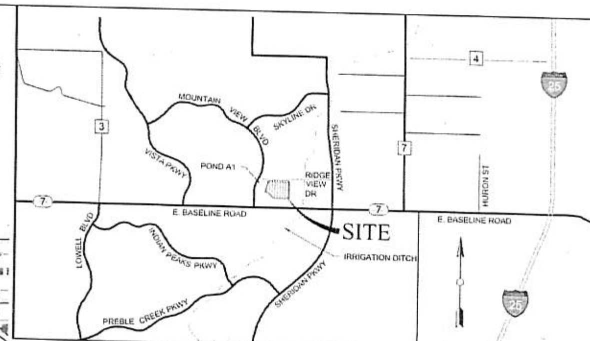
VISTA RIDGE FILING NO. 12, 2ND AMENDMENT

A PORTION OF TRACT A, VISTA RIDGE FILING NO. 12 AND A PORTION OF PARCEL 33, VISTA RIDGE MASTER FINAL PLAT: LOCATED IN THE SOUTHWEST QUARTER OF SECTION 33, TOWNSHIP 1 NORTH, RANGE 68 WEST OF THE 6TH PRINCIPLE MERIDIAN, TOWN OF ERIE, COUNTY OF WELD, STATE OF COLORADO

10.196 ACRES - 37 LOTS / 5 TRACTS

FP-000713-2015





GENERAL LOCATION MAP

SCALE 1"=2000'

LEGEND

- PROPOSED CONDITION DRAINAGE BOUNDARY
- EXISTING CONTOUR
- PROPOSED CONTOUR
- EXISTING STORM DRAIN SYSTEM
- PROPOSED STORM DRAIN SYSTEM
- EXISTING DRAINAGE FLOW ARROW
- PROPOSED DRAINAGE FLOW ARROW
- BASIN IDENTIFICATION / IMPERVIOUSNESS / AREA (ACRES)

DESIGN POINT SUMMARY TABLE

DESIGN POINT	Q_5 (CFS)	Q_{100} (CFS)
1	0.64	3.51
2	1.03	6.82
3	3.19	13.56
4	0.58	2.97
5	0.64	3.02
6	2.61	14.70
7	0.40	2.69
8	0.53	3.38
9	0.70	4.78
10	0.82	5.46
11	1.00	6.85
12	0.89	4.16
13	1.52	6.07
14	1.86	12.33
15	2.13	14.02
16	1.38	5.77
17	3.02	19.89
18	3.15	20.69
19	3.75	25.45
20	4.00	26.77
21	0.36	1.50
22	0.96	4.12
23	4.47	29.76
24	1.11	3.47

BASIN RUNOFF SUMMARY TABLE

BASIN	AREA (AC)	Q_5 (CFS)	Q_{100} (CFS)
A1	0.41	0.73	2.60
A2	0.09	0.27	0.76
A3	0.56	1.00	3.53
A4	0.13	0.22	0.82
A5	0.66	1.65	6.31
A6	0.26	0.48	1.67
A7	0.29	0.48	1.75
A8	0.17	0.22	0.95
A9	0.16	0.02	0.82
B1	0.21	0.23	1.15
B2	0.31	0.45	1.81
B3	0.12	0.23	0.60
B4	0.31	0.46	1.82
B5	0.12	0.25	0.81
B6	0.31	0.45	1.51
B7	0.19	0.22	1.03
B8	0.55	0.88	3.12
B9	0.33	0.59	2.03
B10	0.13	0.22	0.81

BASIN RUNOFF SUMMARY TABLE (CONT.)

BASIN	AREA (AC)	Q_5 (CFS)	Q_{100} (CFS)
B11	0.18	0.56	1.61
B12	0.26	0.46	1.65
B13	0.56	0.88	3.53
B14	0.19	0.44	1.37
B15	0.17	0.17	0.89
B16	0.14	0.43	1.24
B17	1.23	1.19	5.26
B18	0.27	0.55	1.85
B19	0.11	0.24	0.79
B20	0.13	0.22	0.82
B21	0.30	1.03	3.08
B22	0.09	0.05	0.47
B23	0.18	0.24	1.03
C1	0.43	1.13	3.40
C2	0.26	0.20	1.34
OS1	10.46	1.83	32.94
OS2	8.48	1.42	35.56
OS3	0.67	0.78	3.40

MONTEX SOUTH AT VISTA RIDGE PHASE 3 DRAINAGE MAP

N

SCALE: 1"=40'

APPLICANT:
CHARTERED DEVELOPMENT CORP.
3160 VILLAGE VISTA DRIVE, SUITE 104
ERIE, COLORADO 80516
303-545-2554

ENGINEER:
ENERTIA CONSULTING GROUP
1437 LARIMER STREET
DENVER, COLORADO 80202
303-875-7131

DRAWN BY: SM
APRVD BY: SM
DATE: 12/5/2014
REVISED: 6/23/2015
2/9/2016

SHEET 1 OF 2

VISTA RIDGE FILING NO. 12, 2ND AMENDMENT

A PORTION OF TRACT A, VISTA RIDGE FILING NO. 12 AND A PORTION OF PARCEL 33, VISTA RIDGE MASTER FINAL PLAT:
LOCATED IN THE SOUTHWEST QUARTER OF SECTION 33, TOWNSHIP 1 NORTH, RANGE 68 WEST
OF THE 6TH PRINCIPLE MERIDIAN, TOWN OF ERIE, COUNTY OF WELD, STATE OF COLORADO
10.196 ACRES - 37 LOTS / 5 TRACTS
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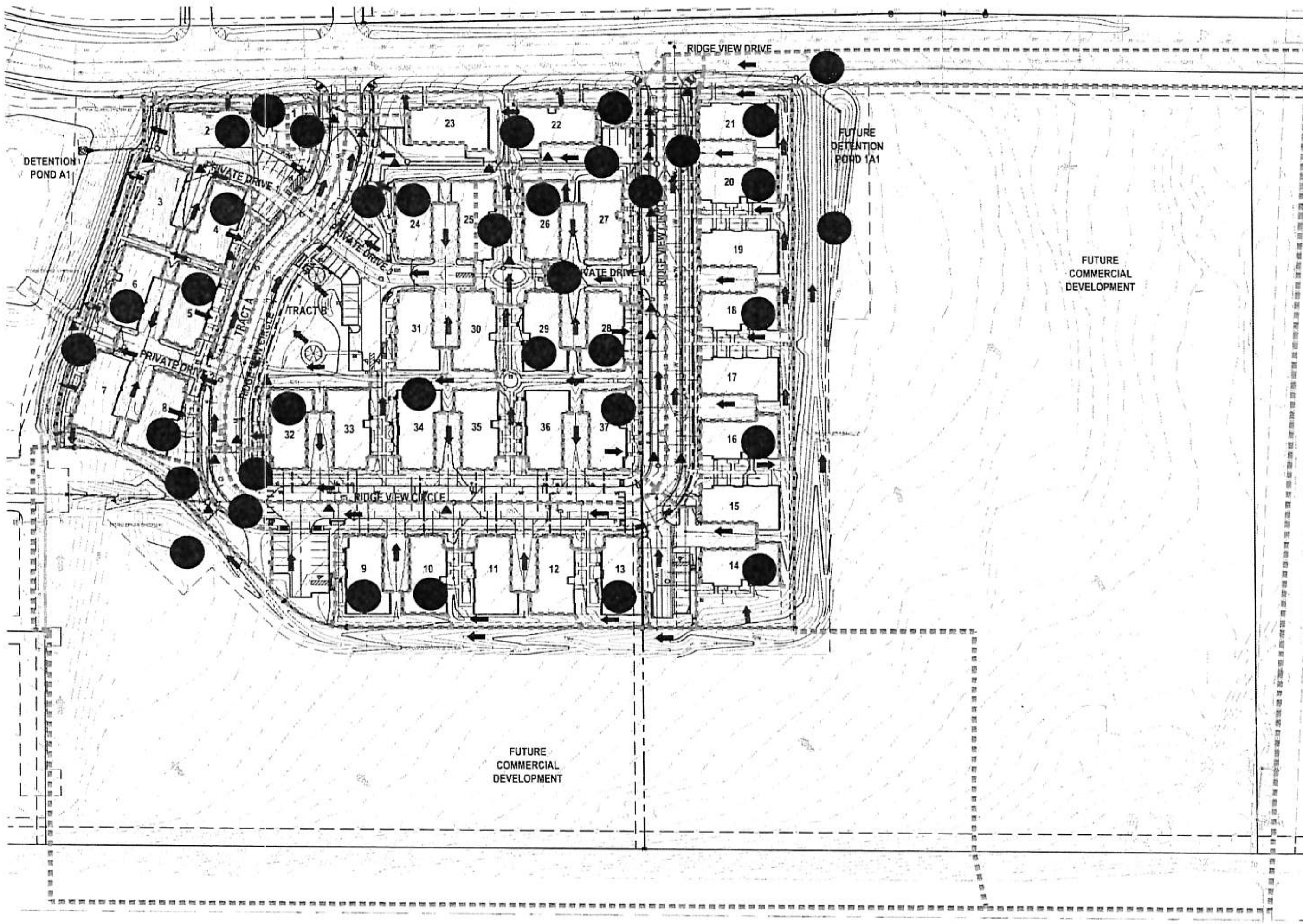
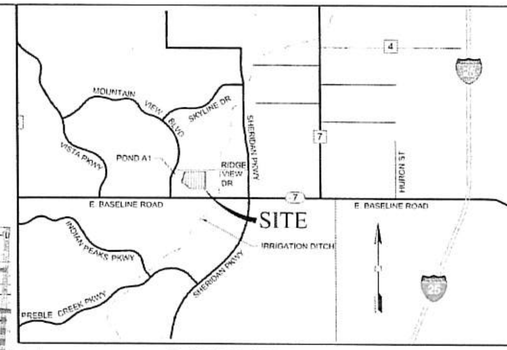




GENERAL LOCATION MAP

SCALE 1"=2000'

LEGEND

PROPOSED CONDITION DRAINAGE BOUNDARY
EXISTING CONTOUR
PROPOSED CONTOUR
EXISTING STORM DRAIN SYSTEM
PROPOSED STORM DRAIN SYSTEM
EXISTING DRAINAGE FLOW ARROW
PROPOSED DRAINAGE FLOW ARROW
BASIN IDENTIFICATION
IMPERVIOUSNESS
AREA (ACRES)

MONTEX SOUTH AT VISTA RIDGE
PHASE 3 OVERALL DRAINAGE MAP

APPLICANT:
CHARTERED DEVELOPMENT CORP
3185 VILLAGE VISTA DRIVE, SUITE 104
ERIE, COLORADO 80516
303-545-2554

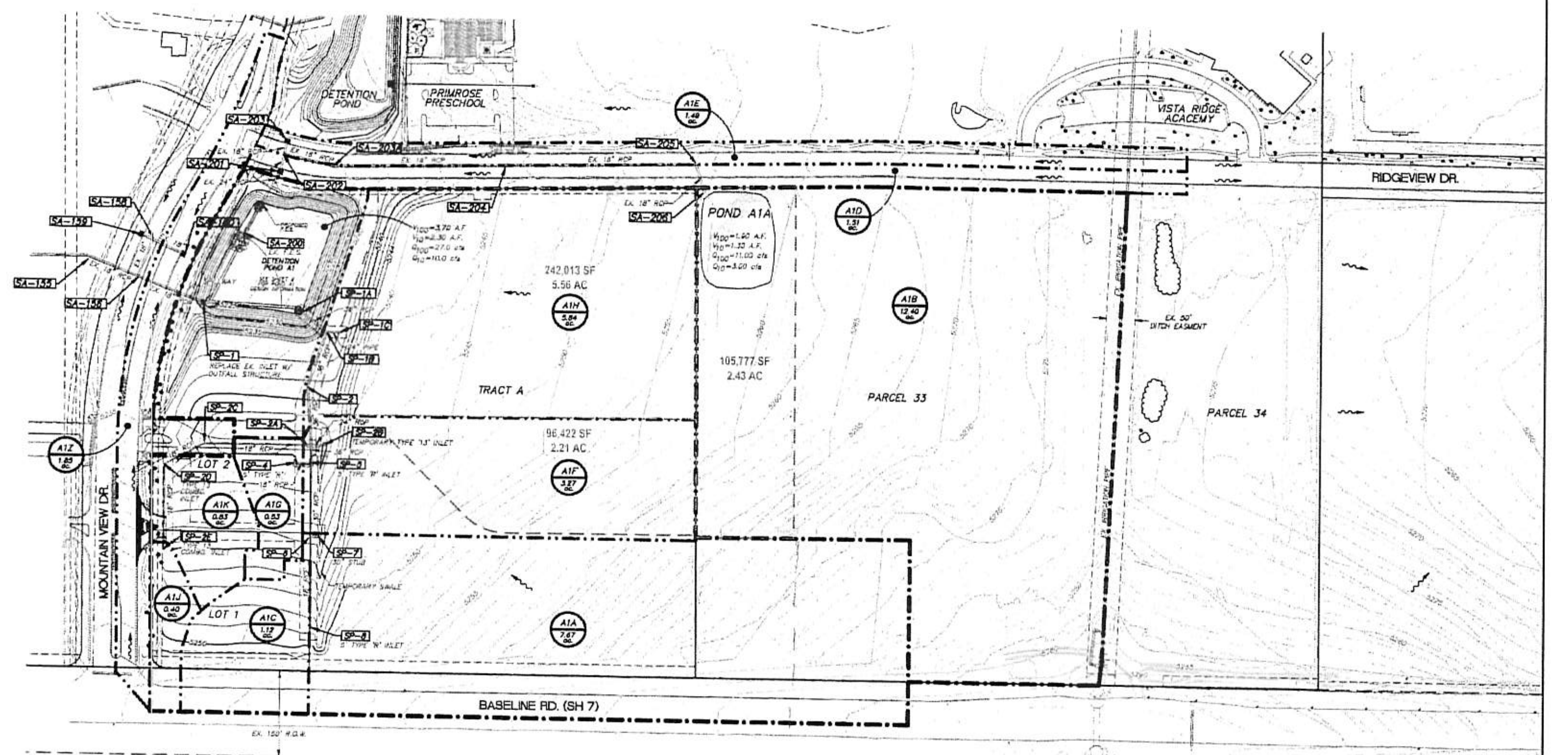
ENGINEER:
ENERTIA CONSULTING GROUP
1437 LARIMER STREET
DENVER, COLORADO 80202
303-875-7131

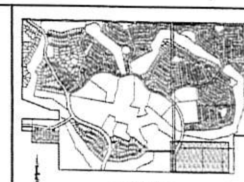
SCALE: 1"=60'

DRAWN BY	SM
APRVD BY	SM
DATE:	12/5/2014
REVISED	6/23/2015
	2/9/2016
SHEET	2 OF 2

Basin	Total Area (acres)	Design Flow Q_5 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
A1A	7.67	33.36	40.49	63.81
A1B	12.40	45.37	56.13	90.21
A1C	1.12	4.87	5.92	9.37
A1D	1.51	3.59	4.67	8.19
A1E	1.49	3.58	4.65	8.11
A1F	3.27	14.21	17.24	27.32
A1G	0.53	2.32	2.82	4.46
A1H	5.84	25.41	30.84	48.86
A1J	0.40	1.73	2.10	3.33
A1K	0.83	3.60	4.37	6.93
A1Z	1.85	5.20	6.72	11.43

Design Point	5-Yr. Design Flow (cfs)
SP-1A	79.51
SP-2A	57.69
SP-2C	5.33
SP-5	39.47
SP-6	38.23





VICINITY MAP
SCALE: 1"=2000'

LEGEND

- Manhole
- Existing Manhole
- 5' Type 'R' Inlet
- 10' Type 'R' Inlet
- 15' Type 'R' Inlet
- Flared End Section
- Storm Pipe
- Existing Storm Pipe
- Existing Contours
- Proposed Contours
- Future Contours
- Master Basin Boundary
- Basin Boundary
- Flow Arrow
- S-28 Structure Number
- A1C 8.61 ac. Basin Designation

NOTE: ALL INLETS ARE 5' TYPE 'R' UNLESS OTHERWISE NOTED

Note: All proposed storm sewer is privately owned and maintained

GRAPHIC SCALE
(IN FEET)
1 inch = 100 ft.

72 HOURS BEFORE YOU DIG
CALL THE UTILITY NOTIFICATION CENTER OF COLORADO (U.N.C.C.)
811
GAS, ELECTRIC, TELEPHONE, CATV AND PANHANDLE EASTERN PIPELINE LOCATIONS

LAST REVISED: 01/26/2009

VISTA RIDGE

MASTER DRAINAGE PLAN
PARCEL 32 – 33

HURST & ASSOCIATES, INC
CONSULTING ENGINEERS

APPENDIX B

Calculations

Master Study Allowable Runoff					
Basin	Basin Area (acres)	Montex South Contribution Area (acres)	% of Basin	Master Study Q_{100} (cfs)	Montex Allowable Runoff Q_{100} to Ponds* (cfs)
A1B	12.40	2.43	19.6%	90.21	**17.68**
A1F	3.27	2.21	67.6%	27.32	**18.46**
A1H	5.64	5.56	98.6%	48.86	**48.17**

* Basin A1F & A1H discharge to Pond A1. A total of 66.63 cfs is allowed to discharge to Pond A1 from the Montex South Site. The entire Montex South Site drains to Pond A1, therefore, runoff to A1A from Basin A1B has been reduced by approximately 17.68 cfs.

ENERTIA CONSULTING GROUP

CALCULATED BY:	SCM
DATE:	18-Jun-15

Montex South at Vista Ridge
Proposed Conditions Imperviousness Calculations

Basin ID	Total Basin Area (sf)	Pervious Area (sf)	Imp. Area (sf)	Imperviousness (%)
A1	17,671	4,636	13,035	**74%**
A2	3,856	0	3,856	**100%**
A3	25,096	6,510	18,586	**74%**
A4	5,729	1,679	4,050	**71%**
A5	41,340	9,204	32,136	**78%**
A6	11,186	2,729	8,457	**76%**
A7	12,540	4,082	8,458	**67%**
A8	7,296	3,133	4,163	**57%**
A9	7,814	7,814	0	**0%**
B1	9,181	4,951	4,230	**46%**
B2	13,323	4,911	8,412	**63%**
B3	5,361	1,312	4,049	**76%**
B4	13,323	4,810	8,513	**64%**
B5	5,019	857	4,162	**83%**
B6	13,323	4,911	8,412	**63%**
B7	8,096	3,880	4,216	**52%**
B8	23,890	6,200	17,690	**74%**
B9	14,470	3,379	11,091	**77%**
B10	5,550	1,540	4,010	**72%**
B11	8,027	0	8,027	**100%**
B12	11,369	3,175	8,194	**72%**
B13	24,294	6,711	17,583	**72%**
B14	8,148	1,075	7,073	**87%**
B15	7,236	4,151	3,085	**43%**
B16	6,184	0	6,184	**100%**
B17	53,675	23,843	29,832	**56%**
B18	11,745	2,366	9,379	**80%**
B19	4,942	904	4,038	**82%**
B20	5,742	1,688	4,054	**71%**
B21	16,754	1,056	15,698	**94%**
B22	4,129	3,445	684	**17%**
B23	7,820	3,249	4,571	**58%**
C1	18,808	1,424	17,384	**92%**

ENERTIA CONSULTING GROUP

CALCULATED BY: SCM
DATE: 18-Jun-15

Montex South at Vista Ridge
Proposed Conditions Imperviousness Calculations

Basin ID	Total Basin Area (sf)	Pervious Area (sf)	Imp. Area (sf)	Imperviousness (%)
C2	11,220	7,846	3,374	**30%**
OS1	455,444			**5%**
OS2	369,348			**5%**
OS3	29,330	12,786	16,544	**56%**

Total Onsite Imperviousness (%) = *69%*

STANDARD FORM SF-2
TIME OF CONCENTRATION

CALCULATED BY: SCM
DATE: 18-Jun-15
CHECKED BY: JMM

PROJECT: Montex South at Vista Ridge
JOB NUMBER:

SUB-BASIN DATA					INITIAL/OVERLAND TIME (T_i)			TRAVEL TIME (T_t)					Tc CHECK (URBANIZED BASINS)			FINAL T_c	REMARKS							
DESIGN: (1)	AREA SF	AREA IMP. SF	AREA Ac (2)	C_5 (3)	LENGTH Ft (4)	SLOPE % (5)	T_i (6)	LENGTH Ft (7)	SLOPE % (8)	C_V (9)	VEL. FPS (10)	T_t (11)	COMP. T_c (12)	TOTAL LENGTH (13)	T_c=(L/180)+10 MIN (14)	MIN (15)		C_2	C_5	C_{100}	K_2	K_5	K_{100}	Imperv. %
A1	17,671	13,035	0.41	0.56									5.0	0	10.0	5.0		0.53	0.56	0.70	0.00	0.04	0.17	74%
A2	3,856	3,856	0.09	0.90									5.0	0	10.0	5.0		0.89	0.90	0.96	0.00	0.01	0.07	100%
A3	25,096	18,586	0.58	0.57	18	2.0%	3.3	453	2.0%	20.0	2.8	2.7	6.0	471	12.6	6.0		0.53	0.57	0.70	0.00	0.04	0.17	74%
A4	5,729	4,050	0.13	0.54									5.0	0	10.0	5.0		0.50	0.54	0.68	0.00	0.04	0.18	71%
A5	41,340	32,136	0.95	0.60									5.0	0	10.0	5.0		0.57	0.60	0.73	0.00	0.03	0.16	78%
A6	11,186	8,457	0.26	0.58	14	25.0%	1.2	654	2.0%	20.0	2.8	3.9	5.1	668	13.7	5.1		0.55	0.58	0.71	0.00	0.03	0.17	76%
A7	12,540	8,458	0.29	0.51									5.0	0	10.0	5.0		0.47	0.51	0.66	0.00	0.04	0.20	67%
A8	7,296	4,163	0.17	0.44									5.0	0	10.0	5.0		0.39	0.44	0.62	0.00	0.05	0.24	57%
A9	7,814	0	0.18	0.15									5.0	0	10.0	5.0		0.04	0.15	0.50	0.00	0.11	0.46	0%
B1	9,181	4,230	0.21	0.38									5.0	0	10.0	5.0		0.31	0.38	0.59	0.00	0.06	0.28	46%
B2	13,323	8,412	0.31	0.48									5.0	0	10.0	5.0		0.43	0.48	0.65	0.00	0.05	0.21	63%
B3	5,361	4,049	0.12	0.58									5.0	0	10.0	5.0		0.55	0.58	0.71	0.00	0.03	0.17	76%
B4	13,323	8,513	0.31	0.48									5.0	0	10.0	5.0		0.44	0.48	0.65	0.00	0.05	0.21	64%
B5	5,019	4,162	0.12	0.66									5.0	0	10.0	5.0		0.63	0.66	0.77	0.00	0.03	0.14	83%
B6	13,323	8,412	0.31	0.48									5.0	0	10.0	5.0		0.43	0.48	0.65	0.00	0.05	0.21	63%
B7	8,096	4,216	0.19	0.41									5.0	0	10.0	5.0		0.35	0.41	0.61	0.00	0.06	0.26	52%
B8	23,890	17,690	0.55	0.57	33	3.2%	3.8	442	1.0%	20.0	2.0	3.7	7.5	475	12.6	7.5		0.53	0.57	0.70	0.00	0.04	0.17	74%
B9	14,470	11,091	0.33	0.59	31	4.5%	3.1	400	1.0%	20.0	2.0	3.3	6.5	431	12.4	6.5		0.56	0.59	0.72	0.00	0.03	0.16	77%
B10	5,550	4,010	0.13	0.55									5.0	0	10.0	5.0		0.51	0.55	0.69	0.00	0.04	0.18	72%
B11	8,027	8,027	0.18	0.90									5.0	0	10.0	5.0		0.89	0.90	0.96	0.00	0.01	0.07	100%
B12	11,369	8,194	0.26	0.55									5.0	0	10.0	5.0		0.51	0.55	0.69	0.00	0.04	0.18	72%
B13	24,294	17,583	0.56	0.55									5.0	0	10.0	5.0		0.51	0.55	0.69	0.00	0.04	0.18	72%
B14	8,148	7,073	0.19	0.70									5.0	0	10.0	5.0		0.68	0.70	0.80	0.00	0.02	0.12	87%
B15	7,236	3,085	0.17	0.36									5.0	0	10.0	5.0		0.29	0.36	0.59	0.00	0.07	0.29	43%
B16	6,184	6,184	0.14	0.90									5.0	0	10.0	5.0		0.89	0.90	0.96	0.00	0.01	0.07	100%
B17	53,675	29,832	1.23	0.43	106	2.5%	9.3	260	1.0%	20.0	2.0	2.2	11.5	366	12.0	11.5		0.37	0.43	0.62	0.00	0.05	0.24	56%
B18	11,745	9,379	0.27	0.62	11	2.0%	2.3	315	1.0%	20.0	2.0	2.6	5.0	326	11.8	5.0		0.59	0.62	0.74	0.00	0.03	0.15	80%
B19	4,942	4,038	0.11	0.64									5.0	0	10.0	5.0		0.62	0.64	0.76	0.00	0.03	0.14	82%
B20	5,742	4,054	0.13	0.54									5.0	0	10.0	5.0		0.50	0.54	0.68	0.00	0.04	0.18	71%
B21	16,754	15,698	0.38	0.80									5.0	0	10.0	5.0		0.78	0.80	0.88	0.00	0.02	0.09	94%
B22	4,129	684	0.09	0.24									5.0	0	10.0	5.0		0.15	0.24	0.55	0.00	0.09	0.40	17%
B23	7,820	4,571	0.18	0.45									5.0	0	10.0	5.0		0.40	0.45	0.63	0.00	0.05	0.23	58%
C1	18,808	17,384	0.43	0.78									5.0	0	10.0	5.0		0.76	0.78	0.86	0.00	0.02	0.10	92%
C2	11,220	3,374	0.26	0.30									5.0	0	10.0	5.0		0.23	0.30	0.57	0.00	0.08	0.34	30%
OS1	455,444	0	10.46	0.18	300	2.9%	20.4	638	2.9%	5.0	0.9	12.5	32.9	938	15.2	15.2		0.08	0.18	0.52	0.00	0.11	0.44	5%
OS2	369,348	0	8.48	0.18	300	4.0%	18.3	900	4.0%	5.0	1.0	15.0	33.3	1200	16.7	16.7		0.08	0.18	0.52	0.00	0.11	0.44	5%
OS3	29,330	16,544	0.67	0.43	18	2.0%	4.1	707	3.1%	20.0	3.5	3.3	7.5	725	14.0	7.5		0.38	0.43	0.62	0.00	0.05	0.24	56%

CALCULATED BY: SCM
DATE: 18-Jun-15
CHECKED BY: JMM

STANDARD FORM SF-3
STORM DRAINAGE SYSTEM DESIGN
(RATIONAL METHOD PROCEDURE)

P_1= 1.01

PROJECT: Montex South at Vista Ridge
JOB NUMBER:
DESIGN STORM: 2-Year

STREET	DIRECT RUNOFF								TOTAL RUNOFF				STREET		PIPE			TRAVEL TIME			REMARKS
	DESIGN POINT	AREA DESIG.	AREA (Ac)	RUNOFF COEFF.	tc (min)	CA (Ac)	I (in/hr)	Q (cfs)	tc (min)	CA (Ac)	I (in/hr)	Q (cfs)	Slope (%)	Flow (cfs)	Flow (cfs)	Slope (%)	Size (in)	Length (ft)	Velocity (fps)	tt (min)	
		A1	0.41	0.53	5.0	0.21	3.43	0.73													
		A2	0.09	0.89	5.0	0.08	3.43	0.27													
		A3	0.58	0.53	6.0	0.31	3.26	1.00													
		A4	0.13	0.50	5.0	0.07	3.43	0.22													
		A5	0.95	0.57	5.0	0.54	3.43	1.85													
		A6	0.26	0.55	5.1	0.14	3.41	0.48													
		A7	0.29	0.47	5.0	0.13	3.43	0.46													
		A8	0.17	0.39	5.0	0.06	3.43	0.22													
		A9	0.18	0.04	5.0	0.01	3.43	0.02													
		B1	0.21	0.31	5.0	0.07	3.43	0.23													
		B2	0.31	0.43	5.0	0.13	3.43	0.45													
		B3	0.12	0.55	5.0	0.07	3.43	0.23													
		B4	0.31	0.44	5.0	0.13	3.43	0.46													
		B5	0.12	0.63	5.0	0.07	3.43	0.25													
		B6	0.31	0.43	5.0	0.13	3.43	0.45													
		B7	0.19	0.35	5.0	0.07	3.43	0.22													
		B8	0.55	0.53	7.5	0.29	3.04	0.88													
		B9	0.33	0.56	6.5	0.19	3.18	0.59													
		B10	0.13	0.51	5.0	0.07	3.43	0.22													
		B11	0.18	0.89	5.0	0.16	3.43	0.56													
		B12	0.26	0.51	5.0	0.13	3.43	0.46													
		B13	0.56	0.51	5.0	0.29	3.43	0.98													
		B14	0.19	0.68	5.0	0.13	3.43	0.44													
		B15	0.17	0.29	5.0	0.05	3.43	0.17													
		B16	0.14	0.89	5.0	0.13	3.43	0.43													
		B17	1.23	0.37	11.5	0.46	2.58	1.19													
		B18	0.27	0.59	5.0	0.16	3.43	0.55													
		B19	0.11	0.62	5.0	0.07	3.43	0.24													

STANDARD FORM SF-3
STORM DRAINAGE SYSTEM DESIGN
(RATIONAL METHOD PROCEDURE)

CALCULATED BY: SCM
DATE: 18-Jun-15
CHECKED BY: JMM

P_1= 1.01

PROJECT: Montex South at Vista Ridge
JOB NUMBER:
DESIGN STORM: 2-Year

STREET	DIRECT RUNOFF								TOTAL RUNOFF				STREET		PIPE			TRAVEL TIME			REMARKS
	DESIGN POINT	AREA DESIG.	AREA (Ac)	RUNOFF COEFF.	tc (min)	CA (Ac)	I (in/hr)	Q (cfs)	tc (min)	CA (Ac)	I (in/hr)	Q (cfs)	Slope (%)	Flow (cfs)	Flow (cfs)	Slope (%)	Size (in)	Length (ft)	Velocity (fps)	tt (min)	
		B20	0.13	0.50	5.0	0.07	3.43	0.22													
		B21	0.38	0.78	5.0	0.30	3.43	1.03													
		B22	0.09	0.15	5.0	0.01	3.43	0.05													
		B23	0.18	0.40	5.0	0.07	3.43	0.24													
		C1	0.43	0.76	5.0	0.33	3.43	1.13													
		C2	0.26	0.23	5.0	0.06	3.43	0.20													
		OS1	10.46	0.08	15.2	0.80	2.28	1.83													
		OS2	8.48	0.08	16.7	0.65	2.18	1.42													
		OS3	0.67	0.38	7.5	0.26	3.04	0.78													

STANDARD FORM SF-3
STORM DRAINAGE SYSTEM DESIGN
(RATIONAL METHOD PROCEDURE)

CALCULATED BY: SCM
DATE: 18-Jun-15
CHECKED BY: JMM

P_1= 2.70

PROJECT: Montex South at Vista Ridge
JOB NUMBER:
DESIGN STORM: 100-Year

STREET	DIRECT RUNOFF								TOTAL RUNOFF				STREET		PIPE			TRAVEL TIME			REMARKS
	DESIGN POINT	AREA DESIG.	AREA (Ac)	RUNOFF COEFF.	tc (min)	CA (Ac)	I (in/hr)	Q (cfs)	tc (min)	CA (Ac)	I (in/hr)	Q (cfs)	Slope (%)	Flow (cfs)	Flow (cfs)	Slope (%)	Size (in)	Length (ft)	Velocity (fps)	tt (min)	
		A1	0.41	0.70	5.0	0.28	9.16	2.60													
		A2	0.09	0.96	5.0	0.08	9.16	0.78													
		A3	0.58	0.70	6.0	0.40	8.72	3.53													
		A4	0.13	0.68	5.0	0.09	9.16	0.82													
		A5	0.95	0.73	5.0	0.69	9.16	6.31													
		A6	0.26	0.71	5.1	0.18	9.12	1.67													
		A7	0.29	0.66	5.0	0.19	9.16	1.75													
		A8	0.17	0.62	5.0	0.10	9.16	0.95													
		A9	0.18	0.50	5.0	0.09	9.16	0.82													
		B1	0.21	0.59	5.0	0.13	9.16	1.15													
		B2	0.31	0.65	5.0	0.20	9.16	1.81													
		B3	0.12	0.71	5.0	0.09	9.16	0.80													
		B4	0.31	0.65	5.0	0.20	9.16	1.82													
		B5	0.12	0.77	5.0	0.09	9.16	0.81													
		B6	0.31	0.65	5.0	0.20	9.16	1.81													
		B7	0.19	0.61	5.0	0.11	9.16	1.03													
		B8	0.55	0.70	7.5	0.38	8.12	3.12													
		B9	0.33	0.72	6.5	0.24	8.51	2.03													
		B10	0.13	0.69	5.0	0.09	9.16	0.81													
		B11	0.18	0.96	5.0	0.18	9.16	1.61													
		B12	0.26	0.69	5.0	0.18	9.16	1.65													
		B13	0.56	0.69	5.0	0.39	9.16	3.53													
		B14	0.19	0.80	5.0	0.15	9.16	1.37													
		B15	0.17	0.59	5.0	0.10	9.16	0.89													
		B16	0.14	0.96	5.0	0.14	9.16	1.24													
		B17	1.23	0.62	11.5	0.76	6.90	5.26													
		B18	0.27	0.74	5.0	0.20	9.16	1.83													
		B19	0.11	0.76	5.0	0.09	9.16	0.79													

STANDARD FORM SF-3
STORM DRAINAGE SYSTEM DESIGN
(RATIONAL METHOD PROCEDURE)

CALCULATED BY: SCM
DATE: 18-Jun-15
CHECKED BY: JMM

P_1= 2.70

PROJECT: Montex South at Vista Ridge
JOB NUMBER:
DESIGN STORM: 100-Year

		DIRECT RUNOFF							TOTAL RUNOFF				STREET		PIPE			TRAVEL TIME			
STREET	DESIGN POINT	AREA DESIG.	AREA (Ac)	RUNOFF COEFF.	tc (min)	CA (Ac)	I (in/hr)	Q (cfs)	tc (min)	CA (Ac)	I (in/hr)	Q (cfs)	Slope (%)	Flow (cfs)	Flow (cfs)	Slope (%)	Size (in)	Length (ft)	Velocity (fps)	tt (min)	REMARKS
		B20	0.13	0.68	5.0	0.09	9.16	0.82													
		B21	0.38	0.88	5.0	0.34	9.16	3.08													
		B22	0.09	0.55	5.0	0.05	9.16	0.47													
		B23	0.18	0.63	5.0	0.11	9.16	1.03													
		C1	0.43	0.86	5.0	0.37	9.16	3.40													
		C2	0.26	0.57	5.0	0.15	9.16	1.34													
		OS1	10.46	0.52	15.2	5.41	6.09	32.94													
		OS2	8.48	0.52	16.7	4.39	5.83	25.56													
		OS3	0.67	0.62	7.5	0.42	8.13	3.40													

Appendix C

Les Schwab Tire Center
Drainage Study

DRAINAGE REPORT ADDENDUM
LES SCHWAB TIRE CENTER
LOT 1, VISTA RIDGE FILING NO. 12
NEC MOUNTAIN VIEW BLVD & HWY 7
ERIE, CO

Prepared for:
SFP-E, LLC (Les Schwab)
P.O. Box 5350
Bend, Oregon 97708-5350

Prepared by:
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Prepared: February 9, 2016

Galloway
Planning. Architecture. Engineering.

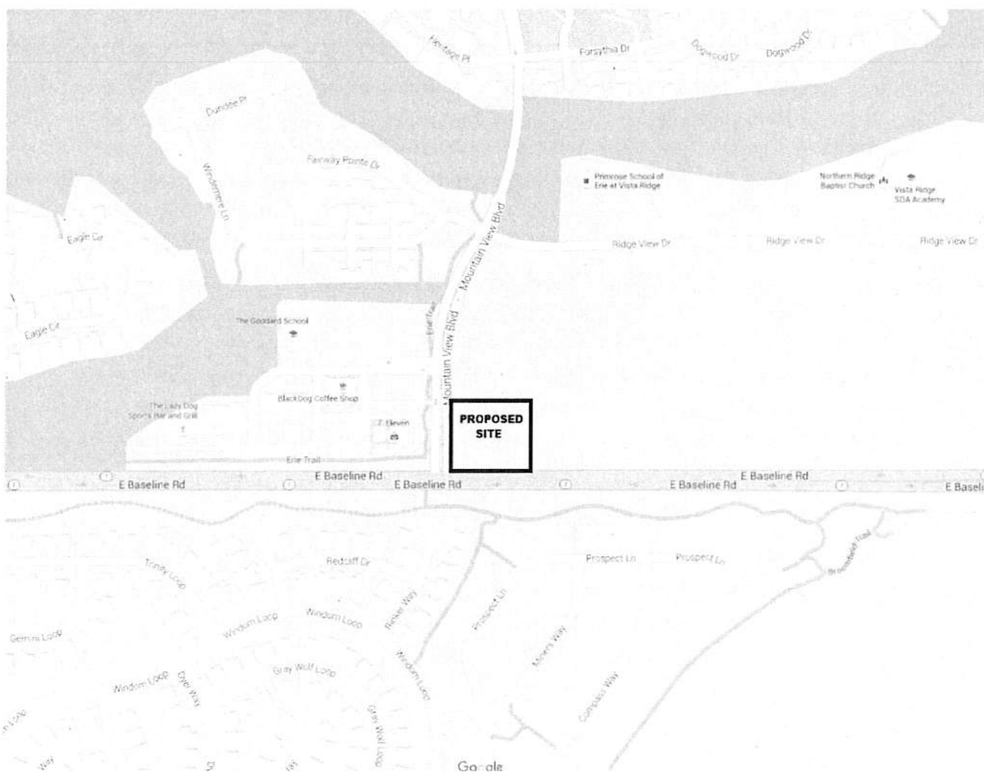
Table of Contents

I. INTRODUCTION

This Drainage Report Addendum has been prepared by Galloway & Company, Inc. for SFP-E, LLC, the developer of the site, to fulfill the drainage requirements of the Town of Erie, Colorado. This addendum establishes the expected drainage characteristics of the developed site as required by the Town's entitlement requirements and demonstrates that those characteristics are in compliance with the previously approved Drainage Report for the Vista Ridge Parcels 32 & 33 (Hurst & Associates, June 3, 2008).

1. LOCATION & GENERAL DESCRIPTION

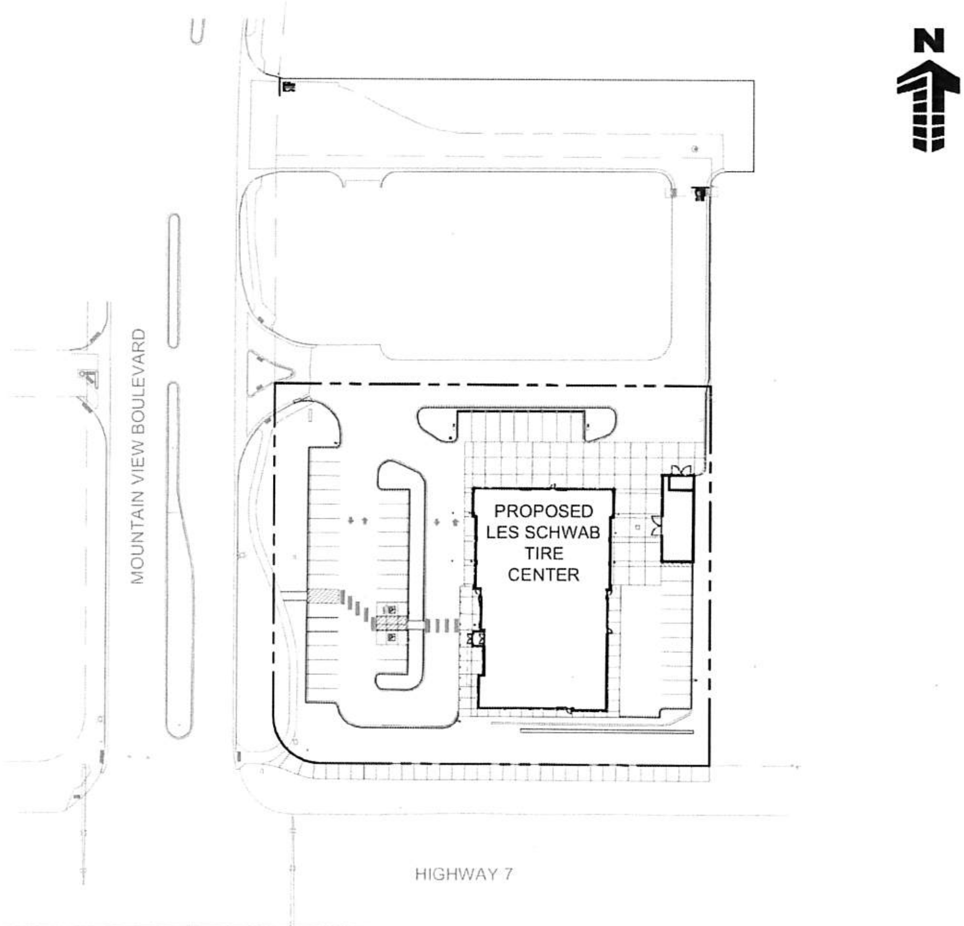
The subject property is located at the northeast corner of the intersection of Mountain View Boulevard and Highway 7 in Erie, Colorado. The site is comprised of all of Lot 1 of Vista Ridge Filing No.1 (hereafter referred to as Lot 1), an area of approximately 1.52 acres. The property is currently undeveloped, although a drive approach off of Mountain View Boulevards been has been fully constructed. The adjacent properties to the north, east and south are currently used as farmland and there is limited commercial development to the west.



VICINITY MAP

2. Proposed Development

The proposed development includes the construction of a Les Schwab Tire Center; an approximately 11,976 square foot facility inclusive of a show room (retail space), offices, storage, and eight servicing bays for automotive work. Also included on the site will be the requisite parking, drive aisles, trash/recycle facilities, landscaping, and utility infrastructure. The development will also include the paving of access drives partially located within Lot 2 of Vista Ridge Filing No. 1 (hereafter referred to as Lot 2).



SITE MAP

II. EXISTING AND PREVIOUSLY APPROVED DRAINAGE

The project site is located within Basin A1A as identified in the approved *Drainage Report for Vista Ridge Parcels 32 & 33,* prepared by Hurst and Associates, Inc (June 3, 2008). This is the Approved Drainage Report (ADR) as referenced hereafter in this report. The stormwater design presented within the ADR anticipates that, post commercial development, stormwater runoff from the subject property will drain unrestrained and untreated to the existing Vista Ridge Extended Detention Water Quality Pond A1 located approximately 510 feet north of Lot 1. The ADR expected that Lots 1 & 2 would have a combined drainage area of 2.43 acres and would connect to the existing storm sewer infrastructure at Design Points SP-3 via an existing 30" storm pipe stub (see the attached Vista Ridge Master Drainage Plan). The anticipated total maximum stormwater flow rates from Lots 1 & 2 post development were calculated to be 8.44 cfs during the 5-year frequency design storm event and 16.85 cfs during the 100-year frequency design storm event (ADR Flow summary sheets are included in the attachments of this report).

Currently, the site slopes from south to north at slopes ranging from 2%-6%.Stormwater runoff flows overland north onto Lot 2. Existing catch basins located within this lot capture a portion of the stormwater; however the majority of the runoff continues to flow north overland. Both the overland flow and the stormwater captured by the existing storm sewer infrastructure flow to the existing Vista Ridge Extended Detention Water Quality Pond A1.

III. DESIGN CRITERIA & METHODOLOGY

As previously mentioned precedence for development of the Lot 1 has been established within the approved *Drainage Report for Vista Ridge Parcels 32 & 33,* prepared by Hurst and Associates, Inc (June 3, 2008).

The storm sewer system for this site has been designed using the Rational methodology as outlined in the Town of Erie Standards and Specifications, Section 800 and the Urban Storm Drainage Criteria Manual.

The Rational Method utilizes the following equation: $Q = CIA$.

Q = storm runoff in cubic feet per second;

C = runoff coefficient based on surface impermeability
(Values provided within Urban Storm Drainage Criteria manual);

I = rainfall intensity in inches per hour (taken from the Town of Erie Standards);

A = drainage basin area in acres.

Calculations included herein demonstrate runoff and flow accumulations for the 5- year and 100-year events as required by the Town of Erie. Inlets capacities are sized with the aid of the UD&FCD spreadsheet design program *UD Inlet v3.12.*

IV. DRAINAGE PLAN

1. GENERAL CONCEPT

The storm drainage design for this property follows the intent of the ADR, however the drainage area of Lots 1 & 2 and the adjacent ROW is slightly larger than the master plan anticipated; 2.62 Acres vs 2.46 Acres. The site has been designed to drain storm runoff flows from the majority of the parking areas to storm drain inlets which will channel the storm runoff to the existing 30" storm sewer line via a proposed storm sewer system. As noted earlier, the existing storm sewer system drains to the existing Detention Pond north of the site which was designed to accommodate storm water runoff from this site. Portions of the proposed on-site drive accesses and all of the proposed off-site drive access improvements will drain to the south onto Lot 2 and further south into the existing Detention Pond via sheet flow. Exhibit EX-1 attached to this Addendum shows the proposed sub-basins and their characteristics. Because the ADR analyzes runoff from both Lot 1 & Lot 2 and gives a summarized expected stormwater runoff rate for both areas at Design Point SP-6, Lot 2 has been analyzed in this report as an additional basin. An assumed post development imperviousness of 90% and time of concentration of 5 minutes were used to approximate peak flow rates from Lot 2.

2. BASIN AND CONVEYANCE DETAIL

Collection and conveyance within and from each of the basins is detailed below; grouped where the specifics are common. Bypass conveyance as explained in each of the descriptions are only for a failed inlet or storm events excess of the 100-year return period. Reference EX-1 attached to this addendum contained in this report for basin delineations and locations. Peak flow rates from each of the basins described can also be found in the attachments of this report, as well as flow rates at key design points as identified on EX-1.

Basin A1 consists of the landscaped area on east edge of the site. This area will sheet flow to the east and drain off-site unrestrained. The existing grades east of the site will channel runoff from this Basin north, and ultimately it will flow into the existing Detention Pond north of the site, in compliance with the ADR design.

Basin A2 consists of the parking and landscape area located in the southeast corner of the site. Stormwater runoff from this area will sheet flow to a proposed catch basin, and be directed into the proposed storm sewer system, ultimately flowing into the existing Detention Pond. This basin will receive off site runoff flows from the basin identified as OS-A2 which consists of the sidewalk and landscaped area south of the site and north of Highway 7. Stormwater runoff from OS-A2 will also sheet flow into the proposed catch basin located within A2. During storms greater than the 100-year design storm, or in the event of catch basin failure, Basins A2 & OS-A2 will overflow into Basins A3 and A4 and stormwater will sheet flow to the north, onto Lot 2.

Basin A3 consists of the drive area located in the north east corner of the site and the recycle/trash enclosure. Stormwater runoff from this area will sheet flow to a proposed catch basin, and be directed into the proposed storm sewer system, ultimately flowing into the existing Detention Pond. During storms greater than the 100-year design storm, or in the event of catch basin failure, Basin will overflow onto Basin A4 and Lot 2.

Basin A4 consists of the access drive areas on the north side of the site. This area will sheet flow to the north and drain onto Lot 2 unrestrained. The existing grades will channel runoff from this Basin north, and ultimately it will flow into the existing Detention Pond north of the site, in compliance with the ADR design.

Basin A5 consists of the parking area located in north central area of the site. Stormwater runoff from this area will sheet flow to a proposed catch basin, and be directed into the proposed storm sewer system, ultimately flowing into the existing Detention Pond. During storms greater than the 100-year design storm, or in the event of catch basin failure, Basin will overflow into Basin A4 and onto Lot2.

Basin R1 consists of the proposed Les Schwab Tire Center building. Stormwater runoff from this area will drain from the roof into proposed roof drain lines connection to the proposed storm sewer system, ultimately flowing into the existing Detention Pond.

Basin A6 & A7 consists of the access drive and parking area directly west of the proposed Les Schwab Tire Center building. Stormwater runoff from this area will sheet flow to two proposed catch basins, and be directed into the proposed storm sewer system, ultimately flowing into the existing Detention Pond. During storms greater than the 100-year design storm, or in the event of catch basin failure, both basins will overflow into Basin A4 and onto Lot2.

Basin A8 consists of the landscaped area in northwest corner of the site and a portion of the existing drive approach. This area will sheet flow to the northwest and into the Mountain View Boulevard Right of Way.

The calculated total of the site stormwater flows through the proposed storm sewer system and into the existing storm system at Design Point 7, as identified on the attached Drainage Map, EX-1, will be 4.7 cfs during the 5-year design storm event and 8.9 cfs during the 100-year design storm event. The total unrestrained overland flow rates off site have been calculated to be 5.0 cfs during the 5-year design storm event and 9.4 cfs during the 100-year design storm event.

As noted earlier in this report the post development peak storm water flows from Lot 2 were also calculated in order to assess compliance with the ADR. The calculated peak flow rates are 4.15 cfs during the 5-year design storm event and 8.32 cfs during the 100-year design storm event.

It is anticipated that the future development of Lot 2 will also include the installation of a storm sewer system which will capture the stormwater runoff which sheet flows from Lot 1 onto Lot 2, at which point the combined peak flow rates from Lot 1 and Lot 2 into the existing Vista Ridge storm sewer infrastructure have been calculated to be 9.3 cfs during the 5-year design storm event and 20.0 cfs during the 100-year design storm event.

V. CONCLUSION

The construction of the proposed Les Schwab Tire Center will result in storm water runoff flow rates below those approved within the ADR. However, the anticipated development of Lot 2 will increase flow rates at design point SP-3 (the connection to the existing 30" storm sewer line) slightly above those calculated within the ADR, as illustrated in Table 1.

	Approved Flow Rate at SP-3	Post Development of Lot 1	Post Development of Lots 1 & 2
5-year Design Storm	8.44 cfs	5.0 cfs	9.4 cfs
100-year Design Storm	16.85 cfs	9.3 cfs	20.0 cfs

Table 1

The combined Lot 1 & Lot 2 drainage area is higher larger than the ADR expected by approximately 0.2 Acres. Per the master plan, this additional area was proposed to drain to the 30" storm drain pipe approximately 150' downstream of design point SP-3. The shift in the approved drainage sub-basin areas will not cause flow rates to exceed the capacity of the existing 30" storm sewer line.

The proposed storm drainage design for the Les Schwab Tire Center on Lot 1 of Vista Ridge Filing No. 12 is in compliance with the intent of the approved *Drainage Report for Vista Ridge Parcels 32 & 33,* prepared by Hurst and Associates, Inc (June 3, 2008) as the development will make no significant alterations to the approved master plan, and will not exceed the capacity of the existing infrastructure. It is anticipated the proposed development will not negatively impact the existing adjacent stormwater systems, waterways, or regional groundwater system during the design storm events analyzed.

VI. REFERENCES

1. Town of Erie Standards and Specifications, Section 800.
2. Drainage Criteria Manual, Urban Drainage and Flood Control District, June 2001, Revised April 2008.
3. Drainage Report, Vista Ridge Parcels 32 & 33 Erie Colorado, Hurst & Associates, June 3, 2008.

LOT 1, VISTA RIDGE FILING NO. 12

SITUATED IN THE SOUTHWEST QUARTER OF SECTION 33,
TOWNSHIP 1 NORTH, RANGE 68 WEST OF THE 6TH PRINCIPAL MERIDIAN
TOWN OF ERIE, COUNTY OF WELD, STATE OF COLORADO
1.528 ACRES
SITE PLAN - SP-00717-2015

LEGAL DESCRIPTION

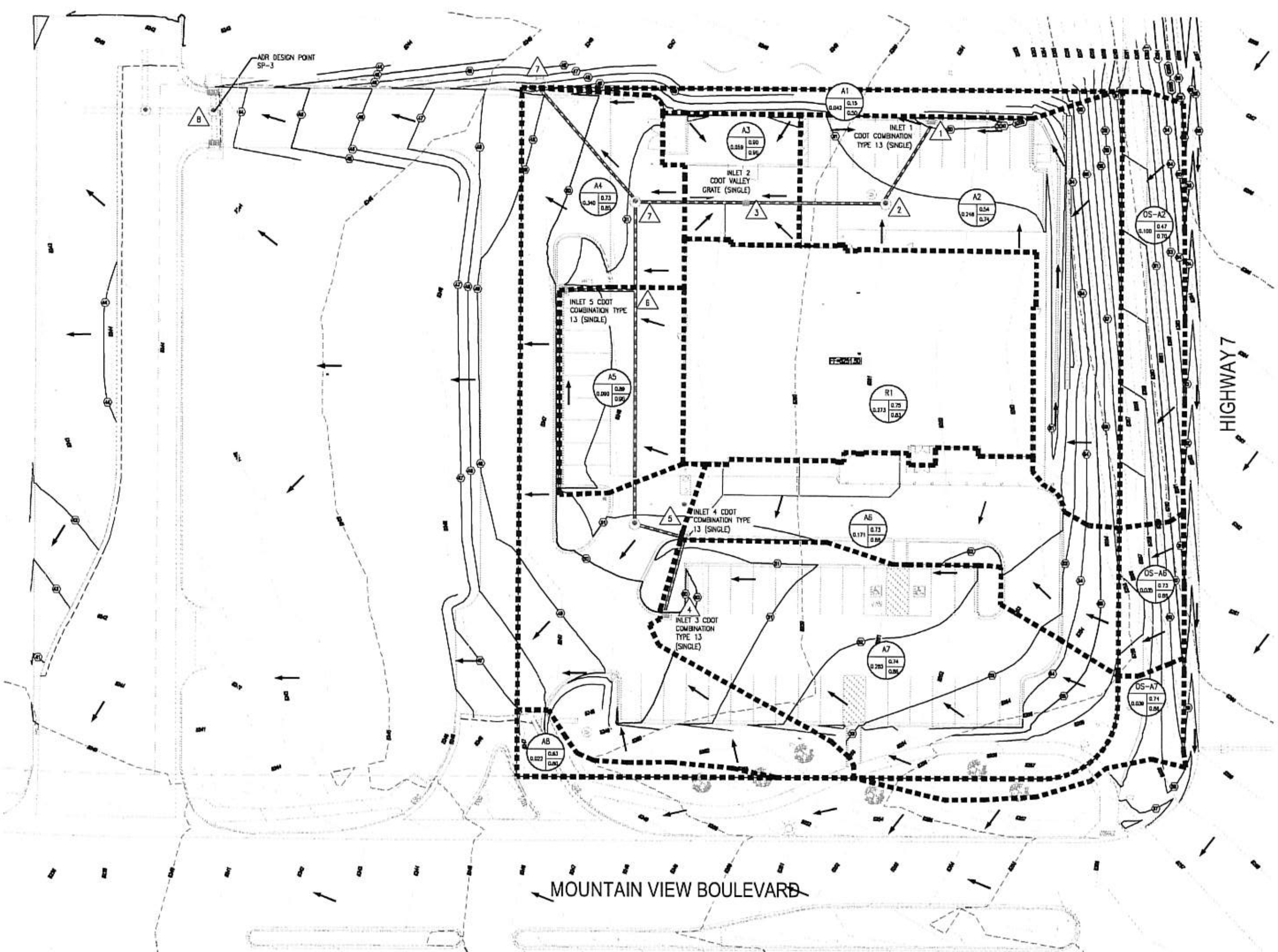
LOT 1, VISTA RIDGE FILING NO. 12, COUNTY OF WELD, STATE OF COLORADO

BASIS OF BEARING

BEARINGS ARE BASED ON THE NORTH RIGHT-OF-WAY LINE OF STATE HIGHWAY NO. 7 BEARING N89°38'31"W AS REFERENCED AND BOUNDED BY THE MONUMENTS SHOWN HEREON.

BENCHMARK

CITY OF AND COUNTY OF BROOMFIELD BM "LUCY" ELEVATION 5097.60 FEET (NAVD 1988 DATUM)



DRAINAGE LEGEND

- EXISTING CONTOUR
- PROPOSED CONTOUR
- EXISTING STORM SEWER
- PROPOSED STORM SEWER
- STORM SEWER PROPOSED BY OTHERS
- BASIN BOUNDARY LINE
- FLOW ARROW
- BASIN DESIGNATION
- 5-YEAR RUNOFF COEFFICIENT
- 100-YEAR RUNOFF COEFFICIENT
- BASIN AREA IN ACRES

CAUTION - NOTICE TO CONTRACTOR

1. ALL UTILITY LOCATIONS SHOWN ARE BASED ON MAPS PROVIDED BY THE APPROPRIATE UTILITY COMPANY AND FIELD SURFACE EVIDENCE AT THE TIME OF SURVEY AND IS TO BE CONSIDERED AN APPROXIMATE LOCATION ONLY. IT IS THE CONTRACTOR'S RESPONSIBILITY TO FIELD VERIFY THE FIELD LOCATION OF ALL UTILITIES, PUBLIC OR PRIVATE, WHETHER SHOWN ON THE PLANS OR NOT, PRIOR TO CONSTRUCTION. REPORT ANY DISCREPANCIES TO THE ENGINEER PRIOR TO CONSTRUCTION.
2. WHERE A PROPOSED UTILITY CROSSES AN EXISTING UTILITY, IT IS THE CONTRACTOR'S RESPONSIBILITY TO FIELD VERIFY THE HORIZONTAL AND VERTICAL LOCATION OF SUCH EXISTING UTILITY, EITHER THROUGH POTHOLING OR ALTERNATIVE METHOD. REPORT INFORMATION TO THE ENGINEER PRIOR TO CONSTRUCTION.

811 Know what's below. Call before you dig.

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Tires LES SCHWAB

NEC MOUNTAIN VIEW & HIGHWAY 7
ERIE, CO
LES SCHWAB TIRE CENTER

#	Date	Issue / Description	Init.

Project No:	
Drawn By:	JAT
Checked By:	KRB
Date:	DECEMBER 2016

DRAINAGE MAP

EX-1

Sheet 1 of 1

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Project: Les Schwab Tire Center

Address: NEC Mountain View & Highway 7

Erie, CO

Date: 2/9/2016

Percent Impervious and Runoff Coefficients

Basin	Land Use	Percent Impervious	Area (FT^2)	Area (Ac.)	Soil Type	Composite C_5	Composite C_{10}	Composite C_{100}
A1	Paved Areas	100%	0	0.00		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	1,842	0.04		0.15	0.25	0.50
	TOTAL	**0%**	**1,842**	**0.042**	**C**	**0.15**	**0.25**	**0.5**
A2	Paved Areas	90%	5,665	0.13		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	5,134	0.12		0.15	0.25	0.50
	TOTAL	**47%**	**10,799**	**0.248**	**C**	**0.54**	**0.6**	**0.74**
OS-A2	Paved Areas	90%	1,862	0.04		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	2,507	0.06		0.15	0.25	0.50
	TOTAL	**38%**	**4,369**	**0.100**	**C**	**0.47**	**0.54**	**0.7**
A3	Paved Areas	90%	2,581	0.06		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	0	0.00		0.15	0.25	0.50
	TOTAL	**90%**	**2,581**	**0.059**	**C**	**0.9**	**0.92**	**0.96**
A4	Paved Areas	90%	11,423	0.26		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	3,389	0.08		0.15	0.25	0.50
	TOTAL	**69%**	**14,812**	**0.340**	**C**	**0.73**	**0.77**	**0.85**
A5	Paved Areas	90%	3,897	0.09		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	30	0.00		0.15	0.25	0.50
	TOTAL	**89%**	**3,927**	**0.090**	**C**	**0.89**	**0.91**	**0.96**
R1	Paved Areas	90%	0	0.00		0.90	0.92	0.96
	Roofs	90%	11,976	0.27		0.75	0.77	0.83
	Landscape	0%	0	0.00		0.15	0.25	0.50
	TOTAL	**90%**	**11,976**	**0.273**	**C**	**0.75**	**0.77**	**0.83**
A6	Paved Areas	90%	5,774	0.13		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	1,670	0.04		0.15	0.25	0.50
	TOTAL	**70%**	**7,444**	**0.171**	**C**	**0.73**	**0.77**	**0.86**
OS-A6	Paved Areas	90%	587	0.01		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	958	0.02		0.15	0.25	0.50
	TOTAL	**34%**	**1,545**	**0.035**	**C**	**0.43**	**0.5**	**0.67**
A7	Paved Areas	90%	9,687	0.22		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	2,715	0.06		0.15	0.25	0.50
	TOTAL	**70%**	**12,402**	**0.285**	**C**	**0.74**	**0.77**	**0.86**
OS-A7	Paved Areas	90%	404	0.01		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	1,287	0.03		0.15	0.25	0.50
	TOTAL	**22%**	**1,691**	**0.039**	**C**	**0.33**	**0.41**	**0.61**
A8	Paved Areas	90%	606	0.01		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	335	0.01		0.15	0.25	0.50
	TOTAL	**58%**	**941**	**0.022**	**C**	**0.63**	**0.68**	**0.8**
On Site Total	Paved Areas	90%	39,633	0.91		0.90	0.92	0.96
	Roofs	90%	11,976	0.27		0.75	0.77	0.83
	Landscape	0%	15,115	0.35		0.15	0.25	0.50
	TOTAL	**70%**	**66,724**	**1.532**	**C**	**0.7**	**0.74**	**0.83**
Total	Paved Areas	90%	42,486	0.98		0.90	0.92	0.96
	Roofs	90%	11,976	0.27		0.75	0.77	0.83
	Landscape	0%	19,867	0.46		0.15	0.25	0.50
	TOTAL	**66%**	**74,329**	**1.706**	**C**	**0.68**	**0.72**	**0.82**
Lot 2	Paved Areas	90%	40,164	0.92		0.90	0.92	0.96
	Roofs	90%	0	0.00		0.75	0.77	0.83
	Landscape	0%	0	0.00		0.15	0.25	0.50
	TOTAL	**90%**	**40,164**	**0.922**	**C**	**0.90**	**0.92**	**0.96**

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Project: Les Schwab Tire Center
Address: NEC Mountain View & Highway 7
Erie, CO
Date: 2/9/2016

Basin Data			Iintial/Overland Time (T_i)			Travel Time (T_t)					T_c Chech Urbanized Basins			Final T_c
Basin	Area (acre)	C_5	Length (ft)	Slope (%)	T_i (min)	Length (ft)	Slope (%)	Conv. Coeff. (Table RO-2)	Vel. (fps)	T_t (min)	Total Length (ft)	Comp. Tc (min)	T_c=(L/180)+10 (min)	Final Tc (min)
A1	0.04	0.15	113	4.2%	11.5						113	11.5	10.6	**10.6**
A2	0.25	0.54	25	20.0%	1.9	175	0.5%	20	1.4	2.1	200	5.0	11.1	**5.0**
OS-A2	0.10	0.47	50	25.0%	2.8	170	0.5%	20	1.4	2.0	220	5.0	11.2	**5.0**
A3	0.06	0.90	35	2.0%	1.7						35	5.0	10.2	**5.0**
A4	0.34	0.73	127	2.0%	6.1						127	6.1	10.7	**6.1**
A5	0.09	0.89	50	2.0%	2.2	75	0.5%	20	1.4	0.9	125	5.0	10.7	**5.0**
R1	0.27	0.75	50	20.0%	1.7						50	5.0	10.3	**5.0**
A6	0.17	0.73	25	25.0%	1.2	150	0.5%	20	1.4	1.8	175	5.0	11.0	**5.0**
OS-A6	0.04	0.43	50	25.0%	3.0	150	0.5%	20	1.4	1.8	200	5.0	11.1	**5.0**
A7	0.28	0.74	35	25.0%	1.3	150	2.1%	20	2.9	0.9	185	5.0	11.0	**5.0**
OS-A7	0.04	0.33	50	25.0%	3.4	150	2.1%	20	2.9	0.9	200	5.0	11.1	**5.0**
A8	0.02	0.63	70	20.0%	2.7						70	5.0	10.4	**5.0**

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Project: Les Schwab Tire Center
Address: NEC Mountain View & Highway 7
Erie, CO
Date: 2/9/2016

5 -YR EVENT ROUTING CALCULATIONS

		DIRECT RUNOFF							TOTAL RUNOFF				
		Basin	AREA (ac)	Runoff Coeff	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Notes
BASIN A1		A1	0.04	0.15	10.6	0.01	3.77	**0 cfs**					
BASIN A2		A2	0.25	0.54	5.0	0.13	5.00	**0.7 cfs**					
BASIN OS-A2		OS-A2	0.10	0.47	5.0	0.05	5.00	**0.24 cfs**					
BASIN A3	INLET 2	A3	0.06	0.90	5.0	0.05	5.00	**0.3 cfs**					
BASIN A4		A4	0.34	0.73	6.1	0.25	4.76	**1.2 cfs**					
BASIN A5	INLET 5	A5	0.09	0.89	5.0	0.08	5.00	**0.4 cfs**					
BASIN A6	INLET 4	A6	0.17	0.73	5.0	0.12	5.00	**0.6 cfs**					
BASIN OS-A6		OS-A6	0.04	0.43	5.0	0.02	5.00	**0.1 cfs**					
BASIN A7		A7	0.28	0.74	5.0	0.21	5.00	**1.05 cfs**					
BASIN OS-A7		OS-A7	0.04	0.33	5.0	0.01	5.00	**0.1 cfs**					
BASIN A8		A8	0.02	0.63	5.0	0.01	5.00	**0.07 cfs**					
BASIN R1		R1	0.27	0.75	5.0	0.20	5.00	**1.02 cfs**					
LOT 2		LOT 2	0.92	0.90	5.0	0.83	5.00	**4.15 cfs**					
DP 1	INLET 1								5.0	0.18	5.00	**0.9 cfs**	Basins A2 & OS-A2
DP 2									5.0	0.28	5.00	**1.4 cfs**	Basins A2, OS-A2 & 0.5 R1
DP 3									5.0	0.3	5.00	**1.7 cfs**	Basins A2, OS-A2, 0.5 R1 & A3
DP 4	INLET 3								5.0	0.22	5.00	**1.1 cfs**	Basins A7 & OS-7
DP 5									5.0	0.47	5.00	**2.3 cfs**	Basins A7, OS-A7, A6, OS-A6, & 0.5R1
DP 6									5.0	0.5	5.00	**2.7 cfs**	Basins A7, OS-A7, A6, OS-A6, 0.5R1 & A5
DP 7									5.0	0.9	5.00	**5.0**	Basins A7, OS-A7, A6, OS-A6, R1, A6, A2, OS-A2 & A3
DP 8									6.1	2.0	4.76	**9.3 cfs**	Basins A2, OS-A2, A3, A4, A5, A6, OS-A6, A7, OS-A7, A8 & LOT 2

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Project: Les Schwab Tire Center

Address: NEC Mountain View & Highway 7
Erie, CO

Date: 2/9/2016

100 -YR EVENT ROUTING CALCULATIONS

	DIRECT RUNOFF								TOTAL RUNOFF				
		Basin	AREA (ac)	Runoff Coeff	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Notes
BASIN A1		A1	0.04	0.50	10.6	0.02	7.07	**0.1 cfs**					
BASIN A2		A2	0.25	0.74	5.0	0.18	9.40	**1.7 cfs**					
BASIN OS-A2		OS-A2	0.10	0.70	5.0	0.07	9.40	**0.66 cfs**					
BASIN A3	INLET 2	A3	0.06	0.96	5.0	0.06	9.40	**0.5 cfs**					
BASIN A4		A4	0.34	0.85	6.1	0.29	8.93	**2.6 cfs**					
BASIN A5	INLET 5	A5	0.09	0.96	5.0	0.09	9.40	**0.8 cfs**					
BASIN A6	INLET 4	A6	0.17	0.86	5.0	0.15	9.40	**1.4 cfs**					
BASIN OS-A6		OS-A6	0.04	0.67	5.0	0.02	9.40	**0.2 cfs**					
BASIN A7		A7	0.28	0.86	5.0	0.24	9.40	**2.3 cfs**					
BASIN OS-A7		OS-A7	0.04	0.61	5.0	0.02	9.40	**0.2 cfs**					
BASIN A8		A8	0.02	0.80	5.0	0.02	9.40	**0.16 cfs**					
BASIN R1		R1	0.27	0.83	5.0	0.23	9.40	**2.13 cfs**					
LOT 2		LOT 2	0.92	0.96	5.0	0.89	9.40	**8.32 cfs**					
DP 1	INLET 1								5.0	0.25	9.40	**2.4 cfs**	Basins A2 & OS-A2
DP 2									5.0	0.37	9.40	**3.4 cfs**	Basins A2, OS-A2 & 0.5 R1
DP 3									5.0	0.4	9.40	**4 cfs**	Basins A2, OS-A2, 0.5 R1 & A3
DP 4	INLET 3								5.0	0.27	9.40	**2.5 cfs**	Basins A7 & OS-7
DP 5									5.0	0.55	9.40	**5.2 cfs**	Basins A7, OS-A7, A6, OS-A6, & 0.5R1
DP 6									5.0	0.6	9.40	**6 cfs**	Basins A7, OS-A7, A6, OS-A6, 0.5R1 & A5
DP 7									5.0	1.1	9.40	**9.4**	Basins A7, OS-A7, A6, OS-A6, R1, A6, A2, OS-A2 & A3
DP 8									6.1	2.2	8.93	**20 cfs**	Basins A2, OS-A2, A3, A4, A5, A6, OS-A6, A7, OS-A7, A8 & LOT 2

Ex. 30" SD Pipe Capacity

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.012	
Channel Slope	0.00500	ft/ft
Normal Depth	2.25	ft
Diameter	2.50	ft

Results

Discharge	33.49	ft^3/s
Flow Area	4.65	ft^2
Wetted Perimeter	6.25	ft
Hydraulic Radius	0.75	ft
Top Width	1.50	ft
Critical Depth	1.97	ft
Percent Full	90.0	%
Critical Slope	0.00614	ft/ft
Velocity	7.20	ft/s
Velocity Head	0.80	ft
Specific Energy	3.05	ft
Froude Number	0.72	
Maximum Discharge	33.80	ft^3/s
Discharge Full	31.42	ft^3/s
Slope Full	0.00568	ft/ft
Flow Type	SubCritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	90.00	%
Downstream Velocity	Infinity	ft/s

Ex. 30" SD Pipe Capacity

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	2.25	ft
Critical Depth	1.97	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.00614	ft/ft

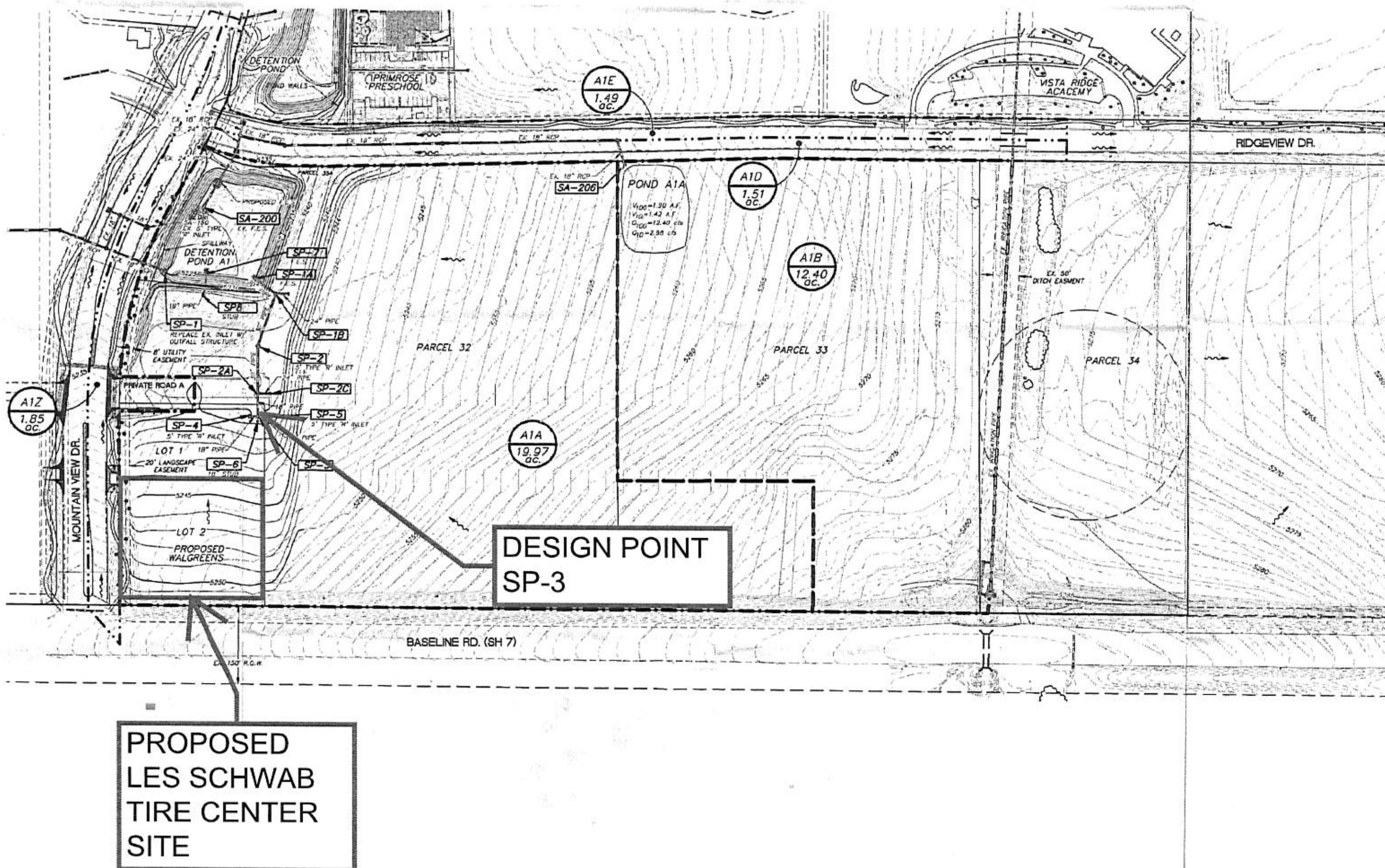
Preliminary Pipe Sizing
Vista Ridge Commercial Parcels 32 & 33
Job Number: 2142-03

Pipe Location	Contributing Basins	Inlets At Full Capacity (cfs)	Q_{10} Pond Release (cfs)	Design Flow (cfs)	Slope (ft/ft)	Manning's n	Pipe Dia. (ft)	Pipe Dia. (in)	Velocity w/ min diameter (fps)	Pipe Sizes (in)
SA-206 to SA-205	A1B		3.65	**3.65**	0.0200	0.013	0.9	**10.6**	5.9	***18***
SA-205 to SA-204	A1B		3.65	**3.65**	0.0307	0.013	0.8	**9.8**	6.9	***18***
SA-203A to SA-202	A1B		3.65	**3.65**	0.0303	0.013	0.8	**9.8**	6.9	***18***
SA-204 to SA-203A	A1B		3.65	**3.65**	0.0260	0.013	0.8	**10.1**	6.5	***18***
SA-203 to SA-202	A1E	4.40	4.69	**4.69**	0.0053	0.013	1.2	**15.0**	3.8	***18***
SA-202 to SA-201	A1B, A1E	0.00	4.69	**4.69**	0.0134	0.013	1.0	**12.6**	5.4	***24***
SA-201 to SA-200	A1B, A1D, A1E	0.00	4.69	**4.69**	0.0082	0.013	1.2	**13.8**	4.5	***24***
SP-1 to SA-156	A1B, A1D, A1E		10.00	**10.00**	0.0060	0.013	1.6	**19.5**	4.8	***18***

PIPE INSTALLED AS 30"

Pipe Location	Contributing Basins	Contributing Area (ac)	Design Flow 5-Yr (cfs)	Design Flow 100-Yr (cfs)	Slope (ft/ft)	Manning's n	Pipe Dia. 5-Yr (ft)	Pipe Dia. 5-Yr (in)	Pipe Dia. 100-Yr (ft)	Pipe Dia. 100-Yr (in)	Pipe Size 5-Yr (in)	Pipe Size 100-Yr (in)
SP-6 to SP-3	A1A	2.27	**7.89**	**15.74**	0.0200	0.013	1.2	**14.2**	1.5	**18.4**	***18***	***24***
SP-5 to SP-4	A1A	0.04	**0.14**	**0.28**	0.0400	0.013	0.2	**2.7**	0.3	**3.6**	***18***	***18***
SP-4 to SP-3	A1A	0.12	**0.42**	**0.83**	0.0400	0.013	0.3	**4.1**	0.4	**5.4**	***18***	***18***
SP-3 to SP-2A	A1A	2.43	**8.44**	**16.85**	0.0235	0.013	1.2	**14.1**	1.5	**18.3**	***18***	***24***
SP-2B to SP-2A	A1A	8.58	**29.81**	**59.49**	0.0180	0.013	2.0	**23.8**	2.6	**30.9**	***24***	***36***
SP-2A to SP-2	A1A	11.01	**38.25**	**76.34**	0.0235	0.013	2.1	**24.9**	2.7	**32.3**	***30***	***36***
SP-2 to SP-1B	A1A	11.01	**38.25**	**76.34**	0.0300	0.013	2.0	**23.8**	2.6	**30.8**	***30***	***36***
SP-1C to SP-1B	A1A	6.43	**22.34**	**44.59**	0.0100	0.013	2.0	**23.9**	2.6	**31.0**	***24***	***36***
SP-1B to SP-1A	A1A	17.44	**60.58**	**120.93**	0.0387	0.013	2.2	**26.9**	2.9	**34.9**	***30***	***36***
SP-8 to SP-7	A1A	1.09	**3.79**	**7.56**	0.1300	0.013	0.6	**7.6**	0.8	**9.8**	***18***	***18***

Basin	Total Area (acres)	Weighted Runoff Coefficients			T_c (mins)	Intensity			Design Flow		
		C_5	C_{10}	C_{100}		I_5 (in/hr)	I_{10} (in/hr)	I_{100} (in/hr)	Q_5 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
A1A	19.97	0.83	0.84	0.87	8.4	4.2	5.10	8.00	69.37	85.80	138.47
A1B	12.40	0.83	0.84	0.87	7.3	4.43	5.38	8.40	45.37	56.13	90.21
A1D	1.51	0.65	0.70	0.79	11.2	3.65	4.45	6.85	3.59	4.66	8.17
A1E	1.49	0.66	0.70	0.80	11.2	3.65	4.45	6.85	3.58	4.64	8.10
A1Z	1.85	0.69	0.73	0.81	8.1	4.10	5.00	7.60	5.20	6.72	11.43
Basins A1A, A1D, & A1E to Pond A1											
	22.96	0.80	0.82	0.86	12.4	3.60	4.40	6.80	66.53	83.23	133.85



VICINITY MAP
SCALE: 1"=2000'

LEGEND

- Manhole
- Existing Manhole
- 5' Type 'R' Inlet
- 10' Type 'R' Inlet
- 15' Type 'R' Inlet
- Flared End Section
- Storm Pipe
- Existing Storm Pipe
- Existing Contours
- Proposed Contours
- Future Contours
- Master Basin Boundary
- Basin Boundary
- Flow Arrow
- S-28 Structure Number
- A1C 6.61 ac. Basin Designation

NOTE: ALL INLETS ARE 5' TYPE 'R' UNLESS OTHERWISE NOTED

GRAPHIC SCALE
(IN FEET)
1 inch = 60 ft.

VISTA RIDGE
MASTER DRAINAGE PLAN
PARCEL 32 – 33

HURST & ASSOCIATES, INC
CONSULTING ENGINEERS

Appendix D

Hydrological IDF Curves and Runoff Coefficients

PROJECT: Vista Ridge Commercial
LOCATION: Mountain View Blvd and Highway 7
Erie, CO

Project No.: SH7000001.01
Date: July 26, 2016
Designed By: Phil Dalrymple
Checked By: Brandon McCrary, PE

PERCENT IMPERVIOUS VALUES	
LANDSCAPE	0
PAVING	100
ROOFING	90
WALKS/DRIVES	90
FUTURE COMMERCIAL	95

*Refer to Table RO-5, Urban Drainage, Storm Drainage Criteria Manual for Runoff Coefficients used

*Group C Soils

Composite Runoff Coefficients and Percent Imperviousness for Developed Drainage Basins

BASIN DESIG.	OVERALL AREA (AC)	LANDSCAPE AREA (AC)	PAVED AREA (AC)	ROOF AREA (AC)	WALKS/ DRIVES (AC)	FUTURE COMMERCIAL (AC)	2-YEAR COEFF.	5-YEAR COEFF.	10-YEAR COEFF.	100-YEAR COEFF.	PERCENT IMPERVIOUS
A-1	7.34	1.36	0.00	0.00	0.00	5.98	-	0.60	0.63	0.72	0.77
Total to Pond	7.34	1.36	0.00	0.00	0.00	5.98	-	0.60	0.63	0.72	0.77
B-1	0.98	0.18	0.00	0.00	0.03	0.77	-	0.60	0.63	0.72	0.77
B-2	0.70	0.12	0.00	0.00	0.03	0.55	-	0.62	0.65	0.74	0.79
B-3	0.99	0.20	0.00	0.00	0.05	0.74	-	0.59	0.63	0.72	0.76
B-4	1.01	0.21	0.00	0.00	0.05	0.75	-	0.58	0.62	0.71	0.75
B-5	0.64	0.12	0.00	0.00	0.03	0.49	-	0.60	0.63	0.72	0.77
B-6	1.38	0.26	0.00	0.00	0.05	1.07	-	0.60	0.63	0.72	0.77
B-7	1.11	0.19	0.00	0.00	0.03	0.89	-	0.62	0.65	0.74	0.79
B-8	0.28	0.00	0.00	0.00	0.00	0.28	-	0.82	0.84	0.89	0.95
B-9	1.40	0.00	0.60	0.00	0.30	0.50	-	0.83	0.85	0.90	0.96
B-10	0.50	0.00	0.50	0.00	0.00	0.00	-	0.90	0.92	0.96	1.00
B-11	0.47	0.00	0.00	0.00	0.00	0.47	-	0.82	0.84	0.89	0.95
B-12	0.98	0.46	0.00	0.00	0.00	0.52	-	0.40	0.46	0.60	0.50
B-13	0.22	0.22	0.00	0.00	0.00	0.00	-	0.15	0.25	0.50	0.00
Total to Pond	10.66	1.96	1.10	0.00	0.57	7.03	-	0.61	0.65	0.73	0.78
OS-1	1.52	0.36	0.91	0.27	0.00	0.00	-	0.63	0.68	0.80	0.76

Composite Runoff Coefficients and Percent Imperviousness for Historic Drainage Basins

H-1	7.34	7.34	0.00	0.00	0.00		0.00	0.15	0.25	0.50	0.00
H-2	10.64	10.64	0.00	0.00	0.00		0.00	0.15	0.25	0.50	0.00

PROJECT: Vista Ridge Commercial
LOCATION: Mountain View Blvd and Highway 7
Erie, CO

Project No.: SH7000001.01
Date: July 26, 2016
Engineer: Phil Dalrymple
Checked By: Brandon McCrary, PE

Galloway
Planning. Architecture. Engineering.

Developed Conditions - Time of Concentration Runoff Calculations

Basin Data			Intial/Overland Time (T_i)			Travel Time (T_t)					T_c Check Urbanized Basins			Final T_c		
Basin	Area (acre)	C_5	Length (ft)	Slope (%)	T_i (min)	Length (ft)	Slope (%)	Conv. Coeff.	Vel. (fps)	T_t (min)	Total Length	Comp. Tc	T_c=(L/180)+10	Final Tc	C_5	C_{100}
A-1	7.34	0.60	50	2.0%	5.2	745	3.1%	20	3.5	3.5	795	8.7	14.4	**8.7**	0.60	0.72
B-1	0.98	0.60	75	10.7%	3.6	266	1.5%	20	2.4	1.8	341	5.4	11.9	**5.4**	0.60	0.72
B-2	0.70	0.62	73	9.6%	3.5	225	2.6%	20	3.2	1.2	298	5.0	11.7	**5.0**	0.62	0.74
B-3	0.99	0.59	70	10.7%	3.6	225	2.8%	20	3.3	1.1	295	5.0	11.6	**5.0**	0.59	0.72
B-4	1.01	0.58	70	10.7%	3.6	225	3.1%	20	3.5	1.1	295	5.0	11.6	**5.0**	0.58	0.71
B-5	0.64	0.60	63	14.0%	3.0	195	3.1%	20	3.5	0.9	258	5.0	11.4	**5.0**	0.60	0.72
B-6	1.38	0.60	83	14.4%	3.5	420	1.7%	20	2.6	2.7	503	6.1	12.8	**6.1**	0.60	0.72
B-7	1.11	0.62	107	13.0%	3.9	225	1.0%	20	2.0	1.9	332	5.7	11.8	**5.7**	0.62	0.74
B-8	0.28	0.82	15	1.00%	2.0	185	1.1%	20	2.1	1.5	200	5.0	11.1	**5.0**	0.82	0.89
B-9	1.40	0.83	20	2.0%	1.7	1575	2.8%	20	3.3	7.8	1595	9.6	18.9	**9.6**	0.83	0.90
B-10	0.50	0.90	10	2.0%	0.9	1330	2.6%	20	3.2	6.8	1340	7.8	17.4	**7.8**	0.90	0.96
B-11	0.47	0.82	20	2.0%	1.8	145	4.2%	20	4.1	0.6	165	5.0	10.9	**5.0**	0.82	0.89
B-12	0.98	0.40	75	3.0%	7.7	230	1.3%	20	2.3	1.7	305	9.4	11.7	**9.4**	0.40	0.60
B-13	0.22	0.15	5	2.0%	3.1	5	20.0%	7	8.9	0.0	10	5.0	10.1	**5.0**	0.15	0.50
OS-1	1.52	0.63												**6.1**	0.63	0.80

Historic Conditions - Time of Concentration Runoff Calculations

Basin Data			Intial/Overland Time (T_i)			Travel Time (T_t)					T_c Check Urbanized Basins			Final T_c		
Basin	Area (acre)	C_5	Length (ft)	Slope (%)	T_i (min)	Length (ft)	Slope (%)	Conv. Coeff.	Vel. (fps)	T_t (min)	Total Length	Comp. Tc	T_c=(L/180)+10	Final Tc	C_5	C_{100}
H-1	7.34	0.15	50	2.0%	9.8	640	3.0%	7	1.2	8.8	690	18.6	13.8	**13.8**	0.15	0.50
H-2	10.64	0.15	50	4.0%	7.8	1610	3.9%	7	3.9	6.8	1660	14.5	19.2	**14.5**	0.15	0.50

PROJECT: Vista Ridge Commercial
LOCATION: Mountain View Blvd and Highway 7
Erie, CO

Project No.: SH7000001.01
Date: July 26, 2016
Engineer: Phil Dalrymple
Checked By: Brandon McCrary, PE

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Planning. Architecture. Engineering.

Developed Condition: Rational Method Routing - 5 Year Storm Event

5 -YR EVENT ROUTING CALCULATIONS
1.43 INCHES/ HOUR POINT RAINFALL (PER TABLE 8700-2, ERIE CRITERIA)

Basin/Sub-Basin	DIRECT RUNOFF								TOTAL RUNOFF				STREET		PIPE			TRAVEL TIME				Description
	Design Point	Basin	Area (ac)	Runoff Coeff.	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Slope (%)	Flow (cfs)	Flow (cfs)	Slope (%)	Size (in)	Length (ft)	Conv. Coef.	Velocity (fps)	TT (min)	
A-1	1	A-1	7.340	0.60	8.7	4.38	4.08	17.9														
B-1	2	B-1	0.980	0.60	5.4	0.58	4.74	2.8														
B-2	3	B-2	0.700	0.62	5.0	0.44	4.85	2.1														
B-3	4	B-3	0.990	0.59	5.0	0.58	4.85	2.8														
B-4	5	B-4	1.010	0.58	5.0	0.59	4.85	2.8														
B-5	6	B-5	0.640	0.60	5.0	0.38	4.85	1.9														
B-6	7	B-6	1.380	0.60	6.1	0.82	4.58	3.8														
B-7	8	B-7	1.110	0.62	5.7	0.69	4.67	3.2														
B-8	9	B-8	0.280	0.82	5.0	0.23	4.85	1.1														
B-9	10	B-9	1.400	0.83	9.6	1.17	3.93	4.6														
B-10	11	B-10	0.500	0.90	7.8	0.45	4.25	1.9														
B-11	12	B-11	0.470	0.82	5.0	0.39	4.85	1.9														
B-12	13	B-12	0.980	0.40	9.4	0.39	3.97	1.6														
B-13	14	B-13	0.220	0.15	5.0	0.03	4.85	0.2														
OS-1	7	OS-1	1.520	0.63	6.1	0.96	4.59	4.4														

Historic Condition: Rational Method Routing - 5 Year Storm Event

5 -YR EVENT ROUTING CALCULATIONS
1.43 INCHES/ HOUR POINT RAINFALL (PER TABLE 8700-2, ERIE CRITERIA)

Basin/Sub-Basin	DIRECT RUNOFF								TOTAL RUNOFF				STREET		PIPE			TRAVEL TIME				Description
	Design Point	Basin	Area (ac)	Runoff Coeff.	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Slope (%)	Flow (cfs)	Flow (cfs)	Slope (%)	Size (in)	Length (ft)	Conv. Coef.	Velocity (fps)	TT (min)	
H-1	1	H-1	7.340	0.15	13.8	1.10	3.37	3.7														
H-2	2	H-2	10.640	0.15	14.5	1.60	3.29	5.3														

PROJECT: Vista Ridge Commercial
LOCATION: Mountain View Blvd and Highway 7
Erie, CO

Project No.: SH7000001.01
Date: July 26, 2016
Engineer: Phil Dalrymple
Checked By: Brandon McCrary, PE

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Developed Condition: Rational Method Routing - 100 Year Storm Event

100	-YR EVENT ROUTING CALCULATIONS
2.7	INCHES/ HOUR POINT RAINFALL (PER TABLE 8700-2, ERIE CRITERIA)

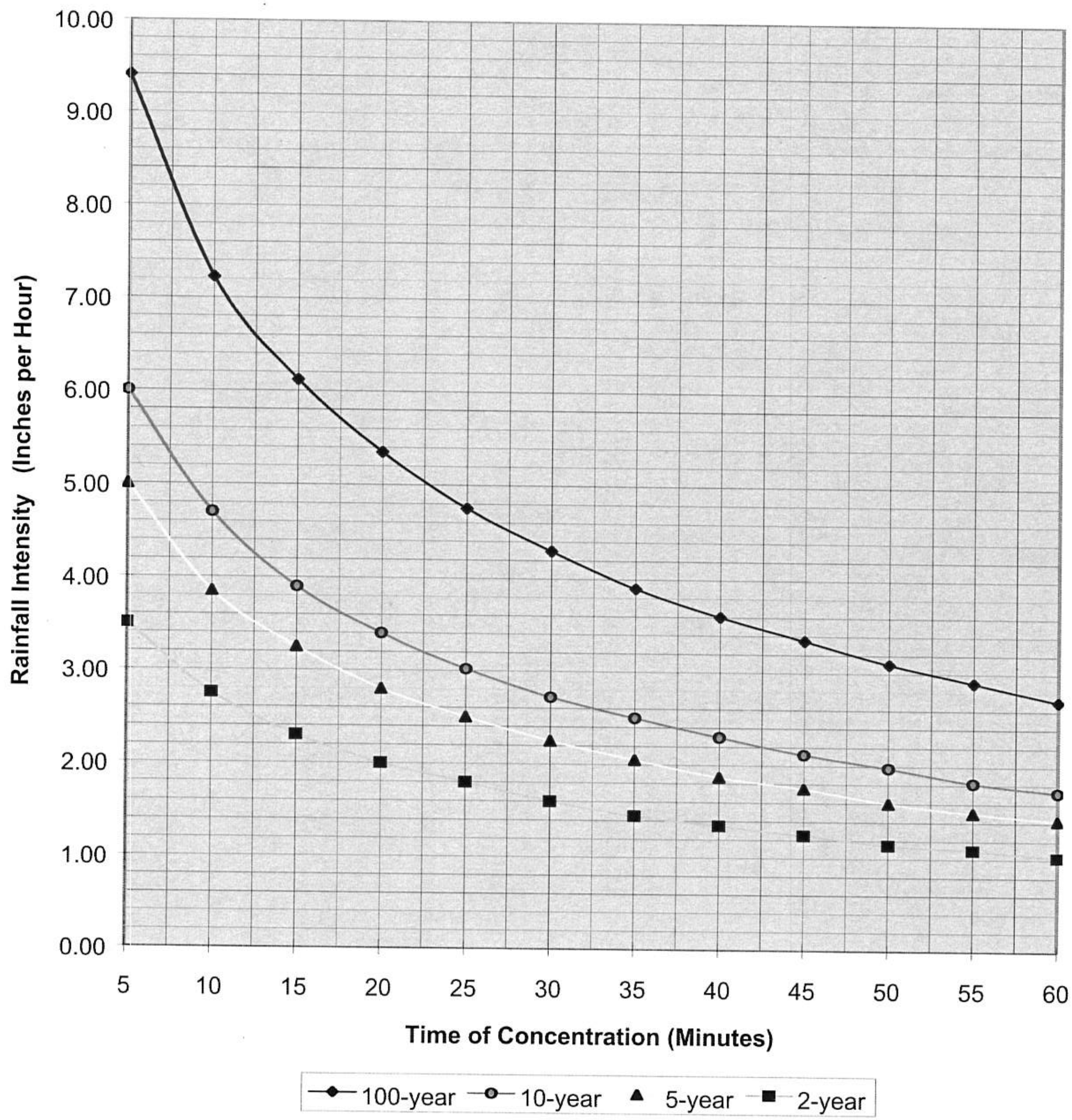
Basin/Sub-Basin	DIRECT RUNOFF								TOTAL RUNOFF				STREET		PIPE			TRAVEL TIME				
	Design Point	Basin	Area (ac)	Runoff Coeff.	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Slope (%)	Flow (cfs)	Flow (cfs)	Slope (%)	Size (in)	Length (ft)	Conv. Coef.	Velocity (fps)	TT (min)	Description
A-1	1	A-1	7.340	0.72	8.7	5.28	7.70	40.7														
B-1	2	B-1	0.980	0.72	5.4	0.71	8.95	6.3														
B-2	3	B-2	0.700	0.74	5.0	0.51	9.16	4.7														
B-3	4	B-3	0.990	0.72	5.0	0.71	9.16	6.5														
B-4	5	B-4	1.010	0.72	5.0	0.73	9.16	6.7														
B-5	6	B-5	0.640	0.71	5.0	0.45	9.16	4.2														
B-6	7	B-6	1.380	0.72	6.1	0.99	8.65	8.6														
B-7	8	B-7	1.110	0.74	5.7	0.82	8.82	7.2														
B-8	9	B-8	0.280	0.89	5.0	0.25	9.16	2.3														
B-9	10	B-9	1.400	0.90	9.6	1.26	7.43	9.4														
B-10	11	B-10	0.500	0.96	7.8	0.48	8.02	3.9														
B-11	12	B-11	0.470	0.89	5.0	0.42	9.16	3.8														
B-12	13	B-12	0.980	0.60	9.4	0.59	7.49	4.4														
B-13	14	B-13	0.220	0.50	5.0	0.11	9.16	1.0														
OS-1	7	OS-1	1.520	0.80	6.1	1.22	8.93	10.9														

Historic Condition: Rational Method Routing - 100 Year Storm Event

100	-YR EVENT ROUTING CALCULATIONS
2.7	INCHES/ HOUR POINT RAINFALL (PER TABLE 8700-2, ERIE CRITERIA)

Basin/Sub-Basin	DIRECT RUNOFF								TOTAL RUNOFF				STREET		PIPE			TRAVEL TIME				
	Design Point	Basin	Area (ac)	Runoff Coeff.	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Tc (min)	CA (ac)	I (in/hr)	Q (cfs)	Slope (%)	Flow (cfs)	Flow (cfs)	Slope (%)	Size (in)	Length (ft)	Conv. Coef.	Velocity (fps)	TT (min)	Description
H-1	1	H-1	7.340	0.50	13.8	3.67	6.36	23.4														
H-2	2	H-2	10.640	0.50	14.5	5.32	6.22	33.1														

Rainfall Intensity Duration Curves



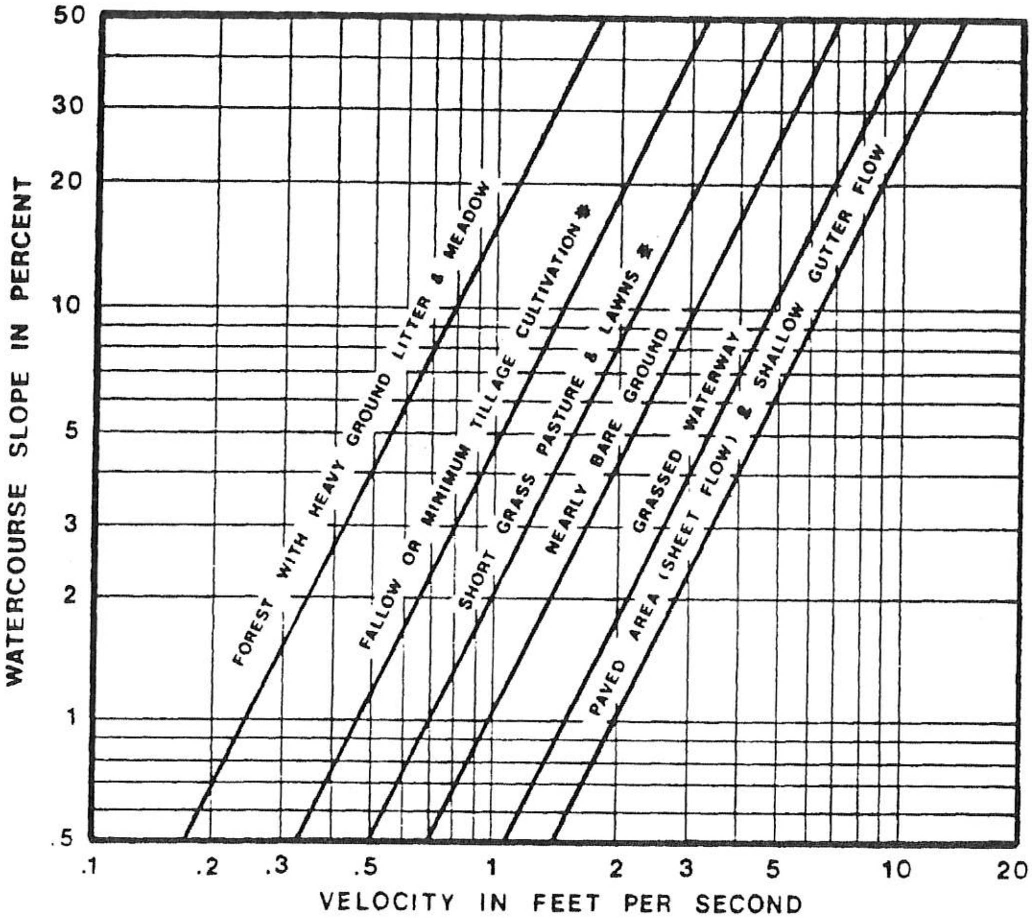


Figure RO-1—Estimate of Average Overland Flow Velocity for Use With the Rational Formula

L = length of overland flow (500 ft maximum for non-urban land uses, 300 ft maximum for urban land uses)

S = average basin slope (ft/ft)

Equation RO-3 is adequate for distances up to 500 feet. Note that, in some urban watersheds, the overland flow time may be very small because flows quickly channelize.

2.4.2 Overland Travel Time

For catchments with overland and channelized flow, the time of concentration needs to be considered in combination with the overland travel time, t_t, which is calculated using the hydraulic properties of the swale, ditch, or channel. For preliminary work, the overland travel time, t_t, can be estimated with the help of Figure RO-1 or the following equation (Guo 1999):

$$V = C_v S_w^{0.5} \qquad \text{(RO-4)}$$

in which:

V = velocity (ft/sec)

C_v = conveyance coefficient (from Table RO-2)

S_w = watercourse slope (ft/ft)

Table RO-2—Conveyance Coefficient, C_v

Type of Land Surface	Conveyance Coefficient, C_v
Heavy meadow	2.5
Tillage/field	5
Short pasture and lawns	7
Nearly bare ground	10
Grassed waterway	15
Paved areas and shallow paved swales	20

The time of concentration, t_c, is then the sum of the initial flow time, t_i, and the travel time, t_t, as per Equation RO-2.

2.4.3 First Design Point Time of Concentration in Urban Catchments

Using this procedure, the time of concentration at the first design point (i.e., initial flow time, t_i) in an urbanized catchment should not exceed the time of concentration calculated using Equation RO-5.

$$t_c = \frac{L}{180} + 10 \qquad \text{(RO-5)}$$

in which:

t_c = maximum time of concentration at the first design point in an urban watershed (minutes)

Table RO-3—Recommended Percentage Imperviousness Values

Land Use or Surface Characteristics	Percentage Imperviousness
Business:	
Commercial areas	95
Neighborhood areas	85
Residential:	
Single-family	*
Multi-unit (detached)	60
Multi-unit (attached)	75
Half-acre lot or larger	*
Apartments	80
Industrial:	
Light areas	80
Heavy areas	90
Parks, cemeteries	5
Playgrounds	10
Schools	50
Railroad yard areas	15
Undeveloped Areas:	
Historic flow analysis	2
Greenbelts, agricultural	2
Off-site flow analysis (when land use not defined)	45
Streets:	
Paved	100
Gravel (packed)	40
Drive and walks	90
Roofs	90
Lawns, sandy soil	0
Lawns, clayey soil	0

* See Figures RO-3 through RO-5 for percentage imperviousness.

$$C_A = K_A + \left(1.31i^3 - 1.44i^2 + 1.135i - 0.12\right) \text{ for } C_A \geq 0\text{, otherwise } C_A = 0 \qquad \text{(RO-6)}$$

$$C_{CD} = K_{CD} + \left(0.858i^3 - 0.786i^2 + 0.774i + 0.04\right) \qquad \text{(RO-7)}$$

$$C_B = \left(C_A + C_{CD}\right)/2$$

Table RO-5— Runoff Coefficients, *C*

Percentage Imperviousness	Type C and D NRCS Hydrologic Soil Groups					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
0%	0.04	0.15	0.25	0.37	0.44	0.50
5%	0.08	0.18	0.28	0.39	0.46	0.52
10%	0.11	0.21	0.30	0.41	0.47	0.53
15%	0.14	0.24	0.32	0.43	0.49	0.54
20%	0.17	0.26	0.34	0.44	0.50	0.55
25%	0.20	0.28	0.36	0.46	0.51	0.56
30%	0.22	0.30	0.38	0.47	0.52	0.57
35%	0.25	0.33	0.40	0.48	0.53	0.57
40%	0.28	0.35	0.42	0.50	0.54	0.58
45%	0.31	0.37	0.44	0.51	0.55	0.59
50%	0.34	0.40	0.46	0.53	0.57	0.60
55%	0.37	0.43	0.48	0.55	0.58	0.62
60%	0.41	0.46	0.51	0.57	0.60	0.63
65%	0.45	0.49	0.54	0.59	0.62	0.65
70%	0.49	0.53	0.57	0.62	0.65	0.68
75%	0.54	0.58	0.62	0.66	0.68	0.71
80%	0.60	0.63	0.66	0.70	0.72	0.74
85%	0.66	0.68	0.71	0.75	0.77	0.79
90%	0.73	0.75	0.77	0.80	0.82	0.83
95%	0.80	0.82	0.84	0.87	0.88	0.89
100%	0.89	0.90	0.92	0.94	0.95	0.96
	TYPE B NRCS HYDROLOGIC SOILS GROUP					
0%	0.02	0.08	0.15	0.25	0.30	0.35
5%	0.04	0.10	0.19	0.28	0.33	0.38
10%	0.06	0.14	0.22	0.31	0.36	0.40
15%	0.08	0.17	0.25	0.33	0.38	0.42
20%	0.12	0.20	0.27	0.35	0.40	0.44
25%	0.15	0.22	0.30	0.37	0.41	0.46
30%	0.18	0.25	0.32	0.39	0.43	0.47
35%	0.20	0.27	0.34	0.41	0.44	0.48
40%	0.23	0.30	0.36	0.42	0.46	0.50
45%	0.26	0.32	0.38	0.44	0.48	0.51
50%	0.29	0.35	0.40	0.46	0.49	0.52
55%	0.33	0.38	0.43	0.48	0.51	0.54
60%	0.37	0.41	0.46	0.51	0.54	0.56
65%	0.41	0.45	0.49	0.54	0.57	0.59
70%	0.45	0.49	0.53	0.58	0.60	0.62
75%	0.51	0.54	0.58	0.62	0.64	0.66
80%	0.57	0.59	0.63	0.66	0.68	0.70
85%	0.63	0.66	0.69	0.72	0.73	0.75
90%	0.71	0.73	0.75	0.78	0.80	0.81
95%	0.79	0.81	0.83	0.85	0.87	0.88
100%	0.89	0.90	0.92	0.94	0.95	0.96

Appendix E
Detention Pond Calculations

DETENTION VOLUME BY THE FULL SPECTRUM METHOD

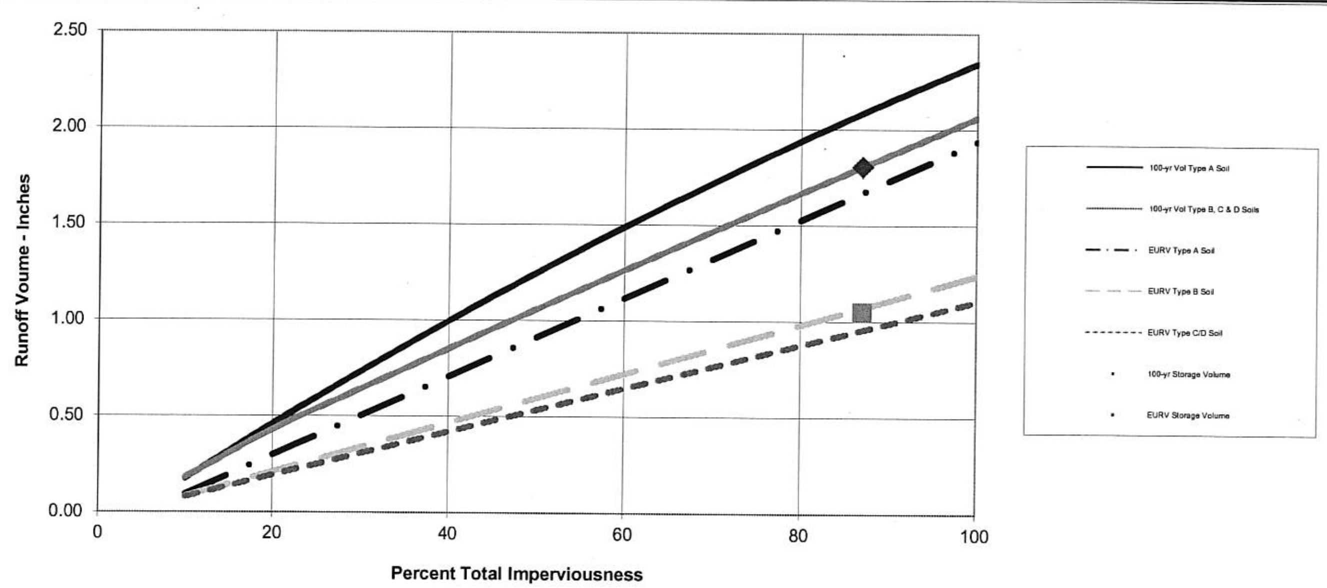
Project: Vista Ridge Commercial
Basin ID: Pond A1 Modified Volume

* User input data shown in blue.

Area of Watershed (acres)	27.72		
Subwatershed Imperviousness	87.0%		
Level of Minimizing Directly Connected Impervious Area (MDCIA)	0	0	
Effective Imperviousness[1]	87.0%		
Hydrologic Soil Type	**Percentage of Area**	**Area (acres)**	
Type A		0.0	
Type B		0.0	
Type C or D	100.0%	27.7	

Recommended Horton's Equation Parameters for CUHP		
Infiltration (inches per hour)		**Decay Coefficient--α**
Initial--f_i	**Final--fo**	
3	0.5	0.0018

	Detention Volumes [2,5]		**Maximum Allowable Release Rate, cfs**[3]
	(watershed inches)	**(acre-feet)**	
Excess Urban Runoff Volume[4]	1.05	2.43	Design Oulet to Empty EURV in 72 Hours
100-year Detention Volume Including WQCV [5]	1.81	4.17	27.72



Notes:

1) Effective imperviousness is based on Figure ND-1 of the Urban Storm Drainage Criteria Manual (USDCM).
2) Results shown reflect runoff reduction from Level 1 or 2 MDCIA and are plotted at the watershed's total imperviousness value; the impact of MDCIA is reflected by the results being below the curves.
3) Maximum allowable release rates for 100-year event are based on Table SO-1. Outlet for the Excess Urban Runoff Volume (EURV) to be designed to empty out the EURV in 72 hours. Outlet design is similar to one for the WQCV outlet of an extended detention basin (i.e., perforated plate with a micro-pool) and extends to top of EURV water surface elevation.
4) EURV approximates the difference between developed and pre-developed runoff volume.
5) 100-yr detention volume includes EURV. No need to add more volume for WQCV or EURV

POND VOLUME CALCULATIONS

Subdivision
Location CO, Erie

Project Name: Vista Ridge Commercial
Project No. SH7000001.01
By: PJD
Checked By: BSM
Date: 5/24/16

Volume=1/3 x Depth x (A+B+(A*B)^0.5)
A - Upper Surface
B - Lower Surface

Pond Name Here

Stage	Stage Elevation	Stage Surface Area (square feet)	Stage Volume (cubic feet)	Cumulative Volume (cubic feet)	Cumulative Volume (acre feet)
0.00	5225.00	10	0	0	0.00
1.00	5226.00	723	273	273	0.01
2.00	5227.00	7,234	3,415	3,688	0.08
3.00	5228.00	35,908	19,753	23,441	0.54
4.00	5229.00	39,113	37,499	60,940	1.40
5.00	5230.00	42,181	40,637	101,577	2.33
6.00	5231.00	45,291	43,727	145,304	3.34
7.00	5232.00	48,575	46,923	192,227	4.41

Volume (cubic feet)	Modified Volume	Water Surface Elevation	Original Volume	Original WSE
WQCV*1.2	45,302.00	5228.59	37461.00	5228.12
EURV	105,850.00	5230.10	104544.00	5230.12*
100-Year Detention	181,645.00	5231.78	161172.00	5231.64

*10-year WSE

DETENTION VOLUME BY THE FULL SPECTRUM METHOD

Project: Vista Ridge Commercial
Basin ID: Pond A1A

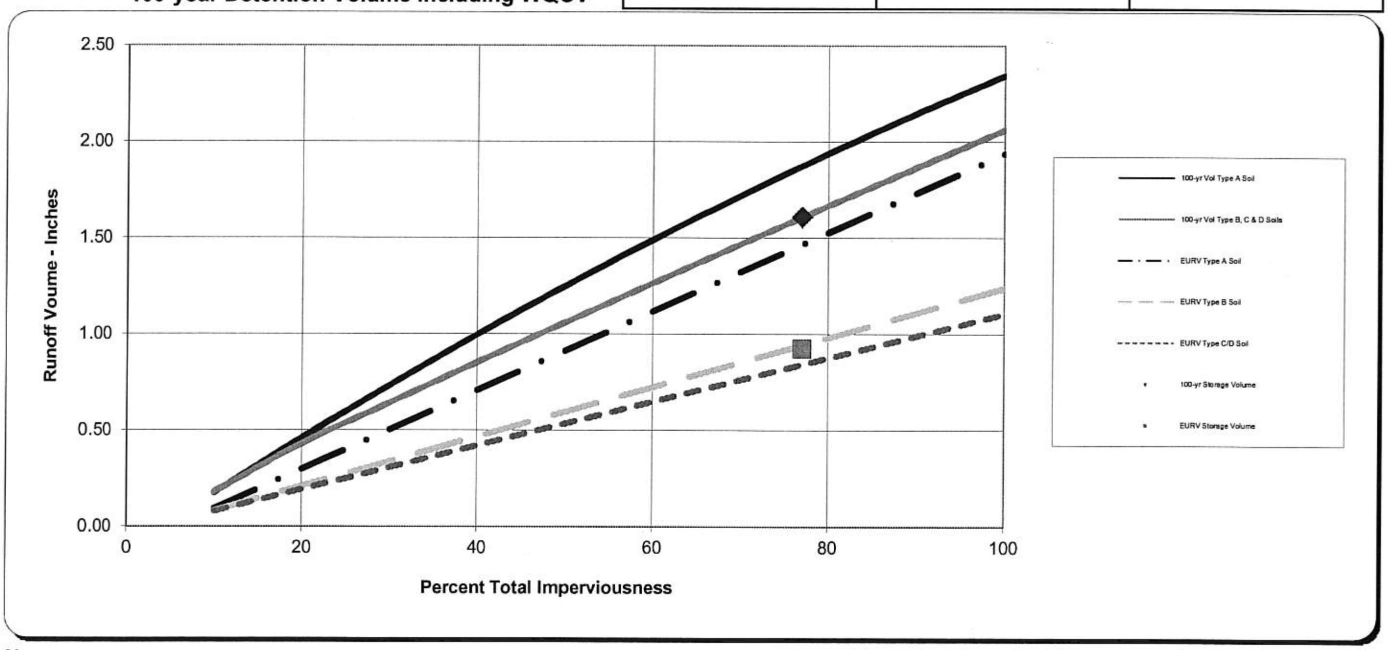
* User input data shown in blue.

Area of Watershed (acres)	7.34	
Subwatershed Imperviousness	77.0%	
Level of Minimizing Directly Connected Impervious Area (MDCIA)	0	0
Effective Imperviousness[1]	77.0%	

Hydrologic Soil Type	**Percentage of Area**	**Area (acres)**
Type A		0.0
Type B		0.0
Type C or D	100.0%	7.3

Recommended Horton's Equation Parameters for CUHP		
Infiltration (inches per hour)		**Decay**
Initial--f_i	**Final--f_o**	**Coefficient--α**
3	0.5	0.0018

	Detention Volumes[2,5]		**Maximum Allowable**
	(watershed inches)	**(acre-feet)**	**Release Rate, cfs**[3]
Excess Urban Runoff Volume[4]	0.93	0.57	Design Oulet to Empty EURV in 72 Hours
100-year Detention Volume Including WQCV[5]	1.61	0.98	7.34



Notes:

1) Effective imperviousness is based on Figure ND-1 of the Urban Storm Drainage Criteria Manual (USDCM).

2) Results shown reflect runoff reduction from Level 1 or 2 MDCIA and are plotted at the watershed's total imperviousness value; the impact of MDCIA is reflected by the results being below the curves.

3) Maximum allowable release rates for 100-year event are based on Table SO-1. Outlet for the Excess Urban Runoff Volume (EURV) to be designed to empty out the EURV in 72 hours. Outlet design is similar to one for the WQCV outlet of an extended detention basin (i.e., perforated plate with a micro-pool) and extends to top of EURV water surface elevation.

4) EURV approximates the difference between developed and pre-developed runoff volume.

5) 100-yr detention volume includes EURV. No need to add more volume for WQCV or EURV

POND VOLUME CALCULATIONS

Subdivision Pond A1A
Location CO, Erie

Project Name: Vista Ridge Commercial
Project No. SH7000001.01
By: PJD
Checked By: BSM
Date: 5/19/16

Volume=1/3 x Depth x (A+B+(A*B)^0.5)
A - Upper Surface
B - Lower Surface

Pond Name Here

Stage	**Stage Elevation**	**Stage Surface Area** (square feet)	**Stage Volume** (cubic feet)	**Cumulative Volume** (cubic feet)	**Cumulative Volume** (acre feet)
0.00	5259.00	10	0	0	0.00
1.00	5260.00	3,737	1,313	1,313	0.03
2.00	5261.00	7,141	5,348	6,661	0.15
3.00	5262.00	9,408	8,248	14,909	0.34
4.00	5263.00	11,727	10,546	25,455	0.58
5.00	5264.00	14,084	12,888	38,343	0.88
6.00	5265.00	16,491	15,272	53,615	1.23
6.50	5265.50	17,705	8,547	62,162	1.43

Volume (cubic feet)	**Volume**	**Water Surface Elevation**
WQCV*1.2	10,018.00	5261.41
EURV	24,829.00	5262.95
100-Year Detention	42,689.00	5264.29

Design Procedure Form: Extended Detention Basin (EDB)

Designer:	Phil Dalrymple
Company:	Galloway & Company, Inc.
Date:	May 20, 2016
Project:	Vista Rdige Commercial
Location:	Erie, Colorado

1. Basin Storage Volume	
A) Effective Imperviousness of Tributary Area, I_a	I_a = 77.0 %
B) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	i = 0.770
C) Contributing Watershed Area	Area = 7.340 ac
D) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm	d_6 = ______ in
E) Design Concept (Select EURV when also designing for flood control)	Choose One ○ Water Quality Capture Volume (WQCV) ◉ Excess Urban Runoff Volume (EURV)
F) Design Volume (1.2 WQCV) Based on 40-hour Drain Time ($V_{DESIGN} = (1.0 * (0.91 * i^3 - 1.19 * i^2 + 0.78 * i) / 12 * Area * 1.2)$)	V_{DESIGN}= 0.228 ac-ft
G) For Watersheds Outside of the Denver Region, Water Quality Capture Volume (WQCV) Design Volume ($V_{WQCV\ OTHER} = (d_6*(V_{DESIGN}/0.43))$)	$V_{DESIGN\ OTHER}$= ______ ac-ft
H) User Input of Water Quality Capture Volume (WQCV) Design Volume (Only if a different WQCV Design Volume is desired)	$V_{DESIGN\ USER}$= ______ ac-ft
I) Predominant Watershed NRCS Soil Group	Choose One ○ A ○ B ◉ C / D
J) Excess Urban Runoff Volume (EURV) Design Volume For HSG A: EURVA = (0.1878i - 0.0104)*Area For HSG B: $EURV_B$ = (0.1178i - 0.0042)*Area For HSG C/D: $EURV_{C/D}$ = (0.1043i - 0.0031)*Area	EURV = 0.567 ac-f t
2. Basin Shape: Length to Width Ratio (A basin length to width ratio of at least 2:1 will improve TSS reduction.)	L : W = 2.1 : 1
3. Basin Side Slopes A) Basin Maximum Side Slopes (Horizontal distance per unit vertical, 4:1 or flatter preferred)	Z = 4.00 ft / ft
4. Inlet A) Describe means of providing energy dissipation at concentrated inflow locations:	

Design Procedure Form: Extended Detention Basin (EDB)

Designer:	**Phil Dalrymple**
Company:	**Galloway & Company, Inc.**
Date:	**May 20, 2016**
Project:	**Vista Rdige Commercial**
Location:	**Erie, Colorado**

5. Forebay

A) Minimum Forebay Volume (V_{FMIN} = 3% of the WQCV)	V_{FMIN} = ______ ac-ft
B) Actual Forebay Volume	V_F = ______ ac-ft
C) Forebay Depth (D_F = 18 inch maximum)	D_F = ______ in
D) Forebay Discharge	
i) Undetained 100-year Peak Discharge	Q_{100} = ______ cfs
ii) Forebay Discharge Design Flow ($Q_F = 0.02 * Q_{100}$)	Q_F = ______ cfs
E) Forebay Discharge Design	Choose One: ○ Berm With Pipe ○ Wall with Rect. Notch ○ Wall with V-Notch Weir
F) Discharge Pipe Size (minimum 8-inches)	Calculated D_P = ______ in
G) Rectangular Notch Width	Calculated W_N = ______ in

6. Trickle Channel

A) Type of Trickle Channel	Choose One: ◉ Concrete ○ Soft Bottom
F) Slope of Trickle Channel	S = 0.0078 ft / ft

7. Micropool and Outlet Structure

A) Depth of Micropool (2.5-feet minimum)	D_M = 2.5 ft
B) Surface Area of Micropool (10 ft^2 minimum)	A_M = 9 sq ft
C) Outlet Type	Choose One: ◉ Orifice Plate ○ Other (Describe):
D) Depth of Design Volume (EURV or 1.2 WQCV) Based on the Design Concept Chosen Under 1.E.	H = 2.15 feet
E) Volume to Drain Over Prescribed Time	EURV = 0.567 ac-ft
F) Drain Time (Min T_D for WQCV= 40 hours; Max T_D for EURV= 72 hours)	T_D = 72 hours
G) Recommended Maximum Outlet Area per Row, (A_o)	A_o = 0.72 square inches
H) Orifice Dimensions: i) Circular Orifice Diameter or	$D_{orifice}$ = 15 / 16 inches
I) Number of Columns	n_c = 1 number
J) Actual Design Outlet Area per Row (A_o)	A_o = 0.69 square inches
K) Number of Rows (nr)	n_r = 6 number
L) Total Outlet Area (A_{ot})	A_{ot} = 4.5 square inches
M) Depth of WQCV (H_{WQCV}) (Estimate using actual stage-area-volume relationship and V_{WQCV})	H_{WQCV} = 3.8 feet DEPTH OF WQCV > DEPTH OF EURV ABOVE LOWEST PERFORATION
N) Ensure Minimum 40 Hour Drain Time for WQCV	$T_{D\ WQCV}$ = 21.4 hours **DRAIN TIME < 40 HOURS**

Design Procedure Form: Extended Detention Basin (EDB)

Designer:	Phil Dalrymple
Company:	Galloway & Company, Inc.
Date:	May 20, 2016
Project:	Vista Rdige Commercial
Location:	Erie, Colorado

8. Initial Surcharge Volume

A) Depth of Initial Surcharge Volume (Minimum recommended depth is 4 inches) — D_{IS} = 6.0 in

B) Minimum Initial Surcharge Volume (Minimum volume of 0.3% of the WQCV) — V_{IS} = 24.8 cu ft

C) Initial Surcharge Provided Above Micropool — V_s = 4.3 cu ft — INCREASE INITIAL SURCHARGE DEPTH OR SURFACE AREA OF MICROPOOL

9. Trash Rack

A) Type of Water Quality Orifice Used

Choose One
- (●) Circular (up to 1-1/4" diameter)
- ( ) Circular (greater than 1-1/4" diameter) OR Rectangular (2" high)

B) Water Quality Screen Open Area: $A_t = Aot * 38.5*(e^{-0.095D})$ — A_t = 157 square inches

C) For 1-1/4"", or Smaller, Circular Opening (See Fact Sheet T-12):

i) Width of Water Quality Screen and Concrete Opening ($W_{opening}$) — $W_{opening}$ = 12.0 inches

ii) Height of Water Quality Screen (H_{TR}) — H_{TR} = 53.8 inches

iii) Type of Screen, Describe if "Other"

Choose One
- ( ) S.S. Well Screen with 60% Open Area*
- ( ) Other (Describe):

Choose One
- ( ) Aluminum Amico-Klemp SR Series (or equal)
- ( ) Other (Describe):

inches

Design Procedure Form: Extended Detention Basin (EDB)

Designer: Phil Dalrymple
Company: Galloway & Company, Inc.
Date: May 20, 2016
Project: Vista Rdige Commercial
Location: Erie, Colorado

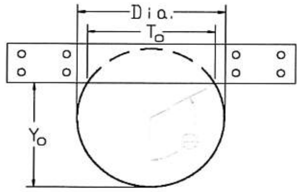
10. Overflow Embankment	
A) Describe embankment protection for 100-year and greater overtopping:	
B) Slope of Overflow Embankment (Horizontal distance per unit vertical, 4:1 or flatter preferred)	Z_E = 0.00 ft / ft TOO STEEP (< 3)
11. Vegetation	Choose One ○ Irrigated ◉ Not Irrigated
12. Access	
A) Describe Sediment Removal Procedures	

Notes:

RESTRICTOR PLATE SIZING FOR CIRCULAR VERTICAL ORIFICES

Project: Vista Ridge Commercial - Pond A1A
Basin ID:

X



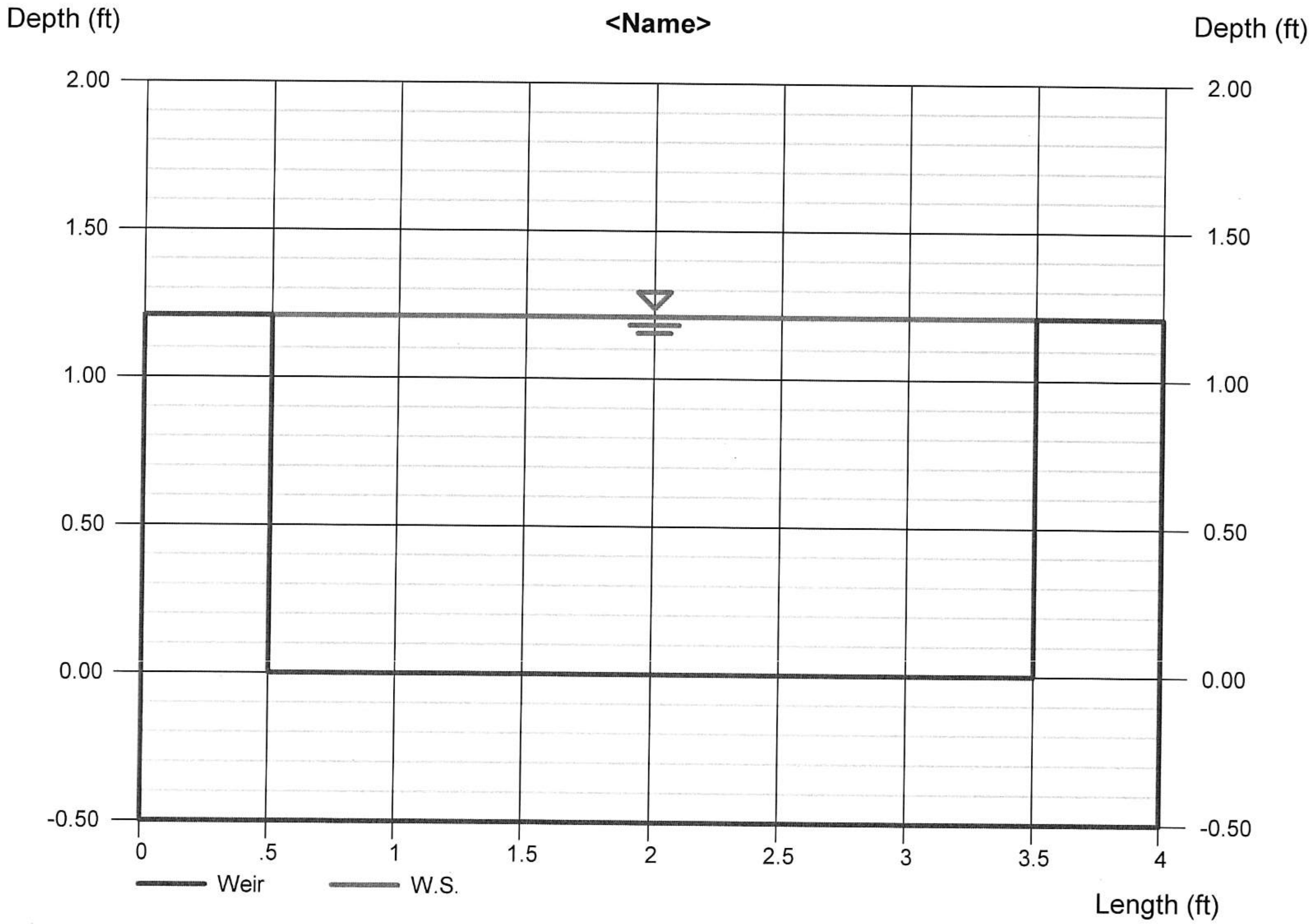
		#1 Vertical Orifice	#2 Vertical Orifice	
Sizing the Restrictor Plate for Circular Vertical Orifices or Pipes (Input)				
Water Surface Elevation at Design Depth	Elev: WS =	64.29		feet
Pipe/Vertical Orifice Entrance Invert Elevation	Elev: Invert =	58.80		feet
Required Peak Flow through Orifice at Design Depth	Q =	7.34		cfs
Pipe/Vertical Orifice Diameter (inches)	Dia =	18.0		inches
Orifice Coefficient	C_o =	0.60		
Full-flow Capacity (Calculated)				
Full-flow area	Af =	1.77		sq ft
Half Central Angle in Radians	Theta =	3.14		rad
Full-flow capacity	Qf =	18.5		cfs
	Percent of Design Flow =	252%		
Calculation of Orifice Flow Condition				
Half Central Angle (0<Theta<3.1416)	Theta =	1.38		rad
Flow area	A_o =	0.67		sq ft
Top width of Orifice (inches)	T_o =	17.67		inches
Height from Invert of Orifice to Bottom of Plate (feet)	Y_o =	0.61		feet
Elevation of Bottom of Plate	Elev Plate Bottom Edge =	59.41		feet
Resultant Peak Flow Through Orifice at Design Depth	Q_o =	7.4		cfs

Weir Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Friday, May 20 2016

<Name>

Rectangular Weir		**Highlighted**	
Crest	= Broad	Depth (ft)	= 1.21
Bottom Length (ft)	= 3.00	Q (cfs)	= 12.38
Total Depth (ft)	= 1.21	Area (sqft)	= 3.63
		Velocity (ft/s)	= 3.41
Calculations		Top Width (ft)	= 3.00
Weir Coeff. Cw	= 3.10		
Compute by:	Known Depth		
Known Depth (ft)	= 1.21		



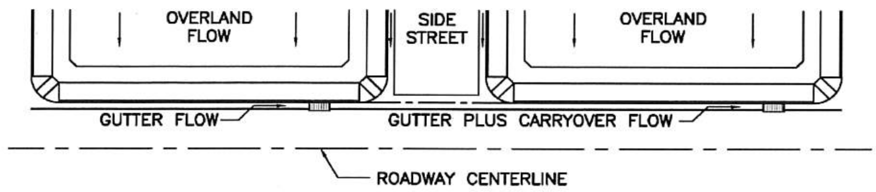
Appendix F

Inlet Calculations

Worksheet Protected

DESIGN PEAK FLOW FOR ONE-HALF OF STREET OR GRASS-LINED CHANNEL BY THE RATIONAL METHOD

Project: Vista Ridge Commercial
Inlet ID: Inlet B-8 (Design Point B-9)



Show Details

Design Flow: ONLY if already determined through other methods:

	Minor Storm	Major Storm	
(local peak flow for 1/2 of street OR grass-lined channel): $^{*}Q_{Known}$ =	4.6	9.4	cfs

* If you enter values in Row 14, skip the rest of this sheet and proceed to sheet Q-Allow or Area Inlet.

<--- FILL IN THIS SECTION OR...

FILL IN THE SECTIONS BELOW. <---

Geographic Information: (Enter data in the blue cells):

Subcatchment Area =		Acres
Percent Imperviousness =		%
NRCS Soil Type =		A, B, C, or D

Site Type:
- Site is Urban
- Site is Non-Urban

Flows Developed For:
- Street Inlets
- Area Inlets in a Median

	Slope (ft/ft)	Length (ft)
Overland Flow =		
Channel Flow =		

Rainfall Information: Intensity I (inch/hr) = $C_1 * P_1 / (C_2 + T_c)$ ^ C_3

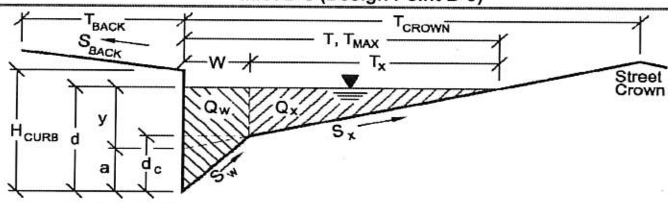
	Minor Storm	Major Storm	
Design Storm Return Period, T_r =			years
Return Period One-Hour Precipitation, P_1 =			inches
C_1 =			
C_2 =			
C_3 =			
User-Defined Storm Runoff Coefficient (leave this blank to accept a calculated value), C =			
User-Defined 5-yr. Runoff Coefficient (leave this blank to accept a calculated value), C_5 =			
Bypass (Carry-Over) Flow from upstream Subcatchments, Q_b =	0.0	0.0	cfs
Total Design Peak Flow, Q =	4.6	9.4	cfs

ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm)

(Based on Regulated Criteria for Maximum Allowable Flow Depth and Spread)

Project: Vista Ridge Commercial

Inlet ID: Inlet B-8 (Design Point B-9)



Gutter Geometry (Enter data in the blue cells)

Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	3.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.020	
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	14.5	ft
Gutter Width	W =	1.00	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.080	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.013	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	14.5	14.5	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	4.0	4.0	inches
Allow Flow Depth at Street Crown (leave blank for no)		☐	☐	check = yes

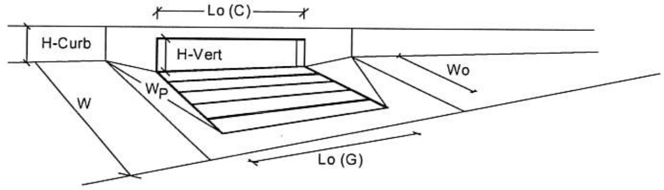
		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion				
MAJOR STORM Allowable Capacity is based on Depth Criterion	Q_{allow} =	**SUMP**	**SUMP**	**cfs**

Minor storm max. allowable capacity GOOD - greater than flow given on sheet 'Q-Peak'

Major storm max. allowable capacity GOOD - greater than flow given on sheet 'Q-Peak'

INLET IN A SUMP OR SAG LOCATION

Project = Vista Ridge Commercial
Inlet ID = Inlet B-8 (Design Point B-9)



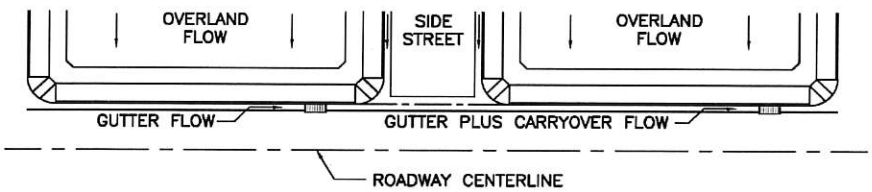
			MINOR	MAJOR	
	Design Information (Input)		MINOR	MAJOR	
	Type of Inlet	Inlet Type =	CDOT Type R Curb Opening		
	Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	3.00	3.00	inches
	Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
	Water Depth at Flowline (outside of local depression)	Ponding Depth =	6.0	6.0	inches ☐ Override Depths
	Grate Information		MINOR	MAJOR	
	Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
	Width of a Unit Grate	W_o =	N/A	N/A	feet
	Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
	Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
	Grate Weir Coefficient (typical value 2.15 - 3.60)	C_w (G) =	N/A	N/A	
	Grate Orifice Coefficient (typical value 0.60 - 0.80)	C_o (G) =	N/A	N/A	
	Curb Opening Information		MINOR	MAJOR	
	Length of a Unit Curb Opening	L_o (C) =	5.00	5.00	feet
	Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
	Height of Curb Orifice Throat in Inches	H_{throat} =	6.00	6.00	inches
	Angle of Throat (see USDCM Figure ST-5)	Theta =	63.40	63.40	degrees
Warning 1	Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
	Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
	Curb Opening Weir Coefficient (typical value 2.3-3.7)	C_w (C) =	3.60	3.60	
	Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)	C_o (C) =	0.67	0.67	
			MINOR	MAJOR	
	Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**6.0**	**6.0**	**cfs**
	WARNING: Inlet Capacity less than Q Peak for MAJOR Storm	$Q_{PEAK\ REQUIRED}$ =	4.6	9.4	cfs

Warning 1: Dimension entered is not a typical dimension for inlet type specified.

Worksheet Protected

DESIGN PEAK FLOW FOR ONE-HALF OF STREET OR GRASS-LINED CHANNEL BY THE RATIONAL METHOD

Project: Vista Ridge Commercial
Inlet ID: Inlet B-9 (Design Point B-9)



Show Details

Design Flow: ONLY if already determined through other methods:

	Minor Storm	Major Storm	
(local peak flow for 1/2 of street OR grass-lined channel): $^*Q_{Known}$ =	1.9	3.9	cfs

* If you enter values in Row 14, skip the rest of this sheet and proceed to sheet Q-Allow or Area Inlet.

<--- FILL IN THIS SECTION OR...

FILL IN THE SECTIONS BELOW. <---

Geographic Information: (Enter data in the blue cells):

Subcatchment Area =		Acres
Percent Imperviousness =		%
NRCS Soil Type =		A, B, C, or D

Site Type:
- Site is Urban
- Site is Non-Urban

Flows Developed For:
- Street Inlets
- Area Inlets in a Median

	Slope (ft/ft)	Length (ft)
Overland Flow =		
Channel Flow =		

Rainfall Information: Intensity I (inch/hr) = $C_1 * P_1 / (C_2 + T_c)^{\wedge} C_3$

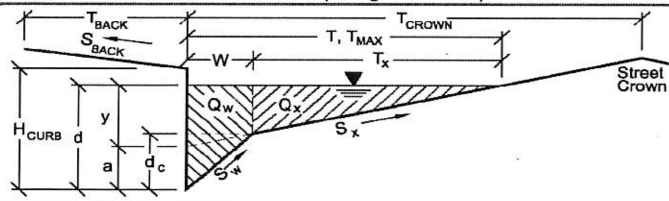
	Minor Storm	Major Storm	
Design Storm Return Period, T_r =			years
Return Period One-Hour Precipitation, P_1 =			inches
C_1 =			
C_2 =			
C_3 =			
User-Defined Storm Runoff Coefficient (leave this blank to accept a calculated value), C =			
User-Defined 5-yr. Runoff Coefficient (leave this blank to accept a calculated value), C_5 =			
Bypass (Carry-Over) Flow from upstream Subcatchments, Q_b =	0.0	0.0	cfs
Total Design Peak Flow, Q =	1.9	3.9	cfs

ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm)

(Based on Regulated Criteria for Maximum Allowable Flow Depth and Spread)

Project: Vista Ridge Commercial

Inlet ID: Inlet B-9 (Design Point B-9)



Gutter Geometry (Enter data in the blue cells)			
Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	3.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.020	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.020	
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	14.5	ft
Gutter Width	W =	1.00	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.080	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.033	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.013	

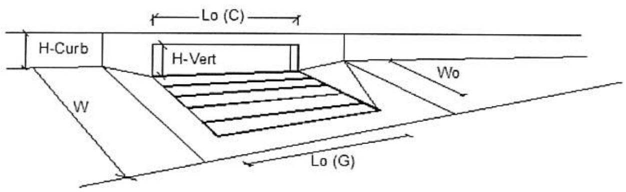
		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	14.5	14.5	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	4.0	4.0	inches
Allow Flow Depth at Street Crown (leave blank for no)		☐	☐	check = yes

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is based on Depth Criterion				
MAJOR STORM Allowable Capacity is based on Depth Criterion	Q_{allow} =	12.7	12.7	cfs

Minor storm max. allowable capacity GOOD - greater than flow given on sheet 'Q-Peak'
Major storm max. allowable capacity GOOD - greater than flow given on sheet 'Q-Peak'

INLET ON A CONTINUOUS GRADE

Project: Vista Ridge Commercial
Inlet ID: Inlet B-9 (Design Point B-9)



Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{LOCAL} =	3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No =	2	2	
Length of a Single Unit Inlet (Grate or Curb Opening)	L_o =	5.00	5.00	ft
Width of a Unit Grate (cannot be greater than W from Q-Allow)	W_o =	N/A	N/A	ft
Clogging Factor for a Single Unit Grate (typical min. value = 0.5)	C_f-G =	N/A	N/A	
Clogging Factor for a Single Unit Curb Opening (typical min. value = 0.1)	C_f-C =	0.10	0.10	
Street Hydraulics: OK - Q < maximum allowable from sheet 'Q-Allow'		MINOR	MAJOR	
Total Inlet Interception Capacity	Q =	1.90	3.71	cfs
Total Inlet Carry-Over Flow (flow bypassing inlet)	Q_b =	0.0	0.2	cfs
Capture Percentage = Q_a/Q_o =	C% =	100	95	%

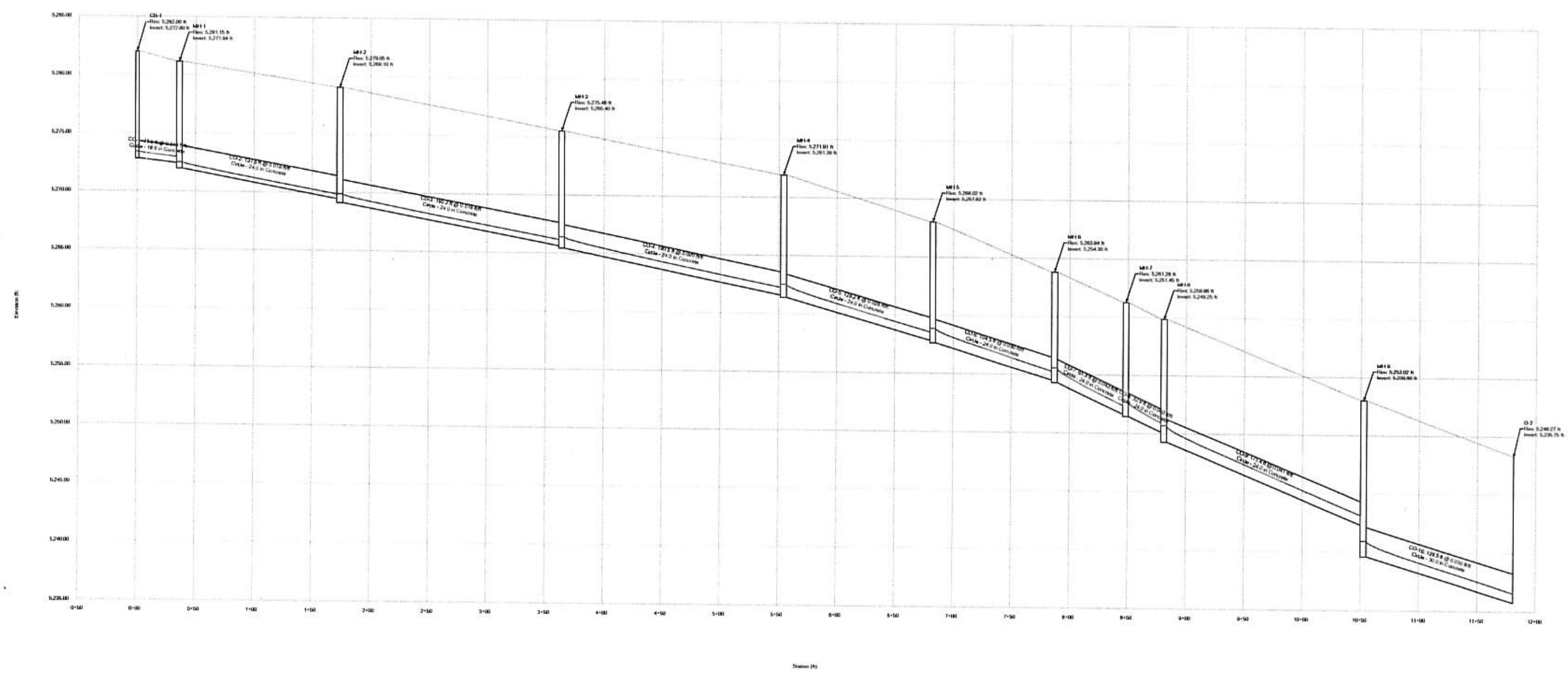
Appendix G

Storm Sewer Profiles (Hydraulic Grade Lines)

5-Year HGL's

Profile Report

Engineering Profile - Profile - 1 (SH7001_StormCAD.stsw)



FlexTable: Conduit Table

Label	Start Node	Invert (Start) (ft)	Stop Node	Invert (Stop) (ft)	Length (User Defined) (ft)	Slope (Calculated) (ft/ft)	Diameter (in)	Manning's n	Flow (cfs)	Velocity (ft/s)	Capacity (Full Flow) (cfs)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)
CO-1	CB-1	5,272.80	MH-1	5,272.44	36.0	0.010	18.0	0.013	2.80	5.03	10.50	5,273.44	5,272.97
CO-2	MH-1	5,271.94	MH-2	5,269.45	137.5	0.018	24.0	0.013	2.80	6.05	30.44	5,272.52	5,269.86
CO-3	MH-2	5,269.10	MH-3	5,265.60	190.2	0.018	24.0	0.013	4.90	7.15	30.69	5,269.88	5,266.39
CO-4	MH-3	5,265.40	MH-4	5,261.59	190.5	0.020	24.0	0.013	7.70	8.38	31.99	5,266.39	5,262.26
CO-5	MH-4	5,261.39	MH-5	5,257.82	128.2	0.028	24.0	0.013	10.50	10.29	37.74	5,262.55	5,258.54
CO-6	MH-5	5,257.62	MH-6	5,254.50	104.5	0.030	24.0	0.013	12.40	11.04	39.09	5,258.89	5,255.28
CO-7	MH-6	5,254.30	MH-7	5,251.65	61.3	0.043	24.0	0.013	12.40	12.63	47.05	5,255.57	5,252.38
CO-8	MH-7	5,251.45	MH-8	5,250.14	32.9	0.040	24.0	0.013	12.40	12.27	45.17	5,252.72	5,250.94
CO-9	MH-8	5,249.25	MH-9	5,242.30	171.4	0.041	24.0	0.013	16.20	13.27	45.55	5,250.70	5,243.12
CO-10	CB-2	5,270.42	MH-2	5,269.70	36.0	0.020	18.0	0.013	2.10	5.94	14.85	5,270.97	5,270.08
CO-11	CB-3	5,266.98	MH-3	5,266.00	36.0	0.027	18.0	0.013	2.80	7.21	17.33	5,267.62	5,266.42
CO-12	CB-4	5,262.81	MH-4	5,262.09	36.0	0.020	18.0	0.013	2.80	6.45	14.85	5,263.45	5,262.54
CO-13	CB-5	5,259.04	MH-5	5,258.32	36.0	0.020	18.0	0.013	1.90	5.77	14.85	5,259.56	5,258.89
CO-14	CB-6	5,250.48	MH-8	5,249.75	37.5	0.019	18.0	0.013	3.80	6.96	14.66	5,251.23	5,250.70
CO-15	CB-7	5,258.80	O-1	5,253.19	43.8	0.128	18.0	0.013	0.00	0.00	37.60	5,258.80	5,253.19
CO-16	MH-9	5,239.60	O-2	5,235.75	128.5	0.030	30.0	0.013	16.20	11.72	70.99	5,240.96	5,236.56

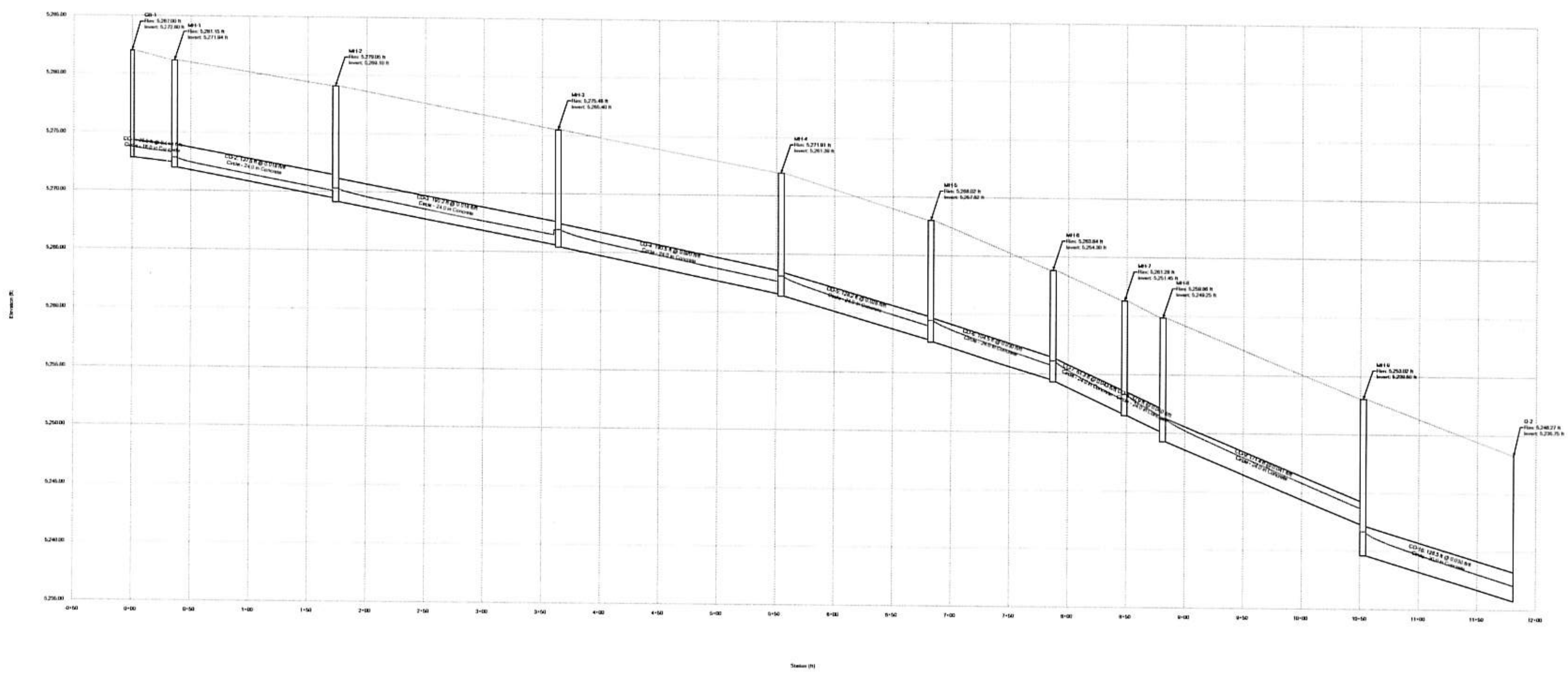
FlexTable: Manhole Table

Label	Elevation (Rim) (ft)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)	System Known Flow (cfs)
MH-1	5,281.15	5,272.52	5,272.52	2.80
MH-2	5,279.05	5,269.88	5,269.88	4.90
MH-3	5,275.48	5,266.39	5,266.39	7.70
MH-4	5,271.91	5,262.55	5,262.55	10.50
MH-5	5,268.02	5,258.89	5,258.89	12.40
MH-6	5,263.84	5,255.57	5,255.57	12.40
MH-7	5,261.28	5,252.72	5,252.72	12.40
MH-8	5,259.86	5,250.70	5,250.70	16.20
MH-9	5,253.02	5,240.96	5,240.96	16.20

100-Year HGL's

Profile Report

Engineering Profile - Profile - 1 (SH7001_StormCAD.stsw)



FlexTable: Conduit Table

Label	Start Node	Invert (Start) (ft)	Stop Node	Invert (Stop) (ft)	Length (User Defined) (ft)	Slope (Calculated) (ft/ft)	Diameter (in)	Manning's n	Flow (cfs)	Velocity (ft/s)	Capacity (Full Flow) (cfs)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)
CO-1	CB-1	5,272.80	MH-1	5,272.44	36.0	0.010	18.0	0.013	6.30	6.21	10.50	5,273.77	5,273.28
CO-2	MH-1	5,271.94	MH-2	5,269.45	137.5	0.018	24.0	0.013	6.30	7.64	30.44	5,272.83	5,270.07
CO-3	MH-2	5,269.10	MH-3	5,265.60	190.2	0.018	24.0	0.013	11.00	8.96	30.69	5,270.29	5,266.91
CO-4	MH-3	5,265.40	MH-4	5,261.59	190.5	0.020	24.0	0.013	17.50	10.41	31.99	5,266.91	5,263.13
CO-5	MH-4	5,261.39	MH-5	5,257.82	128.2	0.028	24.0	0.013	24.20	12.75	37.74	5,263.13	5,259.00
CO-6	MH-5	5,257.62	MH-6	5,254.50	104.5	0.030	24.0	0.013	28.40	13.57	39.09	5,259.46	5,255.80
CO-7	MH-6	5,254.30	MH-7	5,251.65	61.3	0.043	24.0	0.013	28.40	15.68	47.05	5,256.14	5,252.87
CO-8	MH-7	5,251.45	MH-8	5,250.14	32.9	0.040	24.0	0.013	28.40	15.19	45.17	5,253.29	5,251.48
CO-9	MH-8	5,249.25	MH-9	5,242.30	171.4	0.041	24.0	0.013	37.00	16.15	45.55	5,251.19	5,243.68
CO-10	CB-2	5,270.42	MH-2	5,269.70	36.0	0.020	18.0	0.013	4.70	7.45	14.85	5,271.25	5,270.30
CO-11	CB-3	5,266.98	MH-3	5,266.00	36.0	0.027	18.0	0.013	6.50	9.11	17.33	5,267.97	5,266.67
CO-12	CB-4	5,262.81	MH-4	5,262.09	36.0	0.020	18.0	0.013	6.70	8.19	14.85	5,263.81	5,263.13
CO-13	CB-5	5,259.04	MH-5	5,258.32	36.0	0.020	18.0	0.013	4.20	7.23	14.85	5,259.83	5,259.46
CO-14	CB-6	5,250.48	MH-8	5,249.75	37.5	0.019	18.0	0.013	8.60	8.62	14.66	5,251.62	5,251.19
CO-15	CB-7	5,258.80	O-1	5,253.19	43.8	0.128	18.0	0.013	7.34	16.50	37.60	5,259.85	5,253.66
CO-16	MH-9	5,239.60	O-2	5,235.75	128.5	0.030	30.0	0.013	37.00	14.61	70.99	5,241.66	5,237.07

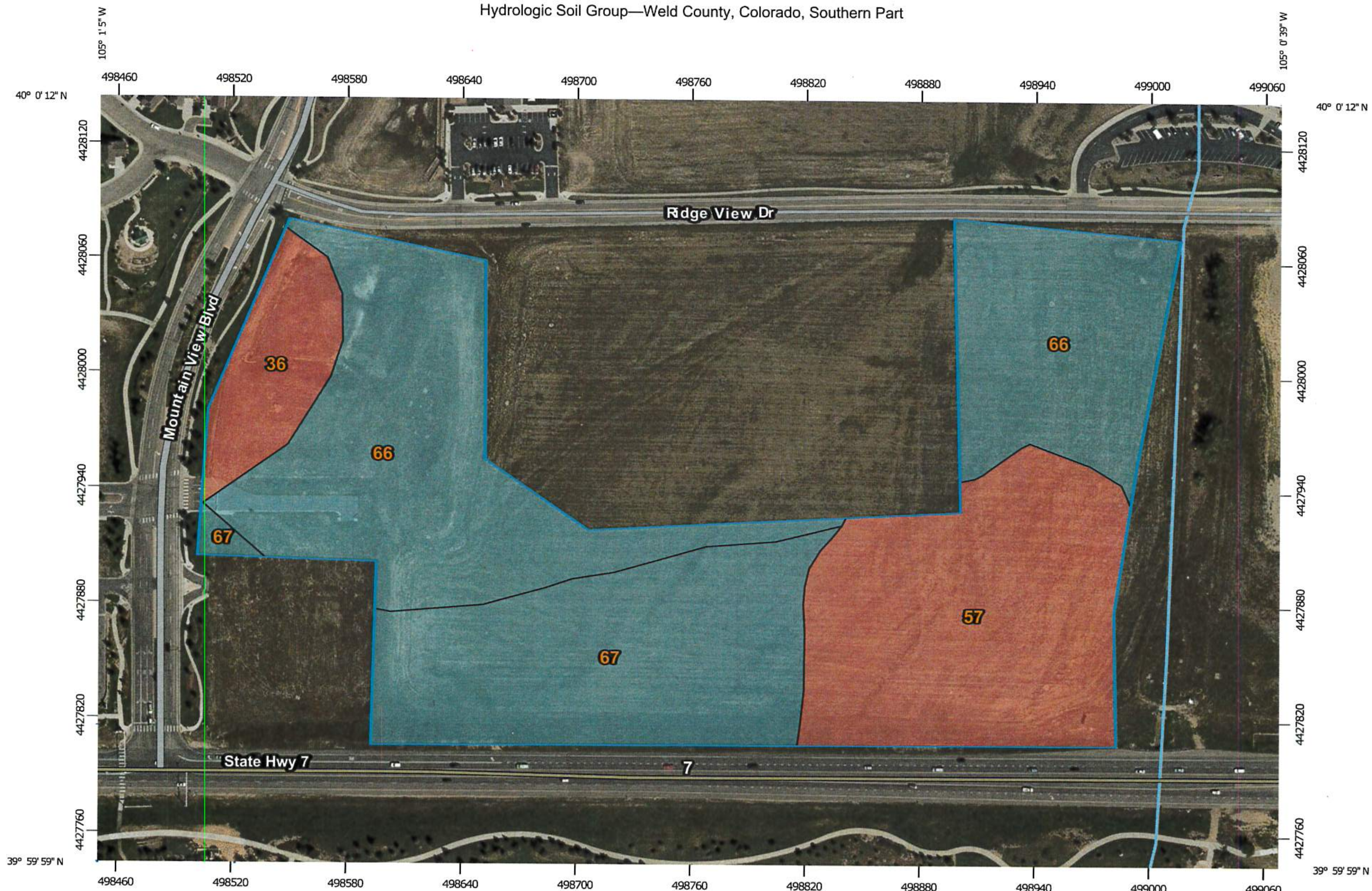
FlexTable: Manhole Table

Label	Elevation (Rim) (ft)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)	System Known Flow (cfs)
MH-1	5,281.15	5,272.83	5,272.83	6.30
MH-2	5,279.05	5,270.29	5,270.29	11.00
MH-3	5,275.48	5,266.91	5,266.91	17.50
MH-4	5,271.91	5,263.13	5,263.13	24.20
MH-5	5,268.02	5,259.46	5,259.46	28.40
MH-6	5,263.84	5,256.14	5,256.14	28.40
MH-7	5,261.28	5,253.29	5,253.29	28.40
MH-8	5,259.86	5,251.19	5,251.19	37.00
MH-9	5,253.02	5,241.66	5,241.66	37.00

Appendix H

Soils Information

Hydrologic Soil Group—Weld County, Colorado, Southern Part



Map Scale: 1:2,820 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0, 40, 80, 160, 240

Feet: 0, 100, 200, 400, 600

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 13N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils

Soil Rating Polygons
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Weld County, Colorado, Southern Part
Survey Area Data: Version 14, Sep 22, 2015

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Aug 30, 2014—Sep 18, 2014

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Hydrologic Soil Group— Summary by Map Unit — Weld County, Colorado, Southern Part (CO618)				
Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
36	Midway-Shingle complex, 5 to 20 percent slopes	D	1.4	6.8%
57	Renohill clay loam, 3 to 9 percent slopes	D	5.4	26.1%
66	Ulm clay loam, 0 to 3 percent slopes	C	8.8	42.6%
67	Ulm clay loam, 3 to 5 percent slopes	C	5.1	24.5%
Totals for Area of Interest			**20.8**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

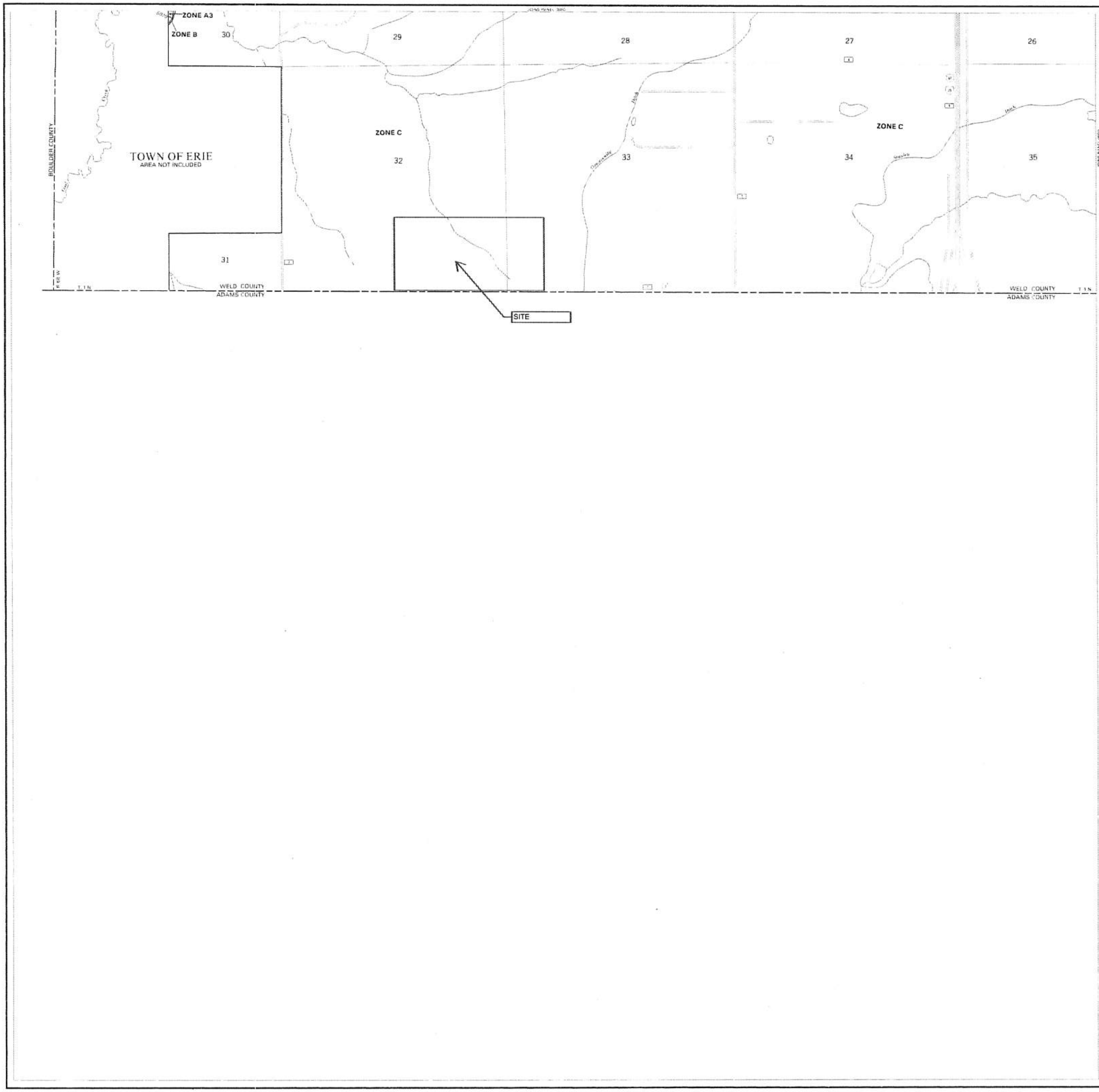
Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: *None Specified*

Tie-break Rule: Higher

Appendix I
FEMA FIRM Map



KEY TO MAP

ZONE B

ZONE A1

ZONE A5

ZONE B

EXPLANATION OF ZONE DESIGNATIONS

NOTES TO USER

APPROXIMATE SCALE IN FEET

1000 0 1000

NATIONAL FLOOD INSURANCE PROGRAM

FIRM
FLOOD INSURANCE RATE MAP

WELD COUNTY,
COLORADO
(UNINCORPORATED AREAS)

PANEL 970 OF 1075
(SEE MAP INDEX FOR PANELS NOT PRINTED)

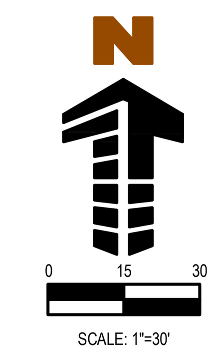
COMMUNITY-PANEL NUMBER
080266 0970 D

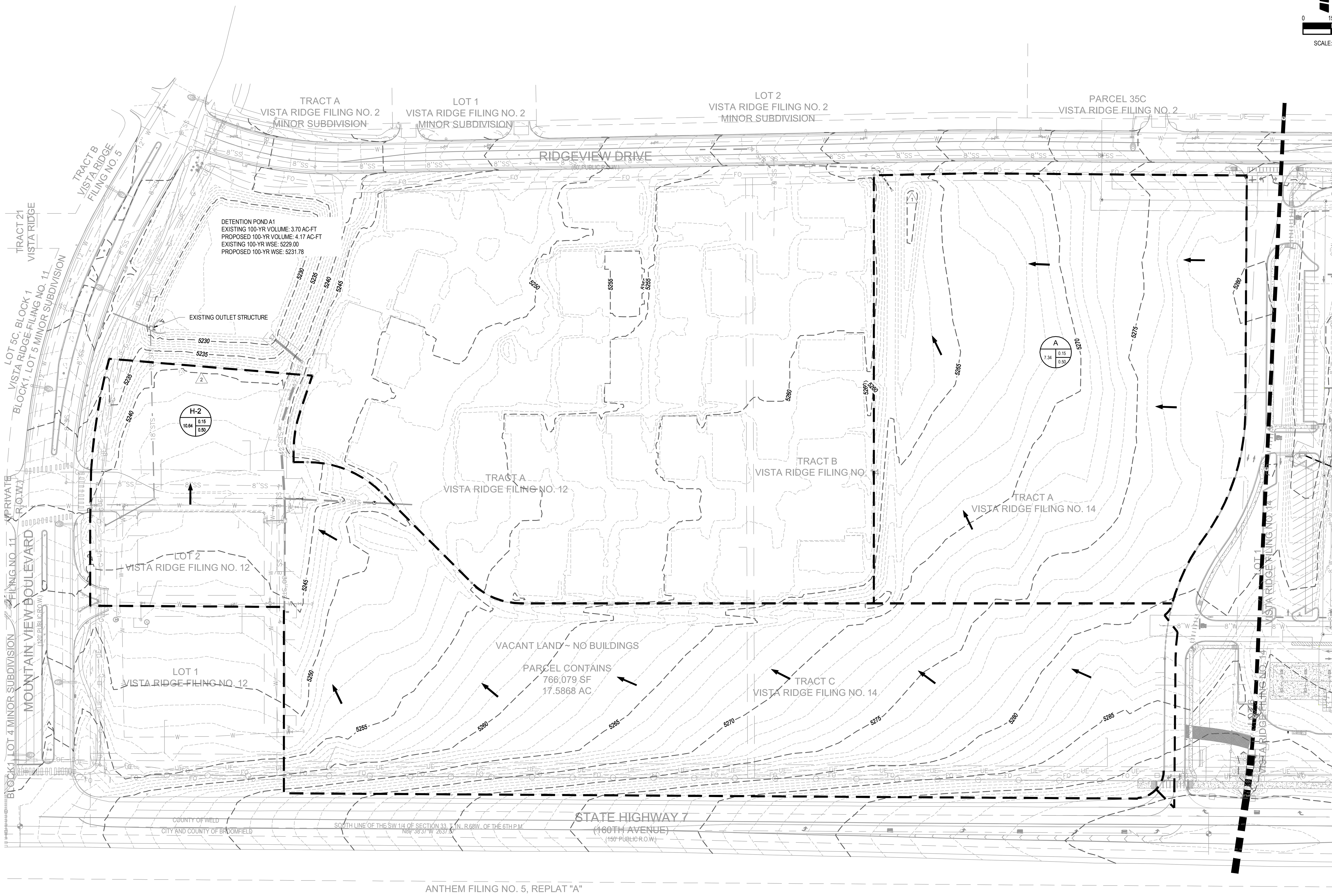
MAP REVISED
SEPTEMBER 28, 1990

Federal Emergency Management Agency

Appendix J

Historic and Proposed Drainage Maps





SITE LEGEND

- PROPERTY BOUNDARY LINE
- ADJACENT PROPERTY BOUNDARY LINE
- EASEMENT BOUNDARY LINE
- EXISTING CURB & GUTTER TO REMAIN
- EXISTING TO BE REMOVED
- PROPOSED CURB & GUTTER
- STREET LIGHT
- PROPOSED INLET

GRADING LEGEND

- 5245 EXISTING CONTOUR
- 45 PROPOSED CONTOUR
- STS EXISTING STORM SEWER
- STS PROPOSED STORM SEWER
- STS PROPOSED STORM SEWER (LESS THAN 12")

DRAINAGE LEGEND

- PROPOSED BASIN BOUNDARY LINE
- FLOW ARROW
- 1 DESIGN POINT
- A BASIN DESIGNATION
- 0.00 5-YEAR RUNOFF COEFFICIENT
- 0.00 100-YEAR RUNOFF COEFFICIENT
- 0.00 BASIN AREA IN ACRES

Runoff Summary Table

Design Point	Basin	Area (Ac)	5-Year Runoff (cfs)	100-Year Runoff (cfs)
1	H-1	7.34	3.70	23.40
2	H-2	10.64	5.30	33.10
Total:		17.98	9.00	56.50

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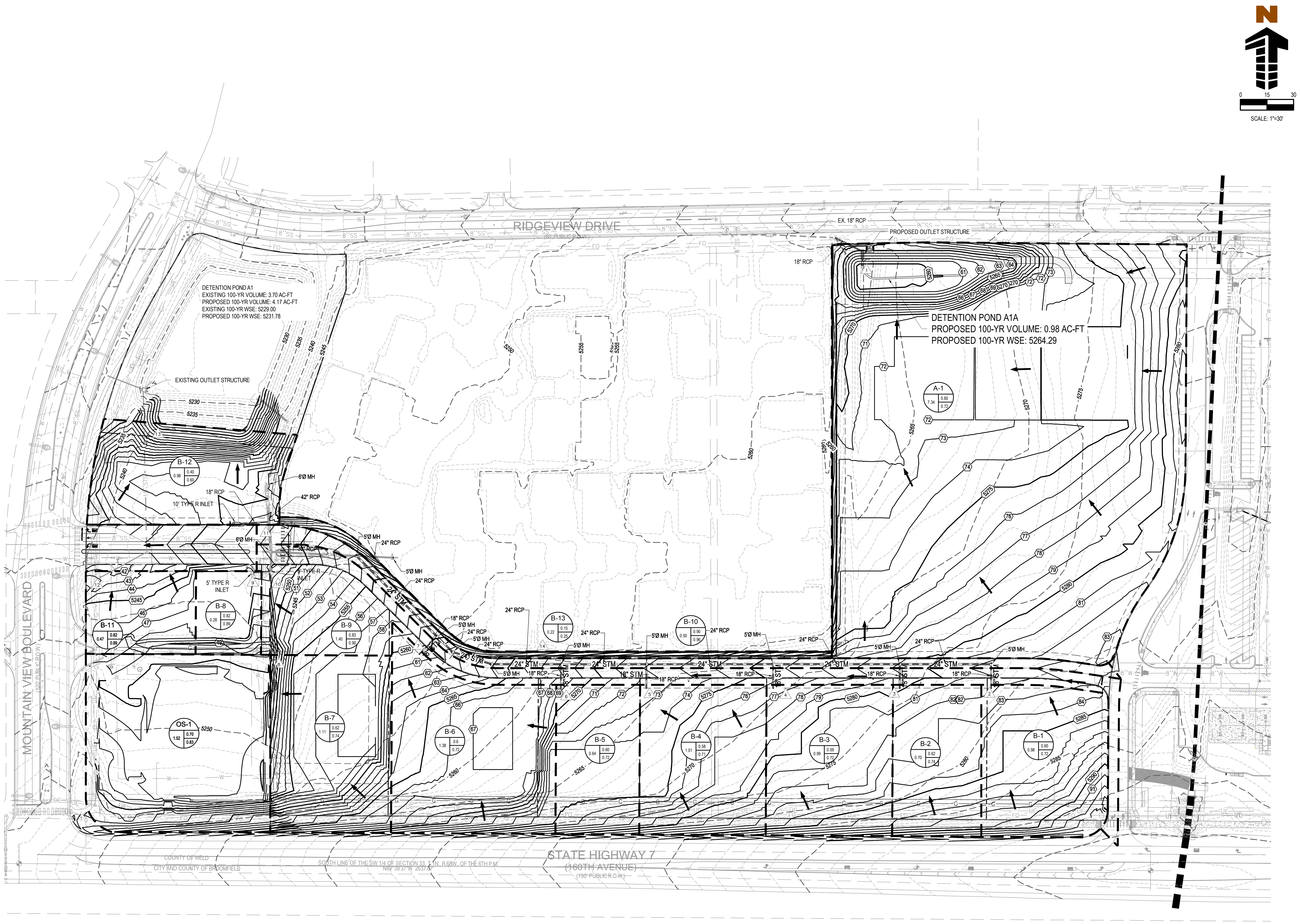
VISTA RIDGE COMMERCIAL WEST CIVIL CONSTRUCTION DRAWINGS

ERIE, COLORADO

#	Date	Issue / Description	Init.
1	6/29/16	BID ADDENDUM #1	PJD
2	7/28/16	TOWN SUBMITTAL #2	PJD

Project No:	SH7000001.01
Drawn By:	DMP
Checked By:	PJD
Date:	5/26/16

HISTORICAL DRAINAGE PLAN



SITE LEGEND

- PROPERTY BOUNDARY LINE
- ADJACENT PROPERTY BOUNDARY LINE
- EASEMENT BOUNDARY LINE
- EXISTING CURB & GUTTER TO REMAIN
- EXISTING TO BE REMOVED
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- STS PROPOSED STORM SEWER
- STS PROPOSED STORM SEWER (LESS THAN 12")

DRAINAGE LEGEND

- PROPOSED BASIN BOUNDARY LINE
- FLOW ARROW
- DESIGN POINT
- BASIN DESIGNATION
- 5-YEAR RUNOFF COEFFICIENT
- 100-YEAR RUNOFF COEFFICIENT
- BASIN AREA IN ACRES

Runoff Summary Table				
Design Point	Basin	Area (Ac)	5-Year Runoff (cfs)	100-Year Runoff (cfs)
1	A-1	7.34	17.90	40.70
2	B-1	0.98	2.80	6.30
3	B-2	0.70	2.10	4.70
4	B-3	0.99	2.80	6.50
5	B-4	1.01	2.80	6.70
6	B-5	0.64	1.90	4.20
7	B-6	1.38	3.80	8.60
8	B-7	1.11	3.20	7.20
8	OS-1	1.52	4.40	10.90
9	B-8	0.28	1.10	2.30
10	B-9	1.40	4.60	9.40
11	B-10	0.50	1.90	3.90
12	B-11	0.47	1.90	3.80
13	B-12	0.98	1.60	4.40
14	B-13	0.22	0.20	1.00
Total:		19.52	53.00	120.6

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VISTA RIDGE COMMERCIAL WEST CIVIL CONSTRUCTION DRAWINGS

ERIE, COLORADO

#	Date	Issue / Description	Init.
1	6/29/16	BID ADDENDUM #1	PJD
2	7/28/16	TOWN SUBMITTAL #2	PJD

Project No:	SH7000001.01
Drawn By:	DMP
Checked By:	PJD
Date:	5/26/16

DRAINAGE PLAN

Storm Water Management Plan (SWMP)
For Construction Activities

VISTA RIDGE COMMERCIAL
VISTA RIDGE FILING NO. 14 – 1ST AMENDMENT
NEC MOUNTAIN VIEW BLVD & HWY 7
ERIE, CO

Prepared for:
State Highway 7 Marketplace, Inc.
9750 W. Cambridge Place
Littleton, CO 80127
Phone (303) 920-9400
Attn: James Spehalski
jspehalski@marathonlc.com

Prepared by:
Galloway & Company, Inc.
6162 S. Willow Drive, Suite 320
Greenwood Village, CO 80111
Phone (303) 770-8884
Fax (303) 770-3636
Attn: Brandon McCrary, PE
BrandonMcCrary@GallowayUS.com

Prepared: May 26, 2016

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STORMWATER MANAGEMENT PLAN (SWMP)
FOR CONSTRUCTION ACTIVITIES
AT

Vista Ridge Commercial

Vista Ridge Filing No. 14, 1st Amendment
NEC Mountain View Blvd and Highway 7
Erie, Colorado

Prepared For:	State Highway 7 Marketplace, Inc. 9750 W. Cambridge Place Littleton, CO 80127	
Preparation Date:	May 26, 2015	
Prepared by (Civil Eng. Firm):	Galloway & Company, Inc. 6162 Willow Drive, Suite 320 Greenwood Village, CO 80111	
Estimated Project Dates:	Construction Start:	August 2016
	Construction Complete:	January 2017
Owner Name and Address:	State Highway 7 Marketplace, Inc. 9750 W. Cambridge Place Littleton, CO 80127	
Developer:	State Highway 7 Marketplace, Inc. 9750 W. Cambridge Place Littleton, CO 80127	
General Contractor:	TBD	
SWMP Administrator:	TBD	

TABLE OF CONTENTS

I. Introduction

The objective of this Stormwater Management Plan (SWMP) is to identify, design, construct, and implement Best Management Practices (BMP's) to reduce to the greatest extent practical pollutants in storm water discharges during the construction of this project.

This SWMP includes all Erosion and Sediment Control Plans in the Contract Drawings including location maps, phasing drawings, detail sheets, and all applicable attachments: General Permit Application, Inspection Checklists, Logs, and Inactivation Notice. This SWMP is a living, breathing document with all updates and modifications during construction made part of the overall plan as they occur.

<u>The EPA and local government agencies that oversee this project are</u>:

Colorado Department of Public Health and Environment
Water Quality Control Division
WQCD-Permits
4300 Cherry Creek Drive South
Denver, Colorado 80246-1560
Ph. (303) 692-3517

Town of Erie
645 Holbrook St
Erie, CO 80516
Ph. 303.926.2870

II. Contact List of Operators

Prior to the commencement of earth disturbing activities a Pre-Construction Meeting is to be held and the attached Pre-Construction Meeting Form will be fully executed listing all required contact names and numbers. Any subcontractor(s) required to be a co-permittee by local jurisdictions must be listed and provide a copy of their General Permit Application or co-permit to the owner and attach to this SWMP.

III. Project Description

A. <u>Project Scope</u>: The proposed development includes the construction of the site infrastructure to support future pad sites within the Vista Ridge Commercial development. Pad sites will not be developed as part of this project, and will be developed individually in the future. The proposed site is approximately 17.6 acres in size. Improvements include the main private roadway through the site (including the water, sanitary sewer, and storm sewer mains through the site), and overlot grading of the entire site. A small portion of an existing detention pond will be modified as part of the site improvements, and is included in the total disturbed area calculation.

B. Location Maps:

C. Site Area:	Vista Ridge Commercial	17.6 Acres ±
	Total Disturbed Area=	17.94 Acres ±

D. Impervious Area: Before Development: 2%
(% to total) Post Development: 77%

E. Runoff Coefficient: Before Development: 0.52 (100-Year)
Post Development: 0.73(100-Year)

F. Existing Site Topography/Use:

The subject property is currently vacant. Native vegetation includes brush, grasses and small trees. Curb, gutter and sidewalks have been installed along the frontage of Mountain View Blvd, and a drive approaches into the site from Mountain View Blvd has been installed. Currently, the site slopes from south to north at slopes ranging from 2%-6%.Stormwater runoff flows overland north into the existing detention pond at the northwest corner of the site Both the overland flow and the stormwater captured by the existing and proposed storm sewer infrastructure flow to the existing Vista Ridge Extended Detention Water Quality Pond north of the site. The existing Detention Pond drains into the Town of Erie storm sewer system, which flows to the Coal Creek.

The grading operations for this project will disturb approximately 17.94 acres. Cuts and fills ranging from -8.4 to +10.8 feet are expected during grading operations. The final grading could see a net fill of 110,700 cubic yards. These quantities are approximate and do not include over-excavation or shrink/swell factors. The contractor is responsible for performing their own calculations for the earthwork. The Contractor is responsible for hauling and disposing of any excess cut in an appropriate manner. Once the grading is complete, the site will be stabilized with permanent landscaping as well as seeding and mulching. Refer to the project construction plans for location and limits of the grading operations.

G. Site Soils: According to the NRCA National Cooperative Soil Survey – Web Soil Survey 2.0, site soils are made up Ulm Clay sandy loams Midway-Shingle complex, and Renohill clay loams which are classified as Hydrologic Soil Groups C and D. These soils are defined as having slow infiltration rates when thoroughly wet and having a slow rate of water transmission.

H. Rainfall information:

	Jan	Feb	March	April	May	June	July	Aug	Sept	Oct	Nov	Dec	Total
Average rainfall in inches	.5	.4	1.4	1.9	2.2	1.8	1.1	1.6	1.1	0.8	0.7	.6	14.1

- The total average annual rainfall for the project area is: 14.1 inches

I. Name of Receiving Waters: Storm runoff flows north to an existing detention basin and is then released to the Town of Erie storm sewer which releases to Coal Creek approximately 2.5 miles northwest of the site.

J. Off-Site Borrow Location (If applicable):

<Location to be Determined, Contractor to Insert Location information below once determined.>

*This can be filled in at anytime during the life of this SWMP. An off-site borrow location for imported soil material that is solely designated to this project must be monitored under this SWMP. If the off-site borrow location services multiple locations it should have it's own NOI and SWMP by the owner/operator of the borrow location. The general contractor is responsible for verifying any and all sources of imported material to be within this SWMP.

K. Endangered Species: The CDPS General Permit does not require evaluation for Threatened and Endangered Species

L. Other Industrial Activities: None.

IV. Erosion and Sediment Controls

A. Sequence of Major Activities**:** The order of activities will be as follows:

Phase 1

Implementation and installation of the following areas: trailer, parking, lay down, porta-potty, wheel wash, concrete washout, mason's area, fuel and material storage containers, solid waste containers, etc., immediately denote them on the site maps and note any changes in location as they occur throughout the construction phases.

1. *Install stabilized construction exit(s) and SWMP entrance sign.*
2. *Install silt fences on the site (clear only those areas necessary to install silt fence).*

3. Prepare temporary parking and storage area.
4. Construct and stabilize sediment/retention basins and sediment traps with appropriate outfall.
5. Install and stabilize hydraulic control structures (dikes, swales, etc.).

Halt all activities and contact the civil engineering consultant to perform inspection and certification of BMPs. General Contractor shall schedule and conduct storm water pre-construction meeting with engineer and all ground-disturbing contractors before proceeding with construction.

6. Clear and grub the site as construction activities require.
7. Begin grading the site.
8. Start construction of building pad and structures.

Phase II

1. Temporarily seed, or apply erosion control blanket throughout construction, any denuded areas that will be inactive for 14 days or more.
2. Maintain silt fence, inlet protection and stabilized construction exits installed during Phase 1.
3. Install utilities, underdrains and storm sewers.
4. Install rip-rap around outlet structures as each structure is installed.
5. Install inlet protection around all storm sewer structures as each inlet structure is installed.
6. Permanently stabilize areas to be vegetated as they are brought to final grade.
7. Prepare site for paving.
8. Pave site.
9. Install appropriate inlet protection devices for paved areas as work progresses.
10. Complete grading and installation of permanent stabilization over all areas including out lots.
11. Contact civil engineering consultant after the site appears to be fully stabilized for an inspection.
12. Remove all temporary erosion and sediment control devices after approval of the civil engineering consultant and stabilize any areas disturbed by the removal of the BMP.
13. Continue daily inspection reports until the final daily inspection is signed off by the construction manager that the site is fully stabilized and the permit may be terminated.

Note: the general contractor may complete construction-related activities concurrently only if all preceding BMPs have been completely installed. BMP-related steps in the above sequence are italicized for clarity.

B. Temporary Stabilization: Soil stockpiles and disturbed portions of the site where construction activity temporarily ceases are to be stabilized within **seven** days. Stabilization as defined in the above "Sequence of Major Activities." Straw mulch is to be tracked into place by machine, disked, or tackified to prevent blowing and washing away of the straw.

C. Dewatering: The area between check dams will not be allowed to discharge except through infiltration or by construction dewatering practices. Dewatering may also be necessary for on-site utility installations and foundation construction. Therefore, construction dewatering is anticipated and the General Contractor will be required to obtain a construction dewatering permit from Colorado Department of Health and Environment. The General Contractor will be required to submit a construction dewatering application at least 30 days prior to the anticipated date of discharge and pay the associated fees.

Discharges from dewatering operations must be directed through an appropriate pollution prevention/treatment measure, such as a pump discharge filter bag, sediment trap or sediment basin prior to being discharged from the site. Locations of pollution prevention/treatment measures shall be shown on the Site Maps once they are determined. Under no circumstances are discharges from dewatering operations to be discharged directly into streams, rivers, lakes or other areas off-site. Likewise, discharges into storm sewer systems that do not drain to a suitable on-site treatment facility, such as a basin, are also prohibited. Discharges from dewatering operations must also be conducted in a manner sufficient to prevent erosion from the discharge runoff.

D. Permanent Stabilization: Disturbed portions of the site where construction activities permanently cease are to be stabilized with permanent seed, mulch, sod, etc. per the final landscaping plan in the Construction Drawings. This permanent stabilization must occur within **seven** days of an area reaching final grade.

E. Structural Practices: The structural practices for this project include, but are not limited to, those specific items shown of the erosion and sediment control drawings listed in Section III. B. Other BMP's may be required or added with Owner's Civil Engineering Consultant's approval letter. All structural BMPS must meet with Urban Drainage and Flood Control District Standards.

1. General Best Method Practices (BMP's) are below:

   a. Diversion Ditches/Berms – They consist of temporary or permanent swales or dikes made of soil material, sometimes with impermeable liners, to control the flow of sediment laden surface water. Most of these BMP's will be coupled with check dams, sediment traps, and or basins.

   b. Check Dam – (Also known as Ditch Checks) Consists of rock, riprap, or other material designed to control concentrated flows of water in a ditch or swale. Water moving at a higher velocity will be pooled by a check dam to allow sediment to settle out before the surface water continues through the device.

   c. Construction Entrance – All access to and from the site will require the appropriately constructed access drive usually consisting of stone on top of a geotextile fabric. When conditions require, a truck wash station will also be utilized to prevent the tracking of sediment off site.

   d. Inlet protection – These devices may consist of a wood frame with silt fence fabric, straw bales, large rock or other pre-manufactured products designed to keep sediment-laden water from entering storm drain inlets.

   e. Sediment Basins / Traps – Consist of a depression created in the earth to collect sediment-laden surface water to allow settlement of suspended soil particles before storm water is allowed to exit the site. The size and construction of these devices are to be shown on the site-specific drawings. Accumulated sediment must be removed to maintain effectiveness.

   f. Silt Fence – This BMP consists of a synthetic permeable woven fabric that must only be used to control small surface water flows within this product's design capability. Silt fence must also be inspected and cleaned per the weekly checklist to maintain its effectiveness.

V. Other Pollutant Controls

A. The following items are pollutant issues (outside of storm water sediment) during the construction process:

1. Dust Control - The general contractor will employ the use of water trucks or other dust control agents to reduce dust generated during construction to levels acceptable by local authorities and the owner's agent. Water trucks or other dust control agents will be used as needed during construction to minimize dust generated on the site. Tackifiers may be used to hold soil in place and prevent dust. Manufacturer recommendations for application locations and rates must be used for dust control applications.

2. Concrete Waste (Washout from Ready Mix Trucks) - All concrete washouts will be in designated locations, noted by the general contractor on the job site erosion control plan. The concrete washout will be isolated and contained from storm water run-off. Excess liquid may be allowed to percolate into the ground on-site; it may not be discharged off site as runoff in any storm drainage conveyance. Off-site disposal, solids or liquids, only allowed to an appropriately licensed facility.

3. Equipment/Vehicle Maintenance – All on-site equipment shall receive regular maintenance by the contractors using the equipment to help prevent leaking of fluids or other pollutant discharges. The general contractor is responsible for overseeing that any onsite vehicle maintenance is handled appropriately and that all fluids and materials are disposed of properly.

4. Fuel Tanks – All onsite fuel tanks must meet all government standards including proper barriers for safety and containment of potential spills. The general contractor must note the location of any fuel tanks on the job site erosion control plan.

5. Hazardous Waste Management and Spill Reporting – Any hazardous or potentially hazardous material that is brought onto the construction site will be handled properly in order to reduce the potential for storm water pollution. All materials used on this construction site will be properly stored, handled, dispensed and disposed of following all applicable label directions. Flammable and combustible liquids will be stored and handled according to 29 CFR 1926.152. Only approved containers and portable tanks shall be used for storage and handling of flammable and combustible liquids.

   Material Safety Data Sheets (MSDS) information will be kept on site for any and all applicable materials.

   In the event of an accidental spill, immediate action will be undertaken by the General Contractor to contain and remove the spilled material. All hazardous materials will be disposed of by the Contractor in the manner specified by federal, state and local regulations and by the manufacturer of such products. As soon as possible, the spill will be reported to the appropriate agencies. As required under the provisions of the Clean Water Act, any spill or discharge entering waters of the United States will be properly reported. The General Contractor will prepare a written record of any spill and associated clean-up activities of petroleum products or hazardous materials in excess of 1 gallon or reportable quantities, which ever is less. The General Contractor will provide notice to Owner immediately upon identification of a reportable spill.

Any spills of petroleum products or hazardous materials in excess of Reportable Quantities as defined by EPA or the state or local agency regulations, shall be immediately reported to the EPA National Response Center (1-800-424-8802) and the Colorado Department of Public Health and Environment (CDPHE) (1-877-518-5608).

The State reportable quantity for petroleum products is 25 gallons or more (or that cause a sheen on nearby surface waters). Spills from regulated aboveground and underground fuel storage tanks must be reported to the State Oil Inspector within 24 hours (after-hours contact CDPHE Emergency Spill Reporting Line). This includes spills from fuel pumps. Spills or releases of hazardous substances from regulated storage tanks in excess of the reportable quantity (40 CFR Part 302.6) must be reported to the National Response Center and the local fire authority immediately and to the State Oil Inspector within 24 hours

The reportable quantity for hazardous materials can be found in 40 CFR 302 and http://a257.g.akamaitech.net/7/257/2422/08aug20031600/edocket.access.gpo.gov/cfr_2003/julqtr/pdf/40cfr302.6.pdf

In order to minimize the potential for a spill of petroleum product or hazardous materials to come in contact with storm water, the following steps will be implemented:

a) All materials with hazardous properties (such as pesticides, petroleum products, fertilizers, detergents, construction chemicals, acids, paints, paint solvents, additives for soil stabilization, concrete, curing compounds and additives, etc.) will be stored in a secure location, under cover, when not in use.

b) The minimum practical quantity of all such materials will be kept on the job site and scheduled for delivery as close to time of use as practical.

c) A spill control and containment kit (containing for example, absorbent material such as kitty litter or sawdust, acid neutralizing agent, brooms, dust pans, mops, rags, gloves, goggles, plastic and metal trash containers, etc.) will be provided on the construction site and location(s) shown on Site Maps.

d) All of the product in a container will be used before the container is disposed of. All such containers will be triple rinsed, with water prior to disposal. The rinse water used in these containers will be disposed of in a manner in compliance with state and federal regulations and will not be allowed to mix with storm water discharges.

e) All products will be stored in and used from the original container with the original product label.

f) All products will be used in strict compliance with instructions on the product label.

g) The disposal of excess or used products will be in strict compliance with instructions on the products label.

6. Misc. Building Materials or Supplies – All materials that will become part of the permanent improvements are to be kept in sealed containers and maintained in an orderly fashion until installed. The general contractor will be responsible for monitoring any and all stockpiles of material and equipment on site.

7. Offsite Vehicle Tracking – Per the Structural Practices section, a stabilized construction entrance will be provided to help reduce vehicle tracking of sediments. The paved streets adjacent to the site are to be swept as necessary to remove any excess mud, dirt or rock tracked from the site. Dump trucks hauling loose material from the construction site are to be covered with a tarpaulin.

8. Sanitary Waste – All on site personnel are to utilize the temporary or permanent sanitary facilities provided on site by the general contractor. Sanitary waste is to be collected from the temporary/portable units a minimum of one time per week by a licensed sanitary waste management contractor, or as required by local regulation. The location of sanitary units is to be noted on the job site erosion control plan by the general contractor.

9. Solid Waste Material (Construction Debris) - No solid waste is to be allowed in storm water discharges. *On site burning or burying of waste material is prohibited.* All trash and construction debris from the site is to be deposited in dumpsters or proper hauling equipment. The dumpsters are to meet local and state solid waste management regulations and emptied as deemed necessary to an approved off site dump. The location of dumpsters is to be noted on the job site erosion control plan by the general contractor. All construction companies working on site will be responsible for the correct procedure in their waste disposal.

10. Non Stormwater Discharges - The General Permit for Storm Water Discharges Associated with Construction Activities prohibits most non-storm water discharges during the construction phase. Allowable non-storm water discharges that occur during construction on this project, which are covered by the General Permit, include:

    1. Emergency fire fighting activities;

    2. Un-contaminated springs;

    3. Landscape irrigation return flows.

    Construction dewatering water can not be discharged to surface waters or to storm sewer systems without separate permit coverage. The discharge of construction dewatering water to the ground, under specific conditions, may be allowed by the Stormwater Construction Permit when appropriate BMPs are implemented. Refer to section 4C for more information on dewatering.

    No other non-stormwater discharges are anticipated, or allowed by coverage of the CDPS General Permit.

11. Asphalt and Concrete Batch Plants – Shall not be permitted on-site.

VI. Inspection and Maintenance Procedures for Construction

A. The cornerstone of the maintenance procedure is the Inspection Report. Qualified owners representatives and general contractor site superintendents will be trained in the inspection and maintenance practices necessary for keeping the pollutant controls used in this SWMP in good working order. The site superintendent will be responsible for the daily oversight of the pollution controls along with the execution of the site inspection report in accordance with this SWMP. The owner’s representative will also have periodic inspection requirements to ensure proper execution of site inspections and maintenance.

VII. Certification of Compliance with Federal, State, and Local Requirements

A. This Stormwater Management Plan reflects State of Colorado and Town of Erie requirements for storm water management and erosion and sediments control. This plan was prepared in accordance with the attached permit text. There are no other known applicable State or Federal requirements for sediment and erosion site plans (or permits); or storm water management site plans (or permits).

VIII. Post Construction Practices

A. Structures and Pollutants

1. The proposed development includes the construction of the site infrastructure to support future pad sites within the Vista Ridge Commercial development. Storm runoff from the future buildings, parking, and drive areas will be conveyed through a permanent underground storm drain system. Piping will route runoff flows to an on-site stormwater detention pond.

2. The expected pollutants to be generated by this site should be typical of a Commercial/Retail center. Some of those sources include fluids from automobiles and trucks like oil, grease, fuel, antifreeze, and brake fluid, plus particulates created by or carried on vehicles and deposited on the site such as brake dust, rubber fragments from tires, and dirt picked up from or carried onto the site. In addition, trash generated by building occupants or blown onto the site may be found at times. Thermal pollution may also occur during rainfall events when the building roof or asphalt pavement is hot from significant sunlight prior to the rainfall.

3. The post construction measures used to minimize pollutants in waterways include regular monitoring and collection of trash and debris, and good housekeeping of delivered and stored operating and retail goods.

B. Maintenance Guidelines for Post Construction Operation

1. Maintenance of all storm water pollution prevention measures will be the responsibility of the on-site management staff. The maintenance guidelines consist mostly of good housekeeping measures. Any grassed or vegetated areas that experience erosion from rainfall events should be repaired and revegetated as soon as possible. Trash or litter should be picked up and properly disposed to prevent it from getting into the storm drainage system and downstream waterways. The detention and retention ponds will be monitored for sediment build up. Periodic removal of sediment should be done to keep the structures effective. Pavement areas should also be monitored for pollutants. Any large quantity of fluids such as oil, antifreeze, brake fluid, etc. found on the pavement should be reported to the office and the source determined, if possible, and removed from the site for maintenance or repair. Pavements should also be monitored for sediment coming from vegetated areas that drain onto the pavement. If sediment is found it should be cleaned off the pavement, and the source of the soil found and repaired as discussed above.

X. Attachments

- *General Permit Application* (State) and *Stormwater Construction Permit Application* (Local)
  Contractor to Add
- *Final Permit, Colorado Discharge Permit System – Stormwater Certification* (State)
  Contractor to Add
- *CDPS General Permit – Stormwater Discharges Associated with Construction Activity*
  Contractor to Add

Final Utility Study
Vista Ridge Commercial West

NEC of Mountain View Boulevard & Hwy 7
Located in the S ½ of Section 33, Township 1 North, Range 68 West, of the 6th Principal Meridian, Town of Erie, County of Weld, State of Colorado

Date: May 26, 2015
Revised July 28, 2016

Prepared for:
State Highway 7 Marketplace
9750 W. Cambridge Place
Littleton, CO 80127
Phone (303) 920-9400
Attn: James Spehalski

Prepared by:
Galloway & Company, Inc.
6162 S. Willow Drive, Suite 320
Greenwood Village, CO 80111
Phone (303) 770-8884
Fax (303) 770-3636
Attn: Brandon S. McCrary, P.E.

Galloway
Planning. Architecture. Engineering.

ENGINEER'S STATEMENT

The enclosed Final Utility Report and exhibits were prepared by me, or under my direct supervision, and are correct to the best of my knowledge and belief. Said Utility Report has been prepared in accordance with applicable Town of Erie criteria. I accept responsibility for any liability caused by negligent acts, errors or omissions on my part in preparing this report.

Brandon S. McCrary, PE
Colorado Registered Professional Engineer

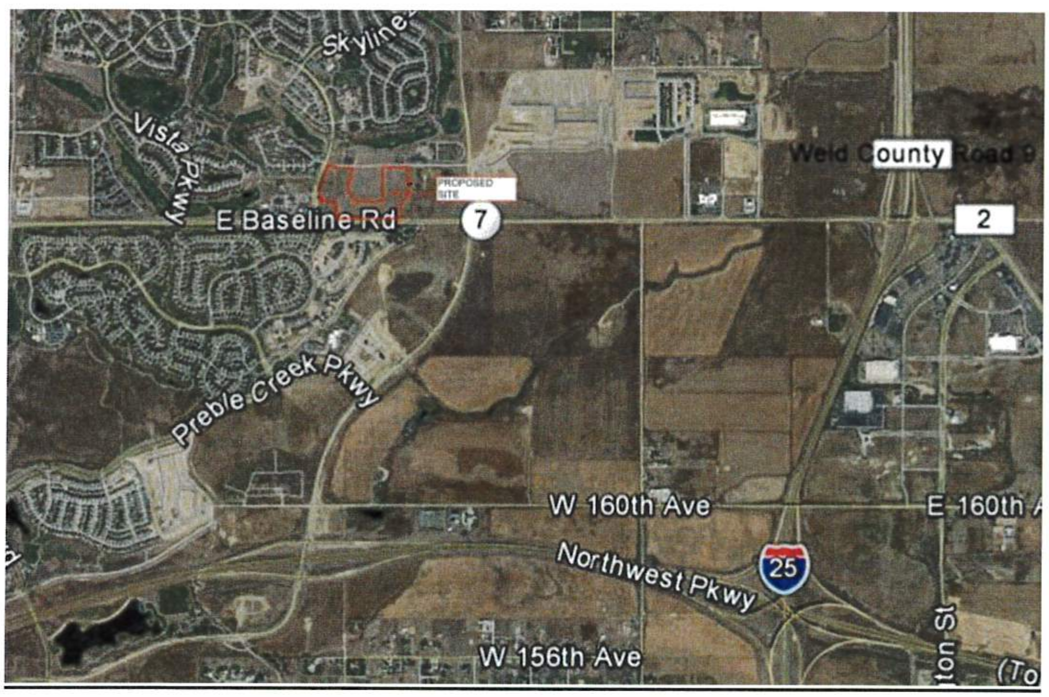
TABLE OF CONTENTS

I. INTRODUCTION

This final utility report has been prepared by Galloway & Company, Inc. for the Vista Ridge Commercial West development which is located at the NEC of Mountain View Boulevard & Hwy 7 located in the SW ¼ of Section 33, Township 1 North, Range 68 West, of the 6th Principal Meridian, Town of Erie, County of Weld, State of Colorado. The site is bounded by Ridge View Drive to the north, Mountain View Boulevard to the west, Highway 7 to the south, and a private roadway and King Soopers to the east.

The site will include multiple pad sites along with a junior anchor parcel on the northeastern portion of the site. The site will be bisected by a private roadway running east-west to provide access to all of the proposed pad sites. The total site area is approximately 17.8 acres. The development will be served by two major access points, one along Mountain View Boulevard, and one along the private roadway between the site and the proposed King Soopers (currently under construction). The project includes associated parking and infrastructure improvements, water, sanitary sewer, and storm sewer improvements.

PROJECT LOCATION MAP.



NOT TO SCALE

Based on the geotechnical report for the site, prepared by Kumar and Associates, dated October 12, 2015, the subsurface conditions consist of a variable thickness top soil overlying overburden man-placed fills and natural soils. The underlying bedrock consisted generally of claystone with frequent zones of interbedded claystone and sandstone ranging from a few inches to about 18 feet below ground surface.

The imported soils encountered on the site ranged from 5 to 8 feet in depth. The degree of compaction of the existing fill material was not determined at this time. Groundwater was encountered in two borings at depths ranging from 8 to 18 feet. Follow up groundwater measurements were taken 14 days after drilling and no groundwater was encountered.

This study is intended to analyze the proposed water and sanitary sewer infrastructure, and describe the process of sizing the public water and sanitary sewer mains associated with the project.

II. EXISTING UTILITY INFRASTRUCTURE

Water

The site will utilize existing water line stubs to serve the site. There is an existing 8" public water main which runs through the proposed King Soopers site to the east and into the site. In addition, there is another existing 8" public water main which has been stubbed into the site from the west off of Mountain View Boulevard. These water line stubs will be located within the future private drive for the site. There is an existing 12" public water main in Mountain View Boulevard as well. This water main runs east and west in Ridgeview Dr.

Sanitary Sewer

An existing 8" public sanitary sewer main has been stubbed into the site form the west off of Mountain View Drive. This stub connects into the existing Sanitary Sewer main within Mountain View Boulevard.

III. PROPOSED UTILITY INFRASTRUCTURE

Proposed Water

The proposed water infrastructure will be designed to accommodate the ten lots within the project. Service sizes for the future buildings are currently unknown, as the end users for the pad sites have not been determined. Table 1 below identifies the proposed lot area for each pad site, and the proposed use.

One water line loop with two connection points is planned to serve the project's water demands. The Town of Erie Specifications (Ref. 1) states that the jurisdiction prefers PVC pipe for all water mains 12" or less.

The proposed loop is to be an 8" PVC. The proposed loop will connect to the existing 8" existing 8" public water main which runs through the proposed King Soopers site to the east and into the site. In addition, there is another existing 8" public water main which has been stubbed into the site from the west off of Mountain View Boulevard. The water main will run just south of the proposed private drive through the site. The water loop is designed to convey anticipated flows from all of the pad sites, as well as anticipated fire flows.

Table 1: Proposed Lot Size

Lot	Lot Size	Use
1	0.84 ac	Retail
2	0.86 ac	Retail
3	1.48 ac	Retail
4	1.20 ac	Retail
5	0.73 ac	Retail
6	0.92 ac	Retail
7	0.92 ac	Retail
8	0.69 ac	Retail
9	0.88 ac	Retail
10	7.34 ac	Retail

Proposed Sanitary

The proposed sanitary infrastructure will be designed to accommodate all of the pad sites within the project. Service sizes for the future buildings are currently unknown, as the end users for the pad sites have not been determined. Table 1 below identifies the proposed lot area for each pad site, and the proposed use.

The proposed 8" sanitary sewer for the site will connect to an existing 8" public sanitary sewer main has been stubbed into the site form the west off of Mountain View Drive. This stub connects into the existing Sanitary Sewer main within Mountain View Boulevard. The sanitary sewer main will parallel portions of the proposed water line loop while maintaining a minimum of 10-feet of separation.

IV. UTILITY SYSTEM DESIGN CRITERIA

Water

The water system was designed and analyzed using the Town of Erie Design Specifications (Ref. 1). Demands for the project were obtained using criteria outlined in Table 2. A sample calculation is as follows for a 0.98-acre Retail/Commercial parcel. The rest of the calculations for the site can be found in Appendix A.

Average Daily Demand (AD) = (0.98-acre) x (1651 GPD/acre) = 1,613 GPD
=1.125 gpm

Max Day Demand =AD x Max Day Factor
=2.0 x 1.125
=2.25 gpm

Max Hour Demand =AD x Max Hr Factor
=(1.125 gpm) x 3.00
=3.37 gpm

Table 2: Estimated Water Loading Criteria

Future Phase	Average Day Demand	Max Day Factor*	Max Hour Factor
Residential	140 GPCD	2.6	3.9
Commercial	1651	2.0	3.0
Industrial	1651	1.32	3.0

The proposed water main was designed using the following constraints:

Table 3: Maximum and Minimum Water Design Constraints

Flow Scenario	Minimum Static Pressure	Maximum Static Pressure	Maximum Velocity	Maximum Headloss
Average Daily*	43 psi	125 psi	10 fps	2 ft/ 1000ft*
Max Day	43 psi	N/A	10 fps	2 ft/ 1000ft*
Max Hour	20 psi**	N/A	10 fps	2 ft/ 1000ft*

*Maximum Headloss for 8"-12" waterlines

**Minimum residual pressure during fire flow

Fire flow requirements were obtained using the Town of Erie Design Specifications. Using these criteria, the required fire flow for the site will be 2,500 gpm. Fire Hydrants will be placed on the pad sites as they develop in the future.

Existing static pressures were obtained for a couple of locations adjacent to the site. The following pressures were obtained:

1. 98.6 psi – at the intersection of Mountain View Blvd. and Ridge View Dr.
2. 86.6 psi – at the intersection of Ridge View Dr. and Sheridan Pkwy.

Sanitary Sewer

The sanitary sewer system was designed and analyzed using the Town of Erie Design Specifications (Ref. 1). Demands for the project were obtained using criteria outlined in Table 4. A sample calculation is as follows for a 0.98-acre Retail/Commercial parcel. The rest of the calculations for the site can be found in Appendix A.

Average Daily Demand (AD) = (98-acre) x (1,000 gal/acre/day) = 980 GPD
=0.0015 cfs

Peaking Factor (PF) $=3.8/(ADF)^{0.17}$ where ADF= annual average daily flow in MGD
PF will not be less than 2.5 or greater than 5.0
$=3.8/(980 \times 4.0E10^{-4})^{0.17}$
=4.53

Peak Flow Demand =AD x PF
=(.002 cfs) x 4.53
=0.007 cfs

Table 4: Estimated Sanitary Sewer Loading Criteria

Future Phase	Average Day Demand
Residential/Multi Family	90 gal/capita/day
Industrial	1,500 gal/acre/day
Commercial	1,000 gal/acre/day
Park/Recreation	50 gal/acre/day
Elementary Schools	13 gal/student/day
Jr. & Sr. High School	20 gal/student/day

Sewers 10" in diameter and smaller are to be designed to carry the peak design flow at a maximum flow depth of 80% of the pipe diameter. The minimum velocity at the peak design flow shall be 2 feet per second.

V. UTILITY ANALYSIS AND RESULTS

<u>WaterCAD Analysis</u>

Bentley WaterCAD Version V8i was used to model the proposed water system. The anticipated demands used in the model can be found in Table 5. Three scenarios were modeled for analysis of the loop. Descriptions of each scenario are as follows:

- Average Daily Demand – Includes average daily demands at each building/lot.
- Max Day Demand – includes max day demands at each building/lot.
- Max-Hour Demand + Fire Flow– includes max-hour demands at each building/lot plus the required fire flow of 2,500 gpm. The proposed system satisfies the fire flow and sprinkler requirement with a 20 psi residual pressure.
- Flows from the adjacent King Soopers Site and Les Schwab site have been included in

the WaterCAD model for the site. Excerpts of the King Soopers utility report has been included and can be found in Appendix C.

- An irrigation demand of a maximum 18 gpm for a 1" irrigation tap for the proposed landscaping north of the private road adjacent to the retaining walls only.

Table 5: Anticipated Water Demands by Building/Lot

Phase	Average Daily Demand (GPM)	Max Daily Demand (GPM)	Max-Hour Demand (GPM)
Lot 1	0.96	1.93	2.89
Lot 2	0.99	1.97	2.96
Lot 3	1.70	3.39	5.09
Lot 4	1.38	2.75	4.13
Lot 5	0.84	1.67	2.51
Lot 6	1.06	2.11	3.17
Lot 7	1.06	2.11	3.17
Lot 8	0.79	1.58	2.37
Lot 9	1.01	2.02	3.03
Lot 10	8.42	16.83	25.25

WaterCAD Results

The results are summarized in this section. Refer to Appendix B for detailed results and figures. Table 6 shows that the proposed water main loop is sufficiently sized with respect to the criteria described in Table 6 with exception to the head loss requirement during Max Hour and Fire Flow. The head loss during this event does not adversely affect the operating pressure.

Table 6: Water Loop Results

Scenario	Minimum Pressure (psi)	Maximum Pressure (psi)	Maximum Velocity (fps)	Maximum Head Loss (ft/1000 ft)
Average Daily Demand	76.8 @ J-33	94.8 @ J-29	1.10 @ P-47	0.41 @ P-54
Max Day Demand	76.7 @ J-33	94.8 @ J-29	1.15 @ P-47	0.38 @ P-54
Max Hour Demand	76.7 @ J-33	94.7 @ J-29	1.21 @ P-47	0.37 @ P-54
Max Hour + Fire Flow*	58.6 @ Lot 9/10	90.5 @ J-29	9.83 @ P-47	12.20 @ P-54

*Fire Flow – 1250 gpm at Lot 7/8, Lot 9/10 Nodes

Sanitary Sewer Analysis

Bentley Flowmaster Version V8i was used to model the proposed sanitary system. The anticipated demands used in the model can be found in Table 7.

Table 7: Anticipated Sanitary Sewer Demands by Building/Lot

Phase	Average Day Demand (cfs)	Peak Factor	Peak Flow (cfs)
Lot 1	0.0013	4.65	0.006
Lot 2	0.0013	4.63	0.006
Lot 3	0.0023	4.22	0.010
Lot 4	0.0019	4.37	0.008
Lot 5	0.0011	4.76	0.005
Lot 6	0.0014	4.57	0.007
Lot 7	0.0014	4.57	0.007
Lot 8	0.0011	4.80	0.005
Lot 9	0.0014	4.61	0.006
Lot 10	0.0114	3.21	0.037

Flowmaster Results

The sanitary sewer results are summarized in this section. Refer to Appendix B for detailed results and figures. Due to constraints associated with the existing sanitary invert elevations and the site grading, the proposed sanitary sewer system is anticipated to slope at a minimum 0.4%. This is to provide the maximum depth possible in order to best serve the buildings, and avoid utility conflicts. Theses constraints result in sanitary sewer velocities below the required 2 feet per second during peak flows.

Calculations are included in the Appendix that calculates the minimum and maximum peak flow through the proposed sanitary sewer system. The proposed system does not reached the maximum 80% capacity, as required by the City standards. The calculations show that the system flows at a maximum of 12.8% full. A summary of the velocities are found in the table below:

Table V-IV: Sanitary Sewer Results

Minimum Velocity	1.07 fps	Lot 9
Maximum Velocity	2.80 fps	Project Site

VI. CONCLUSION

The proposed water infrastructure to be constructed with the Vista Ridge Commercial West project includes one water main loop. The water loop is planned to be 8" PVC connecting to existing stubs on the east and west sides of the site. The results of this study show that, according to the criteria set forth by the Town of Erie, the proposed water infrastructure is adequately sized.

The proposed sanitary sewer infrastructure is 8" PVC and sufficiently serves the project. Due to site constraints, the sanitary sewer system is anticipated to slope at a minimum 0.4%. The minimum slopes result in the sanitary sewer system not meeting the Town's minimum velocity requirement during peak flow.

VII. REFERENCES

1. Standards and Specifications for Design and Construction of Public Improvements; Town of Erie, 2015 Edition.
2. Geotechnical Engineering Study Proposed Commercial Development Northeast Corner of State Highway 7 and Mountain View Drive, Erie, Colorado; Kumar & Associates, Inc., October 12, 2015.
3. International Fire Code; International Code Council, 2015

Appendix A
Demands

Vista Ridge Commercial
Erie, CO
SH7000001.01

Potable Water Distribution System Design Criteria

Hazen Williams	100	8"-12" PIPE
Operating Pressures		
Minimum Static Pressure	43 psi	(per 612.00)
Maximum Static Pressure	125 psi	(per 612.00)
Minimum Dynamic Pressure		
Max Hr Demand + fire flow	20 psi	(per jurisdiction)
Maximum Velocities		
Maximum Pipe Velocity	10 fps	(per 619.01)
Headloss	2 ft per 1000'	(per 619.01)
Fire		
Fire Hydrant Demand	2500 gpm	
Fire Pressure Residual	20 psi	(per 611.00)
Fire Duration	2 Hr	(per IFC / Fire Dept)

Domestic Water Demand per Land Classification (per 611.00)

Land Use	Average Day	Max Day Ratio	Max Hour Ratio
Residential Multi family	140 GPCD*	2.60	3.90
Commercial	1651 GPD/Acre	2.00	3.00
Indust.	1651 GPD/Acre	1.13	3.00

*Gallons Per Capita/Day

Demand

Land Use/Building	Area (Acre)	Average Day (GPM)	Max Day (GPM)	Max Hour (GPM)
Lot 1	0.84	0.96	1.93	2.89
Lot 2	0.86	0.99	1.97	2.96
Lot 3	1.48	1.70	3.39	5.09
Lot 4	1.2	1.38	2.75	4.13
Lot 5	0.73	0.84	1.67	2.51
Lot 6	0.92	1.06	2.11	3.17
Lot 7	0.92	1.06	2.11	3.17
Lot 8	0.69	0.79	1.58	2.37
Lot 9	0.88	1.01	2.02	3.03
Lot 10	7.34	8.42	16.83	25.25

Les Schwab	1.52	1.74	3.49	5.23

Vista Ridge Commercial West
Erie, CO
SH7000001.01

Sanitary Sewer Design Criteria

Manning's n	0.015		(per 714.01)
Minimum Pipe Velocity (Peak Flow)	2	fps	(per 713.00)
Maximum Pipe Velocity (Peak Flow)	10	fps	(per 713.00)
Max Flow Depth of Pipe Dia.	80	%	(per 713.00)

Sanitary Sewer Demand per Land Classification (per 712.00)

Land Use	Average Day
Residential Multi family	90 gal/capita/day
Industrial	1,500 gal/acre/day
Commercial	1,000 gal/acre/day
Park/Recreation	50 gal/acre/day
Elementary Schools	13 gal/student/day
Jr. & Sr. High School	20 gal/student/day

Demand

Land Use/Building	Area (Acre)	Average Day (GPD)	Average Day (cfs)	Peak Factor**	Peak Flow (cfs)
Lot 1	0.84	840	0.0013	4.65	0.006
Lot 2	0.86	860	0.0013	4.63	0.006
Lot 3	1.48	1480	0.0023	4.22	0.010
Lot 4	1.2	1200	0.0019	4.37	0.008
Lot 5	0.73	730	0.0011	4.76	0.005
Lot 6	0.92	920	0.0014	4.57	0.007
Lot 7	0.92	920	0.0014	4.57	0.007
Lot 8	0.69	690	0.0011	4.80	0.005
Lot 9	0.88	880	0.0014	4.61	0.006
Lot 10	7.34	7340	0.0114	3.21	0.037
Total	15.86	15860.00	0.0245	2.82	0.069

*Building demand based upon Kroger requirment of 2,500 GPD
**PF=3.8/(ADF)^0.17 where ADF = annual average daily flow in MGD (PF will not be less than 2.5 or greater than 5.0)

Appendix B

WaterCAD & Flowmaster Results

Average Day Demand

FlexTable: Pipe Table

ID	Label	Length (Scaled) (ft)	Start Node	Stop Node	Diameter (in)	Material	Hazen-Williams C	Flow (gpm)	Velocity (ft/s)	Headloss (ft)
32	P-3	280.54	J-2	J-3	8.0	PVC	100.0	72.97	0.47	0.07
34	P-4	20.92	J-3	J-4	8.0	PVC	100.0	-103.60	0.66	0.01
36	P-5	91.65	J-4	J-1	8.0	PVC	100.0	-103.60	0.66	0.04
38	P-6	96.13	J-1	J-6	8.0	PVC	100.0	-105.31	0.67	0.04
40	P-7	51.54	J-6	J-7	8.0	PVC	100.0	-105.31	0.67	0.02
42	P-8	238.78	J-7	J-8	8.0	PVC	100.0	-106.57	0.68	0.11
44	P-9	37.20	J-8	J-9	8.0	PVC	100.0	-107.83	0.69	0.02
46	P-10	72.01	J-9	J-10	8.0	PVC	100.0	-107.83	0.69	0.03
48	P-11	98.48	J-10	J-11	8.0	PVC	100.0	-107.83	0.69	0.05
50	P-12	444.04	J-11	J-12	8.0	PVC	100.0	-109.18	0.70	0.22
52	P-13	14.27	J-12	J-13	8.0	PVC	100.0	-109.18	0.70	0.01
54	P-14	261.41	J-13	J-14	8.0	PVC	100.0	-110.46	0.71	0.13
57	P-16	42.85	J-14	J-15	12.0	PVC	100.0	515.67	1.46	0.05
59	P-17	147.54	J-15	J-16	12.0	PVC	100.0	515.67	1.46	0.18
66	OS-3	294.44	J-19	R-2	12.0	PVC	100.0	-1,403.46	3.98	2.27
67	P-22	72.24	J-19	J-14	12.0	PVC	100.0	626.13	1.78	0.13
76	P-25	11.78	J-16	J-20	12.0	PVC	100.0	513.93	1.46	0.01
77	P-26	174.16	J-20	J-17	12.0	PVC	100.0	513.93	1.46	0.21
80	P-28	220.07	J-21	J-2	8.0	PVC	100.0	72.97	0.47	0.05
92	P-30	449.91	J-22	J-19	12.0	PVC	100.0	-777.33	2.21	1.16
94	P-31	39.47	J-22	J-23	8.0	PVC	100.0	-440.96	2.81	0.26
96	P-32	75.26	J-24	J-17	12.0	PVC	100.0	-513.93	1.46	0.09
102	P-27	235.87	J-21	J-24	8.0	PVC	150.0	-72.97	0.47	0.03
104	P-38	19.35	J-23	J-26	8.0	PVC	100.0	-440.96	2.81	0.16
105	P-39	17.16	J-26	J-24	8.0	PVC	100.0	-440.96	2.81	0.08
107	P-40	45.97	R-1	J-27	12.0	PVC	100.0	-1,374.91	3.90	0.34
108	P-41	259.78	J-27	J-22	12.0	PVC	100.0	-1,218.29	3.46	1.54
110	P-42	1,513.14	J-27	J-28	12.0	PVC	100.0	-156.62	0.44	0.20
112	P-43	562.05	J-28	J-29	12.0	PVC	100.0	-156.62	0.44	0.07
114	P-44	320.01	J-29	Lot 1-2	12.0	PVC	100.0	-120.57	0.34	0.03
120	P-47	111.23	Lot 9-10	J-33	8.0	PVC	100.0	-176.57	1.13	0.13
121	P-48	239.91	J-33	J-3	8.0	PVC	100.0	-176.57	1.13	0.29
124	P-49	147.81	J-29	J-35	8.0	PVC	100.0	-36.04	0.23	0.01
126	P-50	318.74	J-35	J-36	8.0	PVC	100.0	-36.04	0.23	0.02
127	P-51	145.85	J-36	Lot 1-2	8.0	PVC	100.0	20.42	0.13	0.00
129	P-52	175.83	J-36	J-37	8.0	PVC	100.0	-56.47	0.36	0.03
131	P-53	211.38	Lot 1-2	Lot 3-4	8.0	PVC	100.0	-102.10	0.65	0.09
132	P-54	421.32	Lot 3-4	Lot 5-6	8.0	PVC	100.0	-163.39	1.04	0.44
134	P-55	380.39	Lot 5-6	Lot 7-8	8.0	PVC	100.0	-165.29	1.05	0.40
135	P-56	222.71	Lot 7-8	Lot 9-10	8.0	PVC	100.0	-167.14	1.07	0.24
136	P-57	411.27	J-37	Lot 3-4	8.0	PVC	100.0	-58.21	0.37	0.06

FlexTable: Junction Table

ID	Label	Elevation (ft)	Demand (gpm)	Hydraulic Grade (ft)	Pressure (psi)
29	J-2	5,277.00	0.00	5,460.74	79.5
31	J-3	5,280.21	0.00	5,460.68	78.1
33	J-4	5,280.09	0.00	5,460.69	78.1
35	J-1	5,279.30	1.71	5,460.73	78.5
37	J-6	5,279.56	0.00	5,460.77	78.4
39	J-7	5,279.82	1.26	5,460.79	78.3
41	J-8	5,279.23	1.26	5,460.91	78.6
43	J-9	5,279.04	0.00	5,460.92	78.7
45	J-10	5,277.60	0.00	5,460.96	79.3
47	J-11	5,275.57	1.35	5,461.01	80.2
49	J-12	5,272.09	0.00	5,461.22	81.8
51	J-13	5,271.89	1.28	5,461.23	81.9
53	J-14	5,270.25	0.00	5,461.36	82.7
56	J-15	5,271.19	0.00	5,461.31	82.3
58	J-16	5,273.15	1.74	5,461.13	81.3
60	J-17	5,275.55	0.00	5,460.91	80.2
64	J-19	5,268.85	0.00	5,461.49	83.3
75	J-20	5,273.15	0.00	5,461.12	81.3
78	J-21	5,277.50	0.00	5,460.79	79.3
90	J-22	5,280.41	0.00	5,460.32	77.8
93	J-23	5,280.22	0.00	5,460.58	78.0
95	J-24	5,276.41	0.00	5,460.82	79.8
103	J-26	5,277.00	0.00	5,460.74	79.5
106	J-27	5,279.30	0.00	5,458.78	77.7
109	J-28	5,231.00	0.00	5,458.98	98.6
111	J-29	5,240.00	0.00	5,459.06	94.8
113	Lot 1-2	5,248.00	1.95	5,459.08	91.3
115	Lot 5-6	5,271.98	1.90	5,459.61	81.2
117	Lot 9-10	5,282.65	9.43	5,460.26	76.8
119	J-33	5,283.00	0.00	5,460.39	76.7
123	J-35	5,246.00	0.00	5,459.07	92.2
125	J-36	5,248.00	0.00	5,459.09	91.3
128	J-37	5,252.00	1.74	5,459.11	89.6
130	Lot 3-4	5,257.07	3.08	5,459.18	87.4
133	Lot 7-8	5,279.48	1.85	5,460.01	78.1

Max Day Demand

FlexTable: Pipe Table

ID	Label	Length (Scaled) (ft)	Start Node	Stop Node	Diameter (in)	Material	Hazen-Williams C	Flow (gpm)	Velocity (ft/s)	Headloss (ft)
32	P-3	280.54	J-2	J-3	8.0	PVC	100.0	81.70	0.52	0.08
34	P-4	20.92	J-3	J-4	8.0	PVC	100.0	-102.47	0.65	0.01
36	P-5	91.65	J-4	J-1	8.0	PVC	100.0	-102.47	0.65	0.04
38	P-6	96.13	J-1	J-6	8.0	PVC	100.0	-105.95	0.68	0.04
40	P-7	51.54	J-6	J-7	8.0	PVC	100.0	-105.95	0.68	0.02
42	P-8	238.78	J-7	J-8	8.0	PVC	100.0	-108.47	0.69	0.12
44	P-9	37.20	J-8	J-9	8.0	PVC	100.0	-110.99	0.71	0.02
46	P-10	72.01	J-9	J-10	8.0	PVC	100.0	-110.99	0.71	0.04
48	P-11	98.48	J-10	J-11	8.0	PVC	100.0	-110.99	0.71	0.05
50	P-12	444.04	J-11	J-12	8.0	PVC	100.0	-113.70	0.73	0.23
52	P-13	14.27	J-12	J-13	8.0	PVC	100.0	-113.70	0.73	0.01
54	P-14	261.41	J-13	J-14	8.0	PVC	100.0	-116.27	0.74	0.14
57	P-16	42.85	J-14	J-15	12.0	PVC	100.0	519.96	1.48	0.05
59	P-17	147.54	J-15	J-16	12.0	PVC	100.0	519.96	1.48	0.18
66	OS-3	294.44	J-19	R-2	12.0	PVC	100.0	-1,412.64	4.01	2.30
67	P-22	72.24	J-19	J-14	12.0	PVC	100.0	636.23	1.80	0.13
76	P-25	11.78	J-16	J-20	12.0	PVC	100.0	516.48	1.47	0.01
77	P-26	174.16	J-20	J-17	12.0	PVC	100.0	516.48	1.47	0.21
80	P-28	220.07	J-21	J-2	8.0	PVC	100.0	81.70	0.52	0.06
92	P-30	449.91	J-22	J-19	12.0	PVC	100.0	-776.41	2.20	1.16
94	P-31	39.47	J-22	J-23	8.0	PVC	100.0	-434.78	2.78	0.25
96	P-32	75.26	J-24	J-17	12.0	PVC	100.0	-516.48	1.47	0.09
102	P-27	235.87	J-21	J-24	8.0	PVC	150.0	-81.70	0.52	0.03
104	P-38	19.35	J-23	J-26	8.0	PVC	100.0	-434.78	2.78	0.15
105	P-39	17.16	J-26	J-24	8.0	PVC	100.0	-434.78	2.78	0.08
107	P-40	45.97	R-1	J-27	12.0	PVC	100.0	-1,356.51	3.85	0.33
108	P-41	259.78	J-27	J-22	12.0	PVC	100.0	-1,211.19	3.44	1.53
110	P-42	1,513.14	J-27	J-28	12.0	PVC	100.0	-145.33	0.41	0.18
112	P-43	562.05	J-28	J-29	12.0	PVC	100.0	-145.33	0.41	0.06
114	P-44	320.01	J-29	Lot 1-2	12.0	PVC	100.0	-111.94	0.32	0.02
120	P-47	111.23	Lot 9-10	J-33	8.0	PVC	100.0	-184.17	1.18	0.14
121	P-48	239.91	J-33	J-3	8.0	PVC	100.0	-184.17	1.18	0.31
124	P-49	147.81	J-29	J-35	8.0	PVC	100.0	-33.39	0.21	0.01
126	P-50	318.74	J-35	J-36	8.0	PVC	100.0	-33.39	0.21	0.02
127	P-51	145.85	J-36	Lot 1-2	8.0	PVC	100.0	18.59	0.12	0.00
129	P-52	175.83	J-36	J-37	8.0	PVC	100.0	-51.98	0.33	0.02
131	P-53	211.38	Lot 1-2	Lot 3-4	8.0	PVC	100.0	-96.24	0.61	0.08
132	P-54	421.32	Lot 3-4	Lot 5-6	8.0	PVC	100.0	-157.86	1.01	0.41
134	P-55	380.39	Lot 5-6	Lot 7-8	8.0	PVC	100.0	-161.63	1.03	0.39
135	P-56	222.71	Lot 7-8	Lot 9-10	8.0	PVC	100.0	-165.32	1.06	0.24
136	P-57	411.27	J-37	Lot 3-4	8.0	PVC	100.0	-55.47	0.35	0.06

FlexTable: Junction Table

ID	Label	Elevation (ft)	Demand (gpm)	Hydraulic Grade (ft)	Pressure (psi)
29	J-2	5,277.00	0.00	5,460.69	79.5
31	J-3	5,280.21	0.00	5,460.60	78.0
33	J-4	5,280.09	0.00	5,460.61	78.1
35	J-1	5,279.30	3.48	5,460.65	78.5
37	J-6	5,279.56	0.00	5,460.70	78.4
39	J-7	5,279.82	2.52	5,460.72	78.3
41	J-8	5,279.23	2.52	5,460.84	78.6
43	J-9	5,279.04	0.00	5,460.86	78.7
45	J-10	5,277.60	0.00	5,460.89	79.3
47	J-11	5,275.57	2.71	5,460.94	80.2
49	J-12	5,272.09	0.00	5,461.18	81.8
51	J-13	5,271.89	2.57	5,461.19	81.9
53	J-14	5,270.25	0.00	5,461.33	82.7
56	J-15	5,271.19	0.00	5,461.28	82.2
58	J-16	5,273.15	3.48	5,461.10	81.3
60	J-17	5,275.55	0.00	5,460.87	80.2
64	J-19	5,268.85	0.00	5,461.46	83.3
75	J-20	5,273.15	0.00	5,461.08	81.3
78	J-21	5,277.50	0.00	5,460.75	79.3
90	J-22	5,280.41	0.00	5,460.30	77.8
93	J-23	5,280.22	0.00	5,460.55	78.0
95	J-24	5,276.41	0.00	5,460.78	79.8
103	J-26	5,277.00	0.00	5,460.70	79.5
106	J-27	5,279.30	0.00	5,458.77	77.6
109	J-28	5,231.00	0.00	5,458.95	98.6
111	J-29	5,240.00	0.00	5,459.01	94.8
113	Lot 1-2	5,248.00	2.90	5,459.04	91.3
115	Lot 5-6	5,271.98	3.78	5,459.53	81.1
117	Lot 9-10	5,282.65	18.85	5,460.15	76.8
119	J-33	5,283.00	0.00	5,460.29	76.7
123	J-35	5,246.00	0.00	5,459.02	92.2
125	J-36	5,248.00	0.00	5,459.04	91.3
128	J-37	5,252.00	3.49	5,459.06	89.6
130	Lot 3-4	5,257.07	6.14	5,459.12	87.4
133	Lot 7-8	5,279.48	3.69	5,459.91	78.1

Max Hour Demand

FlexTable: Pipe Table

ID	Label	Length (Scaled) (ft)	Start Node	Stop Node	Diameter (in)	Material	Hazen-Williams C	Flow (gpm)	Velocity (ft/s)	Headloss (ft)
32	P-3	280.54	J-2	J-3	8.0	PVC	100.0	91.83	0.59	0.10
34	P-4	20.92	J-3	J-4	8.0	PVC	100.0	-102.11	0.65	0.01
36	P-5	91.65	J-4	J-1	8.0	PVC	100.0	-102.11	0.65	0.04
38	P-6	96.13	J-1	J-6	8.0	PVC	100.0	-107.24	0.68	0.05
40	P-7	51.54	J-6	J-7	8.0	PVC	100.0	-111.02	0.71	0.03
42	P-8	238.78	J-7	J-8	8.0	PVC	100.0	-111.02	0.71	0.12
44	P-9	37.20	J-8	J-9	8.0	PVC	100.0	-114.80	0.73	0.02
46	P-10	72.01	J-9	J-10	8.0	PVC	100.0	-114.80	0.73	0.04
48	P-11	98.48	J-10	J-11	8.0	PVC	100.0	-114.80	0.73	0.05
50	P-12	444.04	J-11	J-12	8.0	PVC	100.0	-118.86	0.76	0.26
52	P-13	14.27	J-12	J-13	8.0	PVC	100.0	-118.86	0.76	0.01
54	P-14	261.41	J-13	J-14	8.0	PVC	100.0	-122.71	0.78	0.16
57	P-16	42.85	J-14	J-15	12.0	PVC	100.0	525.12	1.49	0.05
59	P-17	147.54	J-15	J-16	12.0	PVC	100.0	525.12	1.49	0.18
66	OS-3	294.44	J-19	R-2	12.0	PVC	100.0	-1,423.85	4.04	2.33
67	P-22	72.24	J-19	J-14	12.0	PVC	100.0	647.83	1.84	0.13
76	P-25	11.78	J-16	J-20	12.0	PVC	100.0	519.90	1.47	0.01
77	P-26	174.16	J-20	J-17	12.0	PVC	100.0	519.90	1.47	0.21
80	P-28	220.07	J-21	J-2	8.0	PVC	100.0	91.83	0.59	0.08
92	P-30	449.91	J-22	J-19	12.0	PVC	100.0	-776.02	2.20	1.16
94	P-31	39.47	J-22	J-23	8.0	PVC	100.0	-428.06	2.73	0.24
96	P-32	75.26	J-24	J-17	12.0	PVC	100.0	-519.90	1.47	0.09
102	P-27	235.87	J-21	J-24	8.0	PVC	150.0	-91.83	0.59	0.04
104	P-38	19.35	J-23	J-26	8.0	PVC	100.0	-428.06	2.73	0.15
105	P-39	17.16	J-26	J-24	8.0	PVC	100.0	-428.06	2.73	0.08
107	P-40	45.97	R-1	J-27	12.0	PVC	100.0	-1,320.23	3.75	0.32
108	P-41	259.78	J-27	J-22	12.0	PVC	100.0	-1,204.08	3.42	1.51
110	P-42	1,513.14	J-27	J-28	12.0	PVC	100.0	-116.14	0.33	0.12
112	P-43	562.05	J-28	J-29	12.0	PVC	100.0	-116.14	0.33	0.04
114	P-44	320.01	J-29	Lot 1-2	12.0	PVC	100.0	-88.51	0.25	0.01
120	P-47	111.23	Lot 9-10	J-33	8.0	PVC	100.0	-193.94	1.24	0.16
121	P-48	239.91	J-33	J-3	8.0	PVC	100.0	-193.94	1.24	0.34
124	P-49	147.81	J-29	J-35	8.0	PVC	100.0	-27.64	0.18	0.01
126	P-50	318.74	J-35	J-36	8.0	PVC	100.0	-27.64	0.18	0.01
127	P-51	145.85	J-36	Lot 1-2	8.0	PVC	100.0	20.40	0.13	0.00
129	P-52	175.83	J-36	J-37	8.0	PVC	100.0	-48.04	0.31	0.02
131	P-53	211.38	Lot 1-2	Lot 3-4	8.0	PVC	100.0	-91.96	0.59	0.08
132	P-54	421.32	Lot 3-4	Lot 5-6	8.0	PVC	100.0	-154.44	0.99	0.39
134	P-55	380.39	Lot 5-6	Lot 7-8	8.0	PVC	100.0	-160.12	1.02	0.38
135	P-56	222.71	Lot 7-8	Lot 9-10	8.0	PVC	100.0	-165.66	1.06	0.24
136	P-57	411.27	J-37	Lot 3-4	8.0	PVC	100.0	-53.27	0.34	0.05

FlexTable: Junction Table

ID	Label	Elevation (ft)	Demand (gpm)	Hydraulic Grade (ft)	Pressure (psi)
29	J-2	5,277.00	0.00	5,460.62	79.4
31	J-3	5,280.21	0.00	5,460.52	78.0
33	J-4	5,280.09	0.00	5,460.52	78.1
35	J-1	5,279.30	5.13	5,460.56	78.4
37	J-6	5,279.56	3.78	5,460.61	78.3
39	J-7	5,279.82	0.00	5,460.64	78.2
41	J-8	5,279.23	3.78	5,460.76	78.5
43	J-9	5,279.04	0.00	5,460.78	78.6
45	J-10	5,277.60	0.00	5,460.82	79.3
47	J-11	5,275.57	4.06	5,460.87	80.2
49	J-12	5,272.09	0.00	5,461.12	81.8
51	J-13	5,271.89	3.85	5,461.13	81.9
53	J-14	5,270.25	0.00	5,461.29	82.7
56	J-15	5,271.19	0.00	5,461.24	82.2
58	J-16	5,273.15	5.22	5,461.05	81.3
60	J-17	5,275.55	0.00	5,460.83	80.2
64	J-19	5,268.85	0.00	5,461.43	83.3
75	J-20	5,273.15	0.00	5,461.04	81.3
78	J-21	5,277.50	0.00	5,460.69	79.3
90	J-22	5,280.41	0.00	5,460.27	77.8
93	J-23	5,280.22	0.00	5,460.51	78.0
95	J-24	5,276.41	0.00	5,460.73	79.7
103	J-26	5,277.00	0.00	5,460.66	79.5
106	J-27	5,279.30	0.00	5,458.76	77.6
109	J-28	5,231.00	0.00	5,458.87	98.6
111	J-29	5,240.00	0.00	5,458.92	94.7
113	Lot 1-2	5,248.00	23.85	5,458.93	91.3
115	Lot 5-6	5,271.98	5.68	5,459.40	81.1
117	Lot 9-10	5,282.65	28.28	5,460.02	76.7
119	J-33	5,283.00	0.00	5,460.17	76.7
123	J-35	5,246.00	0.00	5,458.92	92.1
125	J-36	5,248.00	0.00	5,458.93	91.3
128	J-37	5,252.00	5.23	5,458.95	89.5
130	Lot 3-4	5,257.07	9.22	5,459.01	87.4
133	Lot 7-8	5,279.48	5.54	5,459.78	78.0

Max Hour Plus Fire Flow Demand

FlexTable: Pipe Table

ID	Label	Length (Scaled) (ft)	Start Node	Stop Node	Diameter (in)	Material	Hazen-Williams C	Flow (gpm)	Velocity (ft/s)	Headloss (ft)
32	P-3	280.54	J-2	J-3	8.0	PVC	100.0	923.15	5.89	7.18
34	P-4	20.92	J-3	J-4	8.0	PVC	100.0	-587.05	3.75	0.23
36	P-5	91.65	J-4	J-1	8.0	PVC	100.0	-587.05	3.75	1.01
38	P-6	96.13	J-1	J-6	8.0	PVC	100.0	-592.18	3.78	1.08
40	P-7	51.54	J-6	J-7	8.0	PVC	100.0	-595.96	3.80	0.59
42	P-8	238.78	J-7	J-8	8.0	PVC	100.0	-595.96	3.80	2.72
44	P-9	37.20	J-8	J-9	8.0	PVC	100.0	-599.74	3.83	0.43
46	P-10	72.01	J-9	J-10	8.0	PVC	100.0	-599.74	3.83	0.83
48	P-11	98.48	J-10	J-11	8.0	PVC	100.0	-599.74	3.83	1.13
50	P-12	444.04	J-11	J-12	8.0	PVC	100.0	-603.80	3.85	5.18
52	P-13	14.27	J-12	J-13	8.0	PVC	100.0	-603.80	3.85	0.17
54	P-14	261.41	J-13	J-14	8.0	PVC	100.0	-607.65	3.88	3.09
57	P-16	42.85	J-14	J-15	12.0	PVC	100.0	632.06	1.79	0.08
59	P-17	147.54	J-15	J-16	12.0	PVC	100.0	632.06	1.79	0.26
66	OS-3	294.44	J-19	R-2	12.0	PVC	100.0	-1,954.43	5.54	4.20
67	P-22	72.24	J-19	J-14	12.0	PVC	100.0	1,239.71	3.52	0.44
76	P-25	11.78	J-16	J-20	12.0	PVC	100.0	626.84	1.78	0.02
77	P-26	174.16	J-20	J-17	12.0	PVC	100.0	626.84	1.78	0.30
80	P-28	220.07	J-21	J-2	8.0	PVC	100.0	923.15	5.89	5.64
92	P-30	449.91	J-22	J-19	12.0	PVC	100.0	-714.72	2.03	1.00
94	P-31	39.47	J-22	J-23	8.0	PVC	100.0	296.31	1.89	0.12
96	P-32	75.26	J-24	J-17	12.0	PVC	100.0	-626.84	1.78	0.13
102	P-27	235.87	J-21	J-24	8.0	PVC	150.0	-923.15	5.89	2.85
104	P-38	19.35	J-23	J-26	8.0	PVC	100.0	296.31	1.89	0.07
105	P-39	17.16	J-26	J-24	8.0	PVC	100.0	296.31	1.89	0.04
107	P-40	45.97	R-1	J-27	12.0	PVC	100.0	649.19	1.84	0.08
108	P-41	259.78	J-27	J-22	12.0	PVC	100.0	-418.40	1.19	0.21
110	P-42	1,513.14	J-27	J-28	12.0	PVC	100.0	1,067.59	3.03	7.04
112	P-43	562.05	J-28	J-29	12.0	PVC	100.0	1,067.59	3.03	2.61
114	P-44	320.01	J-29	Lot 1-2	12.0	PVC	100.0	822.66	2.33	0.92
120	P-47	111.23	Lot 9-10	J-33	8.0	PVC	100.0	-1,510.21	9.64	7.09
121	P-48	239.91	J-33	J-3	8.0	PVC	100.0	-1,510.21	9.64	15.29
124	P-49	147.81	J-29	J-35	8.0	PVC	100.0	244.93	1.56	0.32
126	P-50	318.74	J-35	J-36	8.0	PVC	100.0	244.93	1.56	0.70
127	P-51	145.85	J-36	Lot 1-2	8.0	PVC	100.0	-134.17	0.86	0.10
129	P-52	175.83	J-36	J-37	8.0	PVC	100.0	379.09	2.42	0.87
131	P-53	211.38	Lot 1-2	Lot 3-4	8.0	PVC	100.0	664.65	4.24	2.95
132	P-54	421.32	Lot 3-4	Lot 5-6	8.0	PVC	100.0	1,029.29	6.57	13.20
134	P-55	380.39	Lot 5-6	Lot 7-8	8.0	PVC	100.0	1,023.61	6.53	11.79
135	P-56	222.71	Lot 7-8	Lot 9-10	8.0	PVC	100.0	-231.93	1.48	0.44
136	P-57	411.27	J-37	Lot 3-4	8.0	PVC	100.0	373.86	2.39	1.97

FlexTable: Junction Table

ID	Label	Elevation (ft)	Demand (gpm)	Hydraulic Grade (ft)	Pressure (psi)
29	J-2	5,277.00	0.00	5,449.85	74.8
31	J-3	5,280.21	0.00	5,442.66	70.3
33	J-4	5,280.09	0.00	5,442.89	70.4
35	J-1	5,279.30	5.13	5,443.91	71.2
37	J-6	5,279.56	3.78	5,444.99	71.6
39	J-7	5,279.82	0.00	5,445.58	71.7
41	J-8	5,279.23	3.78	5,448.30	73.1
43	J-9	5,279.04	0.00	5,448.72	73.4
45	J-10	5,277.60	0.00	5,449.55	74.4
47	J-11	5,275.57	4.06	5,450.69	75.8
49	J-12	5,272.09	0.00	5,455.87	79.5
51	J-13	5,271.89	3.85	5,456.03	79.7
53	J-14	5,270.25	0.00	5,459.12	81.7
56	J-15	5,271.19	0.00	5,459.04	81.3
58	J-16	5,273.15	5.22	5,458.78	80.3
60	J-17	5,275.55	0.00	5,458.46	79.1
64	J-19	5,268.85	0.00	5,459.56	82.5
75	J-20	5,273.15	0.00	5,458.76	80.3
78	J-21	5,277.50	0.00	5,455.48	77.0
90	J-22	5,280.41	0.00	5,458.57	77.1
93	J-23	5,280.22	0.00	5,458.44	77.1
95	J-24	5,276.41	0.00	5,458.33	78.7
103	J-26	5,277.00	0.00	5,458.37	78.5
106	J-27	5,279.30	0.00	5,458.35	77.5
109	J-28	5,231.00	0.00	5,451.32	95.3
111	J-29	5,240.00	0.00	5,448.70	90.3
113	Lot 1-2	5,248.00	23.85	5,447.78	86.4
115	Lot 5-6	5,271.98	5.68	5,431.64	69.1
117	Lot 9-10	5,282.65	1,278.28	5,420.29	59.5
119	J-33	5,283.00	0.00	5,427.38	62.5
123	J-35	5,246.00	0.00	5,448.38	87.6
125	J-36	5,248.00	0.00	5,447.68	86.4
128	J-37	5,252.00	5.23	5,446.81	84.3
130	Lot 3-4	5,257.07	9.22	5,444.84	81.2
133	Lot 7-8	5,279.48	1,255.54	5,419.85	60.7

Flowmaster Results

Worksheet for Top Of System

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.015	
Channel Slope	0.01920	ft/ft
Diameter	8.00	in
Discharge	0.006	cfs

Results

Normal Depth	0.03	ft
Flow Area	0.01	ft²
Wetted Perimeter	0.29	ft
Hydraulic Radius	0.02	ft
Top Width	0.28	ft
Critical Depth	0.03	ft
Percent Full	4.7	%
Critical Slope	0.01217	ft/ft
Velocity	1.01	ft/s
Velocity Head	0.02	ft
Specific Energy	0.05	ft
Froude Number	1.23	
Maximum Discharge	1.56	ft³/s
Discharge Full	1.45	ft³/s
Slope Full	0.00000	ft/ft
Flow Type	SuperCritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	4.69	%
Downstream Velocity	Infinity	ft/s

Worksheet for Top Of System

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	0.03	ft
Critical Depth	0.03	ft
Channel Slope	0.01920	ft/ft
Critical Slope	0.01217	ft/ft

Worksheet for Total Peak Flow

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.015	
Channel Slope	0.03970	ft/ft
Diameter	8.00	in
Discharge	0.069	cfs

Results

Normal Depth	0.08	ft
Flow Area	0.03	ft^2
Wetted Perimeter	0.48	ft
Hydraulic Radius	0.05	ft
Top Width	0.44	ft
Critical Depth	0.12	ft
Percent Full	12.4	%
Critical Slope	0.00888	ft/ft
Velocity	2.76	ft/s
Velocity Head	0.12	ft
Specific Energy	0.20	ft
Froude Number	2.04	
Maximum Discharge	2.24	ft^3/s
Discharge Full	2.09	ft^3/s
Slope Full	0.00004	ft/ft
Flow Type	SuperCritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	12.44	%
Downstream Velocity	Infinity	ft/s

Worksheet for Total Peak Flow

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	0.08	ft
Critical Depth	0.12	ft
Channel Slope	0.03970	ft/ft
Critical Slope	0.00888	ft/ft

Appendix C

Excerpts from King Soopers Utility Report

Galloway
Planning. Architecture. Engineering.

Final Utility Study King Soopers Store #129 at Vista Ridge Marketplace

NWC of East Sheridan Pkwy & Hwy 7
Located in the S ½ of Section 33, Township 1 North, Range 68 West, of the 6th Principal Meridian, Town of Erie, County of Weld, State of Colorado

Date: June 4, 2015

Prepared for:

King Soopers Inc.
65 Tejon Street
Denver, Colorado 80223

Prepared by:

Galloway & Company, Inc.
6162 S. Willow Drive, Suite 320
Greenwood Village, CO 80111
Phone (303) 770-8884
Fax (303) 770-3636
Attn: Gary Iwata, P.E.

Table V-I: Anticipated Water Demands by Building/Lot

Phase	Average Daily Demand (GPM)	Max Daily Demand (GPM)	Max-Hour Demand (GPM)
Lot 1/King Soopers	1.74	3.48	5.22
Lot 2	1.28	2.57	3.85
Lot 3	1.35	2.71	4.06
Lot 4	1.26	2.52	3.78
Lot 5	1.26	2.52	3.78
Lot 6	1.71	3.42	5.13

WaterCAD Results

The results are summarized in this section. Refer to Appendix B for detailed results and figures. Table V-II shows that the proposed water main loop is sufficiently sized with respect to the criteria described in Table IV-II with exception to the head loss requirement during Max Hour and Fire Flow. The head loss during this event does not adversely affect the operating pressure.

Table V-II: Water Loop Results

Scenario	Minimum Pressure (psi)	Maximum Pressure (psi)	Maximum Velocity (fps)	Maximum Head Loss (ft/1000 ft)
Average Daily Demand	78.5 @ J-18	84.0 @ J-19	1.13 @ P-22	0.10 @ P-26
Max Day Demand	78.5 @ J-18	84.0 @ J-19	1.14 @ P-22	0.10 @ P-26
Max Hour Demand	78.4 @ J-18	84.0 @ J-19	1.16 @ P-22	0.10 @ P-26
Max Hour + Fire Flow*	65.4 @ J-3, Lot 5	78.6 @ J-19	7.94 @ P-22	9.28 @ P-28

*Fire Flow – 1000 gpm at J-2, J-12, J-15 & J-17; 850 gpm sprinkler at J-20

Sanitary Sewer Analysis

Bentley Flowmaster Version V8i was used to model the proposed sanitary system. The anticipated demands used in the model can be found in Table V-III.

King Soopers-City Market
Erie, CO
KSS129.01

Potable Water Distribution System Design Criteria

Hazen Williams	100	8"-12" PIPE
Operating Pressures		
Minimum Static Pressure	43 psi	(per 612.00)
Maximum Static Pressure	125 psi	(per 612.00)
Minimum Dynamic Pressure		
Max Hr Demand + fire flow	20 psi	(per jurisdiction)
Maximum Velocities		
Maximum Pipe Velocity	10 fps	(per 619.01)
Headloss	2 ft per 1000'	(per 619.01)
Fire		
Fire Hydrant Demand	3750 gpm	(per IFC 2015, 123,000 sf Type II B)
Fire Pressure Residual	20 psi	(per 611.00)
Fire Duration	2 Hr	(per IFC / Fire Dept)
King Soopers Fire Sprinkler System	850 gpm	(per Kroger requirment)

Domestic Water Demand per Land Classification (per 611.00)

Land Use	Average Day	Max Day Ratio	Max Hour Ratio
Residential Multi family	140 GPCD*	2.60	3.90
Commercial	1651 GPD/Acre	2.00	3.00
Indust.	1651 GPD/Acre	1.13	3.00

*Gallons Per Capita/Day

Demand

Land Use/Building	Area (Acre)	Average Day (GPM)	Max Day (GPM)	Max Hour (GPM)
Kings Soopers*		1.74	3.48	5.22
Lot 2	1.12	1.28	2.57	3.85
Lot 3	1.18	1.35	2.71	4.06
Lot 4	1.1	1.26	2.52	3.78
Lot 5	1.1	1.26	2.52	3.78
Lot 6	1.49	1.71	3.42	5.13

*Building demand based upon Kroger requirment of 2,500 GPD

Appendix B

WaterCAD & Flowmaster Results

AVERAGE DAY

FlexTable: Junction Table (KSS129.wtg)

Current Time: 0.000 hours

ID	Label	Elevation (ft)	Demand Collection	Demand (gpm)	Hydraulic Grade (ft)	Pressure (psi)
27	J-1	5,277.56	<Collection: 0 items>	0.00	5,462.62	80.1
29	J-2	5,277.00	<Collection: 0 items>	0.00	5,462.66	80.3
31	J-3	5,280.21	<Collection: 0 items>	0.00	5,462.70	79.0
33	J-4	5,280.09	<Collection: 0 items>	0.00	5,462.70	79.0
37	J-6	5,279.56	<Collection: 0 items>	0.00	5,462.73	79.2
43	J-9	5,279.04	<Collection: 0 items>	0.00	5,462.77	79.5
45	J-10	5,277.60	<Collection: 0 items>	0.00	5,462.78	80.1
49	J-12	5,272.09	<Collection: 0 items>	0.00	5,462.86	82.5
53	J-14	5,270.25	<Collection: 0 items>	0.00	5,462.91	83.4
56	J-15	5,271.19	<Collection: 0 items>	0.00	5,462.88	82.9
60	J-17	5,275.55	<Collection: 0 items>	0.00	5,462.70	81.0
63	J-18	5,281.21	<Collection: 0 items>	0.00	5,462.56	78.5
64	J-19	5,268.85	<Collection: 0 items>	0.00	5,462.96	84.0
75	J-20	5,273.15	<Collection: 0 items>	0.00	5,462.80	82.1
78	J-21	5,277.50	<Collection: 0 items>	0.00	5,462.64	80.1
58	KS	5,273.15	<Collection: 1 items>	1.74	5,462.80	82.1
51	Lot 2	5,271.89	<Collection: 1 items>	1.28	5,462.87	82.6
47	Lot 3	5,275.57	<Collection: 1 items>	1.35	5,462.80	81.0
41	Lot 4	5,279.23	<Collection: 1 items>	1.26	5,462.77	79.4
39	Lot 5	5,279.82	<Collection: 1 items>	1.26	5,462.73	79.1
35	Lot 6	5,279.30	<Collection: 1 items>	1.71	5,462.71	79.4

AVERAGE DAY

FlexTable: Pipe Table (KSS129.wtg)

Current Time: 0.000 hours

ID	Label	Length (Scaled) (ft)	Start Node	Stop Node	Diameter (in)	Hazen-Williams C	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	Has User Defined Length?	Length (User Defined) (ft)
69	OS-1	210.32	J-18	R-1	12.0	100.0	788.61	2.24	4.12	True	1,553.00
65	OS-2	530.95	J-18	J-19	12.0	100.0	-399.87	1.13	0.40	False	0.00
66	OS-3	294.44	J-19	R-2	12.0	100.0	-797.21	2.26	0.80	False	0.00
70	P-1	76.02	J-18	J-1	12.0	100.0	-388.74	1.10	0.05	False	0.00
32	P-3	280.54	J-2	J-3	8.0	100.0	-52.50	0.34	0.04	False	0.00
34	P-4	20.92	J-3	J-4	8.0	100.0	-52.50	0.34	0.00	False	0.00
36	P-5	91.65	J-4	Lot 6	8.0	100.0	-52.50	0.34	0.01	False	0.00
38	P-6	96.13	Lot 6	J-6	8.0	100.0	-54.21	0.35	0.01	False	0.00
40	P-7	51.54	J-6	Lot 5	8.0	100.0	-54.21	0.35	0.01	False	0.00
42	P-8	238.78	Lot 5	Lot 4	8.0	100.0	-55.47	0.35	0.03	False	0.00
44	P-9	37.20	Lot 4	J-9	8.0	100.0	-56.73	0.36	0.01	False	0.00
46	P-10	72.01	J-9	J-10	8.0	100.0	-56.73	0.36	0.01	False	0.00
48	P-11	98.48	J-10	Lot 3	8.0	100.0	-56.73	0.36	0.01	False	0.00
50	P-12	444.04	Lot 3	J-12	8.0	100.0	-58.08	0.37	0.07	False	0.00
52	P-13	14.27	J-12	Lot 2	8.0	100.0	-58.08	0.37	0.00	False	0.00
54	P-14	261.41	Lot 2	J-14	8.0	100.0	-59.36	0.38	0.04	False	0.00
57	P-16	42.85	J-14	J-15	12.0	100.0	337.98	0.96	0.02	False	0.00
59	P-17	147.54	J-15	KS	12.0	100.0	337.98	0.96	0.08	False	0.00
62	P-19	155.89	J-17	J-1	12.0	100.0	336.24	0.95	0.09	False	0.00
67	P-22	72.24	J-19	J-14	12.0	100.0	397.34	1.13	0.05	False	0.00
76	P-25	11.78	KS	J-20	12.0	100.0	336.24	0.95	0.01	False	0.00
77	P-26	174.16	J-20	J-17	12.0	100.0	336.24	0.95	0.10	False	0.00
79	P-27	154.43	J-1	J-21	8.0	100.0	-52.50	0.34	0.02	False	0.00
80	P-28	220.07	J-21	J-2	8.0	100.0	-52.50	0.34	0.03	False	0.00

MAX DAY

FlexTable: Junction Table (KSS129.wtg)

Current Time: 0.000 hours

ID	Label	Elevation (ft)	Demand Collection	Demand (gpm)	Hydraulic Grade (ft)	Pressure (psi)
27	J-1	5,277.56	<Collection: 0 items>	0.00	5,462.60	80.1
29	J-2	5,277.00	<Collection: 0 items>	0.00	5,462.64	80.3
31	J-3	5,280.21	<Collection: 0 items>	0.00	5,462.67	78.9
33	J-4	5,280.09	<Collection: 0 items>	0.00	5,462.68	79.0
37	J-6	5,279.56	<Collection: 0 items>	0.00	5,462.70	79.2
43	J-9	5,279.04	<Collection: 0 items>	0.00	5,462.75	79.5
45	J-10	5,277.60	<Collection: 0 items>	0.00	5,462.76	80.1
49	J-12	5,272.09	<Collection: 0 items>	0.00	5,462.84	82.5
53	J-14	5,270.25	<Collection: 0 items>	0.00	5,462.89	83.3
56	J-15	5,271.19	<Collection: 0 items>	0.00	5,462.87	82.9
60	J-17	5,275.55	<Collection: 0 items>	0.00	5,462.69	81.0
63	J-18	5,281.21	<Collection: 0 items>	0.00	5,462.55	78.5
64	J-19	5,268.85	<Collection: 0 items>	0.00	5,462.95	84.0
75	J-20	5,273.15	<Collection: 0 items>	0.00	5,462.78	82.0
78	J-21	5,277.50	<Collection: 0 items>	0.00	5,462.62	80.1
58	KS	5,273.15	<Collection: 1 items>	3.48	5,462.79	82.0
51	Lot 2	5,271.89	<Collection: 1 items>	2.57	5,462.85	82.6
47	Lot 3	5,275.57	<Collection: 1 items>	2.71	5,462.77	81.0
41	Lot 4	5,279.23	<Collection: 1 items>	2.52	5,462.74	79.4
39	Lot 5	5,279.82	<Collection: 1 items>	2.52	5,462.71	79.1
35	Lot 6	5,279.30	<Collection: 1 items>	3.42	5,462.69	79.3

MAX DAY

FlexTable: Pipe Table (KSS129.wtg)

Current Time: 0.000 hours

ID	Label	Length (Scaled) (ft)	Start Node	Stop Node	Diameter (in)	Hazen-Williams C	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	Has User Defined Length?	Length (User Defined) (ft)
69	OS-1	210.32	J-18	R-1	12.0	100.0	787.04	2.23	4.11	True	1,553.00
65	OS-2	530.95	J-18	J-19	12.0	100.0	-401.04	1.14	0.40	False	0.00
66	OS-3	294.44	J-19	R-2	12.0	100.0	-804.26	2.28	0.81	False	0.00
70	P-1	76.02	J-18	J-1	12.0	100.0	-385.99	1.09	0.05	False	0.00
32	P-3	280.54	J-2	J-3	8.0	100.0	-49.50	0.32	0.03	False	0.00
34	P-4	20.92	J-3	J-4	8.0	100.0	-49.50	0.32	0.00	False	0.00
36	P-5	91.65	J-4	Lot 6	8.0	100.0	-49.50	0.32	0.01	False	0.00
38	P-6	96.13	Lot 6	J-6	8.0	100.0	-52.92	0.34	0.01	False	0.00
40	P-7	51.54	J-6	Lot 5	8.0	100.0	-52.92	0.34	0.01	False	0.00
42	P-8	238.78	Lot 5	Lot 4	8.0	100.0	-55.44	0.35	0.03	False	0.00
44	P-9	37.20	Lot 4	J-9	8.0	100.0	-57.96	0.37	0.01	False	0.00
46	P-10	72.01	J-9	J-10	8.0	100.0	-57.96	0.37	0.01	False	0.00
48	P-11	98.48	J-10	Lot 3	8.0	100.0	-57.96	0.37	0.02	False	0.00
50	P-12	444.04	Lot 3	J-12	8.0	100.0	-60.67	0.39	0.07	False	0.00
52	P-13	14.27	J-12	Lot 2	8.0	100.0	-60.67	0.39	0.00	False	0.00
54	P-14	261.41	Lot 2	J-14	8.0	100.0	-63.24	0.40	0.05	False	0.00
57	P-16	42.85	J-14	J-15	12.0	100.0	339.97	0.96	0.02	False	0.00
59	P-17	147.54	J-15	KS	12.0	100.0	339.97	0.96	0.08	False	0.00
62	P-19	155.89	J-17	J-1	12.0	100.0	336.49	0.95	0.09	False	0.00
67	P-22	72.24	J-19	J-14	12.0	100.0	403.21	1.14	0.06	False	0.00
76	P-25	11.78	KS	J-20	12.0	100.0	336.49	0.95	0.01	False	0.00
77	P-26	174.16	J-20	J-17	12.0	100.0	336.49	0.95	0.10	False	0.00
79	P-27	154.43	J-1	J-21	8.0	100.0	-49.50	0.32	0.02	False	0.00
80	P-28	220.07	J-21	J-2	8.0	100.0	-49.50	0.32	0.02	False	0.00

MAX HOUR

FlexTable: Junction Table (KSS129.wtg)

Current Time: 0.000 hours

ID	Label	Elevation (ft)	Demand Collection	Demand (gpm)	Hydraulic Grade (ft)	Pressure (psi)
27	J-1	5,277.56	<Collection: 0 items>	0.00	5,462.58	80.1
29	J-2	5,277.00	<Collection: 1 items>	0.00	5,462.62	80.3
31	J-3	5,280.21	<Collection: 0 items>	0.00	5,462.65	78.9
33	J-4	5,280.09	<Collection: 0 items>	0.00	5,462.65	79.0
37	J-6	5,279.56	<Collection: 0 items>	0.00	5,462.67	79.2
43	J-9	5,279.04	<Collection: 0 items>	0.00	5,462.72	79.5
45	J-10	5,277.60	<Collection: 0 items>	0.00	5,462.73	80.1
49	J-12	5,272.09	<Collection: 0 items>	0.00	5,462.82	82.5
53	J-14	5,270.25	<Collection: 0 items>	0.00	5,462.88	83.3
56	J-15	5,271.19	<Collection: 1 items>	0.00	5,462.85	82.9
60	J-17	5,275.55	<Collection: 1 items>	0.00	5,462.67	81.0
63	J-18	5,281.21	<Collection: 0 items>	0.00	5,462.53	78.4
64	J-19	5,268.85	<Collection: 0 items>	0.00	5,462.94	84.0
75	J-20	5,273.15	<Collection: 1 items>	0.00	5,462.77	82.0
78	J-21	5,277.50	<Collection: 1 items>	0.00	5,462.60	80.1
58	KS	5,273.15	<Collection: 1 items>	5.22	5,462.77	82.0
51	Lot 2	5,271.89	<Collection: 1 items>	3.85	5,462.83	82.6
47	Lot 3	5,275.57	<Collection: 1 items>	4.06	5,462.75	81.0
41	Lot 4	5,279.23	<Collection: 1 items>	3.78	5,462.71	79.4
39	Lot 5	5,279.82	<Collection: 1 items>	3.78	5,462.68	79.1
35	Lot 6	5,279.30	<Collection: 1 items>	5.13	5,462.66	79.3

MAX HOUR

FlexTable: Pipe Table (KSS129.wtg)

Current Time: 0.000 hours

ID	Label	Length (Scaled) (ft)	Start Node	Stop Node	Diameter (in)	Hazen-Williams C	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	Has User Defined Length?	Length (User Defined) (ft)
69	OS-1	210.32	J-18	R-1	12.0	100.0	785.44	2.23	4.09	True	1,553.00
65	OS-2	530.95	J-18	J-19	12.0	100.0	-402.25	1.14	0.40	False	0.00
66	OS-3	294.44	J-19	R-2	12.0	100.0	-811.26	2.30	0.82	False	0.00
70	P-1	76.02	J-18	J-1	12.0	100.0	-383.19	1.09	0.05	False	0.00
32	P-3	280.54	J-2	J-3	8.0	100.0	-46.42	0.30	0.03	False	0.00
34	P-4	20.92	J-3	J-4	8.0	100.0	-46.42	0.30	0.00	False	0.00
36	P-5	91.65	J-4	Lot 6	8.0	100.0	-46.42	0.30	0.01	False	0.00
38	P-6	96.13	Lot 6	J-6	8.0	100.0	-51.55	0.33	0.01	False	0.00
40	P-7	51.54	J-6	Lot 5	8.0	100.0	-51.55	0.33	0.01	False	0.00
42	P-8	238.78	Lot 5	Lot 4	8.0	100.0	-55.33	0.35	0.03	False	0.00
44	P-9	37.20	Lot 4	J-9	8.0	100.0	-59.11	0.38	0.01	False	0.00
46	P-10	72.01	J-9	J-10	8.0	100.0	-59.11	0.38	0.01	False	0.00
48	P-11	98.48	J-10	Lot 3	8.0	100.0	-59.11	0.38	0.02	False	0.00
50	P-12	444.04	Lot 3	J-12	8.0	100.0	-63.17	0.40	0.08	False	0.00
52	P-13	14.27	J-12	Lot 2	8.0	100.0	-63.17	0.40	0.00	False	0.00
54	P-14	261.41	Lot 2	J-14	8.0	100.0	-67.02	0.43	0.05	False	0.00
57	P-16	42.85	J-14	J-15	12.0	100.0	342.00	0.97	0.02	False	0.00
59	P-17	147.54	J-15	KS	12.0	100.0	342.00	0.97	0.08	False	0.00
62	P-19	155.89	J-17	J-1	12.0	100.0	336.78	0.96	0.09	False	0.00
67	P-22	72.24	J-19	J-14	12.0	100.0	409.01	1.16	0.06	False	0.00
76	P-25	11.78	KS	J-20	12.0	100.0	336.78	0.96	0.01	False	0.00
77	P-26	174.16	J-20	J-17	12.0	100.0	336.78	0.96	0.10	False	0.00
79	P-27	154.43	J-1	J-21	8.0	100.0	-46.42	0.30	0.02	False	0.00
80	P-28	220.07	J-21	J-2	8.0	100.0	-46.42	0.30	0.02	False	0.00

MAX HR + FIRE FLOW

FlexTable: Junction Table (KSS129.wtg)

Current Time: 0.000 hours

ID	Label	Elevation (ft)	Demand Collection	Demand (gpm)	Hydraulic Grade (ft)	Pressure (psi)
27	J-1	5,277.56	<Collection: 0 items>	0.00	5,447.71	73.6
29	J-2	5,277.00	<Collection: 1 items>	1,000.00	5,431.93	67.0
31	J-3	5,280.21	<Collection: 0 items>	0.00	5,431.47	65.4
33	J-4	5,280.09	<Collection: 1 items>	0.00	5,431.44	65.5
37	J-6	5,279.56	<Collection: 1 items>	0.00	5,431.14	65.6
43	J-9	5,279.04	<Collection: 1 items>	1,000.00	5,430.65	65.6
45	J-10	5,277.60	<Collection: 1 items>	0.00	5,432.08	66.8
49	J-12	5,272.09	<Collection: 1 items>	0.00	5,442.93	73.9
53	J-14	5,270.25	<Collection: 0 items>	0.00	5,448.50	77.1
56	J-15	5,271.19	<Collection: 1 items>	1,000.00	5,447.87	76.4
60	J-17	5,275.55	<Collection: 1 items>	1,000.00	5,447.22	74.3
63	J-18	5,281.21	<Collection: 0 items>	0.00	5,448.93	72.6
64	J-19	5,268.85	<Collection: 0 items>	0.00	5,450.50	78.6
75	J-20	5,273.15	<Collection: 1 items>	850.00	5,447.23	75.3
78	J-21	5,277.50	<Collection: 1 items>	0.00	5,441.20	70.8
58	KS	5,273.15	<Collection: 1 items>	5.22	5,447.28	75.3
51	Lot 2	5,271.89	<Collection: 1 items>	3.85	5,443.22	74.1
47	Lot 3	5,275.57	<Collection: 1 items>	4.06	5,434.04	68.6
41	Lot 4	5,279.23	<Collection: 1 items>	3.78	5,430.71	65.5
39	Lot 5	5,279.82	<Collection: 1 items>	3.78	5,431.06	65.4
35	Lot 6	5,279.30	<Collection: 1 items>	5.13	5,431.29	65.8

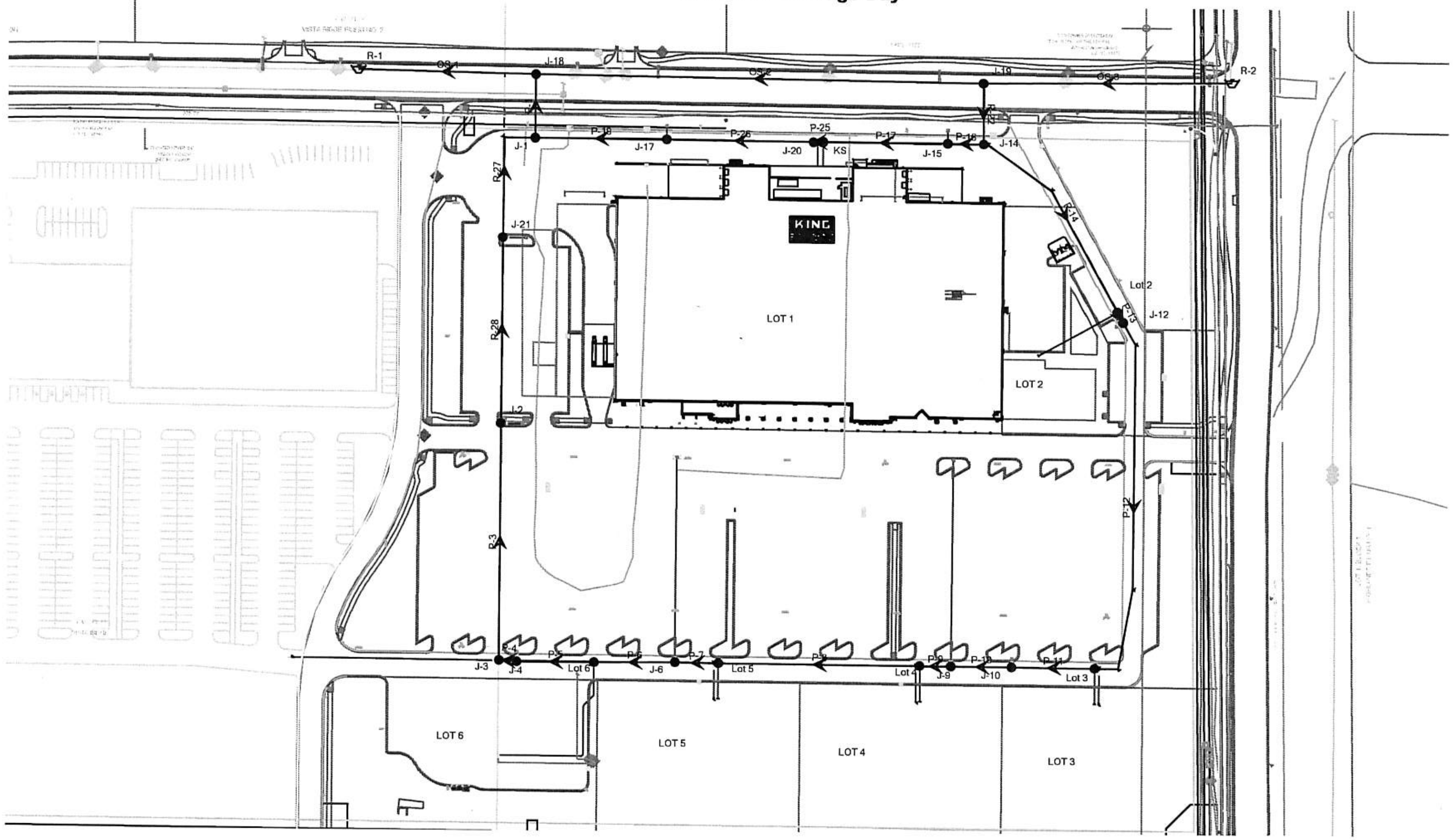
MAX HR + FIRE FLOW

FlexTable: Pipe Table (KSS129.wtg)

Current Time: 0.000 hours

ID	Label	Length (Scaled) (ft)	Start Node	Stop Node	Diameter (in)	Hazen-Williams C	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	Has User Defined Length?	Length (User Defined) (ft)
69	OS-1	210.32	J-18	R-1	12.0	100.0	-1,238.88	3.51	9.51	True	1,553.00
65	OS-2	530.95	J-18	J-19	12.0	100.0	-838.41	2.38	1.58	False	0.00
66	OS-3	294.44	J-19	R-2	12.0	100.0	-3,636.94	10.32	13.26	False	0.00
70	P-1	76.02	J-18	J-1	12.0	100.0	2,077.29	5.89	1.21	False	0.00
32	P-3	280.54	J-2	J-3	8.0	100.0	208.13	1.33	0.46	False	0.00
34	P-4	20.92	J-3	J-4	8.0	100.0	208.13	1.33	0.03	False	0.00
36	P-5	91.65	J-4	Lot 6	8.0	100.0	208.13	1.33	0.15	False	0.00
38	P-6	96.13	Lot 6	J-6	8.0	100.0	203.00	1.30	0.15	False	0.00
40	P-7	51.54	J-6	Lot 5	8.0	100.0	203.00	1.30	0.08	False	0.00
42	P-8	238.78	Lot 5	Lot 4	8.0	100.0	199.22	1.27	0.36	False	0.00
44	P-9	37.20	Lot 4	J-9	8.0	100.0	195.44	1.25	0.05	False	0.00
46	P-10	72.01	J-9	J-10	8.0	100.0	-804.56	5.14	1.43	False	0.00
48	P-11	98.48	J-10	Lot 3	8.0	100.0	-804.56	5.14	1.96	False	0.00
50	P-12	444.04	Lot 3	J-12	8.0	100.0	-808.62	5.16	8.90	False	0.00
52	P-13	14.27	J-12	Lot 2	8.0	100.0	-808.62	5.16	0.29	False	0.00
54	P-14	261.41	Lot 2	J-14	8.0	100.0	-812.47	5.19	5.28	False	0.00
57	P-16	42.85	J-14	J-15	12.0	100.0	1,986.07	5.63	0.63	False	0.00
59	P-17	147.54	J-15	KS	12.0	100.0	986.07	2.80	0.59	False	0.00
62	P-19	155.89	J-17	J-1	12.0	100.0	-869.15	2.47	0.50	False	0.00
67	P-22	72.24	J-19	J-14	12.0	100.0	2,798.53	7.94	2.00	False	0.00
76	P-25	11.78	KS	J-20	12.0	100.0	980.85	2.78	0.05	False	0.00
77	P-26	174.16	J-20	J-17	12.0	100.0	130.85	0.37	0.02	False	0.00
79	P-27	154.43	J-1	J-21	8.0	100.0	1,208.13	7.71	6.51	False	0.00
80	P-28	220.07	J-21	J-2	8.0	100.0	1,208.13	7.71	9.28	False	0.00

Scenario: Average Day

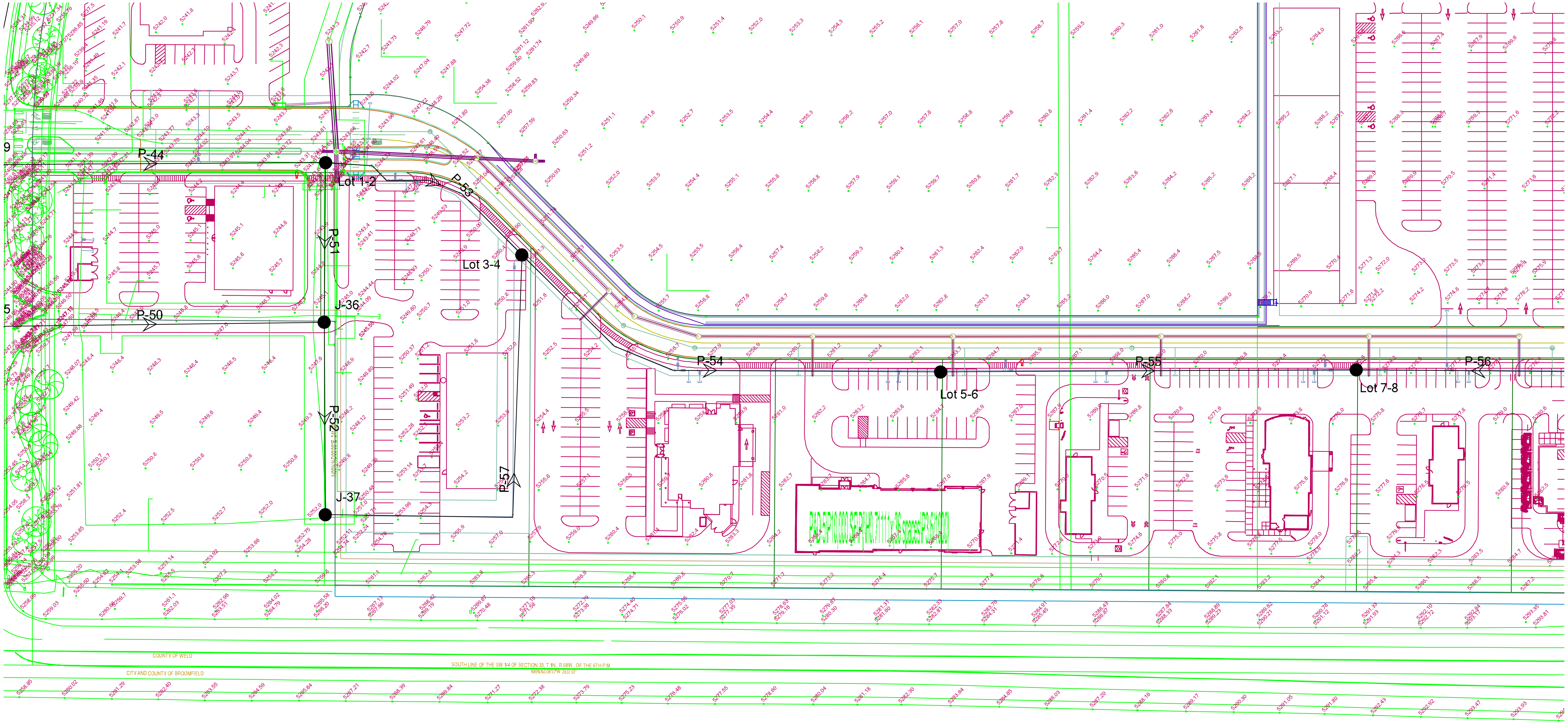


KSS129.wtg
5/19/2015

Appendix D

Maps/Plans

Scenario: Max Hr+FF



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GEOTECHNICAL ENGINEERING STUDY
PROPOSED COMMERCIAL DEVELOPMENT
NORTHEAST CORNER OF STATE HIGHWAY 7
AND MOUNTAIN VISTA DRIVE
ERIE, COLORADO

Prepared By:

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Project No. 15-3-164

October 12, 2015

TABLE OF CONTENTS

SUMMARY

1. The subsurface conditions encountered at the site were evaluated by drilling 31 exploratory borings to depths ranging from about 10 to 30 feet below existing ground surface. The borings generally encountered a variable thickness of topsoil overlying man-placed fills and natural overburden soils underlain by claystone and sandstone bedrock. Existing fill was encountered in four of the borings and extended to depths estimated to range from about 5 to 8 feet.

   Natural overburden lean to fat clay was encountered near the ground surface or beneath the fill in the exploratory borings and ranged in thickness from nil to about 16 feet. The natural soils encountered in the borings generally were light brown to brown, and moist. Occasional calcareous zones were noted within the overburden soils.

   Bedrock was encountered in all of the borings at depths ranging from a few inches to about 18 feet below ground surface. The bedrock encountered in the borings consisted generally of claystone bedrock with occasional zones of interbedded sandstone and claystone. The claystone was moist, and brown to gray. Based on sampler penetration resistance, the bedrock was medium hard to very hard. The interbedded sandstone and claystone bedrock was fine to medium grained, firm to very hard, and light brown to brown to gray. The sandstone also had nil to weak cementation. The bedrock surface elevations ranged from about 5224 to 5283 feet.

   Groundwater was encountered in two of the borings during drilling at depths ranging from about 8 to 18 feet. The borings were left open in order to measure stabilized groundwater levels, where present. Follow-up groundwater level measurements made 14 days after drilling did not encountered groundwater.

2. The project site has highly varied subsurface conditions. Considering the magnitude of the planned mass grading efforts, we recommend shallow spread footing foundations placed on properly compacted structural fill material be used to support the buildings at the site. Site grading should be planned accordingly, as discussed in more detail in the “Site Grading” section of the report.

   Swelling soils require overexcavation and replacement to create a pad of suitable bearing material for shallow foundations. Spread footings bearing directly on a minimum of 10 feet of properly compacted structural fill should be designed for an allowable soil bearing pressure of 3,000 psf

3. Slab-on-grade construction will be acceptable across the site. Overexcavation and replacement will be necessary due to high to very high swell potential.

4. Drilled shaft and structural floor options are viable on the project site. Discussion of drilled shafts and structurally supported floor systems along with design criteria are presented herein.

5. We recommend that all pavement sections be underlain by at least 3 feet of properly compacted fill material. The following table presents the minimum pavement thickness recommendations for this facility and roadways.

Paved Area	**Full Depth Asphalt (inches)**	**Composite Section Asphalt/ABC (inches)**	**PCCP (inches)**
Light Duty	6.5	3.5 / 8.0	6.0
Heavy Duty	7.5	5.0 /10.0	7.0

ABC – Aggregate Base Course
PCCP – Portland Cement Concrete Pavement

PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical engineering study for the proposed commercial development being planned for the northeast corner of State Highway 7 (SH 7) and Mountain Vista Drive in Erie, Colorado. The project site is generally shown on Fig. 1. The study was conducted in accordance with the scope of work in our Proposal No. P3-15-207 dated August 21, 2015 and revised August 26, 2015.

A field exploration program consisting of exploratory borings was conducted to obtain information on subsurface conditions. Samples of soils and bedrock obtained during the field exploration were tested in the laboratory to determine their strength, compressibility or swell characteristics, and classification. Results of the field exploration and laboratory testing were analyzed to develop recommendations for the building foundations and floor slabs, exterior flatwork areas, and pavements. The results of the field exploration and laboratory testing are presented herein.

This report has been prepared to summarize the data obtained during this study and to present our conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to construction of the proposed facility are included in the report.

PROPOSED CONSTRUCTION

We have not been provided with the exact layout and type of structures to be constructed on the site; however, we anticipate that the structures will likely consist of single story retail and/or commercial storefronts. Paved surfaces and minor landscaping will likely be provided in and around the site.

Initial site grading plans indicate that the proposed ground surfaces will require minor cuts across the site as deep as about 2 to 3 feet and major fills as deep as about 12 to 15 feet.

If the proposed construction varies significantly from that generally described above or depicted in this report, we should be notified to reevaluate the conclusions and recommendations provided herein.

SITE CONDITIONS

At the time of drilling, the site was being used as an actively farmed agricultural field. The project site lies between SH 7 on the south and Ridgeview Drive on the north. Mountain View Drive lies on the western property boundary. The project site extends approximately 1,700 feet east of Mountain View Drive. This project includes portions of the property within the limits described above.

The site slopes gently down to the west and north. Maximum difference of elevation across the subject site of 50 to 60 feet.

SUBSURFACE CONDITIONS

As requested by the Kentro Group, the subsurface conditions encountered at the site were evaluated by drilling a total of 31 exploratory borings to depths ranging from about 10 to 30 feet below existing ground surface. Specifically, eight borings were located within areas of proposed structures, seven borings within proposed drive lanes and parking areas, and sixteen borings in future pad sites. The approximate locations of the borings are shown on Fig. 1. The logs of the exploratory borings are presented on Figs. 2 through 5, and a legend and associated explanatory notes are also presented on Fig. 6.

Subsurface Soil and Bedrock Conditions: The borings generally encountered a variable thickness of topsoil overlying man-placed fills and natural overburden soils underlain by claystone and sandstone bedrock. Existing fill was encountered in four of the borings and extended to depths estimated to range from about 5 to 8 feet. The fill generally consisted of lean clay with occasional fine to medium grained sand lenses. The fill was generally moist and light brown to brown. The lateral extent, depth and degree of compaction of the existing fill were not determined as part of this study.

Natural overburden lean to fat clay was encountered near the ground surface or beneath the fill in the exploratory borings and ranged in thickness from nil to about 16 feet. The natural soils encountered in the borings generally were light brown to brown, and moist. Occasional calcareous zones were noted within the overburden soils. Based on sampler penetration resistance, the natural overburden soils were generally very stiff to hard.

Bedrock was encountered in all of the borings at depths ranging from a few inches to about 18 feet below ground surface. The bedrock encountered in the borings consisted generally of claystone bedrock with occasional zones of interbedded sandstone and claystone. The claystone was moist, and brown to gray. Based on sampler penetration resistance, the bedrock was medium hard to very hard. The interbedded sandstone and claystone bedrock was fine to medium grained, firm to very hard, and light brown to brown to gray. The sandstone also had nil to weak cementation. The bedrock surface elevations ranged from about 5224 to 5283 feet.

Groundwater Conditions: Groundwater was encountered in two of the borings during drilling at depths ranging from about 8 to 18 feet. The borings were left open in order to measure stabilized groundwater levels, where present. Follow-up groundwater level measurements made 14 days after drilling did not encountered groundwater. Development of perched groundwater on top of or within the fractured zones of the bedrock will occur, particularly after wet weather and landscape irrigation subsequent to development.

LABORATORY TESTING

Laboratory testing was performed on selected soil and bedrock samples obtained from the borings to determine in-situ soil moisture content and dry density, Atterberg limits, swell-consolidation characteristics, gradation, and concentration of water soluble sulfates. The results of the laboratory tests are shown to the right of the logs on Figs. 2 through 5 and summarized in Table 1. The results of specific tests are graphically plotted on Figs. 7 through 20. The testing was conducted in general accordance with recognized test procedures, primarily those of the American Society for Testing of Materials (ASTM).

Swell-Consolidation: Swell-consolidation tests were conducted on samples of the existing fill, the natural lean clay, and the claystone bedrock. The swell-consolidation tests were performed in order to determine the compressibility and swell characteristics of the samples under loading and when submerged in water. Each sample was prepared and placed in a confining ring between porous discs, subjected to a surcharge pressure of 200 or 1,000 psf, and allowed to consolidate before being submerged. The sample height was monitored until deformation practically ceased under each load increment.

Results of the swell-consolidation tests are plotted as a curve of the final strain at each increment of pressure against the log of the pressure, and are presented on Figs. 7 through 20. Based on the results of swell-consolidation tests, the fill and natural soil samples exhibited low to very high swell potential upon wetting at surcharge pressures of both 200 and 1,000 psf. The bedrock samples generally exhibited low to very high swell potential upon wetting under the surcharge pressures of both 200 and 1,000 psf. One sample of claystone bedrock indicated low to moderate consolidation potential; however, we believe that this does not reflect the on-site bedrock materials. It is highly probable that the apparent consolidation is likely the result of sample disturbance prior to or during the testing procedure.

<u>Index Properties</u>: Samples were classified into categories of similar engineering properties in general accordance with the Unified Soil Classification System. This system is based on index properties, including liquid limit and plasticity index and grain size distribution. Values for moisture content, dry density, liquid limit and plasticity index, and the percent of soil passing the U.S. No. 4 and 200 sieves are presented in Table I and adjacent to the corresponding sample on the boring logs.

GEOTECHNICAL CONSIDERATIONS

As previously discussed, site subsurface conditions generally consist of variable depths of fill and natural overburden soils underlain by claystone and sandstone bedrock. The existing fills are considered non-engineered and unsuitable in their current state for support of foundations and slab-on-grade. The existing soils, including the fills, and the underlying bedrock exhibited a tendency to swell. Our experience in the area also indicates that very high swell potential soils and bedrock are prominent.

With proper site preparation, shallow spread footing foundations and slab-on-grade construction should be feasible. Proper site preparation should include complete removal of existing non-engineered fills where present within the proposed building footprint and beneath other structures, down to the natural soils or bedrock and replacement with compacted structural fill. Although complete removal would be preferable beneath pavement areas, partial removal of existing fills below planned pavement subgrade may be considered with the understanding that unwanted pavement settlement and associated distress could occur over time if the deeper fills left in place were not properly compacted.

Site preparation should also include providing a layer of compacted structural fill below spread footing foundations and slab on grade floors. Including a minimum thickness of structural fill would result in more uniform bearing conditions beneath footings and floor slab support, and a more predictable foundation settlement. Depending on site finished grades, overexcavation into natural soils and/or bedrock will be required. In general, a minimum of 10 feet of properly compacted structural fill should be provided below spread footings and 13 feet of properly compacted structural fill should be provided below floor slabs.

Excavated existing fills, natural overburdens soils, and sandstone bedrock, should suitable for use as site grading fill and may be suitable for use as structural fill beneath buildings and other structures provided they can be properly moisture conditioned and compacted. Claystone may be suitable for use as site grading fill but should not be used for structural fill beneath buildings or other structures, and should not be used as compacted fill within three feet of the subgrade elevation in pavement or exterior flatwork areas.

A deep foundation alternative is feasible (and possibly desirable) for the structures on the site. The primary reasoning for selecting a deep foundation alternative would be the elimination of risks associated with heaving movements of the floor slabs or where small structures will be constructed that would not be included in a larger site-wide earthwork scheme to provide the required thickness of structural fill below structures. Creating a deep excavation for relatively small structures may be more costly than using different construction techniques. If structures are founded on deep foundations, we recommend that the floor slabs be structurally supported by foundation elements and isolated from the underlying soils to prevent heaving movements. We recommend that a minimum 12-inch void be provided beneath the structurally supported floor slabs.

SITE GRADING

In general, the currently proposed site grades will be raised from the current elevations over most of the site. Given the proposed site grades, it appears that significant amounts of imported fill materials will be required.

<u>Cut and Fill Slopes</u>: The site specifically and the area in general is gently to moderately sloping. Major stability problems are not anticipated if site grading is carefully planned and cut and fill slopes do not exceed approximately 15 feet in height.

Permanent unretained cuts in the overburden soils less than 10 feet in height should be sloped at 3 horizontal to 1 vertical, or flatter. Permanent unretained cuts in the hard to very hard bedrock should be sloped at 2 horizontal to 1 vertical, or flatter. The risk of slope instability will be significantly increased if seepage is encountered in cuts. For shallow cuts in the existing overburden soils, we do not anticipate seepage will be encountered. However, cuts extending into bedrock may encounter seepage from water perched at the interface zone between the overburden soils and bedrock. Where groundwater seepage is anticipated or encountered during construction, a stability analysis should be conducted to determine if the seepage will adversely affect the cut.

Permanent fills up to 20 feet in height can be used if the fill slopes do not exceed 3 horizontal to 1 vertical and the fills are properly compacted and drained. The ground surface underlying all fills should be carefully prepared by removing all organic matter, scarification to a depth of 12 inches and compacting to 95% of the standard Proctor (ASTM D698) maximum dry density at moisture contents near optimum. Fills should be benched into existing slopes that exceeding 4 horizontal to 1 vertical.

Good surface drainage should be provided around all permanent cuts and fills to direct surface runoff away from the slope faces. Fill slopes, cut slopes and other stripped areas should be protected against erosion by vegetation or other methods.

No formal stability analyses were performed to evaluate the slopes recommended above. Published literature and our experience with similar cuts and fills indicate the recommended slopes should have adequate factors of safety. If a detailed stability analysis is required, we should be notified.

Temporary Excavations: We assume that the site excavations will be constructed by generally over-excavating the side slopes to a stable configuration where enough space is available. All excavations greater than 4 feet and less than 20 feet in depth should be constructed in accordance with OSHA requirements, as well as state, local and other applicable requirements. OSHA requires excavations or trenching over 20 feet deep be designed by a registered professional engineer.

The existing fills generally classify as OSHA Type C soils and the natural clayey soils generally classify as OSHA Type B. The bedrock underlying the site is anticipated to classify as OSHA Type A soil, although fractured bedrock and non- to weakly-cemented sandstone bedrock would classify as OSHA Type B soils and may classify as Type C soils depending on the degree of fracturing and/or cementation. If unstable soil conditions or groundwater are encountered, the geotechnical engineer should be notified so that additional recommendations can be provided, if necessary.

Excavated slopes may soften or loosen due to construction traffic and erode from surface runoff. Measures to keep surface runoff from excavation slopes, including diversion berms, should be considered.

Existing Fill: The current level of compaction and moisture content of the existing fill materials appears to be erratic. The existing fills should not be considered suitable for support of any structure or flatwork and should be completely removed and recompacted as necessary.

Material Specifications: The following material specifications are presented for fills on the project site. A geotechnical engineer should evaluate the suitability of all proposed import fill material, if required, for the project prior to placement.

1. *Structural Fill Beneath Buildings*: Fill placed beneath building structures should consist of 6 feet of imported Select Fill below the proposed subgrade elevation is preferred. Fill materials used below the structural fill should consist of Common Fill. (See Material Suitability below for Common and Select Fill description and criteria).

2. *Pavement Subgrade*: Materials placed within 3 feet of the pavement subgrade elevation may consist of the on-site soils exclusive of claystone (Common Fill). Excavated, on-site claystone bedrock material may be used in deeper fills, outside of building limits, at depths greater than 3 feet below the proposed pavement subgrade elevation. Claystone bedrock, as well as other on-site materials not suitable for use as structural fill, may be used in fill areas outside of building footprints and pavement subgrades.

3. *Pipe Bedding Material*: Pipe bedding material should be a free draining, coarse grained sand and/or fine gravel. The on-site soils are generally non to very cohesive, fine grained soils and are susceptible to erosion and scour.

4. *Aggregate Base Course*: Material should be crushed stone, crushed slag, recycled concrete, crushed gravel or natural gravel which conforms to CDOT Specifications for Class 6 or Class 5 criteria for aggregate base course.

5. *Utility Trench Backfill*: Material excavated from the utility trenches may be used for backfill provided it does not contain unsuitable material or particles larger than 4 inches.

6. *Material Suitability*: The upper 13 feet of material placed in the building pad overexcavation zone should consist of 6 feet of Select Fill overlying a minimum of 7 feet of Common Fill consisting of any of the overburden soils on site exclusive of claystone bedrock, although sandstone bedrock may be included. A second class of fill to be used for grading will be Claystone Fill. Claystone Fill may be used outside of building areas, either in landscaped areas or more than 3 feet below pavement subgrade elevation.

Claystone encountered in cut areas should be overexcavated 2 feet below the pavement subgrade elevation, the base of subexcavated area should be scarified 12 inches, moisture conditioned and recompacted, and then the upper 2 feet replaced with Common Fill, resulting a total of 3 feet of moisture conditioned material.

Imported Select Fill should contain less than 40% passing the No. 200 sieve, have a maximum liquid limit of 35 and a maximum plasticity index of 10. Also, the swell potential when remolded to 100% of the ASTM D 698 standard Proctor maximum dry density at optimum moisture content should be less than ½% under a 1,000 pcf surcharge pressure.

Imported Common Fill should contain 20% to 70% passing the No. 200 sieve, have a maximum liquid limit of 40 and a maximum plasticity index of 15, and a maximum swell potential of 1% when remolded as described above.

All fill material should be free of vegetation, brush, sod and other deleterious substances and should not contain rocks, debris or lumps having a diameter of more than 4 inches. Rocks, debris or lumps should be dispersed throughout the fill and "nesting" of these materials should be avoided. The geotechnical engineer should evaluate the suitability of proposed import fill materials prior to placement.

<u>Placement and Compaction Specifications</u>: We recommend the following compaction criteria be used on the project:

1. *Moisture Content*: Compaction of all fill materials should be compacted as outlined below with moisture contents between the optimum moisture content and 3 percentage points above optimum moisture for clayey material and within -2 to +2 percentage points of optimum for granular soils.

   The contractor should be aware that the on-site and/or proposed imported fine-grained soils may become somewhat unstable and deform under wheel loads if placed near the upper end of the moisture range(s). Some fill instability is not a concern in deeper fills provided the required density is achieved; instability is a concern primarily in the upper 2 to 3 feet of pavement subgrade fill.

2. *Placement and Degree of Compaction*: Structural fill beneath foundations and slab-on-grade floors, fill adjacent to shallow spread footing foundations, and wall backfill should be placed in lifts no thicker than 8 inches loose.

The following compaction criteria should be followed during construction:

Fill Location	Percentage of Maximum Standard Proctor Density (ASTM D-698)
Beneath Spread Footing Foundations	98%
Beneath Building Floor Slabs	
Fill less Than 8 Feet below finished grade	95%
Fill more Than 8 Feet below finished grade	98%
Adjacent to Spread Footing Foundations	95%
Wall Backfill	
Backfill Less than 8 Feet below finished grade	95%
Settlement Sensitive Areas	98%[1]
Exterior Backfill More than 8 Feet below finished grade	98%[1]
Beneath Pavements and Settlement-Sensitive Hardscape Areas	
Fill Less Than 8 Feet below finished grade	95%
Fill More Than 8 Feet below finished grade	98%
Utility Trenches	
Interior	95%
Exterior – Backfill Less Than 15 Feet thick	95%
Exterior - Backfill More Than 15 Feet Thick	98%
Landscape and Other Areas	95%

[1] Some difficulty could be encountered achieving adequate compaction with small equipment to avoid exerting excessive compaction stresses on walls.

3. *Subgrade Preparation*: Areas receiving new fill should be prepared as recommended in specific sections of this report to provide a uniform base for fill placement. All other areas to receive new fill not specifically addressed herein should be scarified to a depth of at least 8 inches and recompacted to at least 95% of the standard Proctor (ASTM D 698) maximum dry density at moisture contents recommended above.

Construction Monitoring: A representative of the geotechnical engineer should observe and test fill placement. Structural fills beneath buildings and foundations should be observed and tested on a full-time basis. Full time observation and testing is a critical component to reducing the risk of post-construction settlement of the fills.

FOUNDATION RECOMMENDATIONS

Spread Footings: As discussed previously, we recommend that the commercial and retail buildings near the northeast side of the site (in the area of Borings 1 through 8) be founded on spread footings placed on properly compacted structural fill. It is likely that buildings constructed on the future pad sites (Borings F-1 through F-16) will be able to utilize similar recommendations; however, individual site specific geotechnical engineering studies should be performed to verify or develop geotechnical engineering related recommendations.

The design and construction criteria presented below should be observed for a spread footing foundation system. The construction details should be considered when preparing project documents.

1. Spread footings should be placed on a minimum of 10 feet of structural fill extending to undisturbed natural soils and/or bedrock. The structural fill zone should consist of at least 3 feet of imported Select Fill below the footings overlying a minimum of 7 feet of Common Fill as described in the “Site Grading” section of this report. Areas of loose or soft material or existing fill encountered within the foundation excavation should be removed replaced with structural fill meeting the material and placement requirements outlined in the “Site Grading” section of this report. New structural fill should extend down from the edges of the footings at a 1 horizontal to 1 vertical projection.

2. Footings supported on properly compacted structural fill as recommended herein should be designed for an allowable soil bearing pressure of 3,000 psf and a minimum dead load pressure of 1,000 psf. In order to satisfy the minimum dead load pressure and minimum footing width criteria recommended herein, it may be necessary to concentrate loads by using a grade beam and pad or similar foundation design. If this system is used, a void should be provided beneath the grade beams between pads.

3. Based on experience, we estimate total settlement for footings designed and constructed as discussed in this section will be approximately 1 inch or less. Differential settlements across the building are estimated to be approximately ¾ of the total settlement.

4. Spread footings should have a minimum footing width of 16 inches for continuous footings and of 24 inches for isolated pads.

5. Exterior footings and footings beneath unheated areas should be provided with adequate soil cover above their bearing elevation for frost protection. Placement of foundations at least 36 inches below the exterior grade is typically used in this area.

6. The lateral resistance of a spread footing placed on properly compacted structural fill material will be a combination of the sliding resistance of the footing on the foundation materials and passive earth pressure against the side of the footing. Resistance to sliding at the bottoms of the footings can be calculated based on a coefficient of friction of 0.3. Passive pressure against the sides of the footings can be calculated using an equivalent fluid unit weight of 175 pcf. The above values are working values.

   Compacted fill placed against the sides of the footings to resist lateral loads should meet the material and placement requirements outlined in the "Site Grading" section of this report.

7. Continuous foundation walls should be reinforced top and bottom to span an unsupported length of at least 10 feet.

8. A representative of the geotechnical engineer should observe all footing excavations prior to concrete placement.

Drill Shaft Foundations: If it is deemed by the design team to be more economical or desirable to eliminate the zone of subexcavation below building structures, the structures may be founded on straight shaft drilled piers. The values provided below are for building constructed within the areas of Borings 1 through 8 and may not be appropriate for structures elsewhere on the site. We should be contacted to re-evaluate the design criteria at specific locations on the site.

The design and construction criteria presented below should be observed for a straight-shaft pier foundation system. The construction details should be considered when preparing project documents.

1. Piers should be designed for an allowable end bearing pressure of 18,000 psf and a skin friction of 1,800 psf for the portion of the pier in bedrock. Uplift due to structural loadings on the piers can be resisted by using 75% of the allowable skin friction value plus an allowance for pier weight.

2. Piers should also be designed for a minimum dead load pressure of 30,000 psf calculated as the unfactored dead load applied to the pier cross sectional area. Our experience indicates application of dead load pressure is the most effective way to resist foundation movement due to swelling soils. However, if the minimum dead load

requirement cannot be achieved and the piers are loaded as heavily as practicable, the pier length should be extended beyond the minimum bedrock penetration and minimum length to mitigate the dead load deficit. This can be accomplished by assuming one-half of the skin friction given above acts in the direction to resist uplift caused by swelling soil around the upper portion of the pier. The owner should be aware of an increased potential for foundation movement if the recommended minimum dead load pressure is not met.

3. A minimum penetration of 8 feet into the bedrock and a minimum pier length of 25 feet are recommended.

4. Piers should be designed to resist lateral loads using a modulus of horizontal subgrade reaction in the clay soils of 50 tcf and a modulus of horizontal subgrade reaction of 250 tcf in the bedrock. The modulus values given are linear modulus values intended for use in simplified hand calculations and are for a long one-foot wide pier and must be corrected for pier size. If more rigorous analysis is desired, a computer application such as LPILE should be used.

5. The lateral capacity of the piers may be analyzed using the LPile computer program and the parameters provided in the following table. The strength criteria provided in the table are for use with that software application only and may not be appropriate for other usages.

Material	c (psf)	ø	γ_T	k_s	k_c	ε_{50}	Soil Model Type
Overburden soils / Properly Compacted Fill	750	0	120	500	200	0.007	1
Bedrock	8,000	0	125	2,000	800	0.004	1

c Cohesion intercept (pounds per square foot)
Φ Angle of internal friction (degrees)
γ_T Total unit weight (pounds per cubic foot)
k_s Initial static modulus of horizontal subgrade reaction (pounds per cubic inch)
k_c Initial cyclic modulus of horizontal subgrade reaction (pounds per cubic inch)
ε_{50} Strain at 50 percent of peak shear strength

Soil Types:
1. Stiff clay without free water (Reese)

6. Closely-spaced piers and pier groups will require appropriate reductions of the axial, uplift and lateral capacities based on the effective envelope of the pier group. These reductions can be avoided by spacing the piers at a distance of at least 3 pier diameters

center-to-center for axial loading, 6 pier diameters center-to-center in the direction parallel to lateral loading, and 5 pier diameters center-to-center in the direction perpendicular to lateral loading. More closely spaced piles should be studied on an individual basis to determine the appropriate reduction in axial and lateral load design parameters.

7. If the minimum pier spacings recommended above for lateral loading cannot be achieved, we recommend that the lateral load-displacement curve (p-y curve) for an isolated pier be modified for closely-spaced piers using p-multipliers to reduce all the p-values on the curve. With this approach, the computed load carrying capacity of the pier in a group is reduced relative to the isolated pile capacity. The modified p-y curve should then be reentered into the L-Pile software to calculate the pile deflection. The reduction in capacity for the leading pier, the pier leading the direction of movement of the group, is less than that for the trailing piers.

   For center-to-center spacing of piers in the group in the direction of loading expressed in multiples of the pier diameter, we recommend p-multipliers of 0.7 and 1.0 for pier spacings of 3 and 5 diameters, respectively, for the leading row of piers, 0.4 and 0.85 for pier spacings of 3 and 5 diameters, respectively, for the second row of piers, and 0.3 and 0.7 for pier spacings of 3 and 5 diameters, respectively, for rows 3 and higher. For loading in a direction perpendicular to the row of piers, the p-multipliers are 1.0 for a pier spacing of 5 diameters, 0.7 for a pier spacing of 3 diameters, and 0.5 for a pier spacing of 1 diameter. P-multiplier values for other pier spacing values should be determined by interpolation. These values are consistent with Table 10.7.2.4-1 of the 2012 AASHTO LRFD Bridge Design Specifications. It will be necessary to determine the load distribution between the piers that attain deflection compatibility because the leading pier carries a higher proportion of the group load and the pier cap prevents differential movement between the piers.

8. Piers should be reinforced their full length to resist an unfactored net tensile force from swelling soil pressure of at least 130 kips. The recommended tensile force is for a 1-foot diameter pier and should be increased in proportion to the pier diameter for larger piers. If the design dead load greater than or less than the recommended dead load, the requirement for tension reinforcement should be decreased or increased accordingly to account for the difference.

9. A 12-inch void should be provided beneath the grade beams to concentrate pier loadings and to separate the expansive soil from the grade beams. Absence of a void space will result in a reduction in dead load pressure on the piers which could result in upward movement of the foundation system. A void should also be provided beneath necessary pier caps.

10. The pier length-to-diameter ratio should not exceed 30 to facilitate proper cleaning and observation of the pier hole.

11 Concrete used in the piers should be a fluid mix with sufficient slump so it will fill the void between reinforcing steel and the pier hole. We recommend a concrete slump in the range of 5 to 8 inches be used.

12. Based on the results of our field exploration, laboratory testing, and our experience with similar, properly constructed drilled pier foundations, we estimate pier settlement will be low. Generally, we estimate the settlement of a pier 1 to 3 feet in diameter will be less than 1-inch when designed according to the criteria presented herein. The settlement of closely spaced piers will be larger and should be studied on an individual basis.

13. Pier holes should be properly cleaned prior to the placement of concrete.

14. The presence of water in some of the exploratory borings indicates the use of temporary casing or dewatering equipment in the pier holes may be required to reduce water infiltration. The requirements for casing and dewatering equipment can sometimes be reduced by placing concrete immediately upon cleaning and observing the pier hole. In no case should concrete be placed in more than 3 inches of water unless placed through an approved tremie method.

15. When water and/or drilling slurry is present outside the casing, care should be taken that concrete of sufficiently high slump is placed to a sufficiently high elevation inside the casing to prevent intrusion of the water and/or slurry into the concrete when the casing is withdrawn.

16. The drilled shaft contractor should mobilize equipment of sufficient size and operating condition to achieve the required bedrock penetration.

17. Care should be taken that the pier shafts are not oversized at the top. Mushroomed pier tops can reduce the effective dead load pressure on the piers.

18. Concrete should be placed in piers the same day they are drilled. The presence of water or caving soils may require that concrete be placed immediately after the pier hole is completed. Failure to place concrete the day of drilling will normally result in a requirement for additional bedrock penetration.

19. Difficulty may be encountered in establishing a casing seat in the sandstone to achieve a positive cutoff of groundwater seepage into the hole. Additional bedrock penetration may be required to compensate for the skin friction lost due to disturbance caused by installation of the casing. Skin friction should be neglected in the cased portion of the hole. The amount of additional penetration should be determined in the field at the time of construction. The contract documents should advise potential drilled shaft contractors of these subsurface conditions. In addition, careful consideration should be given to preparing bid items to avoid high costs for potential overruns.

20. A representative of the geotechnical engineer should observe pier drilling operations on a full-time basis to assist in identification of adequate bedrock strata and monitor pier construction procedures.

FLOOR SLABS

We recommend that at least 13 feet of structural fill be placed below slabs on grade. The planned mass site grading activities associated with footing foundation systems should result in at least 13 feet of structural fill placed below the floor slabs on grade. Prior to placing the floor slab, the subgrade should be thoroughly plowed and scarified to a depth of 12 inches, moisture conditioned and compacted as listed below.

General Floor Slab Recommendations: The following measures should be taken to reduce damage which could result from movement should the underslab materials be subjected to moisture changes.

1. Floor slabs should be separated from all bearing walls and columns with expansion joints which allow unrestrained vertical movement.

2. Interior non-bearing partitions resting on floor slabs should be provided with slip joints at the tops or bottoms so that, if the slabs move, the movement cannot be transmitted to the upper structure. This detail is also important for wallboards, stairways and door frames. Slip joints which will allow at least 2 inches of vertical movement are recommended.

   If wood or metal stud partition walls are used, the slip joints should preferably be placed at the bottoms of the walls so differential slab movement won't damage the partition wall. If slab bearing masonry block partitions are constructed, the slip joints will have to be placed at the tops of the walls. If slip joints are provided at the tops of walls and the floors move, it is likely the partition walls will show signs of distress, such as cracking. An alternative, if masonry block walls or other walls without slip joints at the bottoms are required, is to found them on spread footings and to construct the slabs independently of the foundation. If slab bearing partition walls are required, distress may be reduced by connecting the partition walls to the exterior walls using slip channels.

   Floor slabs should not extend beneath exterior doors or over foundation walls, unless saw cut at the wall after construction.

3. Floor slab control joints should be used to reduce damage due to shrinkage cracking. Joint spacing is dependent on slab thickness, concrete aggregate size, and slump, and should be consistent with recognized guidelines such as those of the Portland Cement Association (PCA) or American Concrete Institute (ACI). We suggest joints be provided on the order of 12 to 15 feet apart in both directions. The requirements for slab reinforcement should be established by the designer based on experience and the intended slab use.

4. If moisture-sensitive floor coverings will be used, mitigation of moisture penetration into the slabs, such as by use of a vapor barrier, may be required. If an impervious vapor barrier membrane is used, special precautions will be required to prevent differential curing problems which could cause the slabs to warp. ACI 302.1R addresses this topic.

5. New fill placed within 8 feet of the floor slab subgrade elevation should meet the criteria outlined in the Site Grading section of this report.

6. The bedrock encountered during this study will be expansive when placed in a compacted condition. Consequently, it should not be used as fill beneath floor slabs. The bedrock can be used for fill near the bottom of fills outside the building areas.

7. All plumbing lines should be tested before operation. Where plumbing lines enter through the floor, a positive bond break should be provided. Flexible connections should be provided for slab-bearing mechanical equipment.

Structurally Supported Floors: In the event the drilled piers and structurally supported floor slabs are desired, we recommend that design of a crawl space or underfloor void consider drainage and moisture control. We recommend a minimum 12-inch void beneath floors. Providing a full crawl space (3 feet or more) rather than a 12-inch void beneath the floor has the advantages that utilities can be suspended above the expansive subgrade and crawl space surface drainage can be provided. Utility lines should not be supported on the subgrade, unless adequate measures are taken to account for differential movement between grade supported utilities and slabs. If utilities are connected to the floor or floor openings, void spaces should also be provided below the utility lines. The utility lines should be supported by suitable means such as hangers as necessary. We recommend that void and crawl spaces be designed with positive surface drainage and a collection point or outlet so that free-water introduced into these spaces can be removed. High humidity can develop in crawl spaces due to the transmission of water vapor through moist soils. Crawl space humidity should be controlled through ventilation and/or the use of a vapor barrier on the crawl space floor or on the underside of the structure floor.

It is extremely important that exterior slabs-on-grade and pavements be isolated from the building foundations. Many expansive soil related problems are related to ineffective isolation between pavements/floor slabs and foundation-supported components of structures. Careful design detaining is necessary at locations such as exterior stairway landings and entry points.

Subgrade materials below pavement and exterior flat work adjacent to the building should be placed as described in the Pavement Design section.

FOUNDATION WALLS AND RETAINING STRUCTURES

Foundation walls and retaining structures associated with loading docks which are laterally supported and can be expected to undergo only a moderate amount of deflection should be designed for an at-rest lateral earth pressure computed on the basis of an equivalent fluid unit weight of 72 pcf for backfill consisting of the on-site fine grained soils and 60 pcf for backfill consisting of imported granular materials conforming to CDOT Class 1 Structure Backfill requirements.

Cantilevered retaining structures less than 8 feet in height which can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 50 pcf for backfill consisting of the on-site soils and 40 pcf for backfill consisting of imported granular materials conforming to CDOT Class 1 Structure Backfill.

All foundation and retaining structures should be designed for appropriate hydrostatic and surcharge pressures such as adjacent buildings, traffic, construction materials and equipment. The pressures recommended above assume drained conditions behind the walls and a horizontal backfill surface. The buildup of water behind a wall or an upward sloping backfill surface will increase the lateral pressure imposed on a foundation wall or retaining structure.

Compacted fill placed against the sides of the footings to resist lateral loads should be a granular material meeting the requirements for fill beneath buildings presented in the “Site Grading” section.

SEISMIC DESIGN CRITERIA

The soil profile generally will consist of about 15 to 30 feet of overburden soils underlain by firm to very hard bedrock. The bedrock is considered to extend to a depth of at least 100 feet below ground surface. The existing and anticipated overburden soils will classify as International Building Code (IBC) Site Class D. The underlying bedrock generally classifies as IBC Site Class B or C. The IBC limits the use of Site Class B to profiles where the overburden thickness between the base of the foundations and the rock surface is 10 feet or less. Based on the proposed depth of overburden, we recommend a design soil profile of IBC Site Class C. Based on the subsurface profile, site seismicity, and the anticipated depth of ground water, liquefaction is not a design consideration.

UNDERDRAIN SYSTEM

Our experience indicates that local perched water conditions can develop on relatively shallow bedrock and/or vary during times of heavy precipitation, seasonal runoff, or as a result of site improvements and irrigation. Depending on site grading, overexcavation and backfilling beneath buildings underlain by relatively shallow bedrock could create a potential for water to collect immediately below floor slabs.

To reduce the potential for groundwater to collect at the base of the structural fill zone, we recommend providing an underdrain system for buildings where bedrock will be within 3 feet of the base of the structural fill zone. The underdrain system should consist of a subdrain extending along the perimeter of the structural fill zone. Subdrain pipes should consist of 4-inch diameter, rigid, perforated or slotted, PVC plastic pipes. The pipes should be placed in trenches excavated at least 12 inches below the base of the structural fill zone and covered with drainage aggregate extending up to at least footing subgrade level. Drainage aggregate used in the subdrain systems should consist of a material with a gradation meeting the requirements for a No. 67 coarse aggregate in accordance with ASTM D448. Drain pipe trenches and drainage aggregate should be wrapped with a geotextile filter fabric to prevent migration of fines from the surrounding soil and/or bedrock into the drainage material.

The subdrain system should be sloped at a minimum slope of ½% to a sump or sumps where water can be removed by pumping or gravity drainage. Sumps should be provided with alarms and/or redundant pumps in the event the pumping equipment malfunctions.

WATER SOLUBLE SULFATES

Concentrations of water-soluble sulfates measured in samples of on-site soils ranged from non-detectable levels to 0.16%. These concentrations represent a Class 0 to Class 1 severity exposure to sulfate attack on concrete exposed to these materials. The degree of attack is based on a range of Class 0, Class 1, Class 2, and Class 3 severity exposure as presented in ACI 201 and in Section 601 of the 2011 Colorado Department of Transportation (CDOT) Standards and Specifications.

Based on the laboratory data and our experience with soils on this site and adjacent properties, we recommend all concrete exposed to the on-site materials meet the cement requirements for Class 2 exposure as presented in ACI 201. Alternatively, the concrete could meet the Colorado Department of Transportation’s (CDOT) cement requirements for Class 2 exposure as presented in Section 601.04 of the CDOT Standard Specifications for Road and Bridge Construction (2011).

SURFACE DRAINAGE

Proper surface drainage is very important for acceptable performance of site structures during construction and after the construction has been completed. Drainage recommendations provided by local, state and national entities should be followed based on the intended use of each structure. The following recommendations should be used as guidelines and changes should be made only after consultation with the geotechnical engineer.

1. Excessive wetting or drying of the foundation and slab subgrade(s) should be avoided during construction.

2. Exterior backfill meet the material and placement requirements outlined in the "Site Grading" section of this report.

3. Care should be taken when compacting around the foundation walls and underground structures to avoid damage to the structures. Hand compaction procedures, if necessary, should be used to prevent lateral pressures from exceeding the design values.

4. The ground surface surrounding the exterior of site structures should be sloped to drain away from the foundations in all directions. We recommend a minimum slope of 12 inches in the first 10 feet in unpaved areas. Site drainage beyond the 10-foot zone should be designed to promote runoff and reduce infiltration. A minimum slope of 3 inches in the first 10 feet is recommended in the paved areas. These slopes may be changed as required for handicap access points in accordance with the Americans with Disabilities Act.

5. The upper 2 feet of the backfill should be relatively impervious material compacted as recommended above to limit infiltration of surface runoff.

6. Ponding of water should not be allowed in backfill material or in a zone within 10 feet of the foundations, whichever is greater.

7. Roof downspouts and drains should discharge well beyond the limits of all backfill.

8. Landscaping which requires relatively heavy irrigation and lawn sprinkler heads should be located at least 10 feet from foundations. Irrigation schemes are available which

allow placement of lightly irrigated landscape near foundation walls in moisture sensitive soil areas. Drip irrigation heads with main lines located at least 10 feet from the foundation walls are acceptable provided irrigation quantities are limited.

9. Plastic membranes should not be used to cover the ground surface adjacent to foundation walls.

PAVEMENT DESIGN

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and traffic loadings. Soils are represented for pavement design purposes by means of a soil support value for flexible pavements and a modulus of subgrade reaction for rigid pavements. Both values are empirically related to strength.

Subgrade Materials: Based on the results of the field and laboratory studies, the majority of the subgrade materials at the site classify between A-4 and A-7-6 with group indices between 0 and 42 in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification. Soils classifying as A-4 would generally be considered to provide fair subgrade support, while soils classifying as A-6 and A-7-6 would generally be considered to provide poor subgrade support. The probable subgrade soils are expected to consist primarily of compacted fill composed of material generally classifying as A-6 and A-7-6 soils. For design purposes, a resilient modulus value of 3,025 psi was selected for flexible pavements and a modulus of subgrade reaction of 40 pci was selected for rigid pavements.

Design Traffic: Since anticipated traffic loading information was not available at the time of this report preparation, an 18-kip equivalent single axle loading (ESAL) value of 73,000 was assumed for the paved parking surfaces and an ESAL of 219,000 was assumed for truck routes. The values are selected based on our past experience for facilities of this nature. We believe that the ESAL values of 73,000 and 219,000 should be considered to classify as Light Duty and Heavy Duty pavement sections, respectively. The Light Duty pavement section should be constructed in locations restricted to automobile traffic only and the Heavy Duty pavement section should be constructed in locations of heavy vehicular traffic movements such as truck and tanker routes.

If estimated daily traffic volumes for the development are known to be different from those assumed, we should be provided with this information in order to reevaluate the pavement sections provided below.

Pavement Design: The following table presents the minimum pavement thickness recommendations for this development.

Paved Area	Full Depth Asphalt (inches)	Composite Section Asphalt/ABC (inches)	PCCP (inches)
Light Duty	6.5	3.5 / 8.0	6.0
Heavy Duty	7.5	5.0 /10.0	7.0

ABC – Aggregate Base Course
PCCP – Portland Cement Concrete Pavement

Truck loading dock areas and other areas where truck turning movements are concentrated should be paved with 7 inches of Portland cement concrete. The concrete pavement should contain sawed or formed joints to ¼ of the depth of the slab at a maximum distance of 12 feet on center. Concrete pavements may be a suitable alternative for parking lots, fuel center and delivery areas.

The asphalt binder selected for the proposed pavements should meet criteria for performance graded binders PG 58-28 that conform to requirements outlined in the CDOT Pavement Design Manual. The binder recommendations are based on the design 20-year 18-kip equivalent single axle load ($ESAL_{20}$) application values. The $ESAL_{20}$ values also indicate an N_{DESIGN} value for the gyratory method of compaction and design of 75.

Rigid Pavements: The above Portland cement concrete pavement thicknesses are presented as un-reinforced slabs. Based on projects with similar heavy vehicular loading, we recommend that dowels be provided at transverse joints within the slabs located in the travel lanes of heavily loaded vehicles and tie bars for the longitudinal joints. Additionally, curbs and/or pans should be tied to the slabs. The dowels and tie bars will help minimize the risk for differential movements between slabs to assist in more uniformly transferring axle loads to the subgrade. The Colorado Department of Transportation (CDOT) provides some guidance on dowel and tie bar placement in the current Standard Specifications for Road and Bridge Construction as well as in the current Standard Plans: M&S Standards. It is critical to the performance of the concrete pavement that the joints are properly sealed and maintained to minimize the infiltration of surface water, especially if dowels and tie bars are not installed.

All Portland cement concrete pavement (PCCP) should be based on a mix design established by a qualified engineer. In general, the design mix should consist of aggregate, Portland cement, water and additives that will meet the requirements contained in this section. The fine and coarse aggregate should conform to AASHTO M-6, M-43 and M-80. Cement should be Portland cement conforming to AASHTO M-85 or ASTM C-150 and all additives should be approved by a qualified engineer. Concrete used for drive lanes should meet the requirements established by CDOT for Class P concrete.

Subgrade Preparation: We recommend that areas of pavement be underlain by at least 3 feet of properly moisture conditioned compacted structural fill.

Pavement subgrade materials across significant portions of the site may consist of existing non-engineered fills. Ideally, all non-engineered fill beneath pavements should be removed and replaced with compacted fill consistent the material described in this report. However, a partial fill removal option may be considered, particularly in areas of relatively deep fill. If a partial removal option is selected, we recommend that areas of existing fill within proposed pavement areas be sub-excavated to a minimum depth of 3 feet below the proposed subgrade elevation.

The owner should be aware that partial subexcavation and replacement of existing fills and/or limited subexcavation of claystone bedrock or natural clay soils will reduce but not eliminate potential movement of pavements should moisture levels increase within these materials where present beneath the replacement fill and/or pavement. Also, the owner should be aware that rigid PCCP will be less tolerant of differential settlement- or heave-related movement than flexible pavements. Where rigid PCCP is constructed over existing fills, claystone or natural clay soils, providing reinforcing and doweling as discussed in the previous section of this report would help reduce the risk of pavement distress due to differential settlement- or heave-related movement.

Prior to placing new fill or the pavement section, the entire subgrade area should be scarified to a depth of 8 inches, adjusted to a moisture content near optimum and compacted to at least 95% of the standard Proctor (ASTM D 698) maximum dry density. Fill placed beneath the pavement should meet the material and compaction requirements for structural fill presented in the “Site Grading” section of this report.

The pavement subgrade should be proofrolled with a heavily loaded pneumatic-tired vehicle. Pavement design procedures assume a stable subgrade. Areas that deform excessively under heavy wheel loads are not considered stable and should be removed and replaced to achieve a stable subgrade prior to paving. The contractor should be aware that the clay soils, including on-site and imported materials, may become somewhat unstable and deform under wheel loads if placed near the upper end of the moisture range.

Drainage: The collection and diversion of surface drainage away from paved areas is extremely important to the satisfactory performance of pavement. Drainage design should provide for the removal of water from paved areas and prevent the wetting of the subgrade soils.

DESIGN AND CONSTRUCTION SUPPORT SERVICES

Kumar & Associates, Inc. should be retained to review the project plans and specifications for conformance with the recommendations provided in our report. We are also available to assist the design team in preparing specifications for geotechnical aspects of the project, and performing additional studies if necessary to accommodate possible changes in the proposed construction.

We recommend that Kumar & Associates, Inc. be retained to provide construction observation and testing services to document that the intent of this report and the requirements of the plans and specifications are being followed during construction. This will allow us to identify possible variations in subsurface conditions from those encountered during this study and to allow us to re-evaluate our recommendations, if needed. We will not be responsible for implementation of the recommendations presented in this report by others, if we are not retained to provide construction observation and testing services.

LIMITATIONS

This study has been conducted in accordance with generally accepted geotechnical engineering practices in this area for exclusive use by the client for design purposes. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings at the locations indicated on Fig. 1, and the proposed type of construction. This report may not reflect subsurface variations that occur between the exploratory borings, and the nature and extent of variations across the site may not become evident until site grading and excavations are performed. If during construction, fill, soil, rock or water conditions appear to be different from those described herein, Kumar & Associates, Inc. should be advised at once so that a re-evaluation of the recommendations presented in this report can be made. Kumar & Associates, Inc. is not responsible for liability associated with interpretation of subsurface data by others.

Swelling soils occur on this site. Such soils are stable at their natural moisture content but will undergo high volume changes with changes in moisture content. The extent and amount of perched water beneath the building site as a result of area irrigation and inadequate surface drainage is difficult, if not impossible, to foresee.

The recommendations presented in this report are based on current theories and experience of our engineers on the behavior of swelling soil in this area. The owner should be aware that there is a risk in constructing a building in an expansive soil area. Following the recommendations given by a geotechnical engineer, careful construction practice and prudent maintenance by the owner can, however, decrease the risk of foundation movement due to expansive soils.

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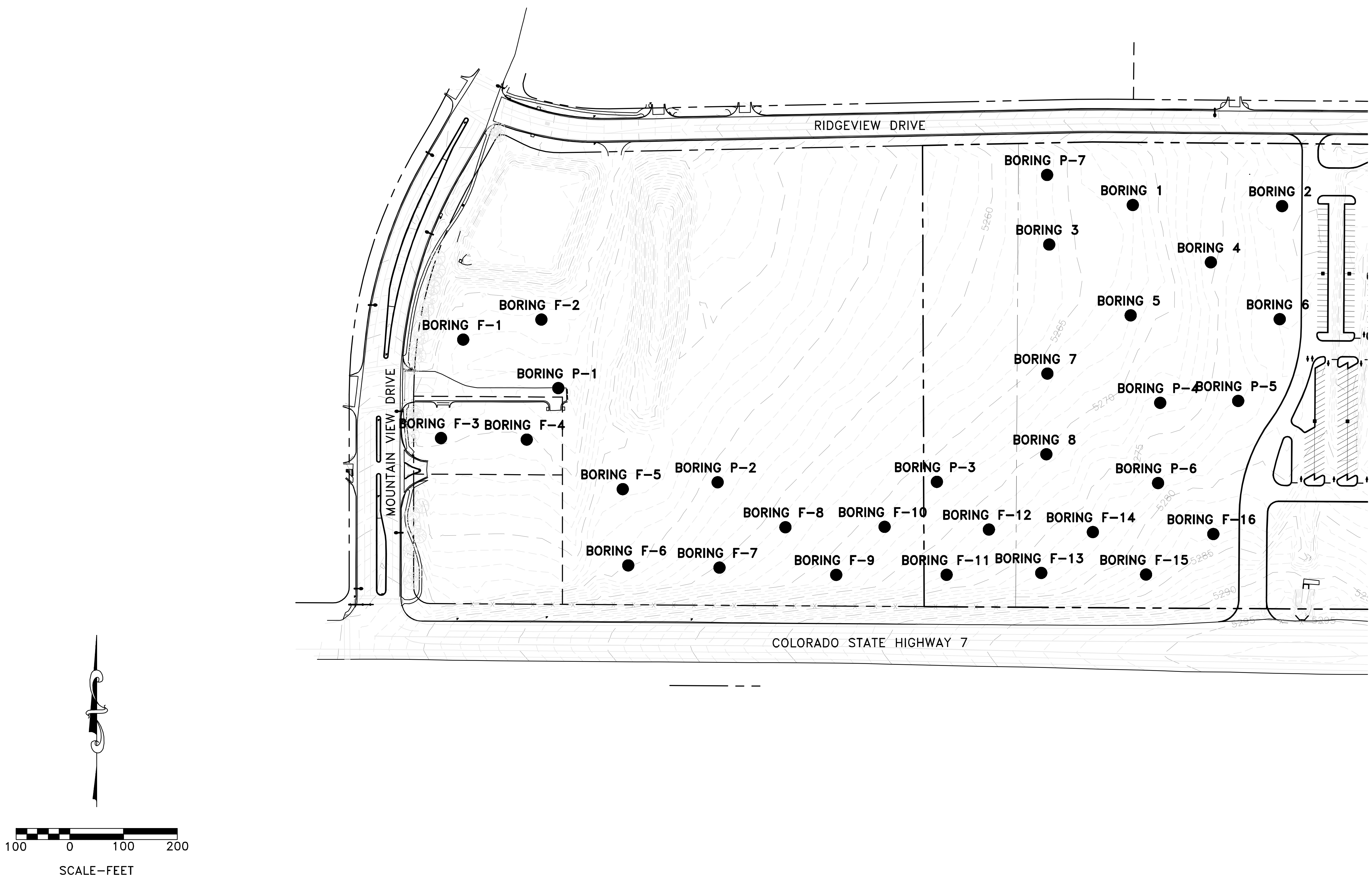

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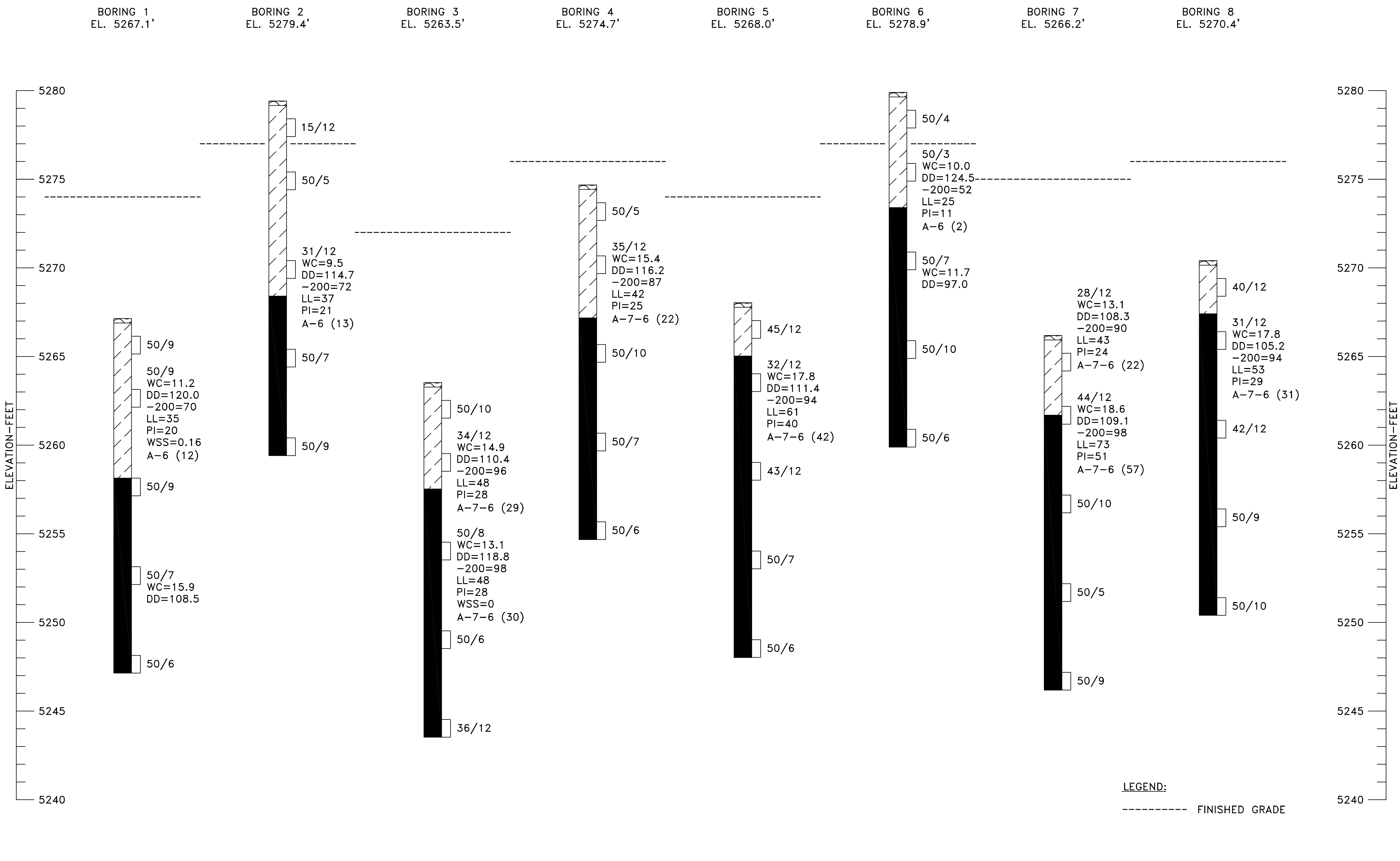
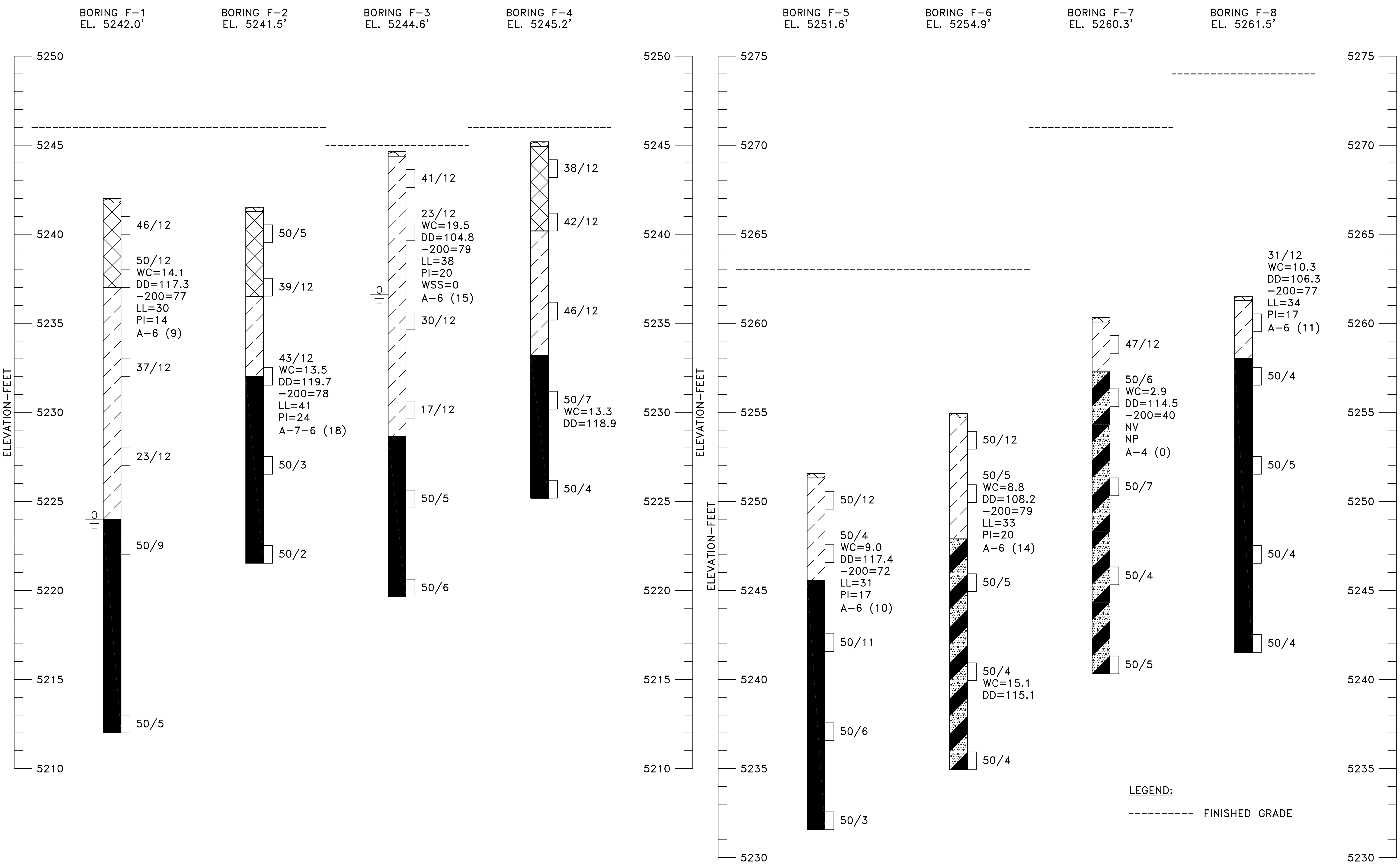
15-3-164	Kumar & Associates	PROPOSED ERIE COMMERCIAL DEVELOPMENT	LOCATIONS OF EXPLORATORY BORINGS	Fig. 1



15-3-164	Kumar & Associates	PROPOSED ERIE COMMERCIAL DEVELOPMENT	LOGS OF EXPLORATORY BORINGS	Fig. 2



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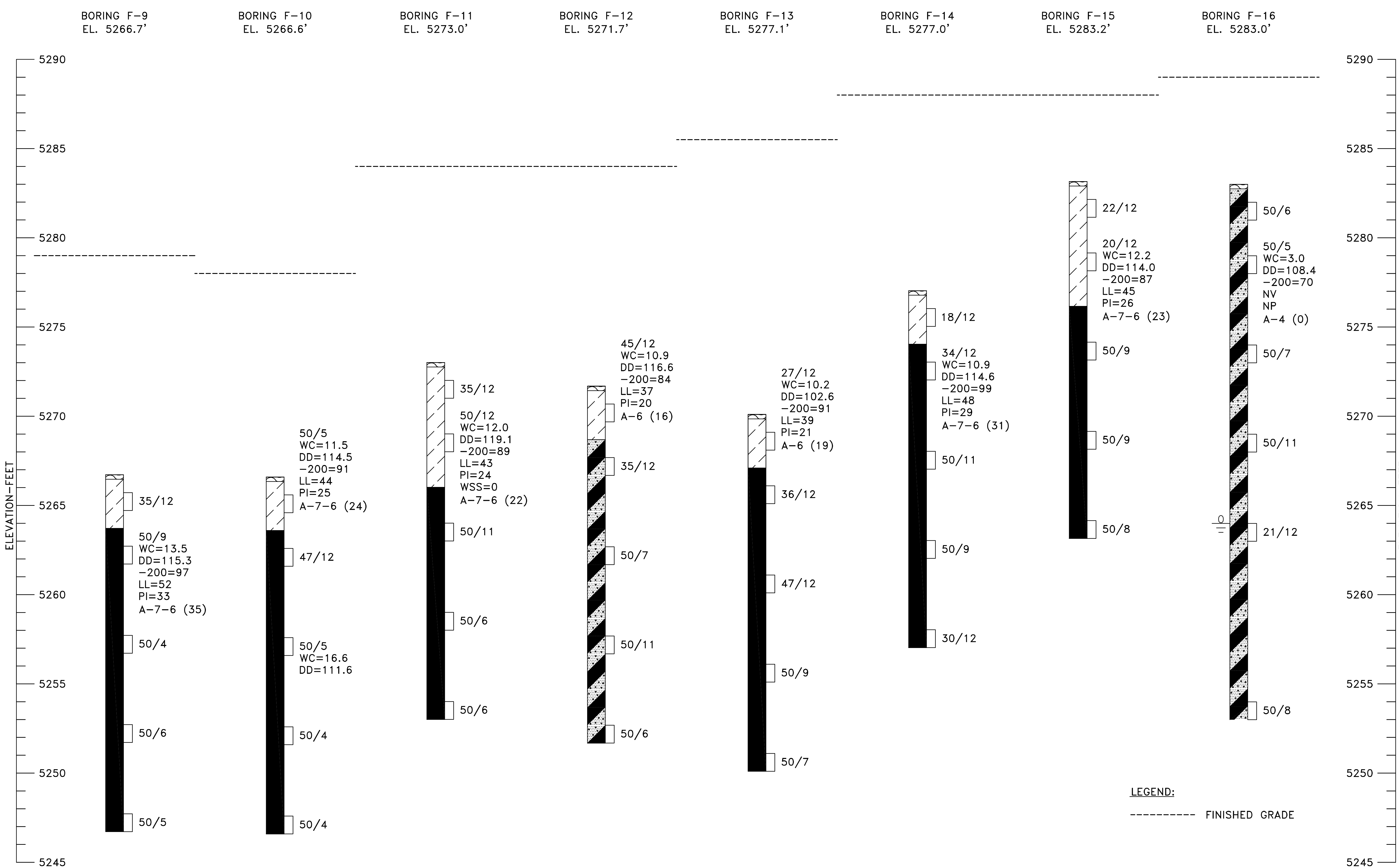
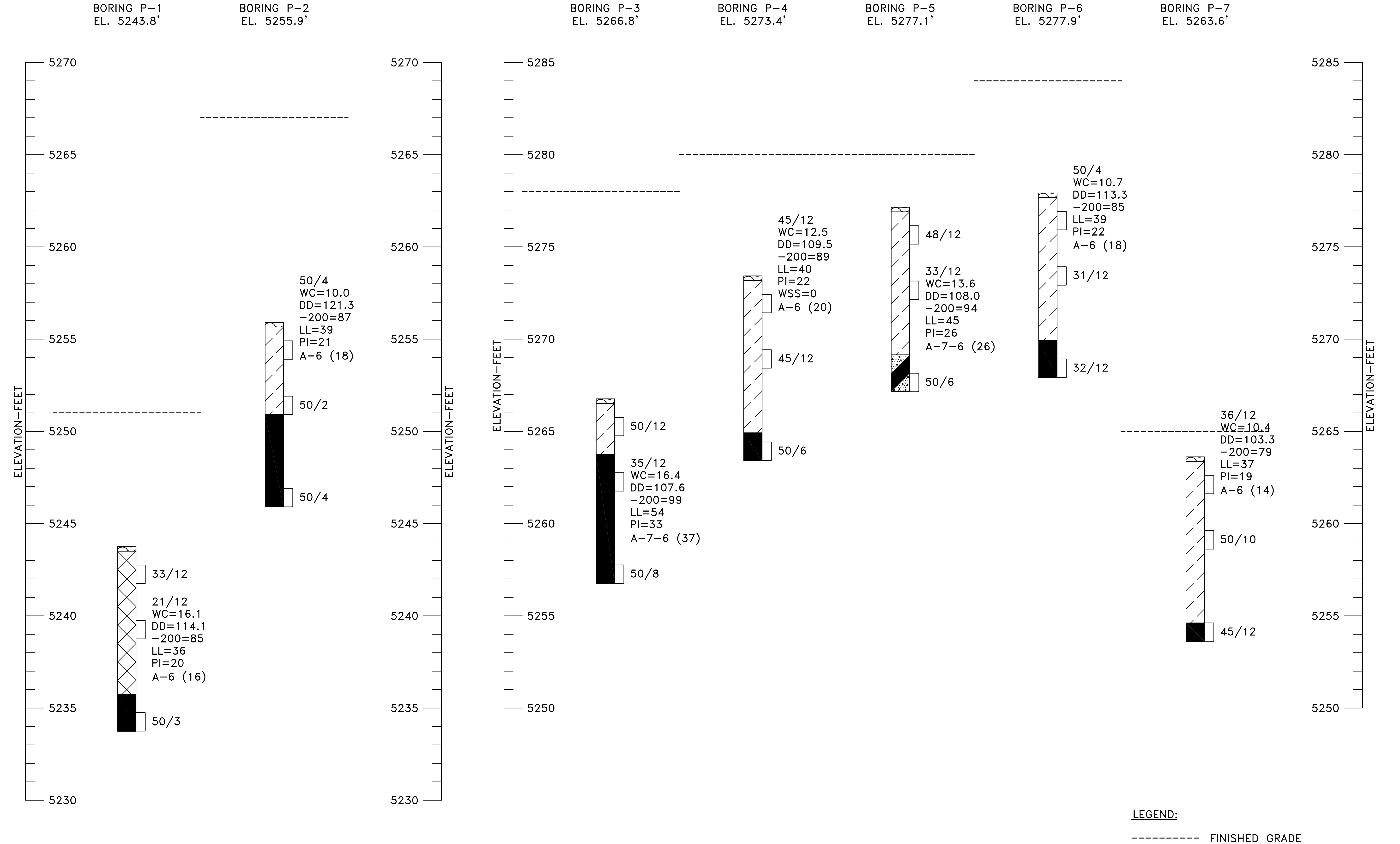
15-3-164	Kumar & Associates	PROPOSED ERIE COMMERCIAL DEVELOPMENT	LOGS OF EXPLORATORY BORINGS	Fig. 3



15-3-164	Kumar & Associates	PROPOSED ERIE COMMERCIAL DEVELOPMENT	LOGS OF EXPLORATORY BORINGS	Fig. 4



15-3-164	Kumar & Associates	PROPOSED ERIE COMMERCIAL DEVELOPMENT	LOGS OF EXPLORATORY BORINGS	Fig. 5

LEGEND

TOPSOIL.

FILL: LEAN CLAY WITH SAND (CL), FINE TO MEDIUM GRAINED, MOIST, LIGHT BROWN TO BROWN.

LEAN CLAY TO SANDY LEAN CLAY (CL), FINE TO COARSE GRAINED, VERY STIFF TO HARD, SLIGHTLY MOIST TO MOIST, LIGHT BROWN TO BROWN, OCCASIONAL CALCAREOUS ZONES.

CLAYSTONE BEDROCK, FINE TO MEDIUM GRAINED, MEDIUM HARD TO VERY HARD, MOIST, GRAY TO BROWN.

INTERBEDDED SANDSTONE AND CLAYSTONE BEDROCK, OCCASIONAL SILTSTONE INTERBEDS, FINE TO MEDIUM GRAINED, FIRM TO VERY HARD, MOIST, GRAY TO BROWN, NIL TO WEAK CEMENTATION.

DRIVE SAMPLE, 2-INCH I.D. CALIFORNIA LINER SAMPLE.

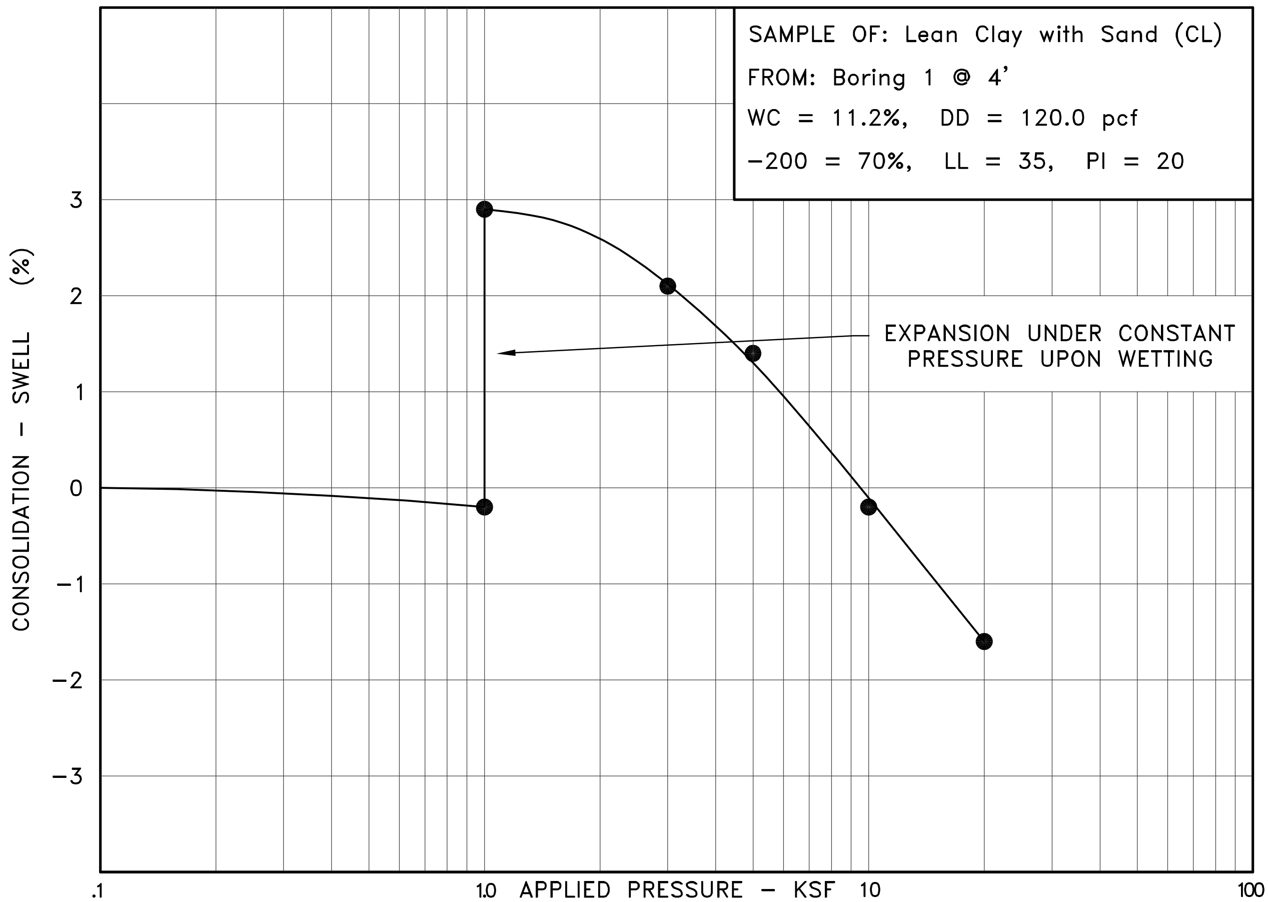
50/9 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 50 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 9 INCHES.

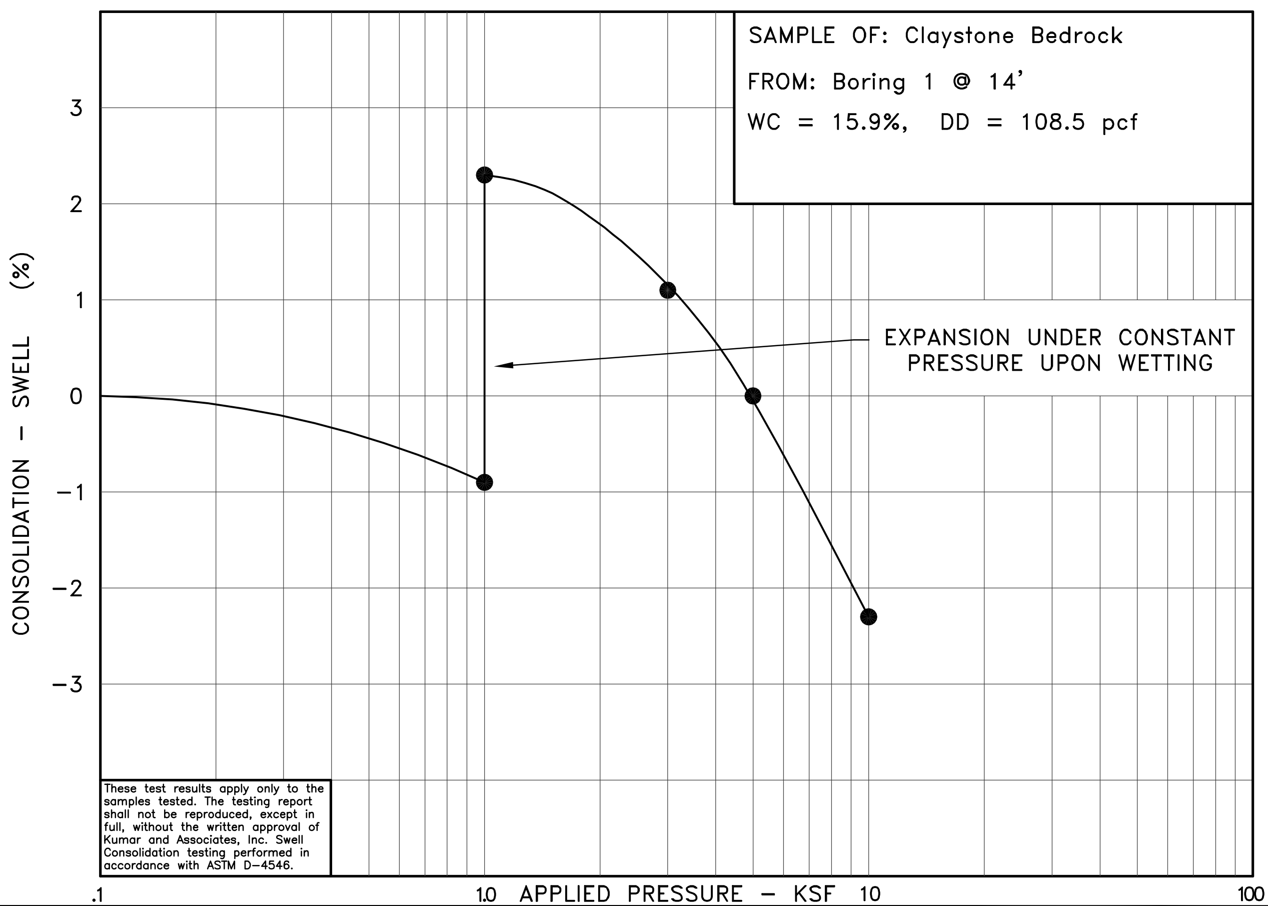
DISTURBED BULK SAMPLE.

DEPTH TO WATER LEVEL ENCOUNTERED AT THE TIME OF DRILLING.

NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED ON SEPTEMBER 8TH AND 10TH, 2015 WITH A 4-INCH DIAMETER CONTINUOUS FLIGHT POWER AUGER.
2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE MEASURED APPROXIMATELY BY PACING FROM FEATURES SHOWN ON THE SITE PLAN PROVIDED.
3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE OBTAINED BY INTERPOLATION BETWEEN CONTOURS ON THE SITE PLAN PROVIDED.
4. THE EXPLORATORY BORING LOCATIONS AND ELEVATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.
5. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.
6. GROUND WATER LEVELS SHOWN ON THE LOGS WERE MEASURED AT THE TIME AND UNDER CONDITIONS INDICATED. FLUCTUATIONS IN THE WATER LEVEL MAY OCCUR WITH TIME.
7. LABORATORY TEST RESULTS:
   WC = WATER CONTENT (%) (ASTM D 2216);
   DD = DRY DENSITY (pcf) (ASTM D 2216);
   -200= PERCENTAGE PASSING NO. 200 SIEVE (ASTM D 1140);
   LL = LIQUID LIMIT (ASTM D 4318);
   PI = PLASTICITY INDEX (ASTM D 4318);
   NP = NON-PLASTIC (ASTM D 4318);
   NV = NO LIQUID LIMIT VALUE (ASTM D 4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP-L 2103);
   A-2-6 (0) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M 145).





These test results apply only to the samples tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar and Associates, Inc. Swell Consolidation testing performed in accordance with ASTM D-4546.

15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 7

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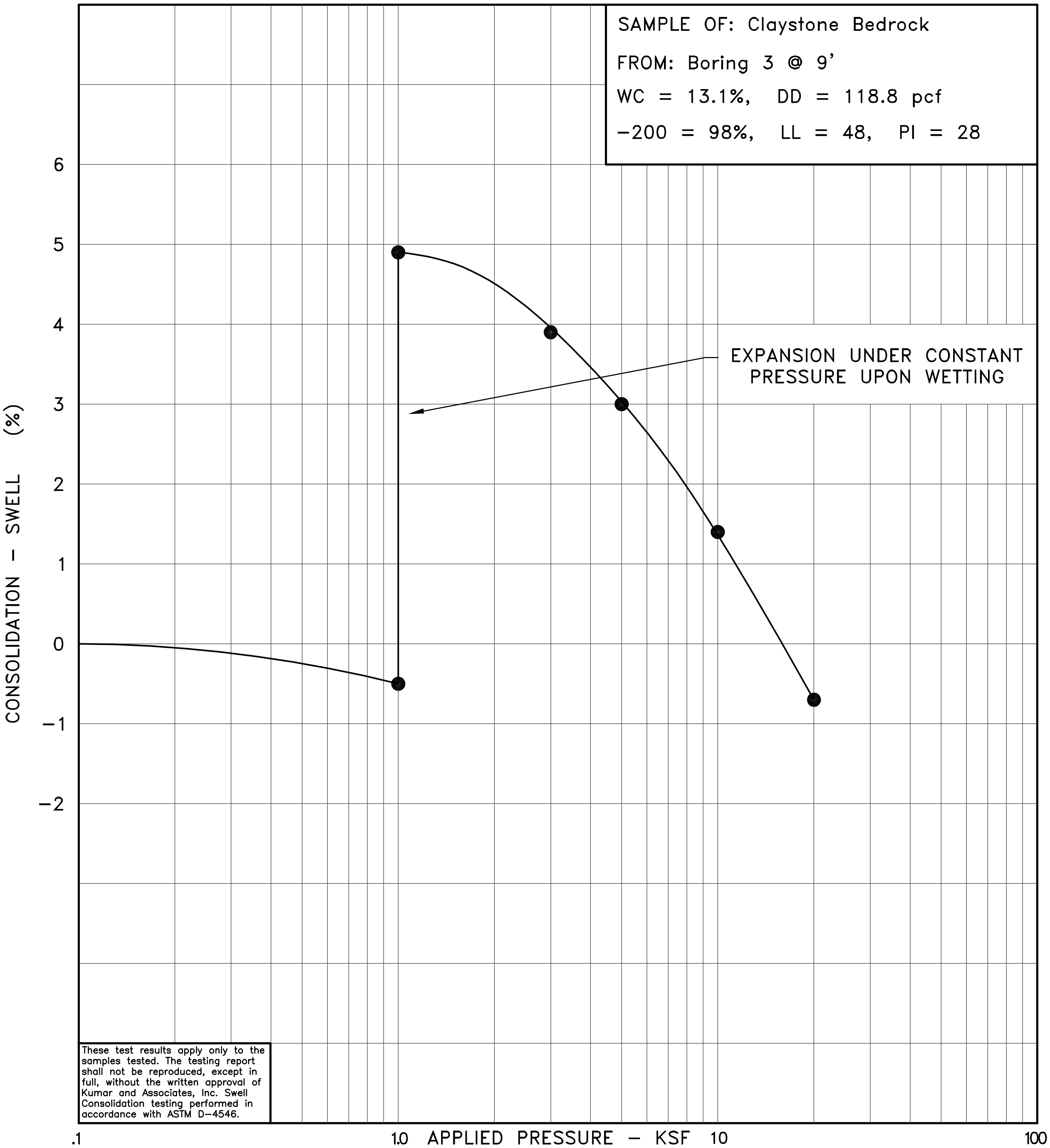

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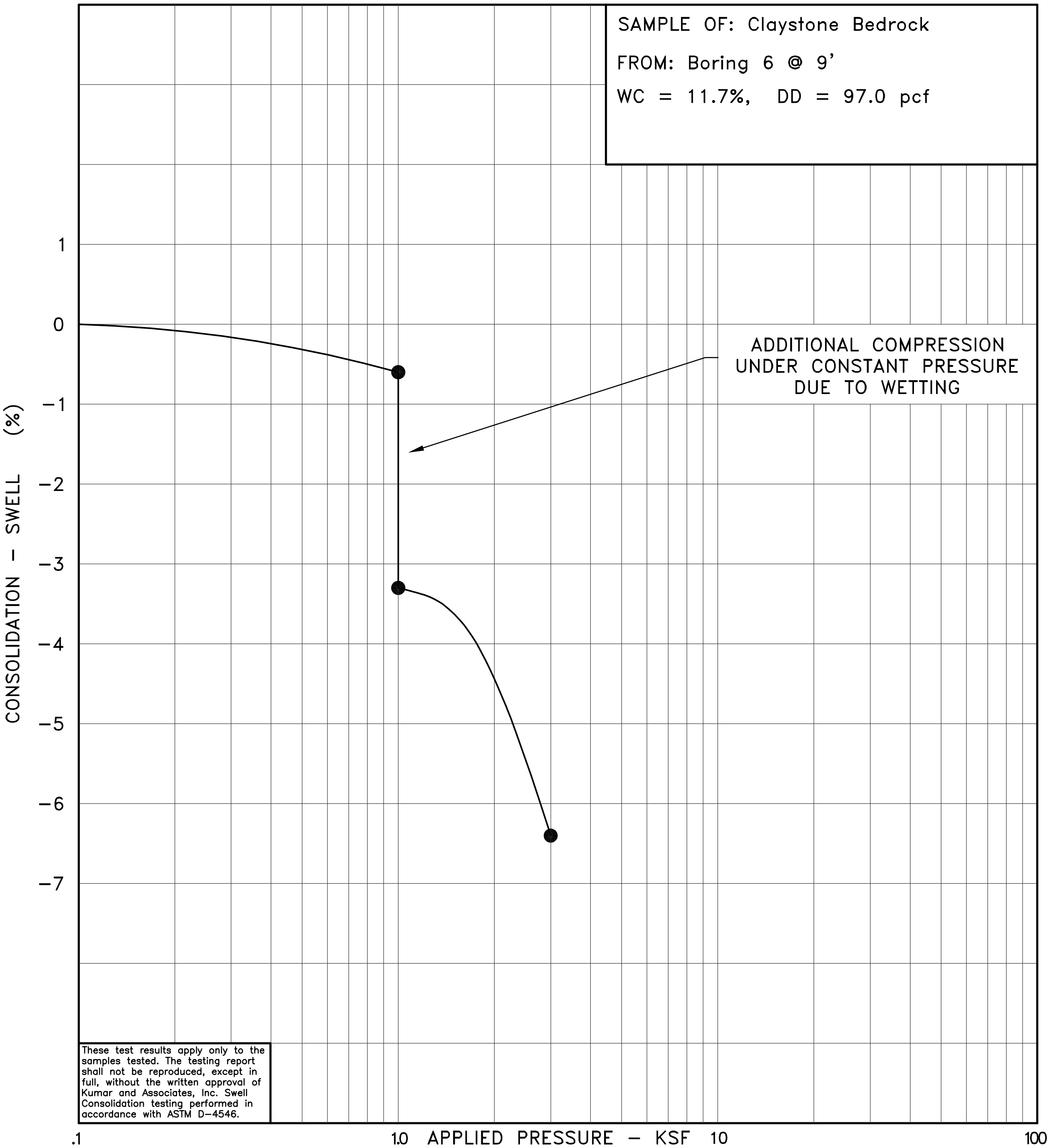
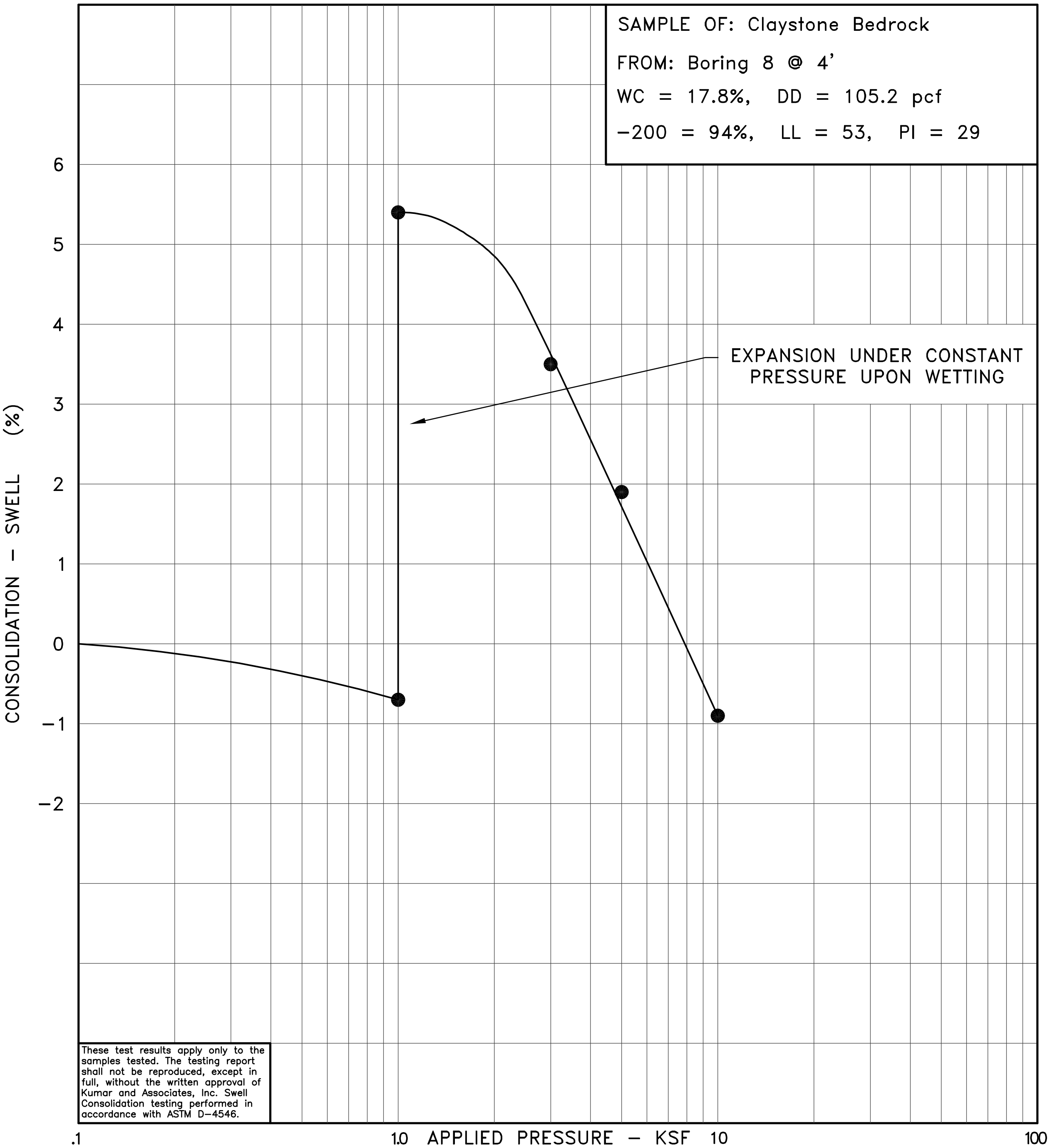
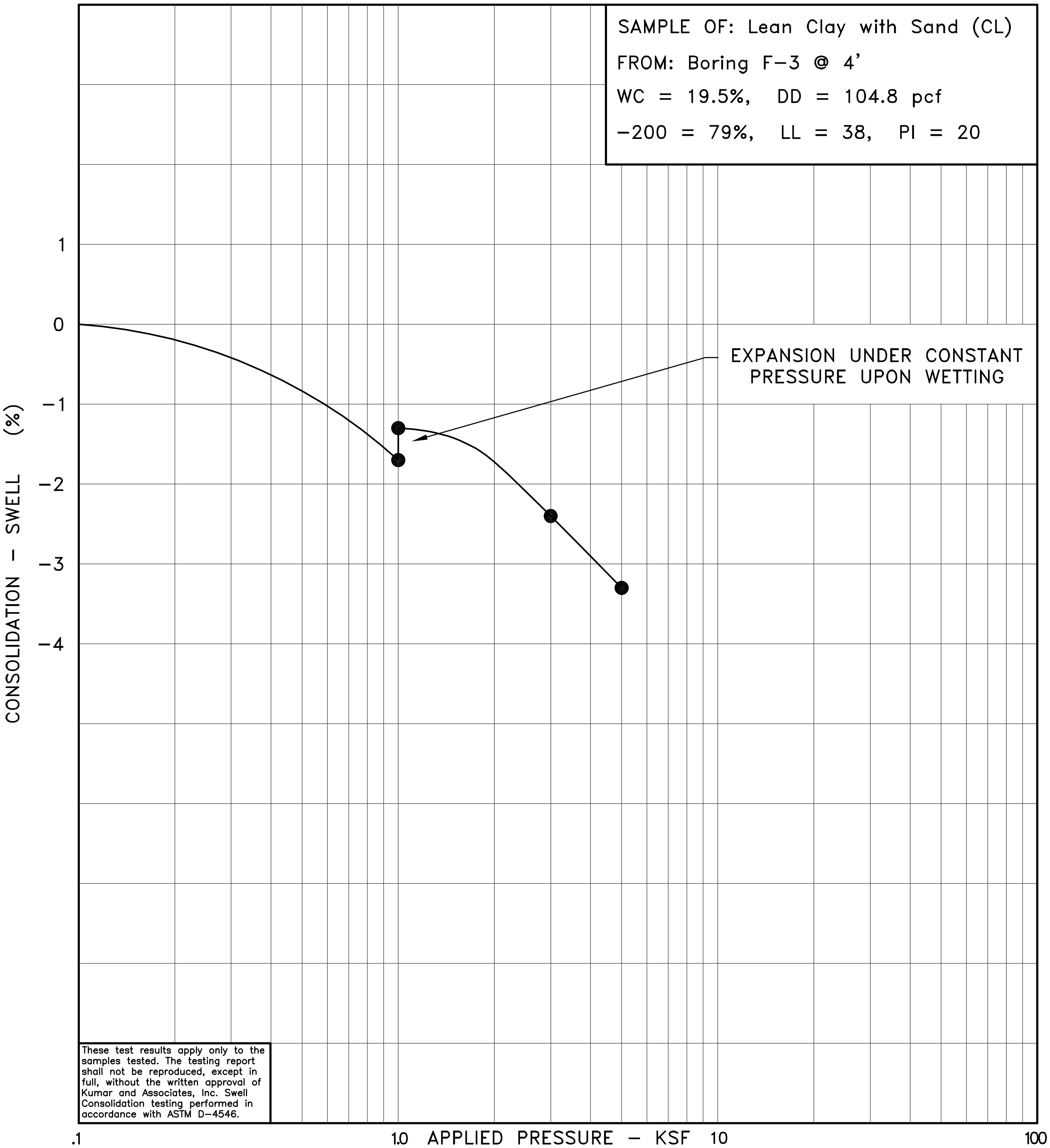
15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 8



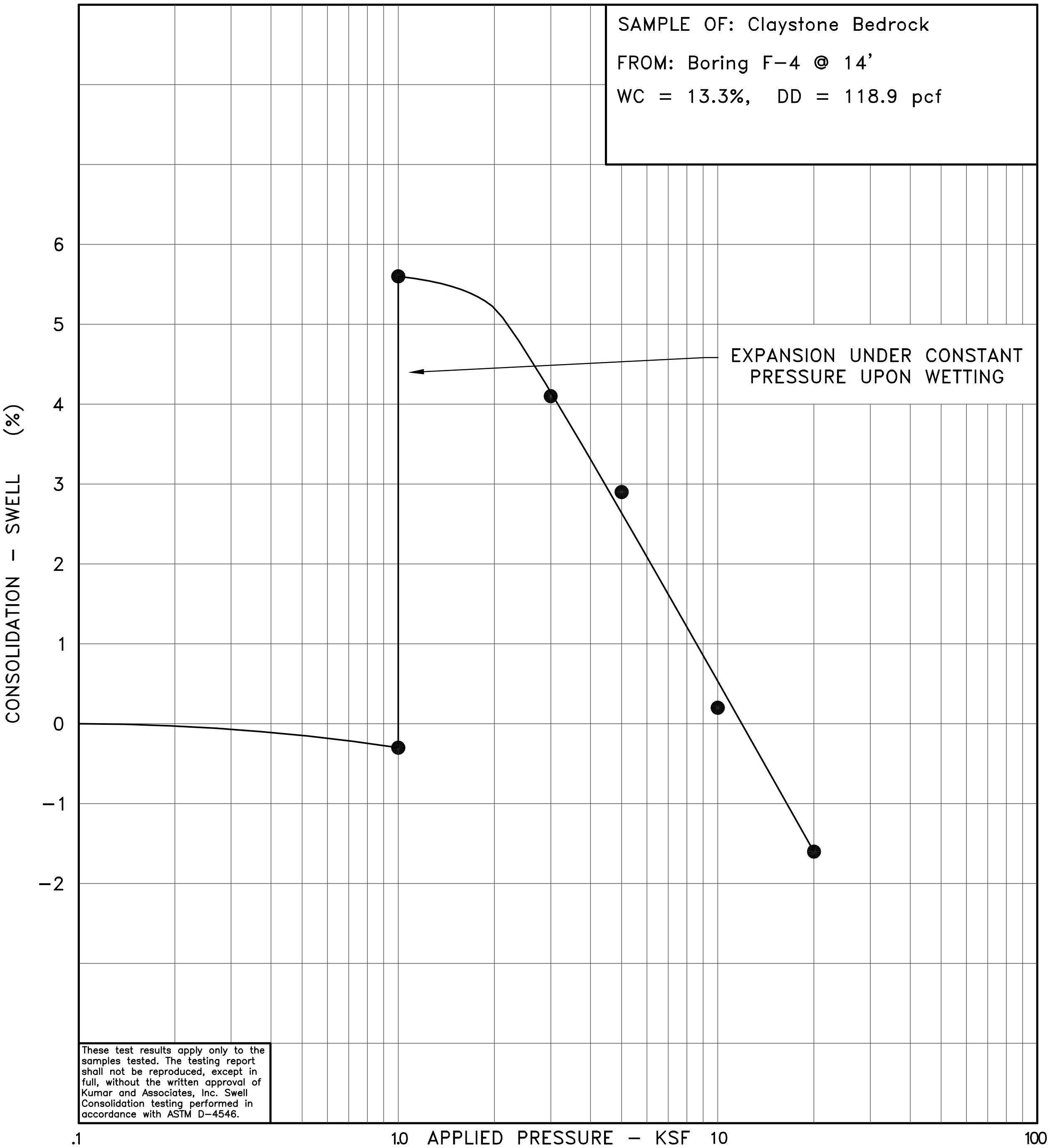
15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 9

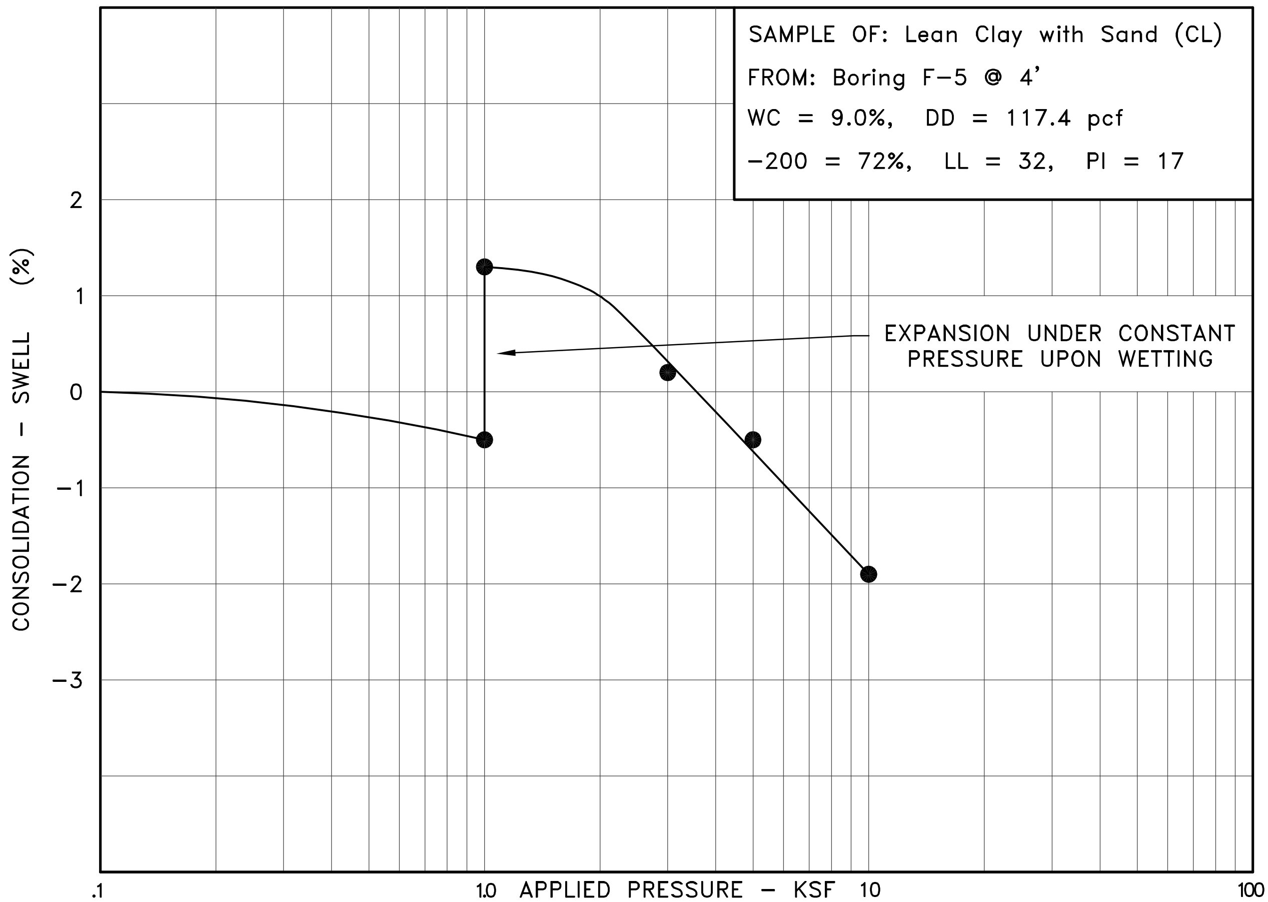


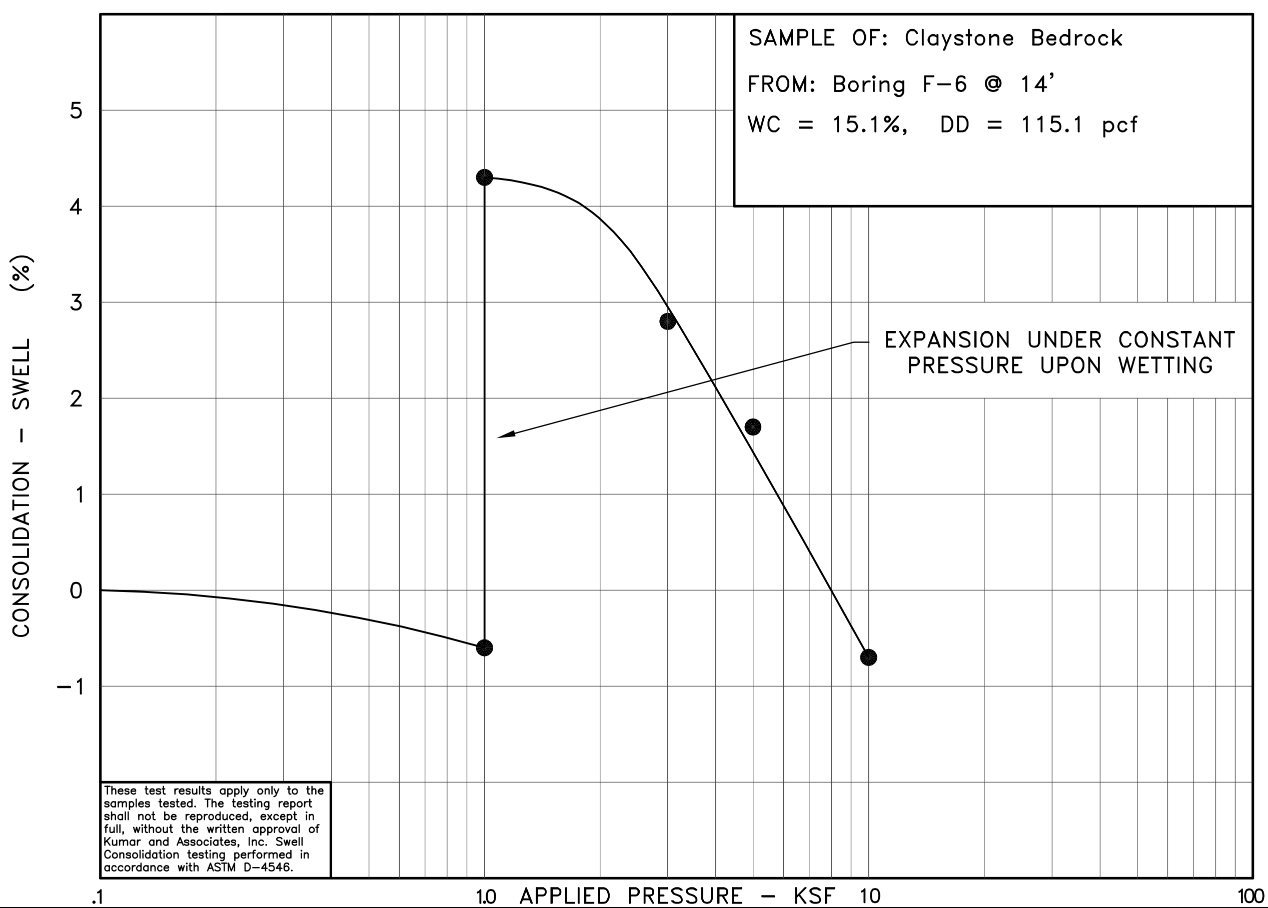
15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 10



15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 11


SAMPLE OF: Claystone Bedrock
FROM: Boring F-4 @ 14'
WC = 13.3%, DD = 118.9 pcf
EXPANSION UNDER CONSTANT PRESSURE UPON WETTING
CONSOLIDATION - SWELL (%)
6
5
4
3
2
1
0
-1
-2
These test results apply only to the samples tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar and Associates, Inc. Swell Consolidation testing performed in accordance with ASTM D-4546.
.1
1.0
APPLIED PRESSURE - KSF
10
100





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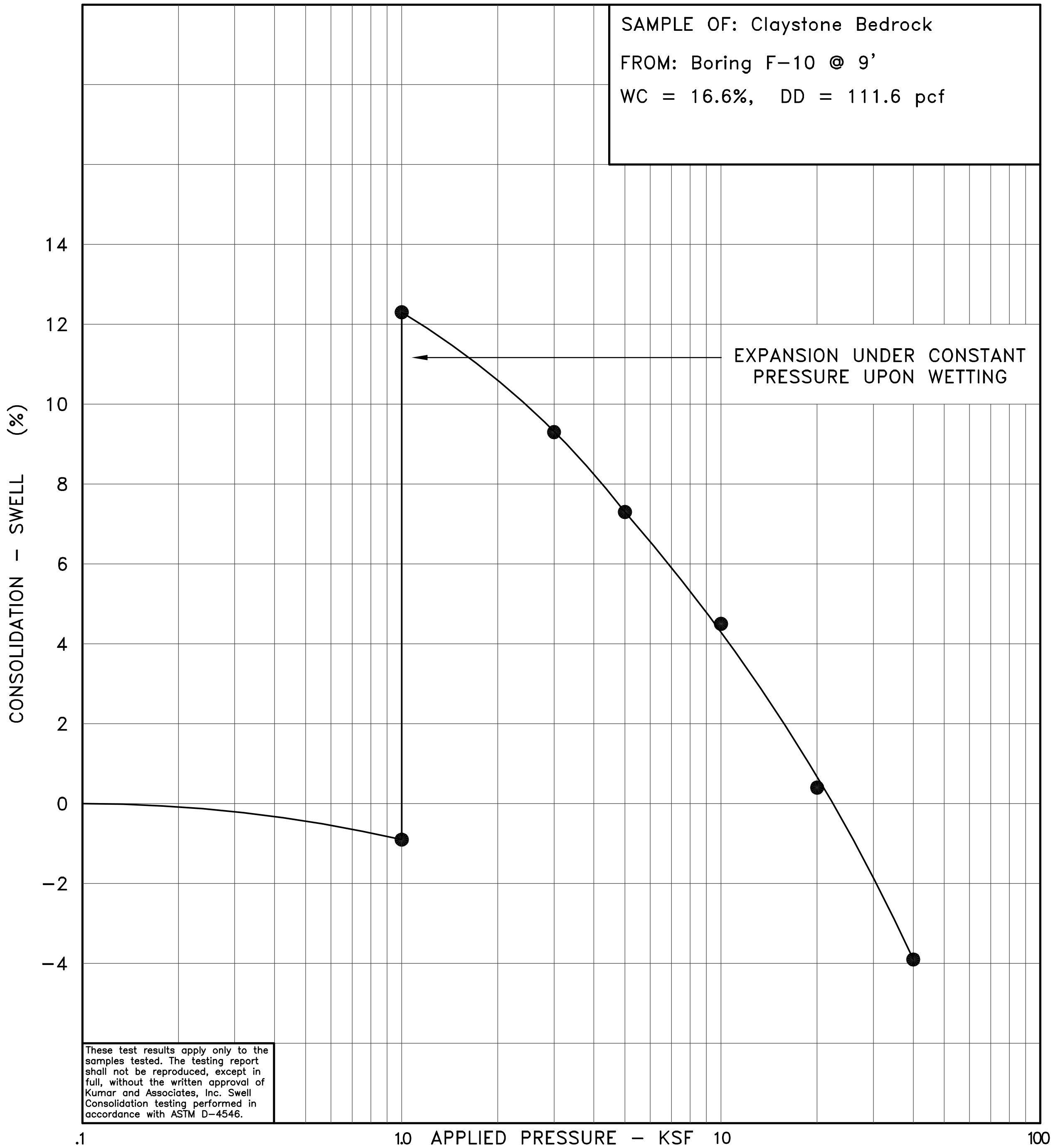
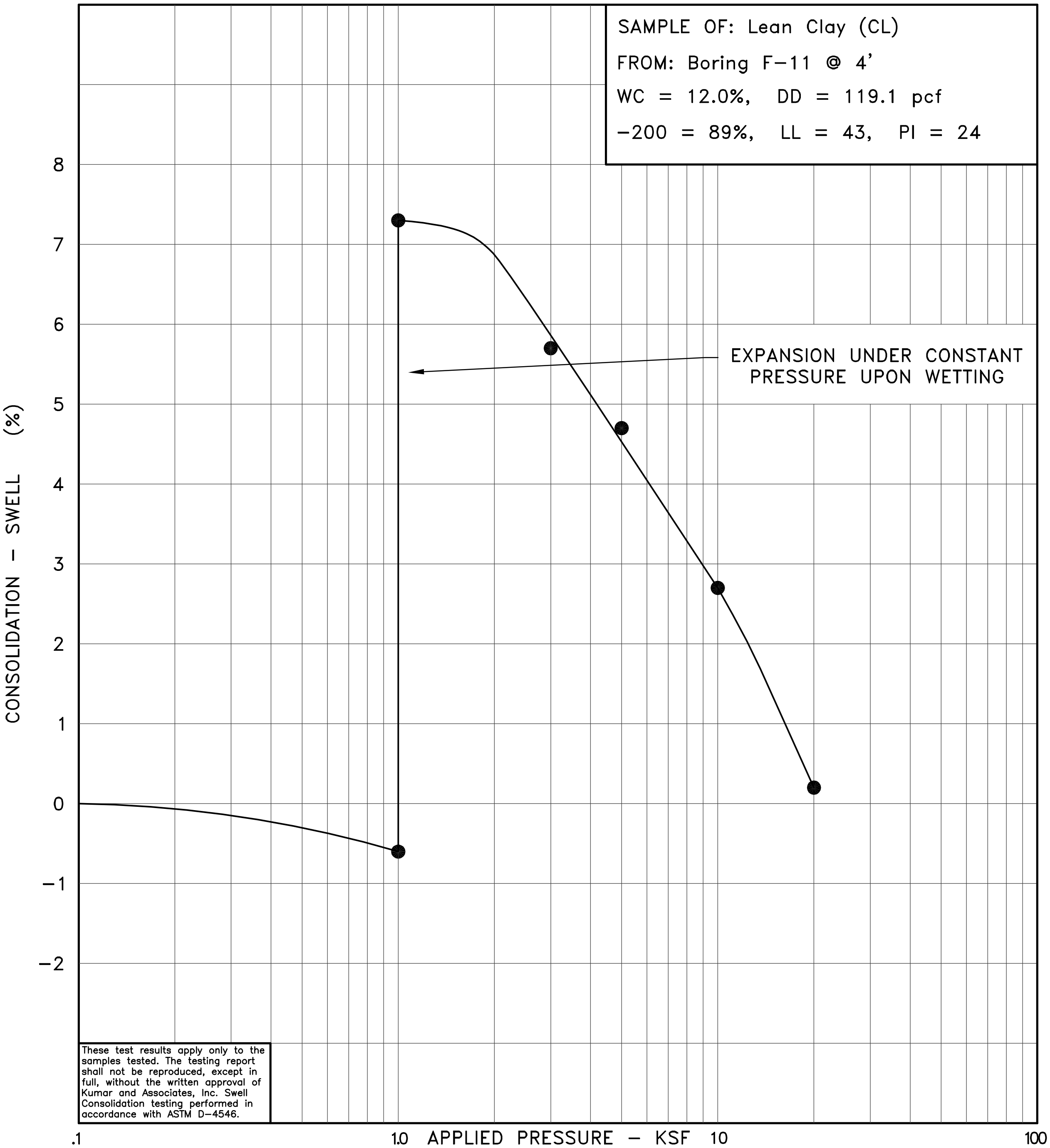
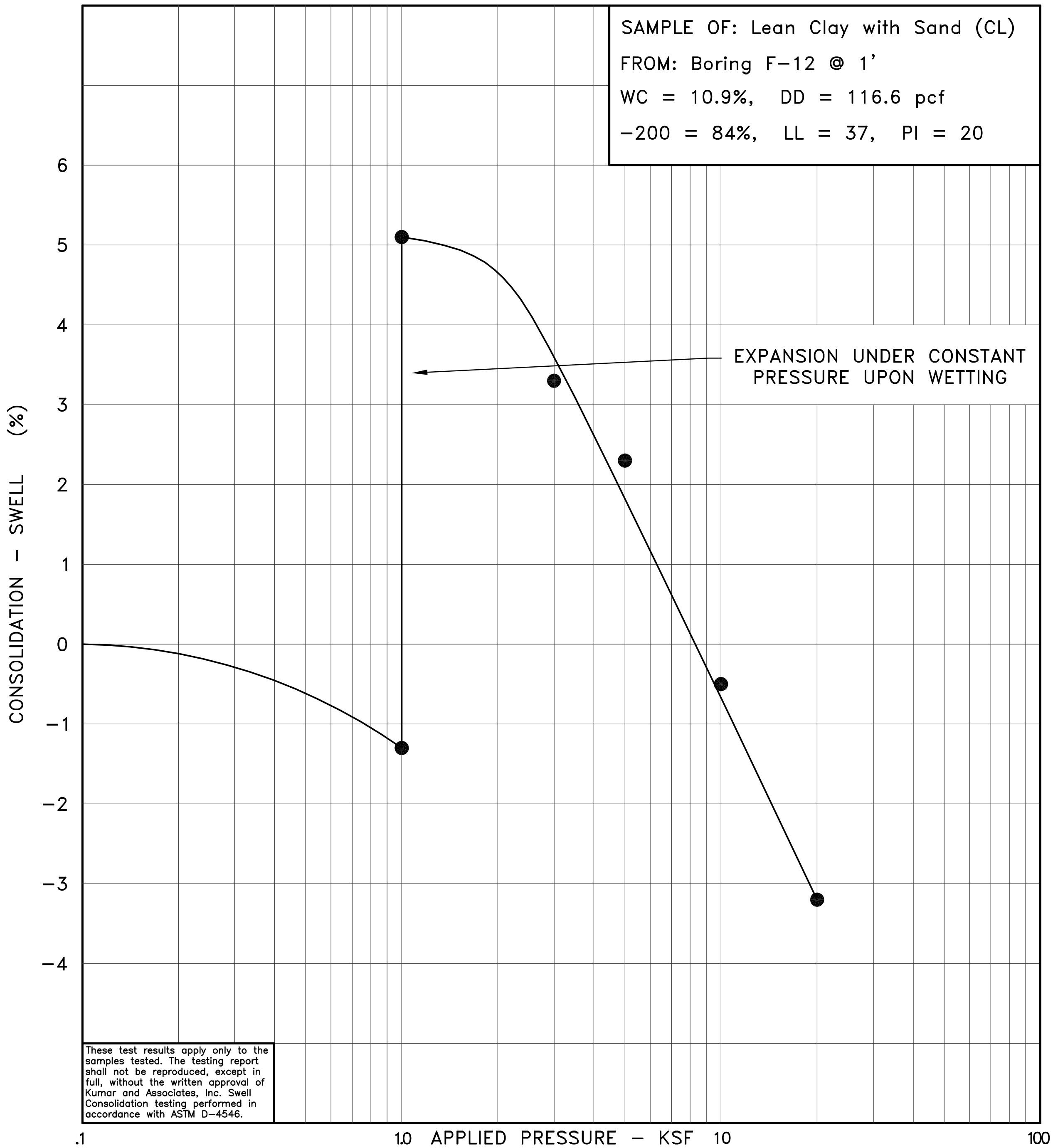
15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 13



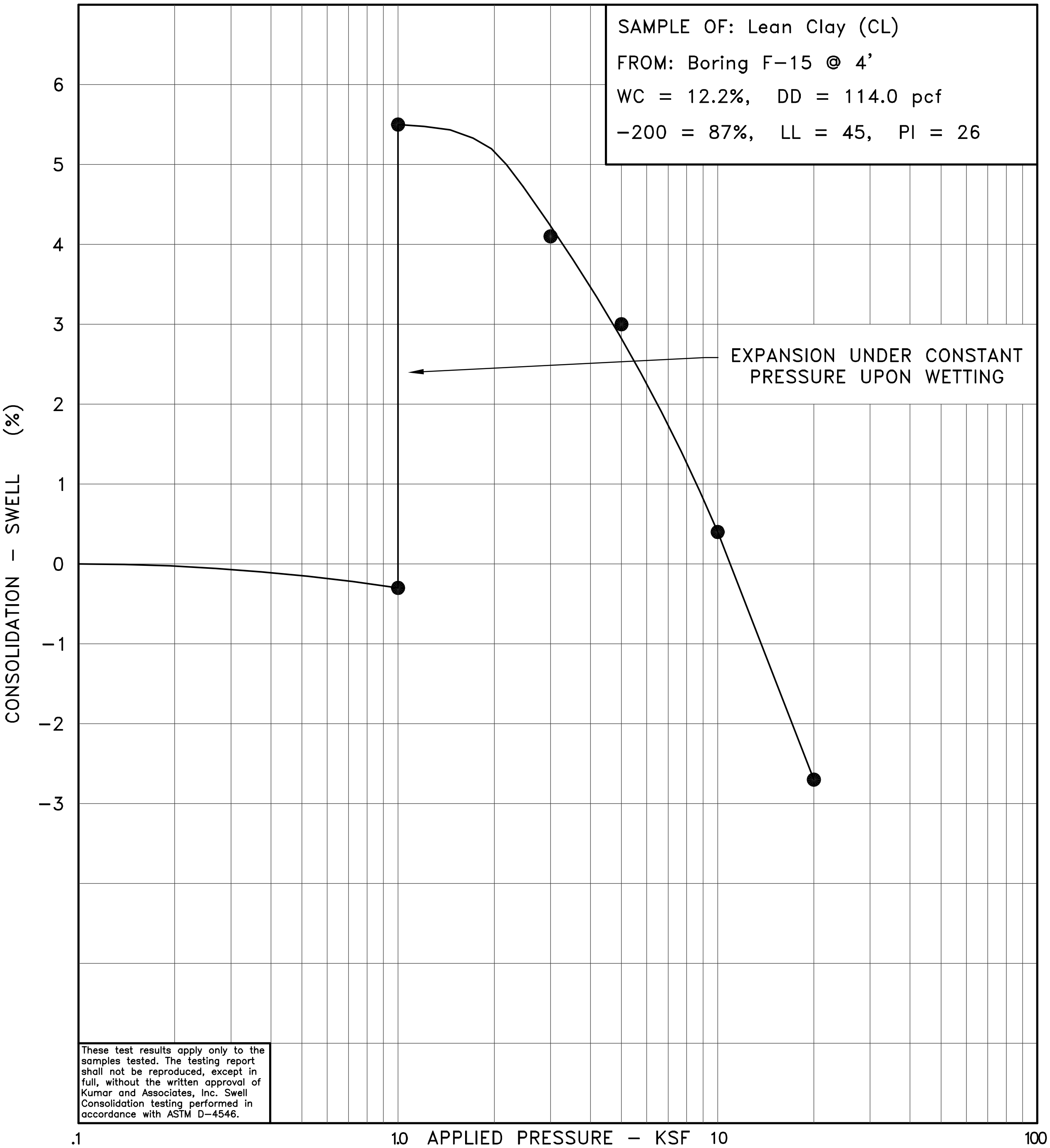
SAMPLE OF: Claystone Bedrock
FROM: Boring F-10 @ 9'
WC = 16.6%, DD = 111.6 pcf
EXPANSION UNDER CONSTANT PRESSURE UPON WETTING
CONSOLIDATION - SWELL (%)
14
12
10
8
6
4
2
0
-2
-4
These test results apply only to the samples tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar and Associates, Inc. Swell Consolidation testing performed in accordance with ASTM D-4546.
.1
1.0
APPLIED PRESSURE - KSF
10
100



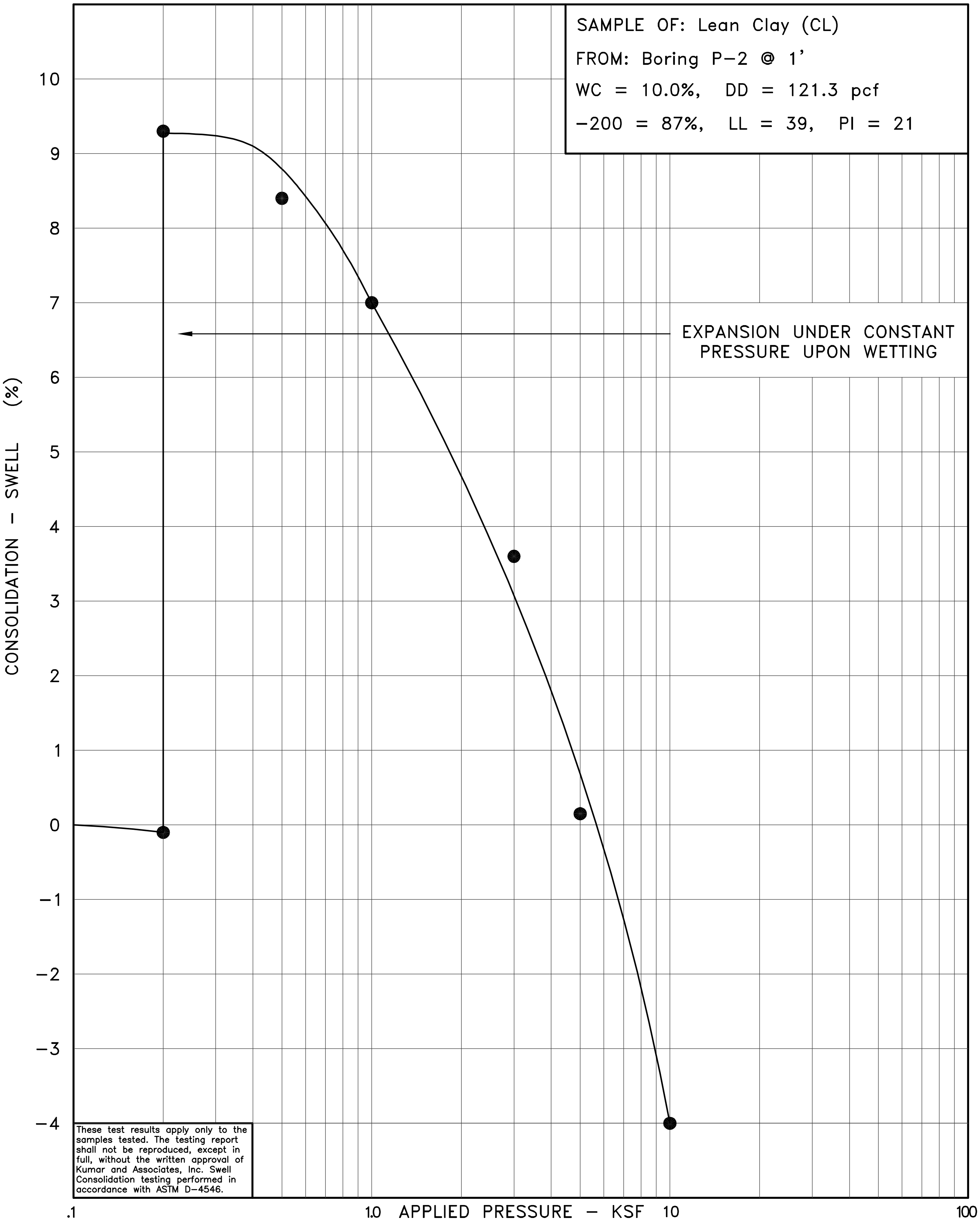
15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 15



15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 16



15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 17



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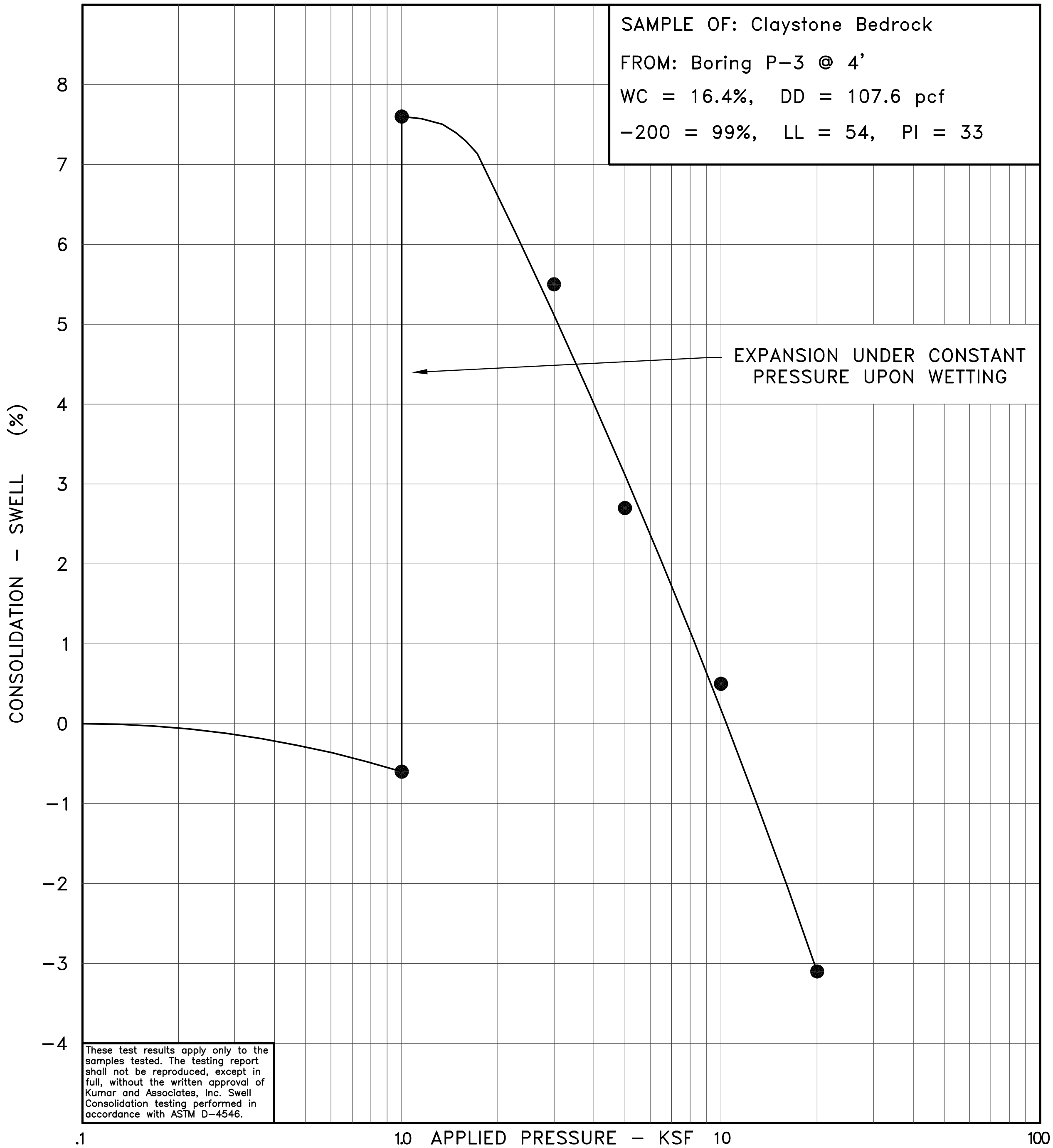
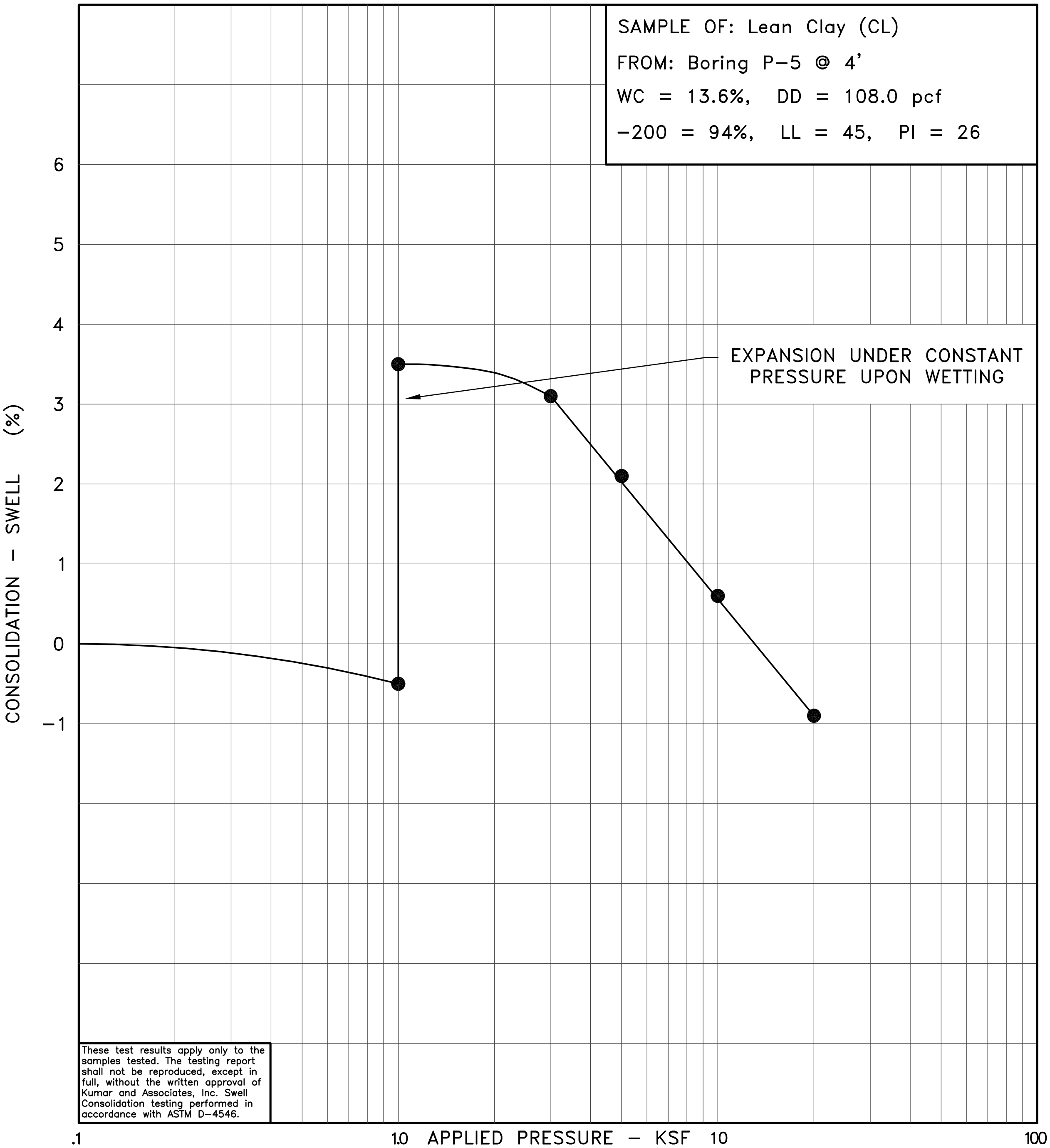
15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 18



15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 19



15-3-164	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 20

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Table I
Summary of Laboratory Test Results

Project No.: 15-3-164
Project Name: Erie Commercial
Date Sampled: September 8 and 10, 2015
Date Received: September 17, 2015

Sample Location						Atterberg Limits				
Boring	Depth (Feet)	Date Tested	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Liquid Limit (%)	Plasticity (%)	Water Soluble Sulfates (%)	AASHTO Classification (Group Index)	Soil or Bedrock Type
1	4	9/21/15	11.2	120.0	70	35	20	0.16	A-6 (12)	Lean Clay with Sand (CL)
1	14	9/21/15	15.9	108.5						Claystone Bedrock
2	9	9/21/15	9.5	114.7	72	37	21		A-6 (13)	Lean Clay with Sand (CL)
3	4	9/21/15	14.9	110.4	96	48	28		A-7-6 (29)	Lean Clay (CL)
3	9	9/21/15	13.1	118.8	98	48	28	0	A-7-6 (30)	Claystone Bedrock
4	4	9/21/15	15.4	116.2	87	42	25		A-7-6 (22)	Lean Clay (CL)
5	4	9/21/15	17.8	111.4	94	61	40		A-7-6 (42)	Claystone B edrock
6	4	9/21/15	10.0	124.5	52	25	11		A-6 (2)	Sandy Lean Clay (CL)
6	9	9/21/15	11.7	97.0						Claystone Bedrock
7	1	9/21/15	13.1	108.3	90	43	24		A-7-6 (22)	Lean Clay (CL)
7	4	9/21/15	18.6	109.1	98	73	51		A-7-6 (57)	Claystone Bedrock
8	4	9/21/15	17.8	105.2	94	53	29		A-7-6 (31)	Claystone Bedrock
F-1	4	9/21/15	14.1	117.3	77	30	14		A-6 (9)	Fill: Lean Clay with Sand (CL)
F-2	9	9/21/15	13.5	119.7	78	41	24		A-7-6 (18)	Claystone Bedrock
F-3	4	9/21/15	19.5	104.8	79	38	20	0	A-6 (15)	Lean Clay with Sand (CL)
F-4	14	9/21/15	13.3	118.9						Claystone Bedrock
F-5	4	9/21/15	9.0	117.4	72	32	17		A-6 (10)	Lean Clay with Sand (CL)
F-6	4	9/21/15	8.8	108.2	79	33	20		A-6 (14)	Lean Clay with Sand (CL)
F-6	14	9/21/15	15.1	115.1						Claystone Bedrock
F-7	4	9/21/15	2.9	114.5	40	NV	NP		A-4 (0)	Sandstone Bedrock
F-8	1	9/21/15	10.3	106.3	77	34	17		A-6 (11)	Lean Clay with Sand (CL)
F-9	4	9/21/15	13.5	115.3	97	52	33		A-7-6 (35)	Claystone Bedrock
F-10	1	9/21/15	11.5	114.5	91	44	25		A-7-6 (24)	Lean Clay (CL)
F-10	9	9/21/15	16.6	111.6						Claystone Bedrock
F-11	4	9/21/15	12.0	119.1	89	43	24	0	A-7-6 (22)	Lean Clay (CL)
F-12	1	9/21/15	10.9	116.6	84	37	20		A-6 (16)	Lean Clay with Sand (CL)
F-13	1	9/21/15	10.2	102.6	91	39	21		A-6 (19)	Lean Clay (CL)
F-14	4	9/21/15	10.9	114.6	99	48	29		A-7-6 (31)	Claystone Bedrock
F-15	4	9/21/15	12.2	114.0	87	45	26		A-7-6 (23)	Lean Clay (CL)
F-16	4	9/21/15	3.0	108.4	70	NV	NP		A-4 (0)	Siltstone Bedrock
P-1	4	9/21/15	16.1	114.1	85	36	20		A-6 (16)	Fill: Lean Clay with Sand (CL)
P-2	1	9/21/15	10.0	121.3	87	39	21		A-6 (18)	Lean Clay (CL)
P-3	4	9/21/15	16.4	107.6	99	54	33		A-7-6 (37)	Claystone Bedrock
P-4	1	9/21/15	12.5	109.5	89	40	22	0	A-6 (20)	Lean Clay (CL)
P-5	4	9/21/15	13.6	108.0	94	45	26		A-7-6 (26)	Lean Clay (CL)
P-6	1	9/21/15	10.7	113.3	85	39	22		A-6 (18)	Lean Clay with Sand (CL)
P-7	1	9/21/15	10.4	103.3	79	37	19		A-6 (14)	Lean Clay with Sand (CL)