Interstate 25 / SH 14
Final Interchange Type Selection Report
April 2016

Prepared for:

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Section 1: Introduction

1.1 Project Background and Purpose

This interchange selection memo provides the Colorado Department of Transportation (CDOT) comparative information on interchange types at this location. When the I-25 project as a whole or selected interchanges move forward toward construction, alternative delivery methods such as design-build are possible. If a contractor team proposes an alternative interchange type, this report will provide CDOT with a basis for comparison when considering the alternative design.

The North I-25 Environmental Impact Statement (EIS), completed in 2011, previously identified and evaluated 61 miles of the Interstate 25 (I-25) corridor from the Fort Collins-Wellington area to Denver. The EIS included an access planning process for I-25 to evaluate the modification of existing interchanges and the potential for new interchanges.

The preferred alternative from the EIS generally included widening of I-25 with an additional general purpose (GP) lane and a managed lane/tolled-express lane (TEL) in each direction with auxiliary lanes between interchanges where warranted. The EIS process evaluated and cleared Standard Diamond Interchanges (SDI) at all locations in the corridor, except those with substantially higher traffic volumes such as the US 34 interchange.

At a minimum, most existing or original crossroad structures would be replaced along with the proposed mainline widening. This work allows the interchange size and type to be modified to meet 2035 traffic forecasts and have flexibility for additional changes beyond 2035 if necessary.

Several of the interchanges in the corridor, including SH 14/Mulberry, had additional analysis and jurisdiction-specific public process work done to verify the design concepts. This process and the resulting alternative were included in a document titled *SH 14 (Mulberry) at I-25 DEIS Interchange Evaluation*, August 20, 2007. At this interchange there was an additional requirement to maintain and enhance frontage road access on the west side of the interchange, with a frontage road underpass to be provided just west of the I-25 interchange, to create a “mini” low-speed interchange. The EIS concept layout of the SH 14 / I-25 interchange is shown in **Figure 1**.

The EIS and Record of Decision (ROD) was completed in 2011 and allowed CDOT to move forward with preliminary design of the I-25 widening and interchange reconstruction. The SDI configuration was used as the base case design, and minor modifications to the design, such as number of turning lanes and ramp lane balance were made as additional detailed design was completed.

It was anticipated that an Interchange Access Request (IAR) would be done concurrently with preliminary design work, however due to uncertainty in the funding of the project, an IAR is not yet scheduled. CDOT has requested some analysis similar to that used in an IAR to verify the interchange type selection of the diamond interchange. Using the FEIS analysis as a baseline, this study evaluates additional interchange configurations, identifying potential improvements or alternatives which would address expected design or operational deficiencies.

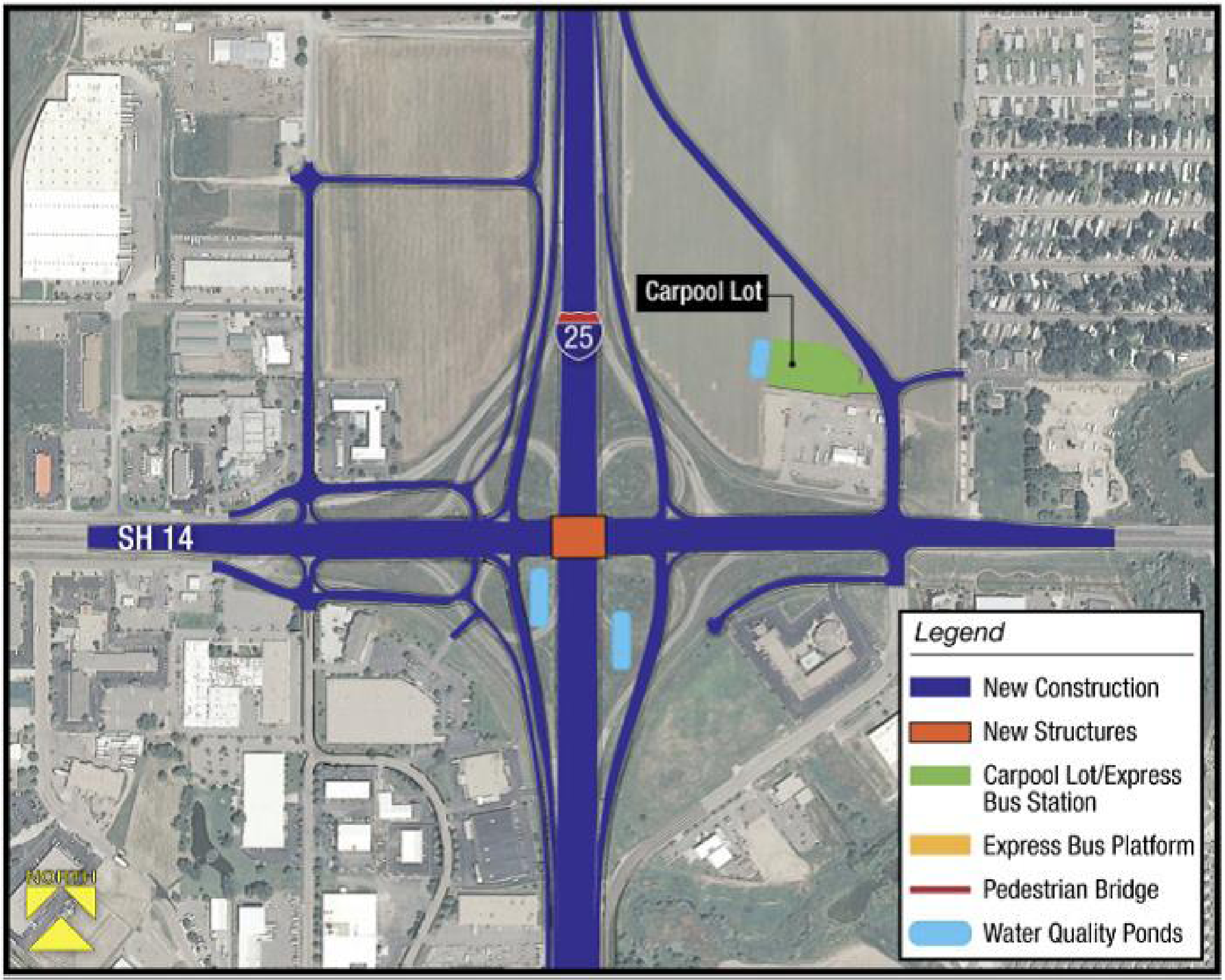


Figure 1.
I-25/SH 14 SDI – North I-25 EIS Preferred Alternative

1.2 Study Area

The I-25/SH 14 (Mulberry Street) interchange is located in unincorporated Larimer County- surrounded by properties annexed by Fort Collins, approximately 4.5 miles east of Colorado State University and one mile north of the Prospect Road interchange. SH 14 is a four-lane principal arterial highway to the west of I-25 and becomes a two-lane undivided principal arterial highway approximately 2,500 feet east of I-25. The Fort Collins transportation plan identifies SH 14 as an ultimate 6-lane arterial to the west of I-25 and a 4-lane arterial to the east. The existing I-25/SH 14 interchange is a partial cloverleaf configuration with loops in all except the southeast quadrant. At the interchange, SH 14 has four through lanes, a WB auxiliary lanes and an EB merge lane passing over I-25. The existing study area is shown in **Figure 2**.

Figure 2.
Study Area

1.3 Existing Land Uses

The area surrounding the interchange is largely industrial and commercial land uses including businesses common at interstate interchanges, such as hotels, restaurants, and other various retail establishments. There are large vacant parcels, particularly in the northeast quadrant. The project team has spoken to developers intending to build on these parcels during project open houses, although there are no known formal development applications as of early 2016.

1.4 Existing Traffic Volumes

The EIS traffic work was done mostly with 2005 data, and then updated prior to completion of the EIS in 2011. For the ongoing design work, existing traffic data was collected in 2014 for the entire I-25 corridor with supplemental data in the interchange area collected in 2015 and 2016. A snapshot of the existing traffic data is shown in **Figure 3**.

No specific Level of Service (LOS) data is reported in this study for existing conditions. The calculated LOS is actually quite good for this interchange. Most of the traffic operations and safety issues at the interchange are due to the 1960s era design standards still in place such as weave sections on the I-25 mainline and on SH 14, the use of low speed loop ramps, and other issues.

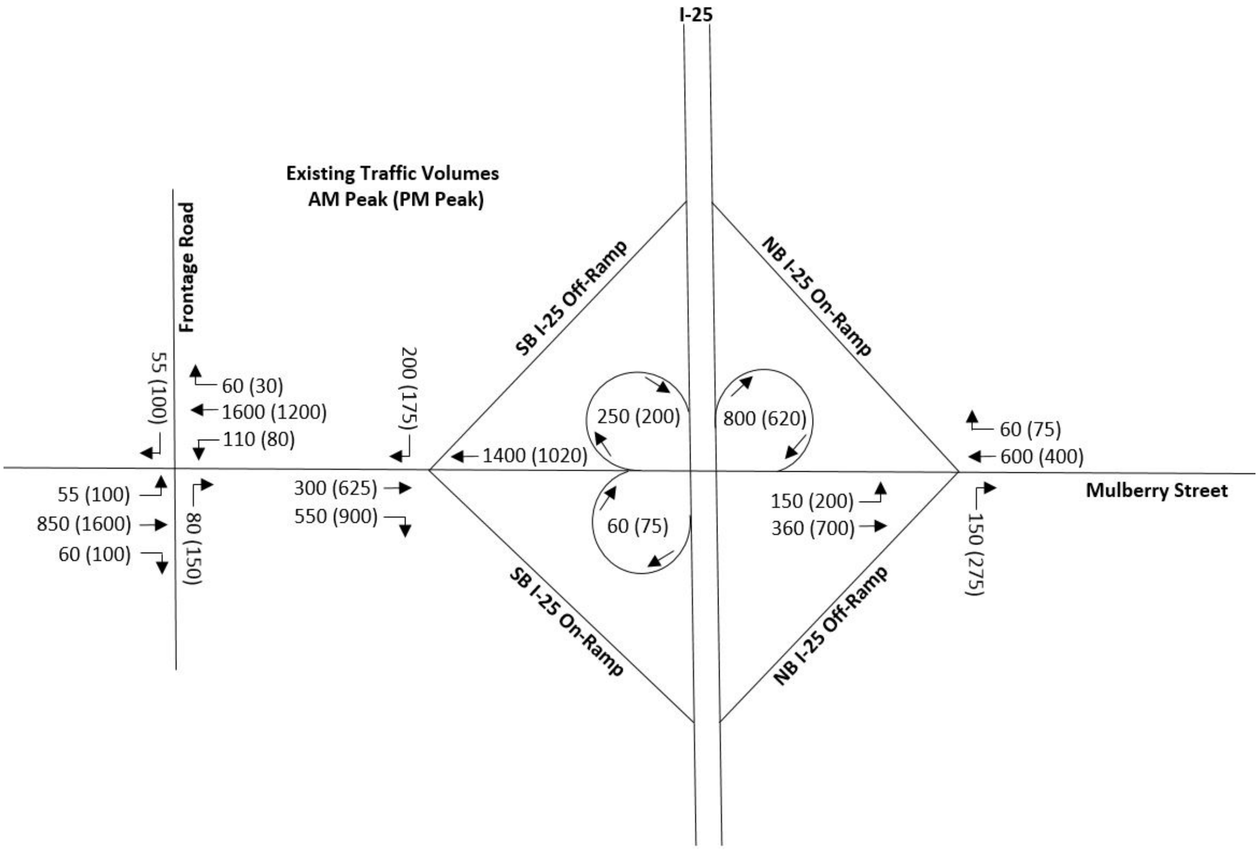


Figure 3.
Existing AM (PM) Peak Hour Traffic Volumes

1.5 Forecasted Traffic and Development at the Interchange

For clarity, definitions of forecasts referenced in this study are described below:

- EIS Forecasts – work done between 2004 and 2011 to initially define the interchange needs, conclusions, and commitments for interchange configuration and adjacent access maintenance (i.e. the frontage road access and underpass west of I-25). Early EIS traffic forecasts were very high based on development trends in 2005-2006. The EIS forecasts were adjusted prior to publication of the EIS in 2011 to reflect the economic downturn of 2008-2009.
- 2014 ROD 1 Revision + ROD 2 – to be referenced as the “Most Recent Forecasts” for this memo. These 2035 forecasts assume the implementation of early phases of I-25 improvements, including a Managed Lane added in each direction. Interchange improvements at Mulberry are also included as occurring by 2035.

The Most Recent Forecasts include ongoing input from the North Front Range Council of Governments (NFRCOG) to reflect development trends and changes, and were adjusted down from the EIS forecasts. The resulting 2035 traffic forecasts for the SH 14 / I-25 interchange area reflect a substantial increase in traffic from 2014 levels, essentially "filling up" Mulberry if it were widened to a six lane arterial west toward Fort Collins. These forecasts are reasonable compared with other six lane arterials in mature developed areas near freeways in the Denver Metro area. The 2035 traffic forecasts used for analysis and comparison of interchange types is shown in **Figure 4**.

Ongoing project discussions with Fort Collins regarding the design of the frontage road access to the west of the interchange resulted in a revised forecast for Mulberry being prepared. This forecast reflects the likely constraint of Mulberry remaining in its existing 4-lane configuration in 2035. These adjusted forecasts are shown in **Figure 5**. These forecasts were only used to evaluate the Phase 1 configuration of the interchange to the west, the ultimate design is still based on the Most Recent Forecasts for 2035.

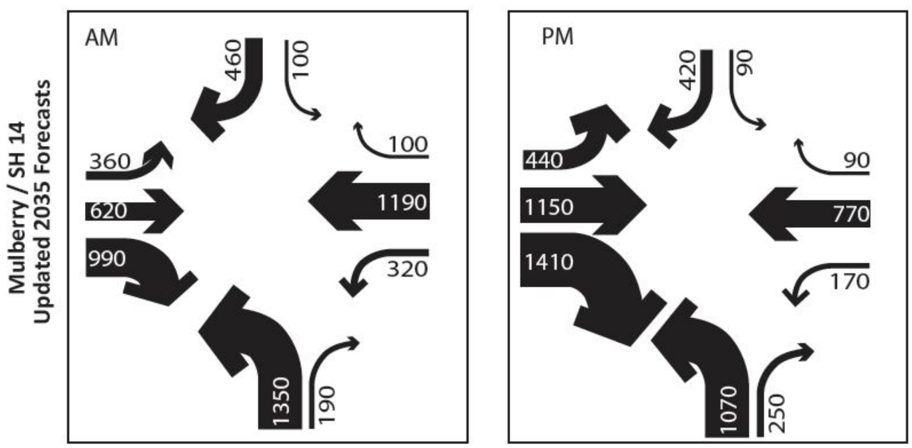


Figure 4.
2035 Traffic Forecasts - 6 Lane Mulberry

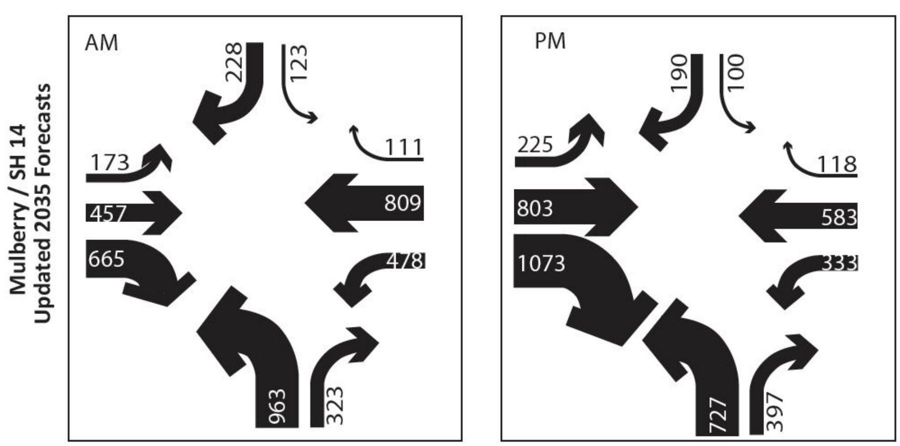


Figure 5.
2035 Traffic Forecasts - 4 Lane Mulberry

Section 2: Preliminary Alternatives and Initial Screening

2.1 Description of Preliminary Alternatives

The EIS evaluation of interchanges utilized several criteria as outlined below:

- LOS D was the minimal acceptable level of operation at existing interchanges.
- At older existing diamond interchanges where there are functional deficiencies or where there is limited structural life or insufficient clearances on the bridges, any I-25 improvement would require a complete reconstruction of the interchange. The reconstructed interchange would, at a minimum, be upgraded to a standard diamond configuration.
- Where the standard diamond interchange did not provide acceptable LOS, an enhanced diamond interchange was evaluated first. The enhanced diamond included additional lanes above and beyond the standard diamond.
- If an enhanced diamond interchange still possesses a LOS of E or F, then an assessment was necessary of both a new interchange and a reconfiguration of the existing interchange.
- The EIS analysis included LOS at the interchange, LOS on mainline, and queueing at the ramps

The purpose of this memo is to identify and examine other interchange types to verify the EIS assumptions and provide a basis of comparison for other interchange types if they are proposed in the future. Five interchange types were evaluated for this study, as listed below:

Alternative 1 – Standard Diamond Interchange with Traffic Signals (SDI)

Alternative 1 is consistent with the interchange configuration recommended in the FEIS. Ramp spacing is 650 feet in the base case design.

Example of a new standard diamond interchange at I-25 and SH 52, showing frontage roads moved away from the interchange.

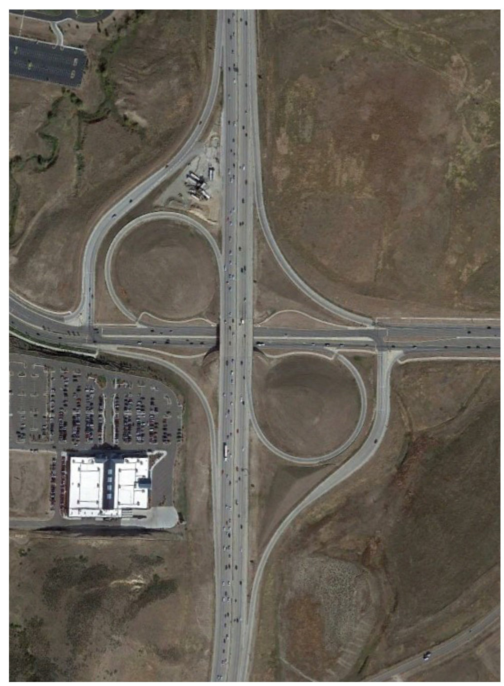
Alternative 2 – Standard Diamond Interchange with Roundabouts

Alternative 2 is based on the same standard diamond configuration, with the exception being that roundabouts would be present at the ramp terminals rather than traffic signals. Roundabout spacing would be similar as in Alternative 1, with adjustments to spacing and ramp geometry made to achieve proper roundabout design criteria.

Example of a new standard diamond interchange with roundabouts on I-70 in Eagle, CO.

Alternative 3 – Partial Cloverleaf Interchange (Parclo)

Because the SH 14 interchange is already a cloverleaf, the existing Right-of-Way (ROW) footprint is larger than at other locations in the corridor. In addition, the SH 14 volumes are higher than many other locations in the corridor, so a higher capacity partial cloverleaf may be a reasonable design. This alternative was only considered with the loop ramps as on-ramps to I-25, the preferred layout for partial cloverleafs.



Example of a partial cloverleaf interchange at I-25 and Ridgegate Parkway in Lone Tree, CO. The preferred loop design is shown where loops are the on-ramps to the freeway.

Alternative 4 – Single Point Urban Interchange (SPUI)

A SPUI is similar to a standard diamond interchange except all approaches converge to a single traffic signal controlled intersection on the structure over I-25. The SPUI would be a typical design that would not facilitate ramp-to-ramp through traffic.

Example of a SPUI interchange at I-25 and US 50 in Pueblo, CO.

Alternative 5 – Diverging Diamond Interchange (DDI)

As of 2016, there are several DDIs in operation or under construction in Colorado, and the implementation of DDIs is increasing rapidly across the US, in particular in Utah, Missouri, and Georgia. This design is advantageous as it allows for two-phase operation at the intersections on each side of the interchange. The project team previously analyzed the DDI configuration for SH 14, as documented in a memorandum – *Updated Interchange Traffic at SH 14 and Prospect Interchanges*, July 24, 2015, located in **Appendix C**. At that time the DDI was determined to be a feasible and potentially preferred alternative for the SH 14 interchange, but the design progressed with the SDI as a larger footprint and more conservative design.

Example of a diverging diamond interchange in Salt Lake City, UT.

Alternatives not evaluated:

There could be the potential for other unique designs at the SH 14 interchange location to address any unique needs or ROW constraints. These were discussed initially with the project team and not carried into this analysis, other than acknowledging them below:

- Full cloverleaf replacement, with collector-distributer roads along I-25 – This alternative was not carried forward since cloverleafs are generally not an appropriate design in urban or suburban areas. Issues with the SH 14 location are that the desirable size of the loops would need to be increased from the existing 150 ft. radius to a 230-250 ft. radius. This would result in ROW impacts in all four quadrants. Additionally, the ramp merges/diverges would be too close to frontage road intersections on either side of I-25. The layout of a full cloverleaf interchange and associated collector/distributor ramps at this location would also infringe on the Prospect interchange, one mile to the south.
- Flyover ramps for specific movements, over a standard diamond – Flyover ramps were not pursued since the specific flyover for the high traffic volume of NB to WB would need to touch down far to the west of I-25, which would interfere with the frontage road access. Also, an SDI or DDI with appropriate laneage can accommodate 2035 traffic levels with good LOS. Flyover ramps are not precluded with the implementation of an SDI or DDI.
- Tight Diamond – A tight diamond interchange (TDI) might be pursued if there were ROW issues forcing tighter ramp geometry. Since the SH 14 area has plenty of ROW, it is unlikely that a TDI would be superior to a properly designed SDI or DDI.
- Three-Level Diamond – This alternative would put SH 14 east-west through movements on a third level over the top of a modified SDI – essentially a "square-about" for the ramp intersection. This might be an appropriate design if there were substantial east-west through traffic, however this is not the case at the SH 14 interchange, most traffic is oriented to-from south I-25.

2.2 Preliminary Screening Criteria

After the preliminary interchange configuration alternatives were identified, a set of criteria for evaluating each alternative was identified. The following criteria was used:

- Addresses Interchange Traffic Operations: The interchange configuration should provide an acceptable LOS at each intersection with a goal of delay LOS C or better (overall intersection). Individual movements or lane groups should not have excessive delays or queuing. The available reserve capacity to accommodate additional traffic growth was also quantified.
- Cost: Initial capital and maintenance costs were taken into consideration and those with lower total costs were viewed more favorable than higher cost alternatives. This was a comparative analysis at this level among the alternatives, focusing on high cost elements like bridge structure.
- Multimodal Accommodations: Safe and efficient pedestrian and bicycle travel throughout the study area is an important consideration for the SH 14/Mulberry Street interchange.

- Right-of-way and Environmental: The base case SDI interchange in the EIS has been cleared for an ultimate ROW footprint, and environmental mitigation has been identified. Other alternatives can be compared to the SDI for whether there can be a reduction of impacts to environmental resources or ROW needs.
- Design Consistency and Driver Expectancy: All interchanges in the FEIS study area are either a standard diamond, tight diamond, or partial cloverleaf interchange. Therefore, interchange types that are consistent with the existing corridor and have high driver familiarity could be given more favorable consideration.
- Interface with frontage road intersections: The west side intersection of the frontage road with Mulberry Street was identified to include an underpass of Mulberry Street and right-in/right-out access points to form a kind of "mini" interchange. The alternatives were compared for how that design and operation can be incorporated.

2.3 Initial Screening of Alternatives

The advantages and disadvantages of each of the five interchange alternatives considered in the initial screening process is summarized in **Table 1**. Alternatives 1 and 5 – the SDI and DDI - showed considerable advantages and few disadvantages compared to the other alternatives and were therefore retained for further analysis. Below are brief descriptions for screening out alternatives 2,3, and 4.

Table 1.
Initial Screening of Alternatives Summary

Interchange Alternative	Advantages	Disadvantages
Preliminary Alternatives – Retained for Detailed Evaluation		
1 – Standard Diamond with Traffic Signals	1) Safe and efficient operation 2) Low construction cost 3) Accommodates multiple modes of transit 4) Accommodates bicycle/pedestrian travel 5) Adaptable 6) Consistent with driver expectancy	1) Higher right-of-way cost
5 – Diverging Diamond	1) Safe and efficient operation 2) Low construction cost 3) Accommodates multiple modes of transit 4) Lower right-of-way cost	1) Not consistent with driver expectancy 2) Not adaptable 3) Can be problematic for bicycles/pedestrians 4) Snow removal/maintenance challenges
Preliminary Alternatives – Not Retained for Detailed Evaluation		
2 – Standard Diamond with Roundabouts	1) Safe and efficient operation 2) Low construction cost 3) Accommodates multiple modes of transit 4) Adaptable 5) Consistent with driver expectancy	1) Less accommodating to bicycle/pedestrian users 2) Higher right-of-way cost 3) Snow removal/maintenance challenges 4) 3-4 lane roundabouts would be too large and not intuitive to drivers
3 – Partial Cloverleaf	1) Safe and efficient operation 2) Accommodates multiple modes of transit 3) Adaptable	1) Higher construction cost 2) Highest right-of-way cost 3) Additional environmental impacts 4) Can be problematic for bicycles/pedestrians
4 – Single Point Diamond (Urban)	1) Safe and efficient operation 2) Accommodates multiple modes of transit 3) Lower right-of-way cost	1) Higher construction cost 2) Not adaptable 3) Less consistent with driver expectancy 4) Snow removal/maintenance challenges 5) Can be problematic for bicycles/pedestrians

Screened Alternative 2 – Standard Diamond with Roundabouts

Traffic volume and lane balance was developed for the roundabout concept for the Mulberry interchange. The primary issue causing the roundabout to be screened out is the need to provide triple-left turns for NB to WB, and double left turns for EB to NB. These overlapping sections cause the overall roundabout size to be a mix between 3 and 4 circulating lanes. While it is possible to design and build a roundabout with these features, roundabouts of this size are not common enough – even in Colorado – to be considered a more acceptable alternative than the SDI or DDI.

Many roundabout interchanges in the United Kingdom that have 3 or 4 lanes are signalized on all approaches. In fact many roundabout interchanges in the UK consist of a single large circle with two separate bridges over the freeway, in addition to signalized entrances. Also, triple-lane roundabout approaches would definitely require pedestrian activated signal control to meet the most current Public Right-of-Way Accessability Guidelines (PROWAG).

The potential cost benefit of a roundabout interchange is that the bridge structure could be narrower than with the SDI. The lane balance exercise determined that a seven lane bridge would be required for an interchange with roundabouts, while a nine lane bridge is needed for the SDI and seven lanes for the DDI. If a single large circle were pursued, each bridge structure would be 3-4 lanes.

Screened Alternative 3 – Partial Cloverleaf, loops as on-ramps

The Partial Cloverleaf Interchange is typically considered when right-of-way impacts are not a key concern, and when the addition of loop ramps would provide significant improvement in capacity over a diamond interchange configuration.

The existing loop ramps at the SH 14 interchange are only 150 ft. radius, meeting a 25 mph design speed. A modern partial cloverleaf interchange would have loops meeting 30mph design criteria, so a minimum 231 ft. radius would be required. This higher radius would cause ROW impacts that would not exist with the SDI or DDI alternatives. In addition, the ramp intersection signals would now be within 500 ft. of the frontage road intersections. The underpass for the frontage road on the west side of I-25 would no longer be feasible due to the size of the loop ramp.

Bicycle and pedestrian accommodations are possible, however some safety issues arise due to the presence of unprotected crosswalks across the loop ramps. Based on the factors described above and the disadvantages when compared to other alternatives, Alternative 3 was not carried forward for further analysis.

Screened Alternative 4 – Single-Point Urban Interchange (SPUI)

The Single Point Urban Interchange (SPUI) is often used in urban locations where limited right-of-way is available. A SPUI can operate efficiently due to all approaches converging at a single intersection, however a volume imbalance on the off-ramps – as there is at SH 14, reduces that efficiency.

Construction costs are higher because of the size and complexity of the bridge structure required, and is further complicated by the requirement of the structure to clear-span I-25. Driver expectancy would be lower at a SPUI as they are uncommon on the I-25 corridor north of Denver.

Pedestrian and bicyclist accommodations are not ideal at a SPUI, as it is not possible to provide a crosswalk across the crossroad. Lack of adaptability is a shortfall of SPUI's due to their geometry; once constructed there is little potential for modification or expansion. Snow removal may also present issues because of the large area of pavement to be cleared.

Section 3: Detailed Evaluation of Alternatives

Upon completion of the preliminary screening process, two alternatives were selected for a more detailed evaluation:

- Alternative 1 – the Standard Diamond Interchange (SDI) as shown in the EIS
- Alternative 5 – the Diverging Diamond Interchange (DDI)

The criteria used in the analysis includes:

1. 2035 Traffic Operations
2. Conceptual Design Development
3. Vehicular Safety/Crash Data
4. Multimodal Operations and Safety
5. Capital Cost
6. ROW/Environmental Impacts
7. Vertical Alignment
8. Public Acceptability/Local Agency Support
9. Maintenance Considerations
10. Heavy Vehicle Operations
11. Constructability
12. Future Flexibility
13. Consistency with Current Planning Documents
14. Life Cycle Cost Analysis

These criteria were evaluated in a quantitative manner if sufficient data was present. Otherwise, a qualitative analysis was used for the criterion evaluation.

3.1 2035 Traffic Operations

2035 most recent forecast traffic volumes as desribed in Section 2 were used to analyze the projected traffic operations at the SH 14/Mulberry Street ramp terminal intersections. Operations at signalized intersections were analyzed at the AM and PM peak hours using the Highway Capacity Manual (HCM) 2000 methodology within Synchro. **Table 2** shows the traffic operations summary for the SDI and DDI configurations. **Appendix A** provides traffic analysis worksheets.

Table 2.
Level of Service Summary

	SDI				DDI			
	LOS		V/C Ratio		LOS		V/C Ratio	
	AM	PM	AM	PM	AM	PM	AM	PM
SB Ramps	C	A	0.71	0.73	C	B	0.71	0.68
NB Ramps	C	C	0.68	0.59	B	B	0.59	0.55

As shown in the table, both interchange types show good LOS and volume-to-capacity (V/C) ratios in each peak hour. It is noted that each interchange concept design was sized to fit the traffic volume and lane balance, so the DDI achieves good LOS similar to the SDI with fewer lanes (and less bridge width).

3.2 Conceptual Design Development

To assist in the detailed evaluation for the remaining alternatives, initial concept designs were created, including elements such as lane configurations and widths and approximate dimensions between ramps terminals and frontage roads. The concepts were also utilized in developing a truck turning analysis.

Figure 6 depicts the concept plan for Alternative 1 – Standard Diamond with Traffic Signals and **Figure 7** depicts Alternative 5 – Diverging Diamond Interchange.

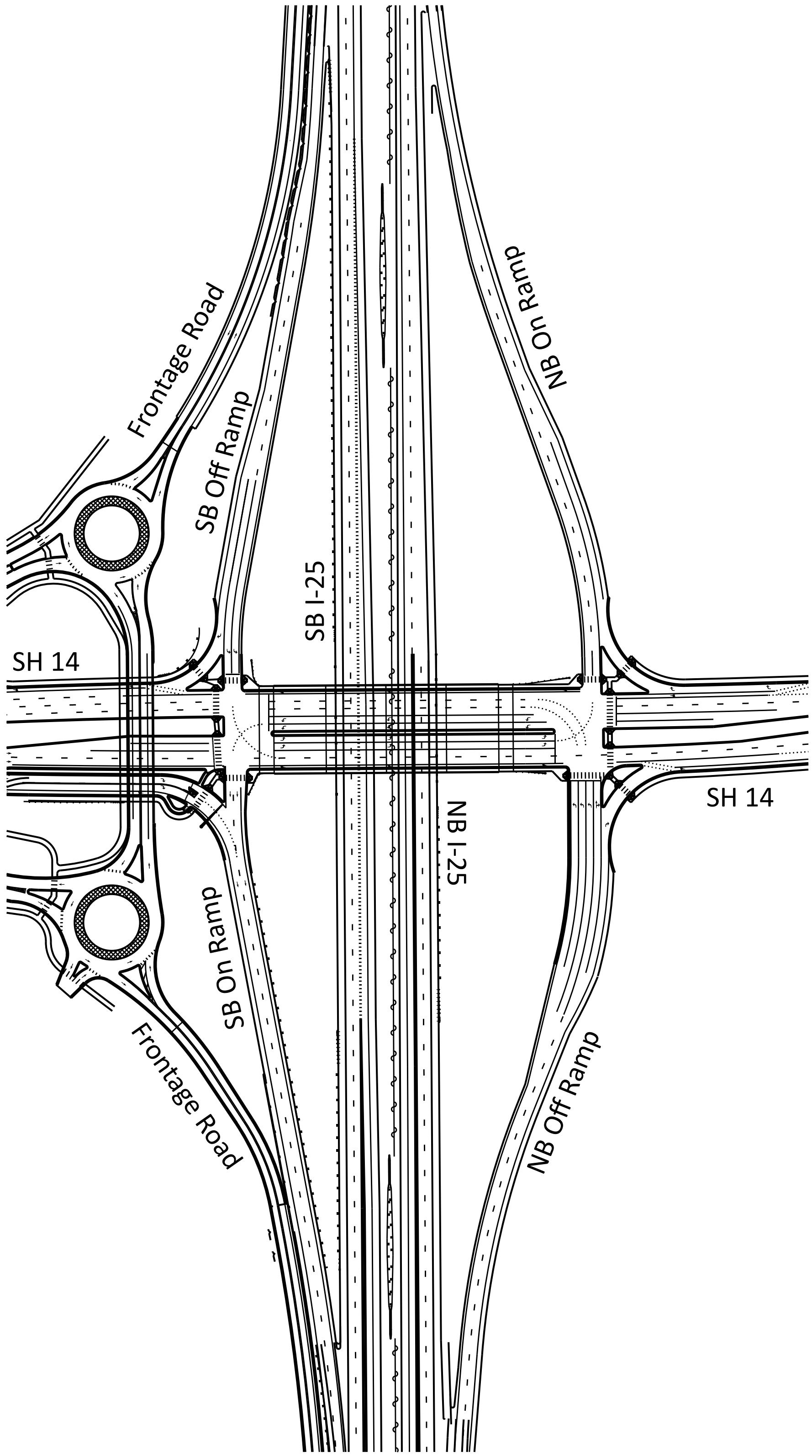


Figure 6.
Standard Diamond Interchange (Alternative 1) Concept

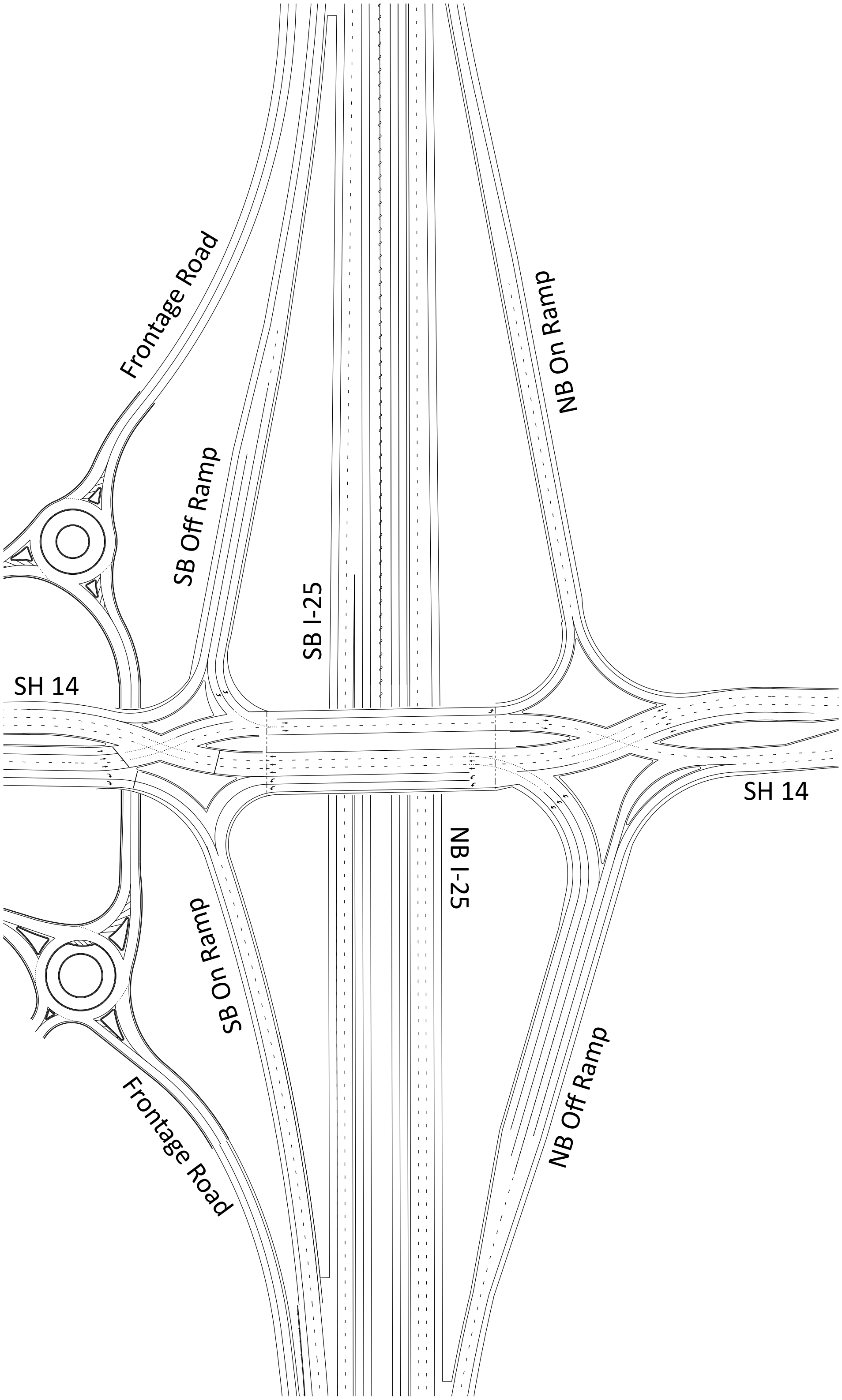


Figure 7.
Diverging Diamond Interchange (Alternative 5) Concept

3.2.1 Phase 1 Conceptual Design Development

During ongoing discussions with the City of Fort Collins, there were concerns brought up about the frontage road access on the west side of I-25. The evolution of this design development and decisions made post-EIS are documented in a Memorandum *Mulberry/I-25 and Prospect/I-25 Bike Lanes, Double Right Turn Lanes, and Mulberry Frontage Road access*, June 14, 2013, located in **Appendix D**. The largest concern in this area was the need in 2035 for double-right turns from EB SH 14 to SB I-25, and the subsequent need to signalize the egress from the south-west frontage road roundabout.

During re-evaluation of this area in 2016, it was determined that the need for double-right turns from EB SH 14 to SB I-25 is only necessary when and if Mulberry is widened to 6 lanes at least to Timberline Road. Without this widening, EB traffic on Mulberry is metered to a level where a single-right turn lane from EB SH 14 to SB I-25 will be adequate, and the egress from the Frontage Road roundabout could also remain free-flow with a weaving section. It was determined with Fort Collins that this would be an appropriate Phase 1 design for EB Mulberry, and that the roundabout design, ROW, and structures would be designed to accommodate the additional width needs if and when Mulberry is widened to six lanes to the west.

The Phase 1 design west of I-25 is shown in **Figure 8** below. It is noted that this design works with either the SDI or DDI design with only minor adjustments necessary to fit either design. The Phase 1 design is a better fit with the DDI design since the WB to SB left turn has more capacity and therefore a longer design life.

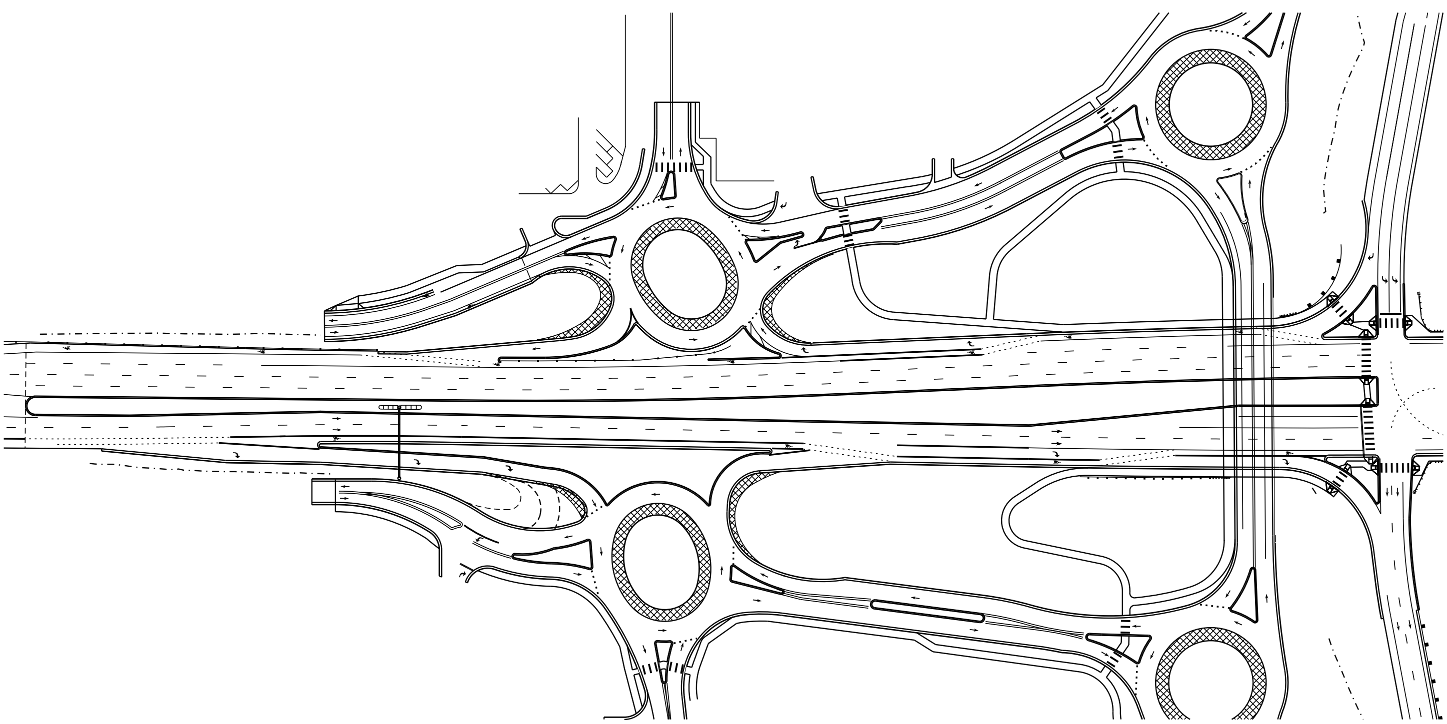


Figure 8.
SH 14 Interchange, Phase 1 Design for EB Approach

3.3 Vehicular Safety/Crash Data

According to the most recent CDOT crash rate book, published in 2012, the SH 14/Mulberry Street segment has a crash rate of 3.40 accidents per million vehicle miles travelled (VMT). The statewide average for state highways classified as principal arterial other is 2.76 accidents per million VMT, indicating that this segment of SH 14/Mulberry Street has an elevated crash rate compared to other similar highways. This higher crash rate is not unexpected due to the older design standard cloverleaf interchange in place since the 1960s. For a comprehensive analysis of vehicular safety and crash data throughout the I-25 corridor, refer to the FEIS.

This interchange analysis report used a more general approach to estimate safety performance between each interchange alternative through the use of comparable conflict diagrams. According to the *Highway Safety Manual* (HSM), one of the most frequent types of errors made by drivers at intersections is "improper lookout", or where drivers "looked but did not see" vehicles on conflicting paths. Each intersection was evaluated based on the number and type of "conflict points", where vehicle paths cross in direct conflict with each other. Conflict points can be classified into one of three types:

- Merging: A merging conflict occurs where two or more separate vehicle paths combine into a single lane in the same direction. Sideswipe and rear end type crashes may occur at these merging conflict points.
- Diverging: A diverging conflict occurs where a vehicle path in one lane separates into multiple paths turning onto different lanes. As vehicles typically slow down to make such maneuvers, rear end type crashes may occur at these diverging conflict points.
- Crossing: A crossing conflict occurs where two or more separate vehicle paths intersect but continue in separate lanes and/or directions. Broadside and approach turn type crashes may occur at crossing conflict points.

Crash severity is also an important metric to consider in safety performance. Crash severity can be described by the level of injury or property damage due to a crash. Conflict type and crash severity are correlated in that crossing conflict points generally experience more severe crashes due to increased vehicle angle and speed.

Table 3 summarizes the number and type of conflicts for each interchange alternative and **Figure 9** illustrates the locations of each conflict point for each interchange alternative.

Table 3.
Conflict Point Comparision by Interchange Alternative

Interchange Type	Merge Conflicts	Diverge Conflicts	Crossing Conflict	Total Conflicts
Standard Diamond with Traffic Signals	6	6	10	22
Diverging Diamond	6	6	2	14

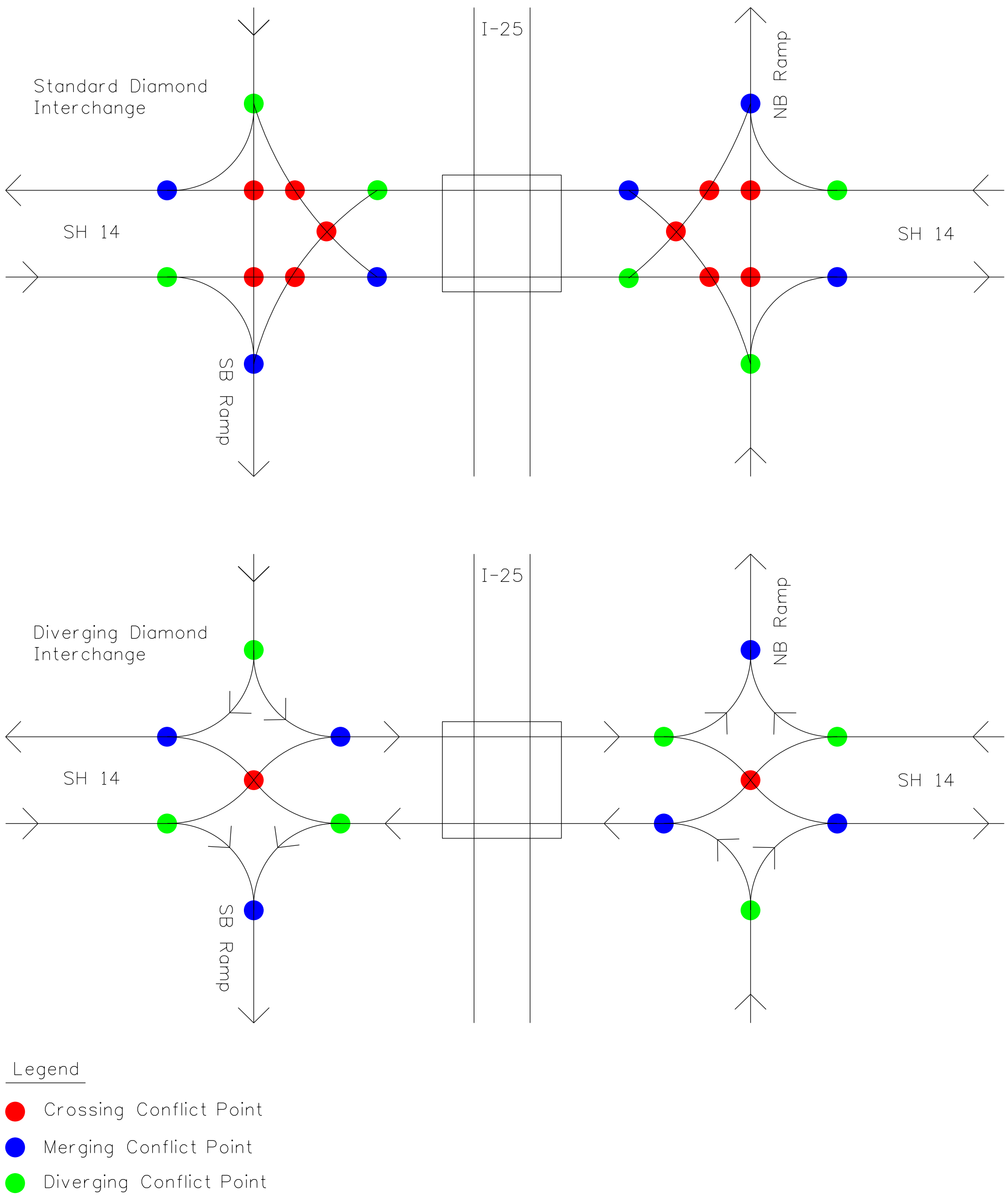


Figure 9.
I-25/SH 14 Interchange Alternative Conflict Diagrams

As shown, the DDI's combination of lower approach speeds at the intersection, fewer total number of conflict points, and improved LOS over the SDI would likely lead to fewer number of accidents, and less severity with the lower speed combination at the remaining conflict points.

3.4 Multimodal Operations and Safety

Alternative 1, the SDI, would have pedestrian and bicycle facilities typical of other SDI layouts, these can be seen in the design concepts in **Figure 6**. All of the crossings of multi-lane roadways would happen concurrent with signalized traffic movements, so there would be no pedestrian conflicts. All four right turn movements at the SDI are likely to be free-flow single right turn lanes, where it is intended that vehicles yield to pedestrians. The EB to SB right turn would open in Phase 1 as a single right turn but would ultimately be converted to a signalized double-right turn when Mulberry is widened to 6 lanes west of I-25.

In Alternative 5, the DDI, the right turn conflicts would be the same as described above for the SDI. If pedestrians remain on the outside of the bridge structure, the DDI may add a potential conflict of a pedestrian crossing a free-flow single left turn lane in two locations, EB to NB and WB to SB. The WB to SB left turn would ultimately be signalized when Mulberry is widened to 6 lanes west of I-25. This conflict is eliminated if a single pedestrian path is created in the center of the bridge structure, which is a typical treatment possible with DDIs.

The on-street bicycle lane along SH 14 with an SDI and DDI would be similar – adjacent to the right-hand outside through lane. This makes bicycle safety relatively equal between the two interchange types. It is noted that the width of the intersections on the DDI is wider than the SDI, so the exposure time for bicycles in the intersection is longer with the DDI.

The FEIS also includes an express bus system running along I-25, and a carpool lot would be constructed northeast of the interchange (outside of the primary interchange area), providing a proposed 150 parking spaces in the Preferred Alternative. Because of the location outside of the interchange area, the type of interchange will not affect safety.

3.5 Capital Cost

While Section 3.2 shows conceptual layouts of the two interchange alternatives, the designs were not developed to a level that a full interchange cost would reasonably be calculated. The most appropriate way to compare costs for the two interchange alternatives is to look at comparable items that change substantially with each interchange type. For example, the crossroad approach roadways and the four ramps are all about the same for the two alternatives in terms of pavement, earthwork, signing, etc. The key items that can have substantial differences between the two alternatives are:

- Bridge structure width (due to number of lanes required). All bridge span lengths over I-25 would be the same due to the I-25 cross section requirements.
- Earthwork fill (due to vertical profile differences). The SDI is being designed for a 50mph vertical curve while the DDI could be designed with a 40mph crest vertical curve.
- Signalization and signing

Right-of-way area and cost could also be a differentiator between the alternatives, however it was previously determined that the ROW footprint from the EIS which accommodates an SDI would be acquired for this project regardless of the interchange type within the footprint.

Table 4 below shows a comparison of quantifies and costs for the items above.

	Standard Diamond Interchange				Diverging Diamond Interchange			
	Unit	Unit Cost	Quantity	Total	Unit	Unit Cost	Quantity	Total
Earthwork	CY	$ 10	52,700	$ 527,000	CY	$ 10	-	$ -
Paving	SF	$ 4.50	545,000	$ 2,452,500	SF	$ 4.50	567,000	$ 2,551,500
Structures	SF	$ 150	46,248	$ 6,937,200	SF	$ 150	38,376	$ 5,756,400
Traffic signals	Each	$ 140,000	2	$ 280,000	Each	$ 140,000	2	$ 280,000
Signing/Striping	-	-	-	$ -	4 OH signs	-	-	$ 200,000
			Subtotal	$ 10,196,700			Subtotal	$ 8,787,900
x 2.0 for all project elements (Design, CE, FA, drainage, walls, other unlisted items)			**Total**	**$ 20,393,400**			**Total**	**$ 17,575,800**

Table 4.
Concept Level Cost Comparison

As shown in the table above, the reduced bridge structure width and the reduced fill/embankment of the DDI can lower the cost of the interchange by approximately $2.8 million. If considered over a base cost of approximately $40 Million, this cost savings would be about 7%.

3.6 ROW/Environmental Impacts

The amount of right-of-way required for Alternative 1 falls within the footprint of the FEIS Preferred Alternative. Environmental impacts are insignificant, as the surrounding land uses do not require relocation or impacts to existing businesses or properties.

The DDI has the advantage of more flexibility with the ramp alignment approaching the intersection, which would generally yield a smaller ROW footprint. However at the SH 14 location, the extra ROW from the former cloverleaf means that ROW is not an issue at this location.

3.7 Vertical Alignment

Alternative 1, SDI, would be designed to have a 50mph crest vertical curve over I-25. This is a typical design speed for arterial crest curves, even though the posted speed will likely remain at 40mph. The 50mph crest curve provides an extra buffer for stopping sight distance.

Alternative 5, DDI, would likely be designed with a 40mph crest vertical curve. The DDI geometry is controlled by the horizontal curves of the crossovers on either side of I-25. Many DDIs designed for new structures are designed with 30mph horizontal curves, with the potential to go to 35mph if there is generous space available to lay out the interchange. DDIs being fit onto existing structures or in areas with existing constraints have design speeds in the 20mph-25mph range.

The SH 14 DDI concept shown in **Figure 7** was designed just above 30mph horizontal design criteria to fit within the constraints of the adjacent frontage road bridge on the west and to maintain spacing to the east frontage road signal. Therefore a 40mph crest vertical curve would be the assumed design speed for the DDI.

Figure 10 below shows the comparison of vertical profiles for SH 14 between 50mph and 40mph crest curve designs. The 40mph design requires 52,700 Cu. Yd. less of embankment fill than the 50mph design.

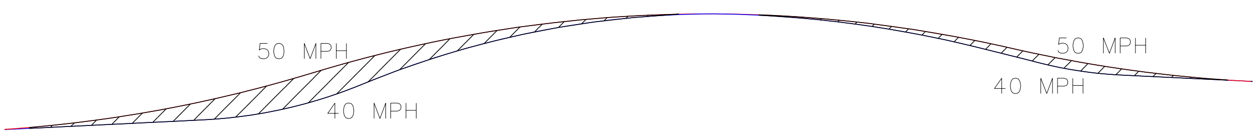


Figure 10.
40 mph vs. 50 mph Crest Vertical Curve Profile

3.8 Public Acceptability/Local Agency Support

The SDI and DDI concepts were presented to City of Fort Collins Engineering staff and CDOT staff in the early stages of the project design during 2012 and 2013. The comparison matrix of the SDI and DDI attributes were presented and discussed. Both Ft. Collins and CDOT staff were supportive of the DDI design, with most questions geared toward bicycle/pedestrian accommodation and maintenance vehicle (snowplows making through movements on the ramps). At the time it was determined to carry forward with the SDI design as the base case since it is more conservative and has a larger footprint for ROW reservation.

The DDI concept has not yet been discussed in any of the public meetings for the project. These public meetings so far have focused on the business access and ROW impacts to the existing businesses surrounding the interchange.

3.9 Maintenance Considerations

The SDI is the basis for comparison when looking at the different interchange types for short-term and long-term maintenance items.

Snow Removal:

CDOT maintenance crews generally drive ramp-to-ramp when removing snow from the freeway ramps. This is not possible at a DDI unless special provisions are made to accommodate snowplows over/around islands.

Signals and Signing:
An SDI is unlikely to have overhead signing since the layout is typical and understood by most motorists. Most urban DDI interchanges in Colorado have added overhead lane use signs within the interchange to provide additional guidance for motorists in these newer interchange types. This additional infrastructure would add to the maintenance burden for CDOT.

DDIs generally include more lighting infrastructure to better light the conflict areas, which are more spread-out than at SDIs.

3.10 Heavy Vehicle Operations
SH 14/Mulberry Street experiences a high percentage of truck traffic since SH 14 to the west connects to US 287, offering a short-cut route to western Wyoming and I-80. Each of the two alternatives would be able to accommodate the volume of heavy vehicles, however.

The DDI design provides for smoother left turn movement paths than the SDI, and similar right turn paths comparing SDI and DDI.

3.11 Constructability
The reconstruction of the Mulberry/SH 14 interchange will result in a substantial change to the horizontal and vertical layout of the interchange. Both the SDI and the DDI are of similar size and shape when considering construction phasing opportunities and impacts. The new bridge width over I-25 is substantially wider than the existing bridge, so the phasing approach will be a relatively straightforward half-half for the structure and the SH 14 mainline.

The only real difference between SDI and DDI for constructability will likely be the additional temporary pavement necessary as the project is phased from an existing cloverleaf - to a temporary diamond - to a final DDI.

3.12 Future Flexibility
The traffic forecasts and the traffic operations analysis showed that the DDI had 4.5% more reserve capacity than the SDI. The future flexibility to expand either interchange alternative is most likely related to the structure over I-25, and the ability to either widen the structure or revise the number of lanes within the initially built structure.

The SDI structure over I-25 requires 9 lanes, and the DDI structure only requires 7 lanes. So while the DDI configuration provides more capacity with fewer lanes, the SDI structure is inherently more flexible due to the additional structure width.

3.13 Consistency with Current Planning Documents

The North I-25 FEIS had the default approach of including an SDI as the assumed design in the preferred alternative. In the 2008-2010 timeframe the DDI did not yet have the same national acceptance as it now does, so it was likely not considered within the EIS document.

The SDI or DDI do not have any changes with how the ramps interact with I-25, only how the traffic control is handled at the ramp/crossroad intersection. Similar projects that focus on the ramp/crossroad intersections are usually treated as a minor interchange modification, so it is unlikely that the use of a DDI would represent a change that would be in conflict with the approved planning documents.

An IAR will ultimately be required for either the SDI or DDI configuration to be implemented, since either is a substantial change to the current partial cloverleaf design. However either design would likely meet the necessary criteria and ultimately gain approval.

3.14 Life Cycle Cost Analysis

An important part of evaluating any alternative involves determining which interchange configuration provides the most cost-effective solution. With that in mind, a life cycle cost analysis was prepared to compare the two alternatives, taking into consideration cost such as right-of-way and construction, design and construction management, environmental document revisions, annual pavement and signal maintenance, and annual user travel time costs for ramp terminal intersections. Cost estimates were based on a lifespan of 25 years and are in 2016 dollars. A breakdown of each cost component by interchange alternative is below in **Table 5**.

Table 5.
Life Cycle Cost Analysis

Interchange Alternative	Design/ Construction	Environmental Document Revisions	25-Year Pavement O&M (1)	25-Year Signal O&M (2)	25-Year Value of Time Cost (3)	Total Cost
1 - Standard Diamond	$ 50,400,000	$ -	$ 2,725,000	$ 250,000	$ 13,865,000	$ 67,240,000
5 - Diverging Diamond	$ 47,200,000	$ 50,000	$ 2,835,000	$ 250,000	$ 10,575,000	$ 60,910,000

Notes:
1. Assume $0.20/SF/year
2. Assume $5,000/signal/year
3. Assume $21/hour of travel time delay costs (from *TTI 2012 Urban Mobility Report* ($16.79/hr*1.25 vehicle occupancy)). See **Appendix B**.
4. All amount are in 2016 dollars

To summarize Section 3, **Table 6** on the following page provides a review of the criteria used in the detailed evaluation of interchange alternatives. The analysis categories where one interchange type is clearly superior to the other are highlighted in green.

Table 6.
Final Alternatives Evaluation Summary

	Standard Diamond Interchange (SDI)	Diverging Diamond Interchange (DDI)
3.1 – 2035 Traffic Operations		
Signal Phasing	Standard 3-phase signals, protected lefts for all movements. Min. cycle length = about 90 sec.	2-phase signals, some lower volume left turn movements could be free-lefts. Min. cycle length = about 50 sec.
Signal Coordination	Likely full cycle lengths (100-120 seconds) needed during peak periods due to adjacent frontage road phasing	Interchange signals could half-cycle (50-60 seconds) even during peak periods, reducing queuing at interchange
3.2 – Conceptual Design	Design fits anticipated ROW, horizontal and vertical constraints, and provides reasonable Phase 1 and ultimate design	Design fits anticipated ROW, horizontal and vertical constraints, and provides reasonable Phase 1 and ultimate design
West-side Frontage Road Accommodation	Same as anticipated with EIS design, Frontage Road structure is just west of ramp intersection	Same as anticipated with EIS design, Frontage Road structure is just west of ramp intersection but would be slightly longer than with SDI due to SH 14 curves
3.3 – Vehicle Safety	6 approach conflicts per intersection. More potential for higher-speed angled collisions. More potential for wrong-way turn onto freeway off-ramp	4 approach conflicts per intersection at lower speeds due to geometry. Main intersection is skewed, but all traffic goes straight at skewed crossing. Wrong way turn onto freeway ramp virtually impossible. Need additional signing on approach road, particularly Mulberry, to bring speed limit down to 30 or 35 from 45 or 55
3.4 – Multimodal Safety	Standard layout for diamond interchanges, most crossings signal protected. Four pedestrian crossings of free-flow right turns	Four pedestrian crossings of free-flow right turns
Bicycles	Right-hand lane adjacent to travel lanes per Ft. Collins preference	Right-hand lane adjacent to travel lanes per Ft. Collins preference. Need to provide good striping thru wide intersection
Pedestrians	Per Ft. Collins, 6 ft. sidewalks on each side of the bridge with 6 ft. separation from driving lane (bike lane). Decision for 40mph speed limit allows no barrier between sidewalk & lanes. Four pedestrian crossings of free-flow right turns	Option for single sidewalk (assume 8 ft.) along inside of one of the two bridges – between opposing traffic flows. Same separation of pedestrians to traffic as SDI. More difficult to convey proper travel direction to visually impaired pedestrians due to angled/non-intuitive travel paths. Four pedestrian crossings of free-flow right turns
3.5 – Capital Cost	$20,393,400	$17,575,800

	Standard Diamond Interchange (SDI)	Diverging Diamond Interchange (DDI)
3.6 – ROW/Environmental	Fits mostly within old cloverleaf ROW, with some narrow acquisitions for the ramps near the gore areas. Fits within area cleared in the EIS	Fits mostly within old cloverleaf ROW, with some narrow acquisitions for the ramps near the gore areas. Fits within area cleared in the EIS
Ramp geometrics	Desirable to bring ramp to intersection with crossroad at or near perpendicular. Requires more area/ROW for ramps	More flexibility with angle of ramp approaching crossroad, since all traffic turns the angle of ramp approach is more flexible. Reduces land area needed for interchange
3.7 – Vertical Alignment	Standard design with 50mph crossroad crest vertical curve over I-25	Potential to lower crest vertical curve design speed to 40-45 mph since horizontal design speed is controlled by 30-35 mph crossovers
3.8 – Public/Local Agency Support	SDI was shown in all EIS public meetings and follow up meetings for design in mid-2013. Design accepted by both CDOT and Fort Collins staff	DDI has not been shown in previous EIS public meetings or local business owner meetings. The DDI concept was considered acceptable by both CDOT and Fort Collins staff
3.9 – Maintenance Considerations	Snowplows often continue straight from ramp-to-ramp, which is accommodated at SDI	DDI would need special lane or drive-over island for ramp-to-ramp to continue, or plowing procedures would need to be revised
3.10 – Heavy Vehicle Operations	All left turns can meet design criteria, but tighter radius causes truck turning paths to widen to over 20 ft., challenging at the triple lefts	Smoother left turn paths allow lane widths to stay closer to 12 ft., even with the triple-lefts
3.11 – Constructability	Typical phased half-half construction as used at other interchanges in the corridor	Typical phased half-half construction as used at other interchanges in the corridor, additional temp paving to operate as temporary SDI until DDI geometry is completed
3.12 – Future Flexibility	Design would be 68% "full" with 2035 traffic. 9-lane bridge has greater flexibility for reconfiguration	Design would be 63% "full" with 2035 traffic, so more capacity with a smaller bridge section. 7-lane bridge has less flexibility for reconfiguration
3.13 – Consistency with Current Planning Documents	Consistent with EIS	Consistent with EIS interchange footprint, with modified intersection traffic control
3.14 – Life Cycle Cost Analysis	$67,240,000	$60,910,000

Section 4: Summary of Findings and Recommendations

The analysis in this memorandum found that both the SDI and the DDI configurations for the SH 14 / I-25 interchange are viable. It was previously determined that for preliminary design the SDI configuration would be included since it results in a larger footprint, and therefore a conservative ROW footprint.

The DDI configuration has several distinct advantages over the SDI in terms of initial implementation cost, and therefore a DDI is likely to be proposed as an alternative delivery approach, assuming that alternative delivery is used.

An Interchange Access Request (IAR) still needs to be processed through FHWA to get final approval for either an SDI or DDI, but the IAR will not likely be pursued until there is reasonable certainty that funding for construction will be available. It is likely that either alternative will be acceptable to FHWA pending the outcome of the full IAR analysis. It is recommended that CDOT portray both the SDI and DDI as acceptable options in future public meetings and jurisdiction meetings for this project.

Appendix A

Traffic Analysis Worksheets

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (vph)	0	615	988	319	2542	0	0	0	0	101	0	458
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)		6.0	6.0	6.0	6.0					6.0	6.0	4.0
Lane Util. Factor		0.81	0.88	0.97	0.91					0.95	0.95	1.00
Frt		1.00	0.85	1.00	1.00					1.00	1.00	0.85
Flt Protected		1.00	1.00	0.95	1.00					0.95	0.95	1.00
Satd. Flow (prot)		7544	2787	3433	5085					1681	1681	1583
Flt Permitted		1.00	1.00	0.95	1.00					0.95	0.95	1.00
Satd. Flow (perm)		7544	2787	3433	5085					1681	1681	1583
Peak-hour factor, PHF	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Adj. Flow (vph)	0	647	1040	336	2676	0	0	0	0	106	0	482
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	0	647	1040	336	2676	0	0	0	0	53	53	482
Turn Type		NA	Perm	Prot	NA					Split	NA	Free
Protected Phases		4		3	8					6	6	
Permitted Phases			4									Free
Actuated Green, G (s)		83.1	83.1	8.0	97.1					10.9	10.9	120.0
Effective Green, g (s)		83.1	83.1	8.0	97.1					10.9	10.9	120.0
Actuated g/C Ratio		0.69	0.69	0.07	0.81					0.09	0.09	1.00
Clearance Time (s)		6.0	6.0	6.0	6.0					6.0	6.0	
Vehicle Extension (s)		3.0	3.0	3.0	3.0					3.0	3.0	
Lane Grp Cap (vph)		5224	1929	228	4114					152	152	1583
v/s Ratio Prot		0.09		c0.10	c0.53					0.03	0.03	
v/s Ratio Perm			0.37									c0.30
v/c Ratio		0.12	0.54	1.47	0.65					0.35	0.35	0.30
Uniform Delay, d1		6.2	9.1	56.0	4.6					51.2	51.2	0.0
Progression Factor		0.76	0.89	0.84	1.77					1.00	1.00	1.00
Incremental Delay, d2		0.0	0.3	231.6	0.3					6.2	6.2	0.5
Delay (s)		4.7	8.3	278.3	8.5					57.4	57.4	0.5
Level of Service		A	A	F	A					E	E	A
Approach Delay (s)		6.9			38.6			0.0			10.8	
Approach LOS		A			D			A			B	

Intersection Summary			
HCM 2000 Control Delay	25.4	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.71		
Actuated Cycle Length (s)	120.0	Sum of lost time (s)	18.0
Intersection Capacity Utilization	65.3%	ICU Level of Service	C
Analysis Period (min)	15		
c Critical Lane Group			

11: I-25 NB OFF RAMP/I-25 NB ON RAMP & MULBERRY ST

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (vph)	360	649	0	0	1190	102	1352	0	186	0	0	0
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	6.0	6.0			6.0	6.0	6.0	6.0	6.0			
Lane Util. Factor	0.97	0.91			0.81	1.00	0.91	0.86	0.95			
Frt	1.00	1.00			1.00	0.85	1.00	0.99	0.85			
Flt Protected	0.95	1.00			1.00	1.00	0.95	0.95	1.00			
Satd. Flow (prot)	3433	5085			7544	1583	3221	1519	1504			
Flt Permitted	0.95	1.00			1.00	1.00	0.95	0.95	1.00			
Satd. Flow (perm)	3433	5085			7544	1583	3221	1519	1504			
Peak-hour factor, PHF	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Adj. Flow (vph)	379	683	0	0	1253	107	1423	0	196	0	0	0
RTOR Reduction (vph)	0	0	0	0	0	81	0	45	90	0	0	0
Lane Group Flow (vph)	379	683	0	0	1253	26	968	430	86	0	0	0
Turn Type	Prot	NA			NA	Perm	Prot	NA	Perm			
Protected Phases	7	4			8		5	2				
Permitted Phases						8			2			
Actuated Green, G (s)	18.2	53.5			29.3	29.3	54.5	54.5	54.5			
Effective Green, g (s)	18.2	53.5			29.3	29.3	54.5	54.5	54.5			
Actuated g/C Ratio	0.15	0.45			0.24	0.24	0.45	0.45	0.45			
Clearance Time (s)	6.0	6.0			6.0	6.0	6.0	6.0	6.0			
Vehicle Extension (s)	3.0	3.0			3.0	3.0	3.0	3.0	3.0			
Lane Grp Cap (vph)	520	2267			1841	386	1462	689	683			
v/s Ratio Prot	c0.11	0.13			c0.17		c0.30	0.28				
v/s Ratio Perm						0.02			0.06			
v/c Ratio	0.73	0.30			0.68	0.07	0.66	0.62	0.13			
Uniform Delay, d1	48.5	21.3			41.1	34.9	25.6	25.0	19.0			
Progression Factor	0.84	0.93			0.44	0.27	1.00	1.00	1.00			
Incremental Delay, d2	5.1	0.1			0.7	0.0	1.1	1.8	0.4			
Delay (s)	46.0	19.9			18.6	9.4	26.7	26.7	19.3			
Level of Service	D	B			B	A	C	C	B			
Approach Delay (s)		29.2			17.9			25.9			0.0	
Approach LOS		C			B			C			A	

Intersection Summary			
HCM 2000 Control Delay	24.1	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.68		
Actuated Cycle Length (s)	120.0	Sum of lost time (s)	18.0
Intersection Capacity Utilization	65.3%	ICU Level of Service	C
Analysis Period (min)	15		
c Critical Lane Group			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (vph)	0	1149	1408	172	1836	0	0	0	0	94	0	416
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)		6.0	6.0	6.0	6.0					6.0	6.0	4.0
Lane Util. Factor		0.81	0.88	0.97	0.91					0.95	0.95	1.00
Frt		1.00	0.85	1.00	1.00					1.00	1.00	0.85
Flt Protected		1.00	1.00	0.95	1.00					0.95	0.95	1.00
Satd. Flow (prot)		7544	2787	3433	5085					1681	1681	1583
Flt Permitted		1.00	1.00	0.95	1.00					0.95	0.95	1.00
Satd. Flow (perm)		7544	2787	3433	5085					1681	1681	1583
Peak-hour factor, PHF	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Adj. Flow (vph)	0	1209	1482	181	1933	0	0	0	0	99	0	438
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	0	1209	1482	181	1933	0	0	0	0	49	50	438
Turn Type		NA	Perm	Prot	NA					Split	NA	Free
Protected Phases		4		3	8					6	6	
Permitted Phases			4									Free
Actuated Green, G (s)		83.4	83.4	8.0	97.4					10.6	10.6	120.0
Effective Green, g (s)		83.4	83.4	8.0	97.4					10.6	10.6	120.0
Actuated g/C Ratio		0.70	0.70	0.07	0.81					0.09	0.09	1.00
Clearance Time (s)		6.0	6.0	6.0	6.0					6.0	6.0	
Vehicle Extension (s)		3.0	3.0	3.0	3.0					3.0	3.0	
Lane Grp Cap (vph)		5243	1936	228	4127					148	148	1583
v/s Ratio Prot		0.16		c0.05	0.38					0.03	0.03	
v/s Ratio Perm			c0.53									c0.28
v/c Ratio		0.23	0.77	0.79	0.47					0.33	0.34	0.28
Uniform Delay, d1		6.6	11.9	55.2	3.4					51.4	51.4	0.0
Progression Factor		0.75	0.88	0.84	1.54					1.00	1.00	1.00
Incremental Delay, d2		0.0	1.7	15.9	0.1					5.9	6.1	0.4
Delay (s)		5.0	12.2	62.5	5.3					57.3	57.5	0.4
Level of Service		A	B	E	A					E	E	A
Approach Delay (s)		9.0			10.2			0.0			10.9	
Approach LOS		A			B			A			B	

Intersection Summary			
HCM 2000 Control Delay	9.7	HCM 2000 Level of Service	A
HCM 2000 Volume to Capacity ratio	0.73		
Actuated Cycle Length (s)	120.0	Sum of lost time (s)	18.0
Intersection Capacity Utilization	72.5%	ICU Level of Service	C
Analysis Period (min)	15		
c Critical Lane Group			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (vph)	442	1243	0	0	769	91	1067	0	249	0	0	0
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	6.0	6.0			6.0	6.0	6.0	6.0	6.0			
Lane Util. Factor	0.97	0.91			0.81	1.00	0.91	0.86	0.95			
Frt	1.00	1.00			1.00	0.85	1.00	0.99	0.85			
Flt Protected	0.95	1.00			1.00	1.00	0.95	0.96	1.00			
Satd. Flow (prot)	3433	5085			7544	1583	3221	1515	1504			
Flt Permitted	0.95	1.00			1.00	1.00	0.95	0.96	1.00			
Satd. Flow (perm)	3433	5085			7544	1583	3221	1515	1504			
Peak-hour factor, PHF	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Adj. Flow (vph)	465	1308	0	0	809	96	1123	0	262	0	0	0
RTOR Reduction (vph)	0	0	0	0	0	78	0	41	41	0	0	0
Lane Group Flow (vph)	465	1308	0	0	809	18	764	344	195	0	0	0
Turn Type	Prot	NA			NA	Perm	Prot	NA	Perm			
Protected Phases	7	4			8		5	2				
Permitted Phases						8			2			
Actuated Green, G (s)	20.1	48.0			21.9	21.9	60.0	60.0	60.0			
Effective Green, g (s)	20.1	48.0			21.9	21.9	60.0	60.0	60.0			
Actuated g/C Ratio	0.17	0.40			0.18	0.18	0.50	0.50	0.50			
Clearance Time (s)	6.0	6.0			6.0	6.0	6.0	6.0	6.0			
Vehicle Extension (s)	3.0	3.0			3.0	3.0	3.0	3.0	3.0			
Lane Grp Cap (vph)	575	2034			1376	288	1610	757	752			
v/s Ratio Prot	c0.14	c0.26			0.11		c0.24	0.23				
v/s Ratio Perm						0.01			0.13			
v/c Ratio	0.81	0.64			0.59	0.06	0.47	0.45	0.26			
Uniform Delay, d1	48.1	29.1			44.9	40.5	19.7	19.4	17.2			
Progression Factor	0.84	0.92			0.55	0.71	1.00	1.00	1.00			
Incremental Delay, d2	8.2	0.7			0.6	0.1	0.2	0.4	0.8			
Delay (s)	48.7	27.5			25.5	28.7	19.9	19.8	18.1			
Level of Service	D	C			C	C	B	B	B			
Approach Delay (s)		33.0			25.9			19.6			0.0	
Approach LOS		C			C			B			A	

Intersection Summary			
HCM 2000 Control Delay	26.8	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.59		
Actuated Cycle Length (s)	120.0	Sum of lost time (s)	18.0
Intersection Capacity Utilization	72.5%	ICU Level of Service	C
Analysis Period (min)	15		
c Critical Lane Group			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (vph)	0	980	988	319	2220	0	0	0	101	0	0	458
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)		4.0	4.0	4.0	4.0				4.0			4.0
Lane Util. Factor		0.91	1.00	1.00	0.91				0.88			1.00
Frt		1.00	0.85	1.00	1.00				0.85			0.86
Flt Protected		1.00	1.00	0.95	1.00				1.00			1.00
Satd. Flow (prot)		5085	1583	1770	5085				2787			1611
Flt Permitted		1.00	1.00	0.95	1.00				1.00			1.00
Satd. Flow (perm)		5085	1583	1770	5085				2787			1611
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	0	1065	1074	347	2413	0	0	0	110	0	0	498
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	0	1065	1074	347	2413	0	0	0	110	0	0	498
Turn Type	Split	NA	Free	Split	NA				Over			Free
Protected Phases	2	2		6	6				6			
Permitted Phases			Free									Free
Actuated Green, G (s)		44.0	120.0	68.0	68.0				68.0			120.0
Effective Green, g (s)		44.0	120.0	68.0	68.0				68.0			120.0
Actuated g/C Ratio		0.37	1.00	0.57	0.57				0.57			1.00
Clearance Time (s)		4.0		4.0	4.0				4.0			
Vehicle Extension (s)		3.0		3.0	3.0				3.0			
Lane Grp Cap (vph)		1864	1583	1003	2881				1579			1611
v/s Ratio Prot		0.21		0.20	c0.47				0.04			
v/s Ratio Perm			c0.68									0.31
v/c Ratio		0.57	0.68	0.35	0.84				0.07			0.31
Uniform Delay, d1		30.4	0.0	14.0	21.4				11.7			0.0
Progression Factor		1.00	1.00	1.01	0.91				1.00			1.00
Incremental Delay, d2		1.3	2.4	0.2	1.9				0.0			0.5
Delay (s)		31.7	2.4	14.3	21.4				11.7			0.5
Level of Service		C	A	B	C				B			A
Approach Delay (s)		17.0			20.5			11.7			0.5	
Approach LOS		B			C			B			A	

Intersection Summary			
HCM 2000 Control Delay	17.2	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.80		
Actuated Cycle Length (s)	120.0	Sum of lost time (s)	8.0
Intersection Capacity Utilization	52.9%	ICU Level of Service	A
Analysis Period (min)	15		
c Critical Lane Group			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (vph)	0	649	360	0	1190	102	0	0	186	0	0	1352
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)		4.0	4.0		4.0	4.0			4.0			4.0
Lane Util. Factor		0.95	1.00		0.86	1.00			1.00			0.76
Frt		1.00	0.85		1.00	0.85			0.86			1.00
Flt Protected		1.00	1.00		1.00	1.00			1.00			1.00
Satd. Flow (prot)		3539	1583		6408	1583			1611			4247
Flt Permitted		1.00	1.00		1.00	1.00			1.00			1.00
Satd. Flow (perm)		3539	1583		6408	1583			1611			4247
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	0	705	391	0	1293	111	0	0	202	0	0	1470
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	0	705	391	0	1293	111	0	0	202	0	0	1470
Turn Type	Split	NA	Free	Split	NA	Free			Free			Over
Protected Phases	2	2		6	6							2
Permitted Phases			Free			Free			Free			
Actuated Green, G (s)		75.7	120.0		36.3	120.0			120.0			75.7
Effective Green, g (s)		75.7	120.0		36.3	120.0			120.0			75.7
Actuated g/C Ratio		0.63	1.00		0.30	1.00			1.00			0.63
Clearance Time (s)		4.0			4.0							4.0
Vehicle Extension (s)		3.0			3.0							3.0
Lane Grp Cap (vph)		2232	1583		1938	1583			1611			2679
v/s Ratio Prot		0.20			c0.20							c0.35
v/s Ratio Perm			0.25			0.07			0.13			
v/c Ratio		0.32	0.25		0.67	0.07			0.13			0.55
Uniform Delay, d1		10.2	0.0		36.6	0.0			0.0			12.5
Progression Factor		0.26	1.00		0.68	1.00			1.00			1.00
Incremental Delay, d2		0.3	0.3		0.5	0.0			0.2			0.8
Delay (s)		2.9	0.3		25.4	0.0			0.2			13.3
Level of Service		A	A		C	A			A			B
Approach Delay (s)		2.0			23.4			0.2			13.3	
Approach LOS		A			C			A			B	

Intersection Summary			
HCM 2000 Control Delay	13.1	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.59		
Actuated Cycle Length (s)	120.0	Sum of lost time (s)	8.0
Intersection Capacity Utilization	55.4%	ICU Level of Service	B
Analysis Period (min)	15		
c Critical Lane Group			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (vph)	0	1149	1408	172	1836	0	0	0	94	0	0	416
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)		4.0	4.0	4.0	4.0				4.0			4.0
Lane Util. Factor		0.91	1.00	1.00	0.91				0.88			1.00
Frt		1.00	0.85	1.00	1.00				0.85			0.86
Flt Protected		1.00	1.00	0.95	1.00				1.00			1.00
Satd. Flow (prot)		5085	1583	1770	5085				2787			1611
Flt Permitted		1.00	1.00	0.95	1.00				1.00			1.00
Satd. Flow (perm)		5085	1583	1770	5085				2787			1611
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	0	1249	1530	187	1996	0	0	0	102	0	0	452
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	0	1249	1530	187	1996	0	0	0	102	0	0	452
Turn Type	Split	NA	Free	Split	NA				Over			Free
Protected Phases	2	2		6	6				6			
Permitted Phases			Free									Free
Actuated Green, G (s)		45.6	120.0	66.4	66.4				66.4			120.0
Effective Green, g (s)		45.6	120.0	66.4	66.4				66.4			120.0
Actuated g/C Ratio		0.38	1.00	0.55	0.55				0.55			1.00
Clearance Time (s)		4.0		4.0	4.0				4.0			
Vehicle Extension (s)		3.0		3.0	3.0				3.0			
Lane Grp Cap (vph)		1932	1583	979	2813				1542			1611
v/s Ratio Prot		0.25		0.11	0.39				0.04			
v/s Ratio Perm			c0.97									0.28
v/c Ratio		0.65	0.97	0.19	0.71				0.07			0.28
Uniform Delay, d1		30.6	0.0	13.4	19.7				12.4			0.0
Progression Factor		1.00	1.00	0.67	0.57				1.00			1.00
Incremental Delay, d2		1.7	15.9	0.1	0.8				0.0			0.4
Delay (s)		32.3	15.9	9.1	12.0				12.4			0.4
Level of Service		C	B	A	B				B			A
Approach Delay (s)		23.3			11.8			12.4			0.4	
Approach LOS		C			B			B			A	

Intersection Summary			
HCM 2000 Control Delay	16.7	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	1.04		
Actuated Cycle Length (s)	120.0	Sum of lost time (s)	8.0
Intersection Capacity Utilization	45.5%	ICU Level of Service	A
Analysis Period (min)	15		
c Critical Lane Group			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (vph)	0	1243	442	0	769	91	0	0	249	0	0	1067
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)		4.0	4.0		4.0	4.0			4.0			4.0
Lane Util. Factor		0.95	1.00		0.86	1.00			1.00			0.76
Frt		1.00	0.85		1.00	0.85			0.86			1.00
Flt Protected		1.00	1.00		1.00	1.00			1.00			1.00
Satd. Flow (prot)		3539	1583		6408	1583			1611			4247
Flt Permitted		1.00	1.00		1.00	1.00			1.00			1.00
Satd. Flow (perm)		3539	1583		6408	1583			1611			4247
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	0	1351	480	0	836	99	0	0	271	0	0	1160
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	0	1351	480	0	836	99	0	0	271	0	0	1160
Turn Type	Split	NA	Free	Split	NA	Free			Free			Over
Protected Phases	2	2		6	6							2
Permitted Phases			Free			Free			Free			
Actuated Green, G (s)		88.1	120.0		23.9	120.0			120.0			88.1
Effective Green, g (s)		88.1	120.0		23.9	120.0			120.0			88.1
Actuated g/C Ratio		0.73	1.00		0.20	1.00			1.00			0.73
Clearance Time (s)		4.0			4.0							4.0
Vehicle Extension (s)		3.0			3.0							3.0
Lane Grp Cap (vph)		2598	1583		1276	1583			1611			3118
v/s Ratio Prot		c0.38			c0.13							0.27
v/s Ratio Perm			0.30			0.06			0.17			
v/c Ratio		0.52	0.30		0.66	0.06			0.17			0.37
Uniform Delay, d1		6.9	0.0		44.3	0.0			0.0			5.8
Progression Factor		0.55	1.00		0.88	1.00			1.00			1.00
Incremental Delay, d2		0.7	0.5		0.7	0.0			0.2			0.3
Delay (s)		4.4	0.5		39.8	0.0			0.2			6.2
Level of Service		A	A		D	A			A			A
Approach Delay (s)		3.4			35.6			0.2			6.2	
Approach LOS		A			D			A			A	

Intersection Summary			
HCM 2000 Control Delay	11.1	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.55		
Actuated Cycle Length (s)	120.0	Sum of lost time (s)	8.0
Intersection Capacity Utilization	42.7%	ICU Level of Service	A
Analysis Period (min)	15		
c Critical Lane Group			

Appendix B

Cost Estimates

User Delay Cost Comparison

Average Intersection Delay

Alternative	Intersection	AM Peak Hour		PM Peak Hour	
		LOS	Delay (sec/veh)	LOS	Delay (sec/veh)
SDI	SB Ramps	C	25.4	A	9.7
	NB Ramps	C	24.1	C	26.8
DDI	SB Ramps	C	20.8	B	16.7
	NB Ramps	B	13.6	B	11.1

Delay Costs

Alternative	Intersection	AM Peak Hour		PM Peak Hour		Total Annual Peak Hour Delay (hrs)	Value of Time
		VPH	VHD	VPH	VHD		
SDI	SB Ramps	5023	9037	5075	3487	12524	$ 6,575,185
	NB Ramps	3839	6553	3861	7329	13883	$ 7,288,553
					Total	**26407**	**$ 13,863,738**
DDI	SB Ramps	5023	7401	5075	6003	13404	$ 7,037,024
	NB Ramps	3839	3698	3861	3036	6734	$ 3,535,323
					Total	**20138**	**$ 10,572,347**

VPH = Vehicles per Hour
VHD = Vehicle Hours of Delay (seconds per vehicle*vph/3600*255 days per year)
Value of Time = $21 per hour*25 years

Appendix C

Memorandum – Updated Interchange Traffic at SH 14 and Prospect Interchanges

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MEMORANDUM

Date: July 24, 2015

To: Nathan Silberhorn, CDOT

cc: Long Nguyen, Mark Connelly - CDOT, Tom Cotton, Atkins

From: David Woolfall, P.E., P.T.O.E.

Project: North I-25, Crossroads to SH 14 Design

Subject: **Updated Interchange Traffic at SH 14 and Prospect Interchanges**

Introduction

In 2012 the project team provided analysis of Diverging Diamond Interchange (DDI) and Standard Diamond Interchange (SDI) designs for the Mulberry (SH 14) and Prospect interchanges on I-25. The interchanges were identified for reconstruction as part of the North I-25 EIS, both as Standard Diamond Interchanges (SDI). Since 2012 the overall traffic forecasts for the I-25 corridor have been updated from the EIS forecasts, resulting in substantial reductions in traffic volume - particularly for the Prospect interchange.

This updated 2015 technical memorandum shows the comparison of the forecasted volumes, updates the analysis of the DDI and SDI concepts, and shows how any design assumptions have changed and what will be incorporated into the 20% design plans.

The Atkins/TSH team has begun preliminary design to implement the EIS concepts. During the preliminary design the required laneage, traffic forecasts, and traffic operations of the SDI at each location was evaluated. The traffic characteristics at these interchanges confirmed that DDI's may be well suited for these two interchanges. The following characteristics of the two locations may make the DDI design a good alternative to the SDI:

- Both interchanges will be completely reconstructed in the future
- Traffic flow served by each interchange is primarily northbound-to-westbound (NB to WB) or eastbound-to-southbound (EB to SB). The greatest proportion of traffic at each interchange is turning, not through traffic, which is well suited to the DDI operations and geometry.
- The DDI design fits within the diamond interchange envelope evaluated and cleared by the EIS.
- The DDI design fits within the planned adjacent local intersections on each crossroad.
- The DDI designs can achieve the same or better LOS as regular diamonds but with 15%-20% less bridge area at each location.

If the SDI concept advances to the Value Engineering (VE) stage near FIR, or if public/private partnership delivery is pursued in the project, the DDI will undoubtedly be brought forward as an alternative due to likely lower costs. This memorandum provides an opportunity for CDOT and the local jurisdictions to review the DDI concept prior to VE and possibly approve the DDI design as the primary alternative for the FIR plans.

A summary matrix that compares the attributes of the SDI and DDI concepts at both Mulberry and Prospect locations is shown in Table 1. The analysis categories where one interchange type is clearly superior to the other are highlighted in green.

Table 1 - Comparison Summary - SDI vs. DDI

	Standard Diamond Interchange (SDI)	Diverging Diamond Interchange (DDI)
Total lanes on structure	**SH 14 / Mulberry = 9 lanes (5+4)** **Prospect = 7 lanes**	**SH 14 / Mulberry = 8 lanes (3+5)** **Prospect = 6 lanes (3+3)**
Structure - other	Likely a single I-25 structure but could be done with two structures. Single structure needs to be done with phased construction. Frontage road bridge is the same width for both alternatives so is not included in total. **Mulberry = 49,350 sf , Prospect = 40,950 sf**	Likely two structures – so phasing is simpler, some extra cost for double shoulders and double bridge rails. North structure can be at higher elevation to facilitate I-25 profile. Frontage road bridge is the same for both alternatives. **Mulberry= 45,500 sf (-8%), Prospect= 37,100 sf (-9.5%)**
Vertical Profile	Per Nov. 15th meeting, posted speed limit for overpasses will be 40mph, design speed for vertical crest will be 50mph. Longer crest vertical requires more earthwork, more ROW, more work on connecting local access ramps at Mulberry	DDI horizontal design requires 30 to 35mph curves at crossovers, vertical design can be lowered to 40 or 45mph crest vertical. Smaller vertical crest requires less earthwork, ties into existing sooner.
Level of Service	LOS at each interchange averages an acceptable LOS C	Overall interchange LOS, delay, and queue lengths are significantly improved due to signal phasing
Signal Phasing	Standard 3-phase signals, protected lefts for all movements. Min. cycle length = about 80 sec. Prospect, 90 sec. Mulberry	2-phase signals, some lower volume left turn movements could be free-lefts. Min. cycle length = about 40 sec. Prospect, 50 sec. Mulberry
Signal Coordination	Likely full cycle lengths (100-120 seconds) needed during peak periods due to adjacent frontage road phasing	Interchange signals could half-cycle (50-60 seconds) even during peak periods, reducing queuing at interchange
Left turn geometry	Triple-lefts radius range from 80' to 120', which constrains left turning vehicles, especially trucks	Triple-lefts radius range from 155' to 200', less constrained so turning traffic moves at more constant speed, less path overlap for large trucks
Ramp geometrics	Desirable to bring ramp to intersection with crossroad at or near perpendicular. Requires more area/ROW for ramps	More flexibility with angle of ramp approaching crossroad, since all traffic turns the angle of ramp approach is more flexible. Reduces land area needed for interchange
Vehicle Queues	Peak hour left and right turn queues are longer since full signal cycles are used	Peak hour left and right turn queues are shorter at full cycle lengths due to two signal phases, and substantially shorter if half-cycles are used in signal timing
Transit	If transit stops are along the ramps, buses continue thru using standard signal phasing. EIS shows transit stops are away from interchange	If bus stops are on the ramps, special lane and signal phase required for thru bus movement. EIS shows transit stops are away from interchange
Maintenance	Snowplows often continue straight from ramp-to-ramp, which is accommodated at SDI	DDI would not allow a snowplow to continue straight from ramp-to-ramp. Special lane or drive-over island would need to be constructed, or plowing procedures revised
Bicycles	Right-hand lane adjacent to travel lanes per Ft. Collins preference	Right-hand lane adjacent to travel lanes per Ft. Collins preference. Should provide good striping thru wide intersect.
Pedestrians	Per Ft. Collins, 6 ft. sidewalks on each side of the bridge with 6 ft. separation from driving lane (bike lane). Decision for 40mph speed limit allows no barrier between walk & lanes	Option for single sidewalk (assume 8 ft.) along inside of one of the two bridges – between opposing traffic flows. Same separation of pedestrians to traffic as SDI. More difficult to convey proper travel direction to visually impaired pedestrians due to angled/non-intuitive travel paths.
Safety - General	6 approach conflicts per intersection. More potential for higher-speed angled collisions. More potential for wrong-way turn onto freeway off-ramp.	4 approach conflicts per intersection at lower speeds due to geometry. Main intersection is skewed, but all traffic goes straight at skewed crossing. Wrong way turn onto freeway ramp virtually impossible. Need additional signing on approach road, particularly Mulberry, to bring speed limit down to 30 or 35 from 45 or 55.
Construction Phasing	Standard method, SDI with reduced lanes during construction.	Standard method, SDI with reduced lanes during construction, temporary paving thru gaps in arterial curves until DDI traffic control is ready to implement

The descriptions in Table 1 show that both interchange types will work at both locations. The comparison categories show that the DDI does have some clear and quantifiable advantages in several areas, including:

- Less bridge structure at each location, accounting for about $0.5 Million in savings at each location
- The DDI has an opportunity for reduced overall earthwork due to the potential lower design speed of the crest vertical curve. Reduction of up to 60,000 cy at each interchange, savings of $300,000 to $500,000 per location.
- Better traffic operations in LOS, but greatly improved flexibility for traffic operations in both peak and off-peak by allowing half-cycling of signals, reducing overall delay and pedestrian delay.
- Better geometric characteristics and flexibility for accommodating the high turning volumes at each interchange.
- Improved safety for the DDI due to lower speeds, fewer conflict points, and reduced potential for wrong-way movements entering the freeway.

The primary issues noted with the DDI are in the areas of pedestrian accommodation and in snow removal maintenance. Final design for each of these issues could likely mitigate these concerns. The need to reduce the speed limit down to 30 or 35mph from the existing 55mph on Mulberry would be a concern, although ultimately Mulberry will be more urbanized and 45mph may be a more appropriate speed limit approaching I-25.

Traffic Forecast Update and Traffic Analysis

The I-25 North EIS work began in about 2004 and continued to about 2011. The 2035 travel forecasts from the EIS (updated 2011 FEIS numbers) were originally used for the 2012 evaluation of the DDI concept and comparison to the SDI at each location. In 2014 CDOT updated the corridor forecasts to reflect the economic slowdown of 2008-2011, resulting in lower traffic forecasts.

The revised 2035 peak hour traffic forecasts for each location compared with the forecasts from the earlier EIS are shown in **Figures 1 and 2**. These updated traffic forecasts show a slight lowering of traffic at Mulberry and a substantial lowering of traffic at Prospect. The lower Prospect forecasts allow for a more conventional right turn design from Prospect to I-25, however Mulberry still requires double-rights to the I-25 ramp.

The existing traffic volumes and traffic forecasts show the traffic split approaching each interchange have predominant traffic flow serving a Fort Collins to South I-25 connection.

The EIS traffic analysis focused on the SDI at each location and showed that each SDI would achieve an acceptable overall LOS. The Mulberry interchange requires triple-lefts from NB to WB, and while both Mulberry and Prospect have high EB to SB right turning traffic at the west ramp intersection, a 2nd right turn lane should be incorparated only at the Mulberry interchange.

The traffic volume figure shows 2035 traffic forecasts for the southern ramps at the Mulberry interchange exceeding 2,000 vph per direction and 1,400 vph per direction at the Prospect interchange. For comparison, the existing volumes at 120th and I-25 - a very busy interchange at the north end of Denver suburbs - do not exceed 1,500 vph for any ramp.

The laneage for each SDI and potential laneage for each DDI is shown in **Figure 3** for the SH 14/Mulberry interchange and **Figure 4** for the Prospect interchange. This is the laneage used to provide a comparison of intersection LOS and preliminary design geometrics at each ramp intersection with the crossroad. The intersection comparison LOS is shown in **Table 2**, which also includes the LOS results from the previous EIS for comparison.

Figure 1 - 2035 Mulberry Street Peak Hour Traffic Forecasts

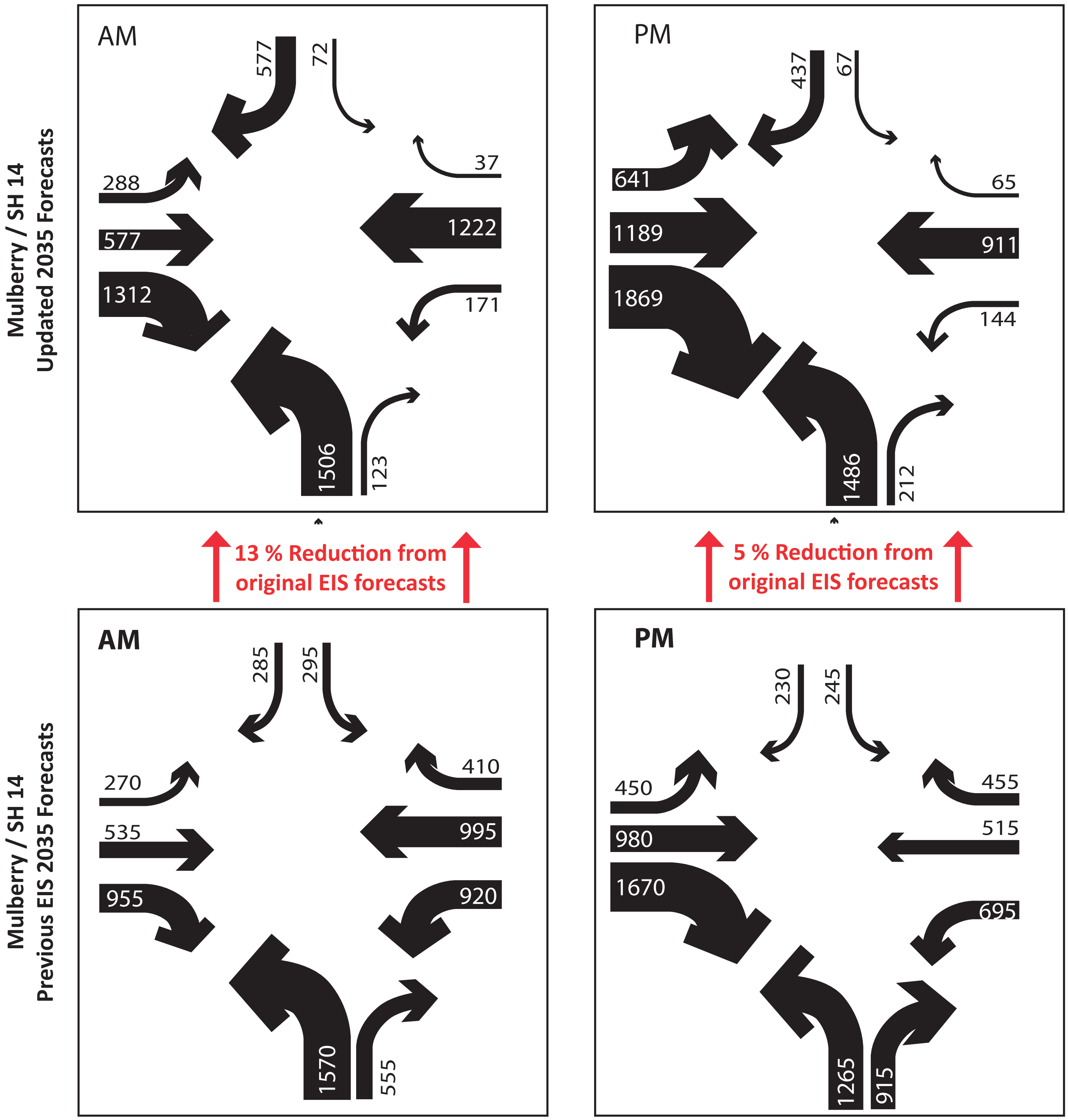


Figure 2 - 2035 Prospect Road Peak Hour Traffic Forecasts

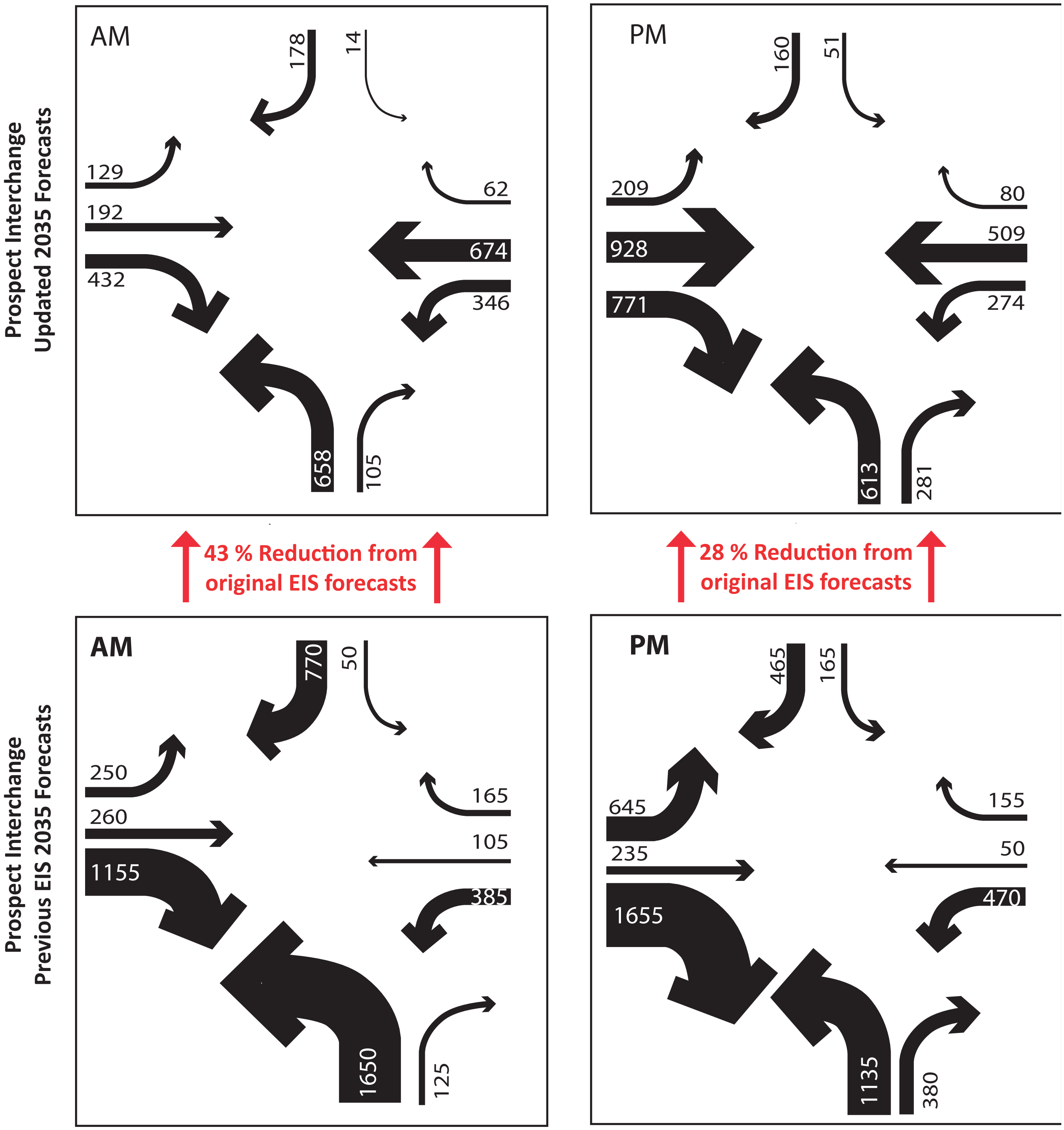
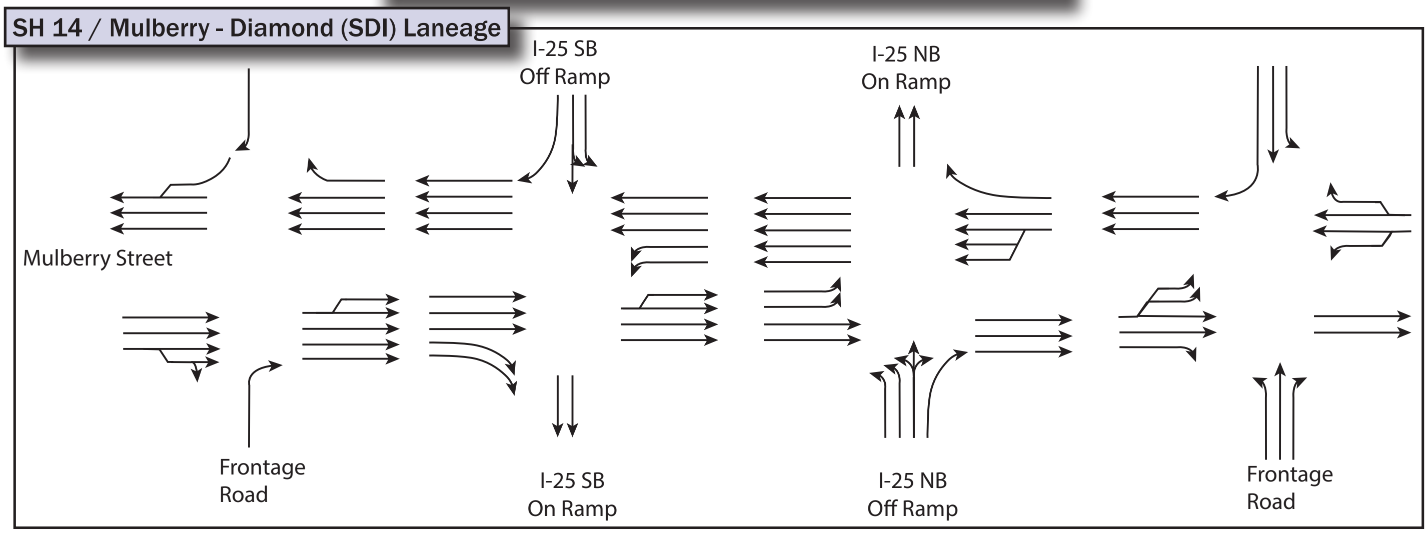


Figure 3 - 2035 Laneage - SH 14 / Mulberry



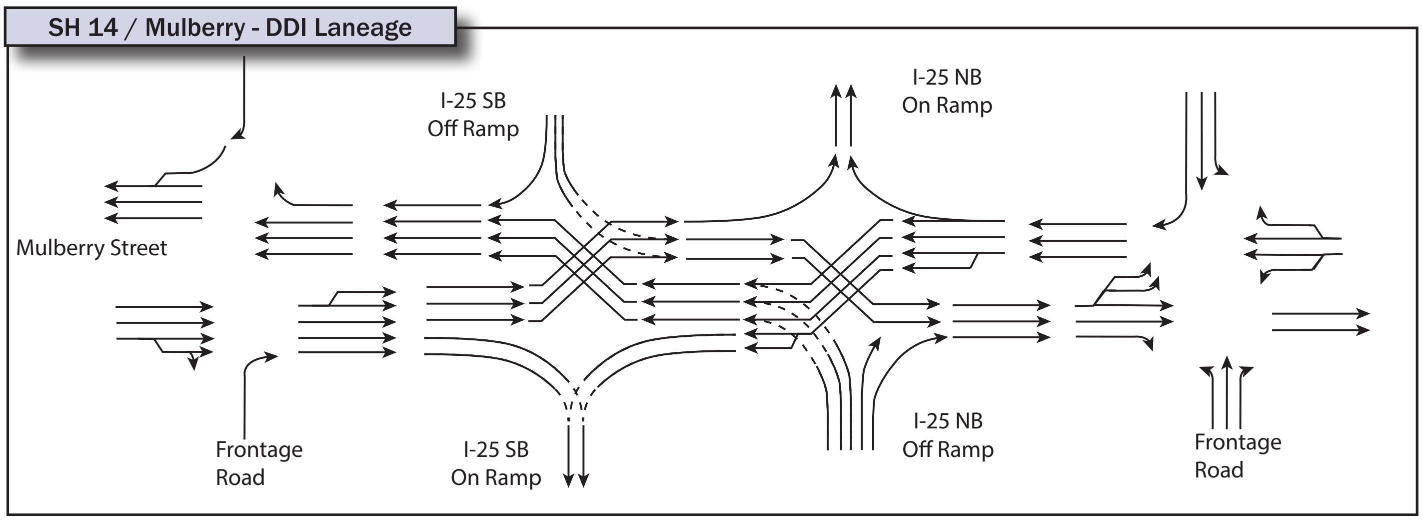


Figure 4 - 2035 Laneage - Prospect

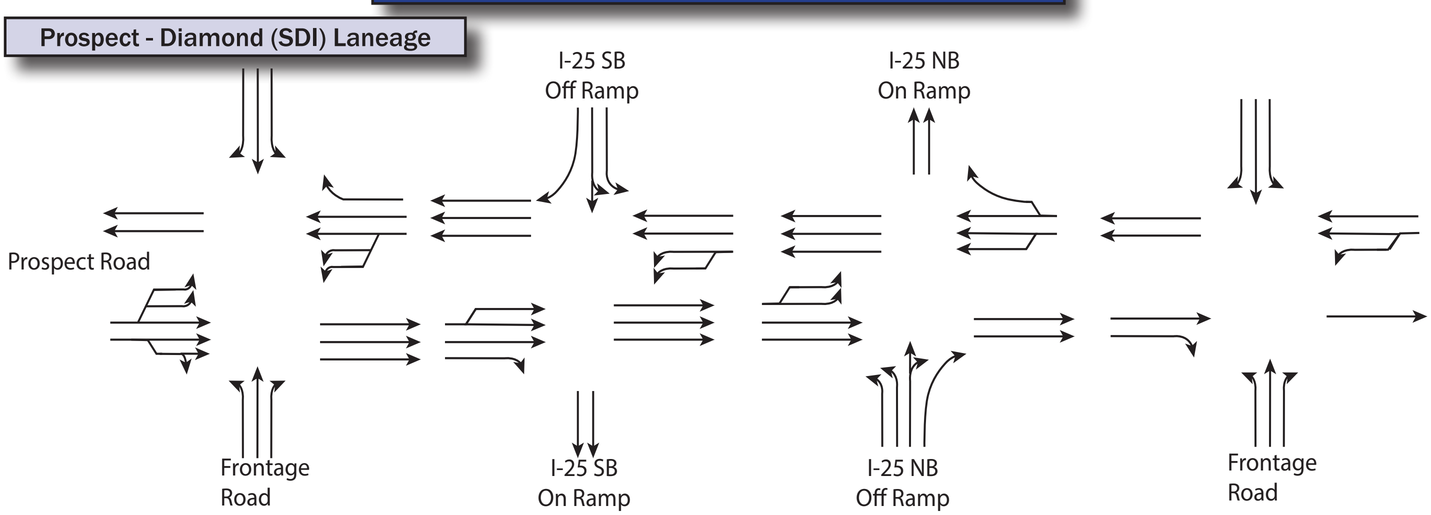


Figure 4 (continued) - 2035 Laneage - Prospect

Prospect - DDI Laneage

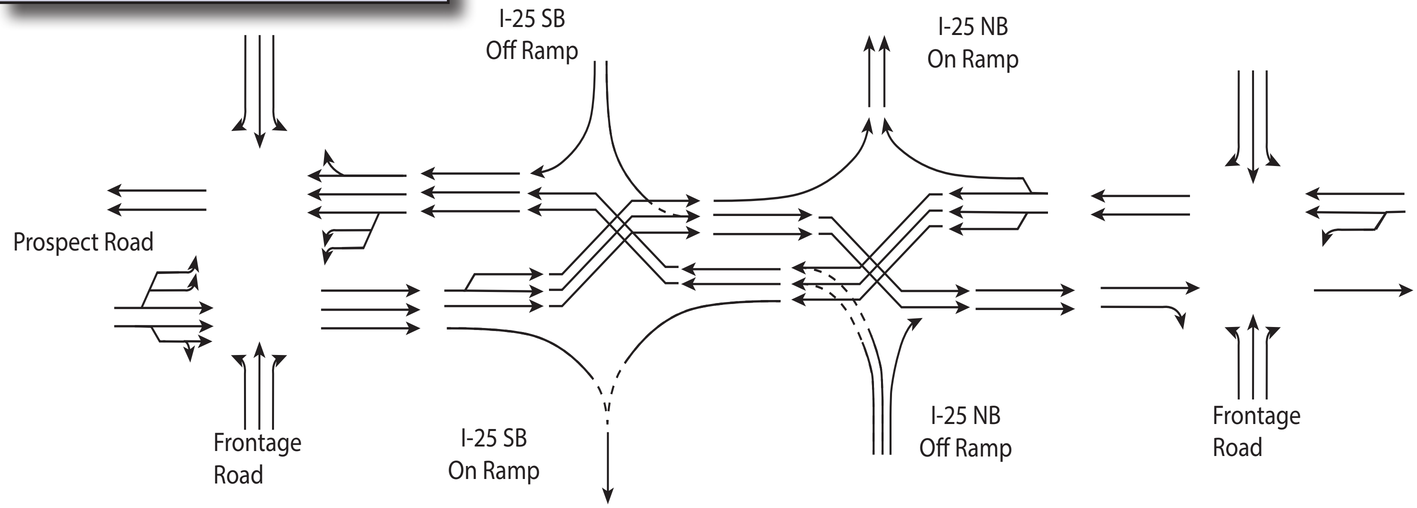


Table 2 - 2035 Level of Service Comparison

AM Peak, 2035 (Revised)

	SDI (120) (2)		DDI (120) (2)		DDI (60) (2)	
	95th % Queue. (ft)	LOS (delay)	95th % Queue. (ft)	LOS (delay)	95th % Queue. (ft)	LOS (delay)
Mulberry Street/I-25						
East Side	NBL - 472	C (25.5) (1)	NBL - 438	B (16.4) (1)	NBL - 228	B (11.1) (1)
West Side	EBR - 400	B (10.2)	EBR - 204	A (6.7) (1)	EBR - 124	A (5.0) (1)
Prospect Road/I-25						
East Side	NBL - 308	C (24.1) (1)	NBL - 272	B (17.8) (1)	NBL - 159	B (12.4) (1)
West Side	EBR - 22	B (17.5)	EBR - 536	B (18.6) (1)	EBR - 443	B (19.4) (1)

FOOTNOTE:

(1) HCS 2000 LOS reported.

(2) Cycle Length (seconds)

AM Peak, 2035 (Previous EIS)

	SDI (120) (2)		DCD (120) (2)		DCD (60) (2)	
	95th % Queue. (ft)	LOS (delay)	95th % Queue. (ft)	LOS (delay)	95th % Queue. (ft)	LOS (delay)
Mulberry Street/I-25						
East Side	NBL - 518	C (27.6) (1)	NBL - 427	B (14.2) (1)	NBL - 243	B (13.7) (1)
West Side	EBR - 456	C (24.5)	EBR - 290	C (20.5) (1)	EBR - 248	B (11.2) (1)
Prospect Road/I-25						
East Side	NBL - 754	C (32.4) (1)	NBL - 675	B (15.1) (1)	NBL - 507	B (14.9) (1)
West Side	EBR - 130	A (7.0)	EBR - 512	B (10.1) (1)	EBR - 303	A (7.1) (1)

FOOTNOTE:

(1) HCS 2000 LOS reported.

(2) Cycle Length (seconds)

Table 2 (continued)- 2035 Level of Service Comparison

PM Peak, 2035 (Revised)

	SDI (120) (2)		DDI (120) (2)		DDI (60) (2)	
	95th % Queue. (ft)	LOS (delay)	95th % Queue. (ft)	LOS (delay)	95th % Queue. (ft)	LOS (delay)
Mulberry Street/I-25						
East Side	NBL - 473	D (36.7) (1)	NBL - 382	B (16.3) (1)	NBL - 191	A (8.1) (1)
West Side	EBR - 1064	B (17.3)	EBR - 611	B (10.4) (1)	EBR - 544	D (37.8) (1)
Prospect Road/I-25						
East Side	NBL - 248	C (26.1) (1)	NBL - 163	B (12.6) (1)	NBL - 114	B (10.3) (1)
West Side	EBR - 205	C (22.0)	EBR - 1153	D (47.7) (1)	EBR - 1054	D (37.7) (1)

FOOTNOTE:

(1) HCS 2000 LOS reported.

(2) Cycle Length (seconds)

AM Peak, 2035 (Previous EIS)

	SDI (120) (2)		DCD (120) (2)		DCD (60) (2)	
	95th % Queue. (ft)	LOS (delay)	95th % Queue. (ft)	LOS (delay)	95th % Queue. (ft)	LOS (delay)
Mulberry Street/I-25						
East Side	NBL - 383	C (20.5) (1)	NBL - 264	A (9.5) (1)	NBL - 150	A (8.7) (1)
West Side	EBR - 1056	D (40.8)	EBR - 728	B (17.5) (1)	EBR - 501	B (19.5) (1)
Prospect Road/I-25						
East Side	NBL - 573	C (34.7) (1)	NBL - 223	A (6.7) (1)	NBL - 223	A (7.1) (1)
West Side	EBR - 778	C (21.6)	EBR - 951	C (22.4) (1)	EBR - 599	D (37.8) (1)

FOOTNOTE:

(1) HCS 2000 LOS reported.

(2) Cycle Length (seconds)

Additionally, the tables also shows a comparison of vehicle queues of the two heaviest traffic movements at each interchange, those being NB left turns and EB right turns. The traffic operations interaction along the arterial corridor is evaluated later in this memo.

As shown in the individual intersection results, the LOS results for the SDI and DDI are similar in most cases. The notable aspect is that the DDI achieves the same result with fewer lanes. In addition, the shorter intersection crossings for the DDI allow the DDI to use half-cycle phasing (50 or 60 seconds vs. 100 or 120 seconds), which is particularly effective in reducing delay during off-peak times. For vehicle queues on key movements, DDI queuing is shorter in all nearly scenarios, and substantially shorter when half-cycles are used for the signals.

Corridor operations and signal progression need to be considered with both the SDI and DDI designs. Synchro was used to provide initial optimization of system operations, which was then translated to progression diagrams to compare the operations of the SDI and the DDI. These preliminary time-space diagrams are shown in the appendix and signal phasing for each DDI concept is shown on the time-space diagram. Time space diagrams were prepared for SDI with 120 second cycles, DDI with 120 second cycles, and DDI with 60 second cycles. In general, the progression band for the key traffic movements is the same for the SDI and the DDI when measured as a percentage of total cycle length.

Table 3 - 2035 Measures of Effectiveness Comparison

2035 Revised - Mulberry

North I-25 Interchange Alternatives under Evaluation at Mulberry

	AM Peak			PM Peak		
Measure of Effectiveness	SDI (1)	DDI (120) (1)	DDI (60) (1)	SDI (1)	DDI (120) (1)	DDI (60) (1)
Total Delay (hr)	43.0	17.4	19.2	53.8	18.2	26.5
Total Delay/Veh (S)	74.2	38.7	40.9	78.6	31.0	32.9
Total Stops	1614	1287	1634	1572	1393	1460
Travel Time (hr)	57.7	34.1	36.2	68.6	36.6	43.9
Fuel Used (gal)	25.3	21.0	22.1	27.9	23.2	24.0

2035 Previous EIS - Mulberry

North I-25 Interchange Alternatives under Evaluation at Mulberry

	AM Peak			PM Peak		
Measure of Effectiveness	SDI (1)	DCD (120) (1)	DCD (60) (1)	SDI (1)	DCD (120) (1)	DCD (60) (1)
Total Delay (hr)	36.9	31.5	26	35.6	23.8	17.3
Total Delay/Veh (S)	392.8	347.5	336.6	406.7	388.7	325.4
Total Stops	2352	1752	1981	2647	1537	1699
Travel Time (hr)	51.6	44.2	38.6	49.7	45.6	44.6
Fuel Used (gal)	25	23	21.8	25.5	23.2	23.5

2035 Revised - Prospect

North I-25 Interchange Alternatives under Evaluation at Prospect

	AM Peak			PM Peak		
Measure of Effectiveness	SDI (1)	DDI (120) (1)	DDI (60) (1)	SDI (1)	DDI (120) (1)	DDI (60) (1)
Total Delay (hr)	13.9	17.2	9.1	13.8	19.8	15.6
Total Delay/Veh (S)	78.5	90.3	50.9	56.5	62.5	52.2
Total Stops	840	930	809	1036	878	985
Travel Time (hr)	22.8	26.6	18.7	26.8	30.5	26.8
Fuel Used (gal)	12.3	13.8	11.8	16.6	15.5	15.0

FOOTNOTE:

(1) Cycle Length (seconds)

2035 Previous EIS - Prospect

North I-25 Interchange Alternatives under Evaluation at Prospect

	AM Peak			PM Peak		
Measure of Effectiveness	SDI (1)	DCD (120) (1)	DCD (60) (1)	SDI (1)	DCD (120) (1)	DCD (60) (1)
Total Delay (hr)	16.2	16.6	15.5	19	21.2	17
Total Delay/Veh (S)	360.2	335.2	299.4	385.6	286.7	334
Total Stops	1067	1125	1016	1735	1776	1483
Travel Time (hr)	26.9	28.7	28.6	33	36.4	28.9
Fuel Used (gal)	14.7	16.4	14.8	17.1	19.2	16.9

FOOTNOTE:

(1) Cycle Length (seconds)

Table 3 shows a comparison of system operations on each arterial as calculated by a Simtraffic simulation. The signal timing parameters were optimized by the computer to attempt to show an unbiased comparison of performance measures. Preliminary time-space progression diagrams based on the simulations are contained in the Appendix. The results are similar to the LOS results in that performance measures are nearly the same or better for most scenarios with the DDI, and the opportunity for shorter cycle lengths with the DDI offers the best operations in all cases. Results from the previous EIS are also included for comparison.

Pedestrians and Bicycles

Pedestrian and bicycle accommodations at a DDI are not very different than at an SDI. Based on the November 15, 2012 meeting with the City of Ft. Collins, on-street bike lanes are preferred on each arterial and carrying through the interchange. At an SDI, the bicycle lanes are essentially straight and remain alongside the right-hand through lane through the interchange. The same is true for a DDI, the on-street bike lane continues alongside the right-hand through lane as the through lanes criss-cross at each side of the interchange. For this reason, there are no notable differences to document for bicycle lanes for either the SDI or the DDI.

The pedestrian accommodation at an SDI was assumed to be 6 ft. attached sidewalks along each side of the bridge over I-25. The route for pedestrians is generally straight with the exception of crossing the right turn lanes at about 45 degree angles. At a DDI the pedestrian route over I-25 can be similar except that there would be twice as many angled crossings (4 per direction) due to the geometrics of the ramp intersections.

The DDI offers the opportunity to install a single sidewalk across I-25, between the opposing lanes of traffic (see the bridge cross section options). This option for pedestrian accommodation still results in the same exposure to adjacent traffic as the SDI, but may increase the amount of traffic volume a pedestrian must cross since all pedestrians would cross the through lanes. Careful design and possibly signing to delineate the intended pedestrian path would also be required since this pedestrian route may not be intuitive to all users.

Pedestrian route options at the Mulberry DDI are shown in the preliminary interchange layouts, the route options at Prospect would be similar.

Preliminary Interchange Geometry

The DDI design requires the arterial lanes to "criss-cross" each other at approximately the ramp intersection location. This generally requires a lower design speed for the arterial, in the range of 30 to 35mph, which results in a similar footprint as an SDI. Using higher design speeds is possible but may widen the earthwork and ROW footprint, and may require the arterial curves to extend onto the bridge.

DDIs for the Mulberry and Prospect interchanges were preliminarily designed for DDIs using 35mph crossroad design speeds. This is an early iteration of design intended to bring forth discussion of the concept and questions. The design will evolve after determining details such as arterial lane balance, phased implementation potential, and location and width of bicycle and pedestrian facilities.

One important modification shown for both SDIs and the DDIs is the need for a 2nd EB to SB right turn lane at the Mulberry interchange due to extremely high right turn volumes. The right turn volumes exceeding 1,500 vph are similar to those at Lincoln and I-25 in south Denver (1,800 vph AM, 1,600 vph PM) and several other locations in Denver including:

- Arapahoe Road and I-25.
- Park Avenue West/Fox St. to I-25
- 20th Street to I-25
- Wadsworth to I-70/I-76

The double right turn lanes should be planned to have signal control to facilitate pedestrian crossings and to meter the conflict between the EB to SB double-rights and the WB to SB double lefts. This approach has the advantage of keeping the on-ramp a maximum of two lanes. Some high volume ramps in Denver have a third lane added with arterial right turns which makes the merge prior to the mainline more problematic.

The preliminary layouts for both the DDIs and SDIs are shown in the attached figures, with notes added for particular items of interest for each interchange.

Phased Implementation

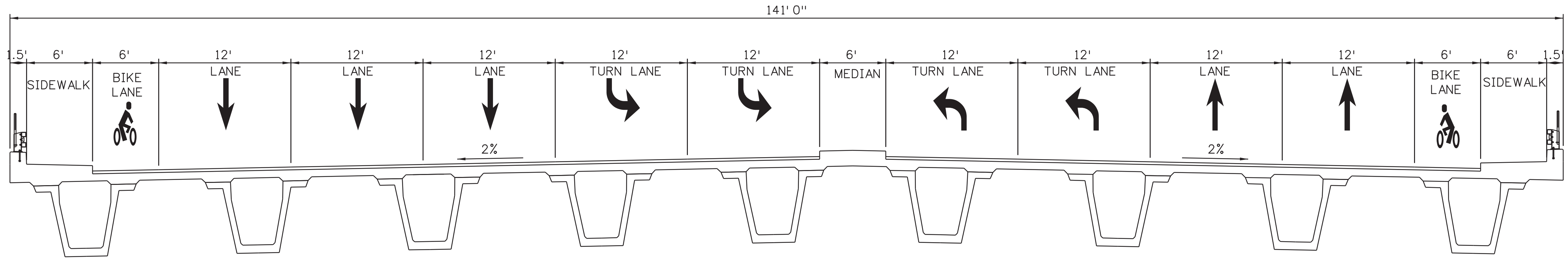
The 2035 traffic forecasts at Mulberry represent a doubling of the traffic volume from existing conditions. The 2035 forecasts require items such as signalized double-rights or triple-lefts at the SH 14 interchange. There are opportunities at both interchanges with either the SDI or DDI concept to design for phased implementation of these higher number of turn lanes. Triple left turns do not need to be implemented immediately, double left turns probably work for up to 20 years of the design life. Similarly, the double-rights are not needed immediately, single rights yielding to the double-lefts probably work for 10 years of the project design life, and signalized double-rights would be implemented when the yielding causes substantial delays and queuing of the right turns.

At this current level of design it is important that the maximum interchange template is designed so that appropriate right-of-way can be acquired. Phased implementation of laneage can be considered post-FIR as more information is learned and in anticipation of updated travel forecasts.

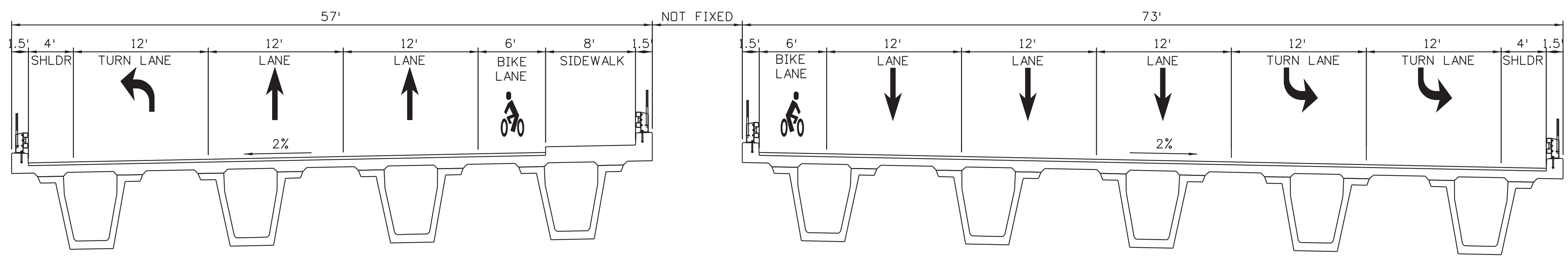
Cross Section Comparisons - Mulberry Interchange

TSIOUVARAS SIMMONS HOLDERNESS
CONSULTING ENGINEERS

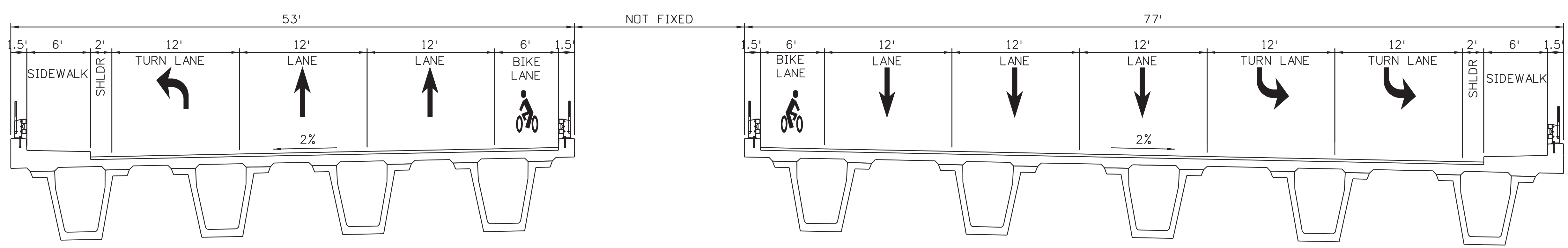
DIAMOND INTERCHANGE TYPICAL SECTION
PER FT. COLLINS MEETING 11/15/2012
BIKE LANES AND ATTACHED SIDEWALK
MULBERRY STREET (SH 14) - LOOKING EAST



DIVERGING DIAMOND INTERCHANGE TYPICAL SECTION
MULBERRY STREET (SH 14) - LOOKING EAST

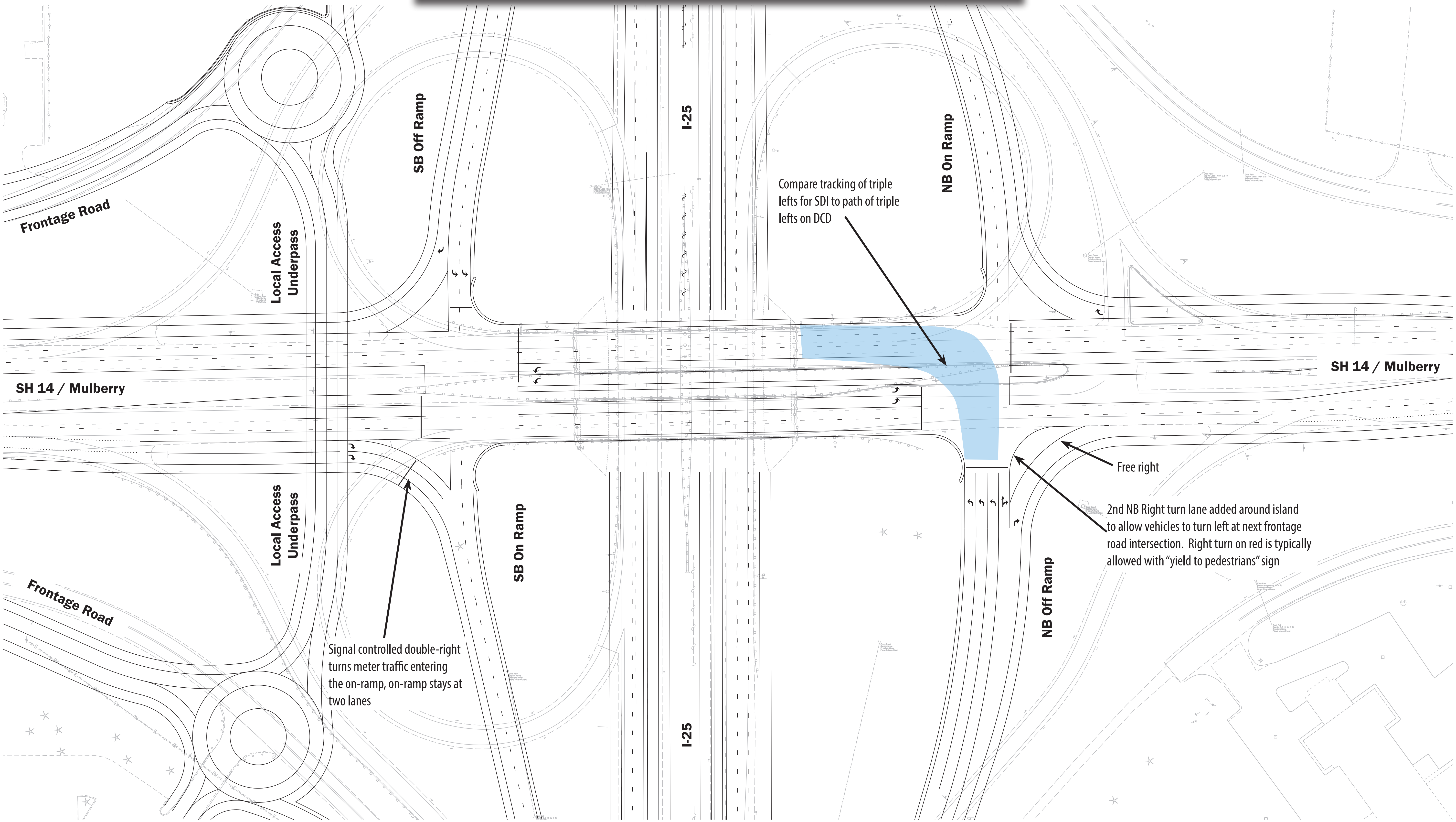


DIVERGING DIAMOND INTERCHANGE TYPICAL SECTION
MULBERRY STREET (SH 14) - LOOKING EAST

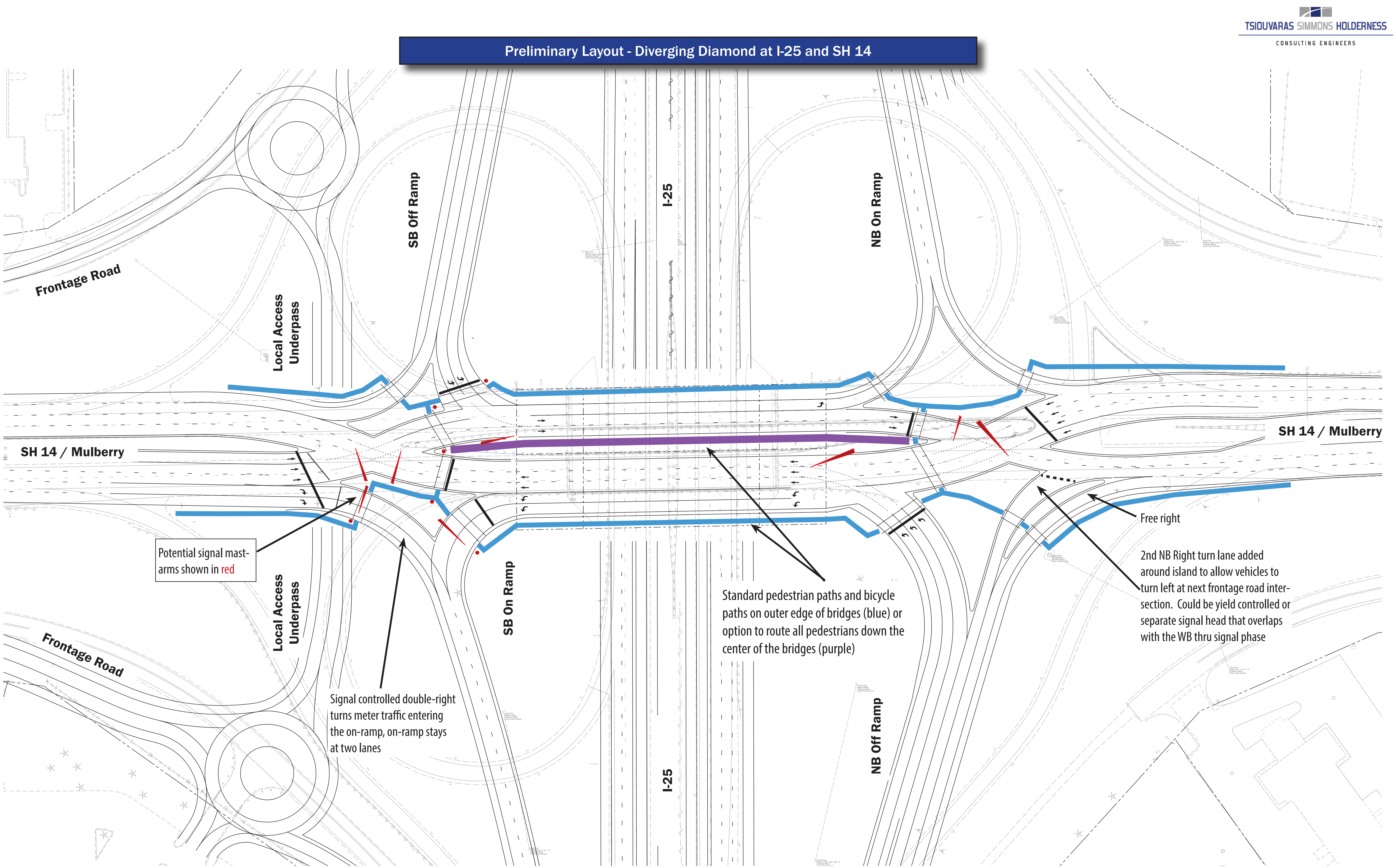


TSIOUVARAS SIMMONS HOLDERNESS
CONSULTING ENGINEERS

Preliminary Layout - Standard Diamond at I-25 and SH 14



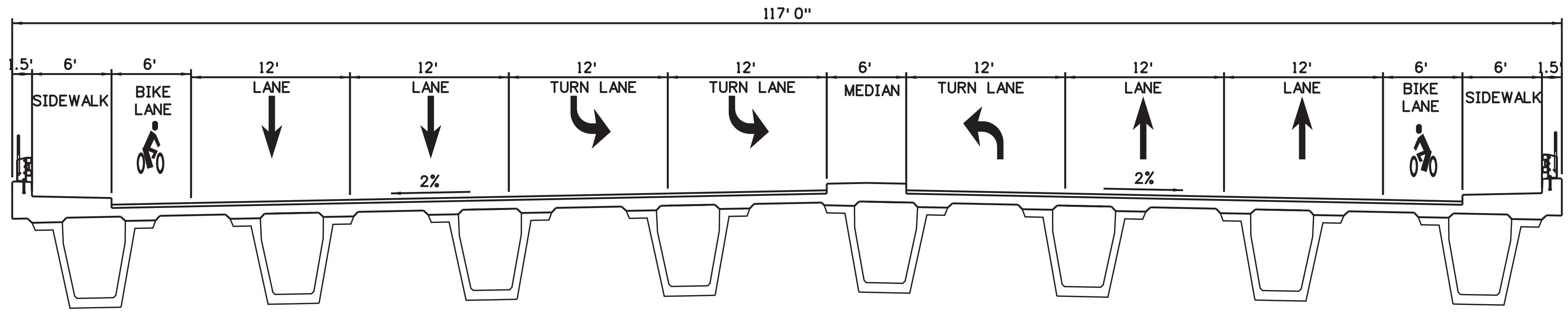
Preliminary Layout - Diverging Diamond at I-25 and SH 14



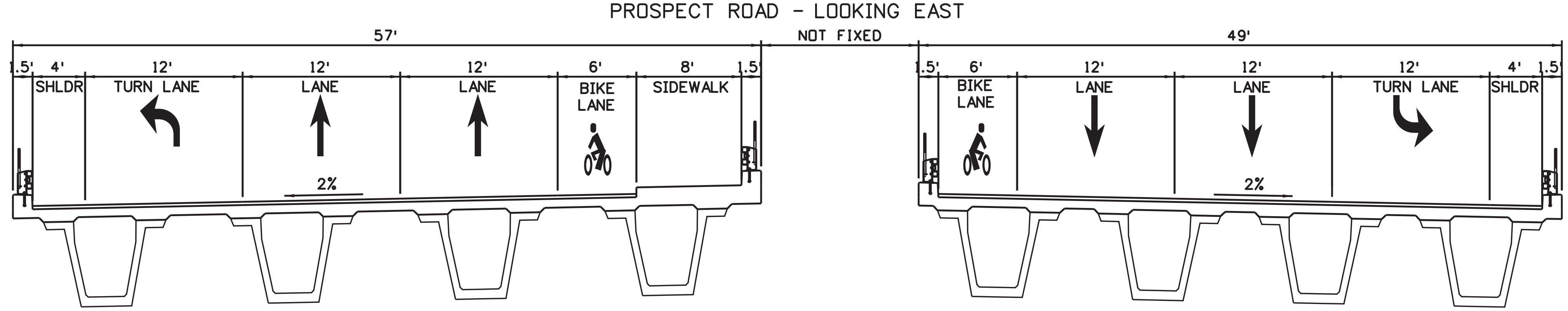
Cross Section Comparisons - Prospect Interchange

TSIOUVARAS SIMMONS HOLDERNESS
CONSULTING ENGINEERS

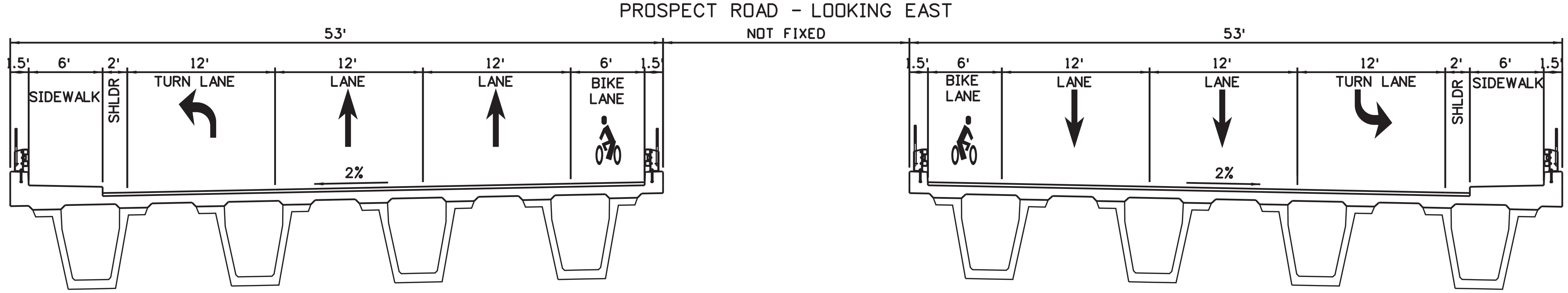
DIAMOND INTERCHANGE TYPICAL SECTION
PER FT. COLLINS MEETING 11/15/2012
BIKE LANES AND ATTACHED SIDEWALK
PROSPECT ROAD - LOOKING EAST



DIVERGING DIAMOND INTERCHANGE TYPICAL SECTION
SIDEWALK INSIDE
PROSPECT ROAD - LOOKING EAST

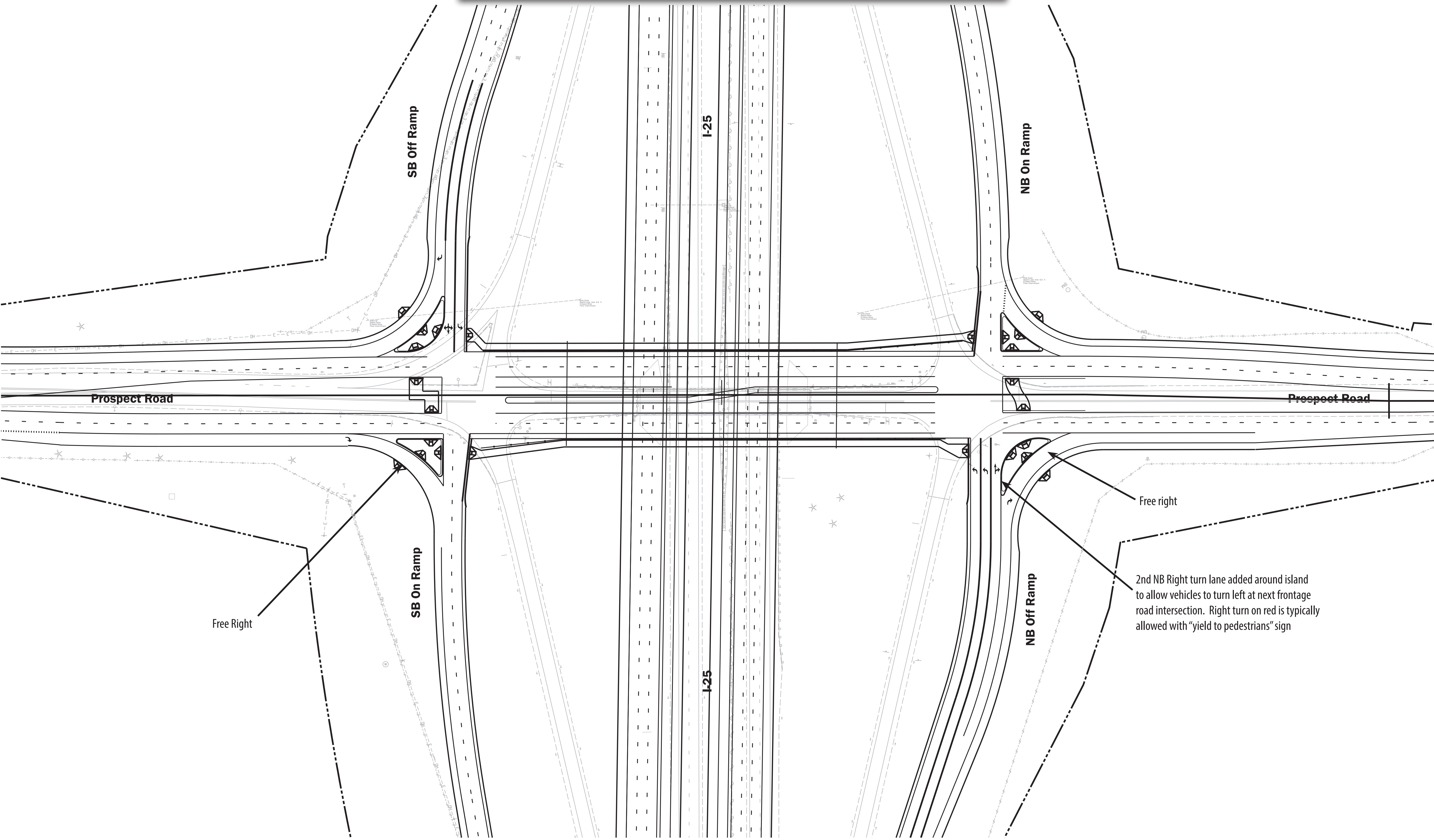


DIVERGING DIAMOND INTERCHANGE TYPICAL SECTION
SIDEWALK OUTSIDE
PROSPECT ROAD - LOOKING EAST



TSIOUVARAS SIMMONS HOLDERNESS
CONSULTING ENGINEERS

Preliminary Layout - Standard Diamond at I-25 and Prospect



TSIOUVARAS SIMMONS HOLDERNESS
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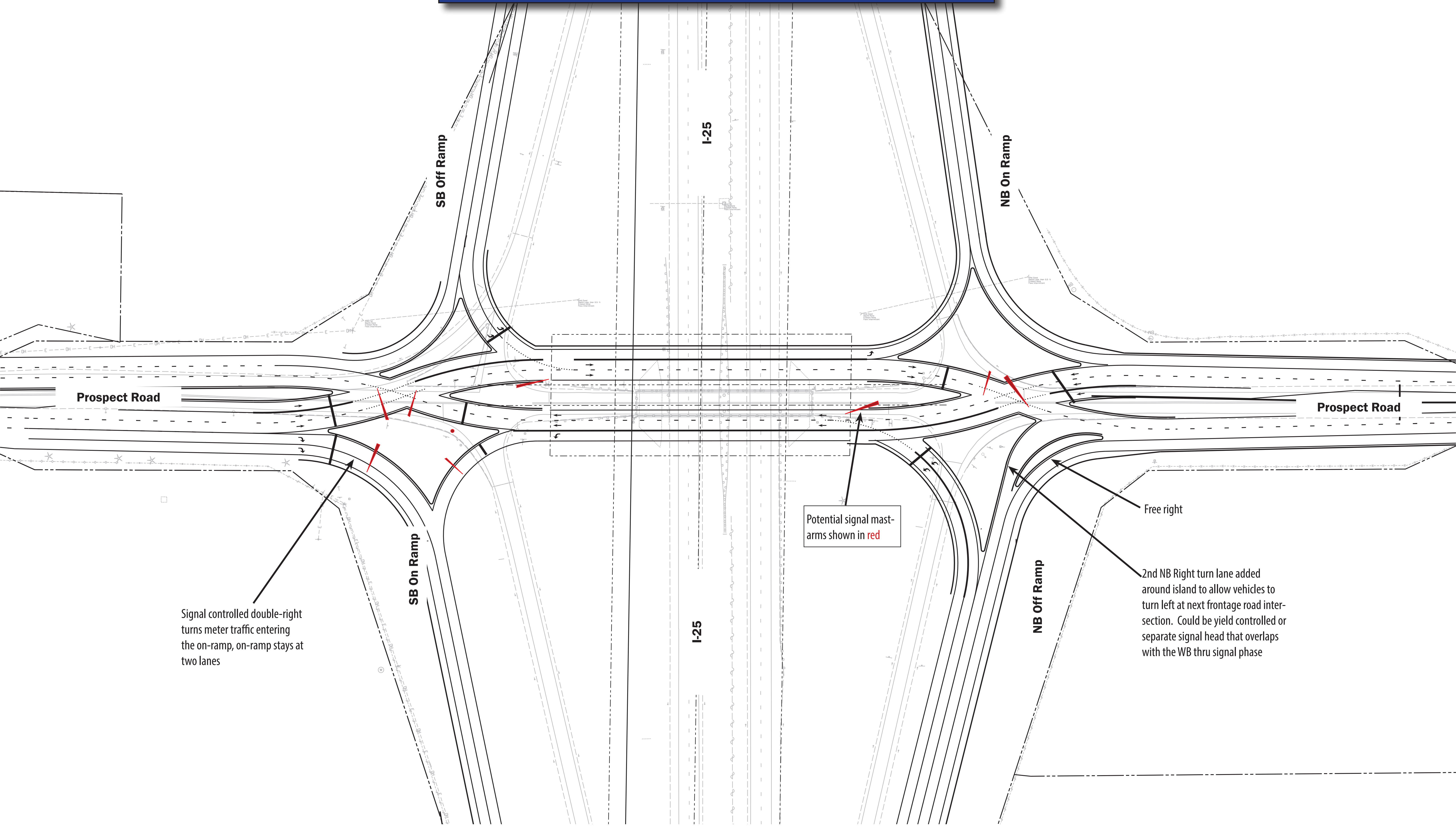
Preliminary Layout - Diverging Diamond at I-25 and Prospect



Appendix D

Memorandum – Mulberry/I-25 and Prospect/I-25 Bike Lanes, Double Right Turn Lanes, and Mulberry Frontage Road access

MEMORANDUM

DATE: June 14, 2013

TO: Sharlene Shadowen, CDOT, Matt Wessel, Atkins,

FROM: David Woolfall, P.E., TSH

SUBJECT: Mulberry/I-25 and Prospect/I-25 Bike Lanes, Double Right Turn Lanes, and Mulberry Frontage Road access

This memorandum summarizes continuing work on the Mulberry/I-25 interchange and the Prospect/I-25 interchange to address the high proportion of right turns at I-25 at both interchanges and how to incorporate on-street bicycle lanes. The Mulberry interchange also has the challenge of incorporating the west-side frontage road intersection which further complicates the bike lane design.

There are several locations in the Denver metro area with similar high proportions of right turns from the arterial to a freeway on-ramp. Most of these locations have been retro-fitted to obtain the additional capacity of a second right turn lane. However the retrofit designs usually do not address the needs of pedestrians and bicycles, except for some locations where grade separations for pedestrians/bicycles have been constructed. In developing the FIR design for the Mulberry and Prospect interchanges, the project team has the opportunity to develop the optimal design for both double-right turns and for good at-grade accommodation of pedestrians and bicycles.

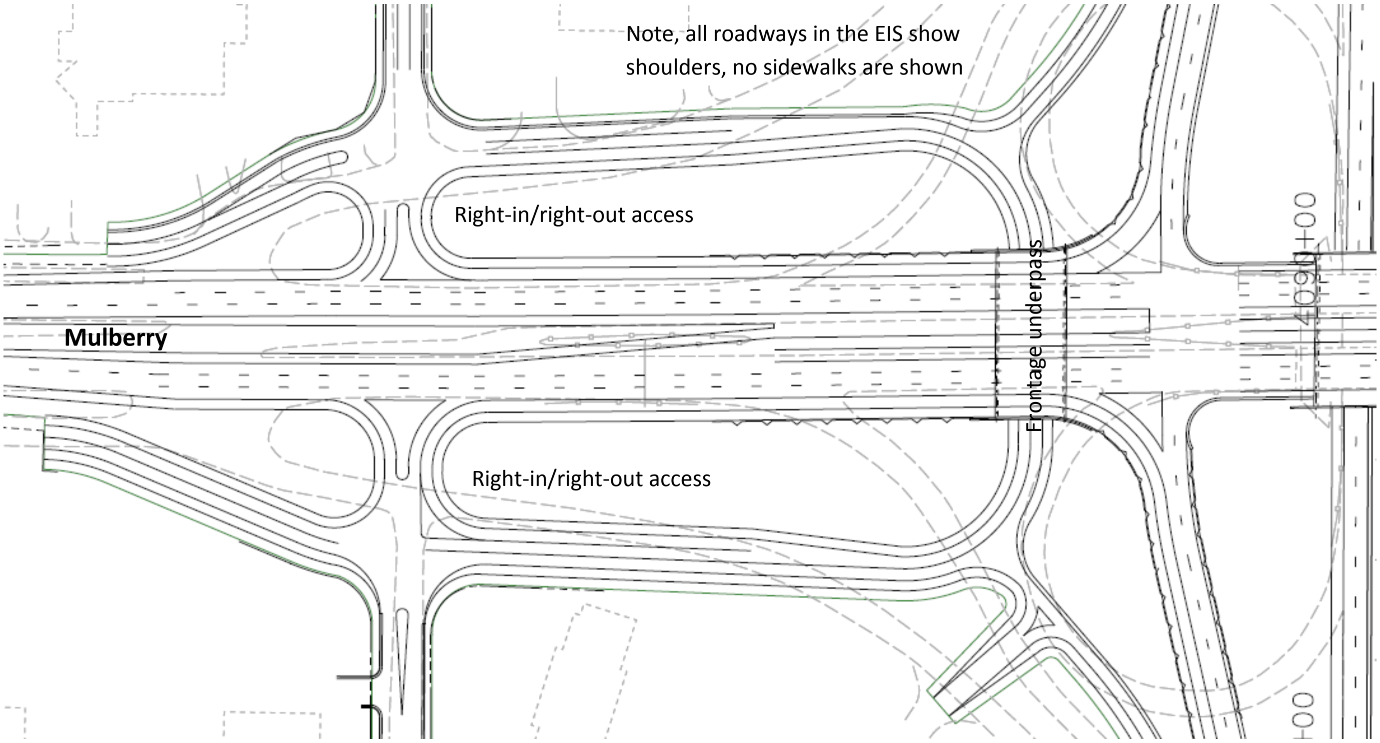
Challenges with the EIS Design

The design concept shown in the EIS for Mulberry is shown on the following page. The EIS design concept was not intended to solve all challenges as is being done with the current design effort, and it had several functional and operational challenges that needed to be addressed, which are discussed below:

- At both the Mulberry and Prospect interchanges in the eastbound direction, over 50% and up to 65% of EB traffic wants to turn right at I-25. Ideally for a 3-lane eastbound cross section, the EB right lane (#3 lane) will drop at I-25, and ideally the #2 lane will allow shared through-rights to allow sufficient capacity and good lane balance for this right turn movement. This traffic volume and lane balance relationship is detailed later.

- At Mulberry, if all of the right turning traffic were in lane #3 approaching I-25, it would likely queue or leave very few gaps for traffic entering Mulberry from the frontage road access.

- At Mulberry there would be a short weave distance between the west-side right-in/right-out (RIRO) access points and the I-25 interchange ramps. This lack of spacing in the EB direction is critical due to the high EB traffic turning south (or right) on the I-25 ramps requiring two lanes.



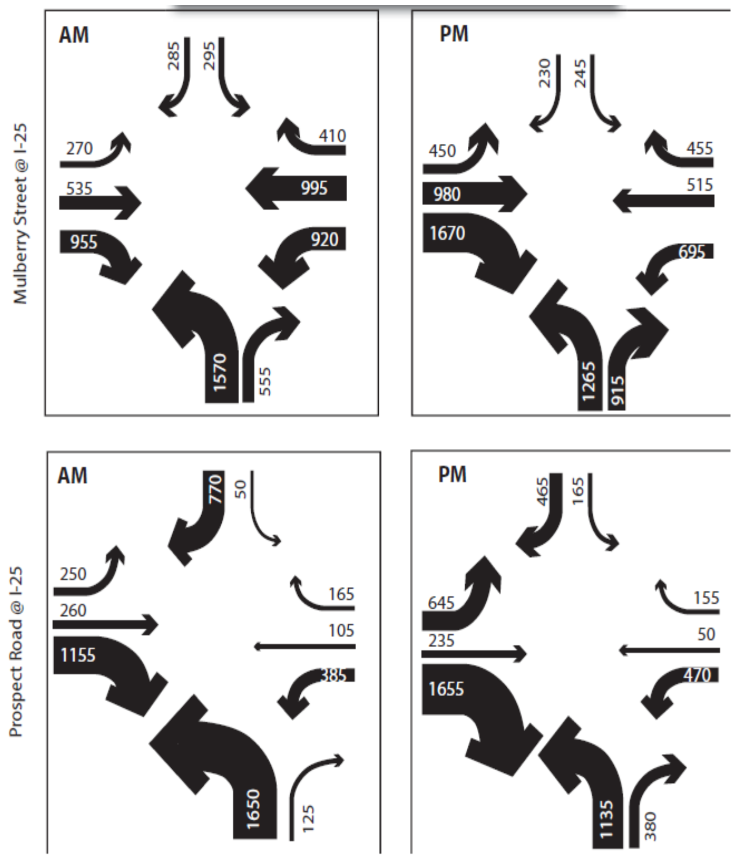
- At Mulberry, the frontage road access intersections serve relatively high traffic volume and numerous large trucks, and the intersections need to be directly adjacent to mainline Mulberry. This combination limits the possible design solutions to address intersection spacing, intersection capacity, intersection sight distance, etc.

- A continuous bike lane on both Mulberry and Prospect are challenging to incorporate safely, particularly in the eastbound direction due to the right turning volume at I-25. The EB right lane (#3 lane) will drop at I-25, and ideally the #2 lane will allow shared through-rights, complicating both the bike lane transition and the traffic weaving from the adjacent frontage road RIRO.

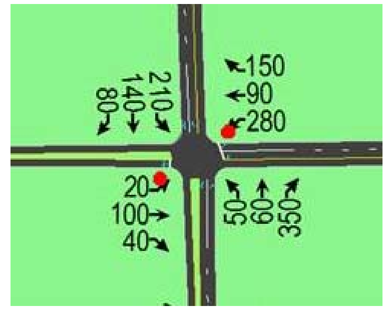
Traffic Volumes

The forecasted 2035 traffic volumes (from FEIS forecasts done in 2011) have been detailed in other project memorandums and are summarized in a figure on the next page. The PM turning volumes at the Mulberry frontage road intersections are also included in the following page.

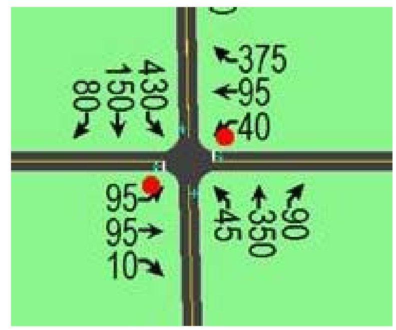
Traffic forecasting updates being done during 2013/2014 are likely to result in slightly lower 2035-2040 forecasts since the 2009/2010 recession will be incorporated. However, the general emphasis of heavy right turning traffic at the interchanges is unlikely to change.



North Frontage - PM



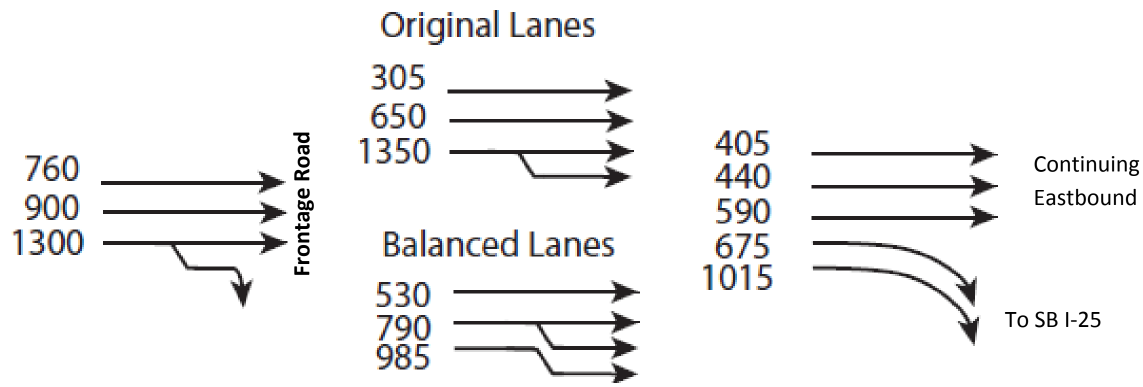
South Frontage-PM



The heavy right turning traffic toward I-25 at both interchanges and the analysis of operations at the ramp intersections led the project team to conclude that double right turns from EB to SB would be necessary at both Mulberry and Prospect. Double-right turns are used at several locations in the Denver/Boulder area, many of those have been retrofit designs. The opportunity exists to properly design the Mulberry and Prospect interchanges to better accommodate the traffic volumes and also incorporate pedestrian and bicycle lanes.

First, the transition of EB arterial lanes to the expanded number of lanes at each interchange was evaluated. In order to optimize the balance of traffic in lanes approaching the interchange, right turn traffic should be split into two EB lanes as soon as possible. If the #1 lane is the inside/median thru lane, the EB right lane (#3 lane) will drop at I-25, and ideally the #2 lane will allow shared through-rights. The approaching volume lane balance for the heavy PM peak traffic is shown on the following page

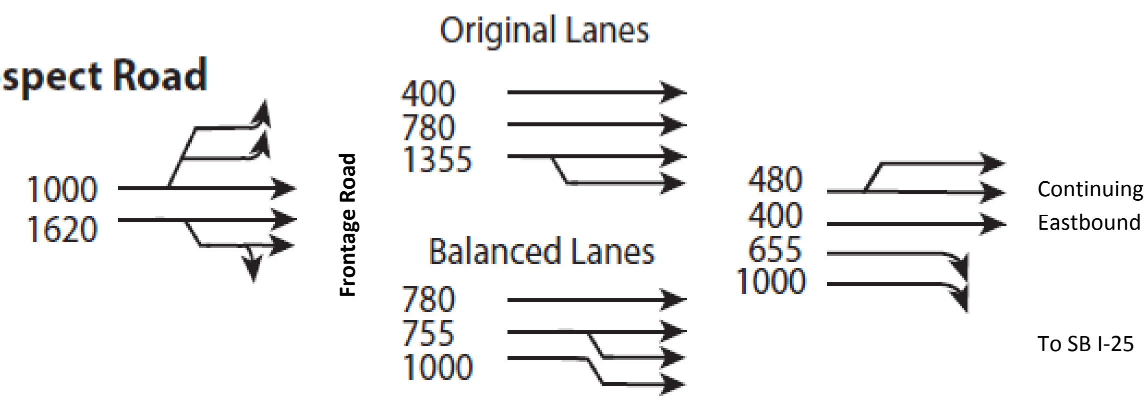
Mulberry Street



Approach volumes per Lane, west of frontage Road

Volume per lane with typical added 4th lane ("Original") and with lane split ("balanced") between Frontage and I-25

Prospect Road



Approach volumes per Lane, west of frontage Road

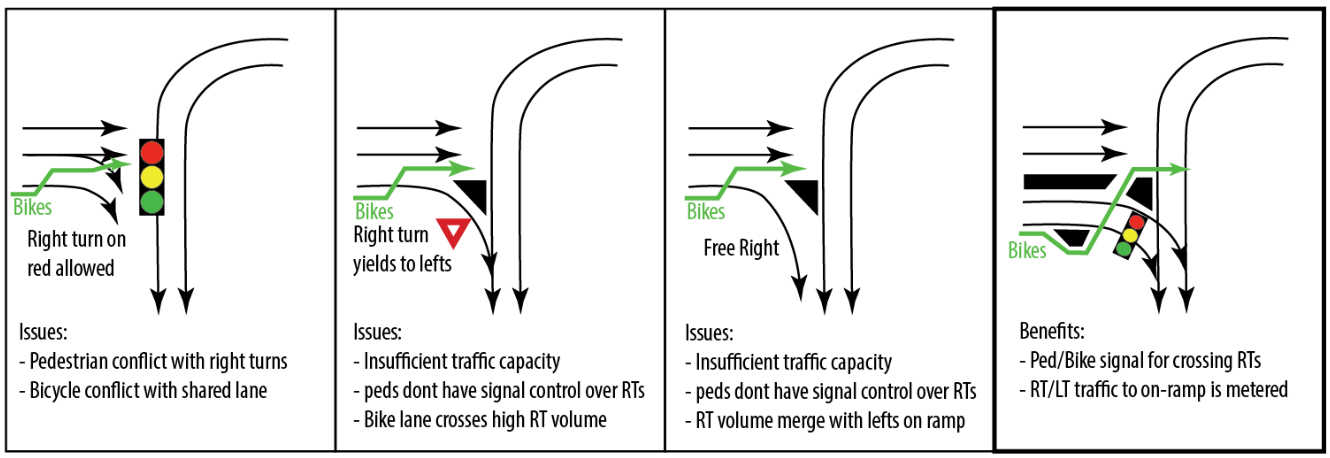
Volume per lane with typical added 4th lane ("Original") and with lane split ("balanced") between Frontage and I-25

Ramp Lanes

The signalized double-rights offer an opportunity to keep the number of ramp lanes for the SB on-ramp at both Mulberry and Prospect at two lanes. The signalized double-rights solve the traffic capacity, bike conflict, pedestrian accommodation, and also meter the on-ramp traffic so that additional ramp lanes would not be necessary.

Many diamond interchange on-ramps are designed for right turns to be added as a 3rd lane to the on-ramp. Typically the 3rd lane merges, and sometimes the two lanes merge to one prior to the gore point. The design options with the preferred on-ramp designs for both the Mulberry and Prospect interchanges is shown below:

Options for left+right turn lanes to freeway on-ramp



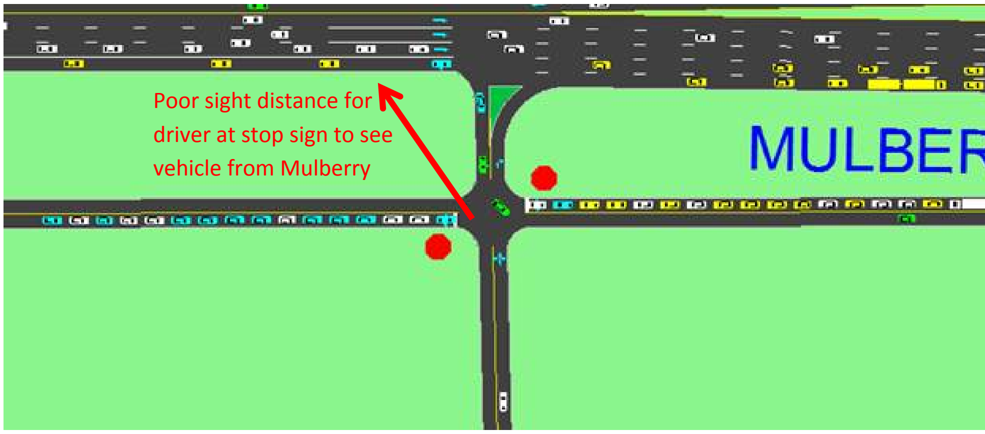
Recommended

Frontage Road Intersection Design and Traffic Control

The perpendicular intersections shown in the EIS concept have several operational flaws which make the perpendicular intersection type undesirable for this location. The RIRO accesses (plus local underpass connection) takes the place of a signalized access that serves interstate-oriented businesses, so traffic levels normally served by double-left turn lanes at a signal are condensed onto single right turn lanes, and intersecting at an unsignalized intersection.

- A two-way stop, as implied by the EIS concept, results in substantial side-street delay, although queuing is reduced back onto mainline Mulberry.
- An all-way stop intersection also does not have sufficient capacity and causes queuing back onto mainline Mulberry.
- Only a roundabout addresses the intersection capacity and queuing issues. This is shown in comparible SimTraffic screen shots below:

Two-Way stop as shown in EIS



LOS F for EB and WB approaches to stop signs

Screen shot of simulation for two-way stop control, south intersection. Queuing on the two stop-controlled approaches, minimal queuing back to Mulberry

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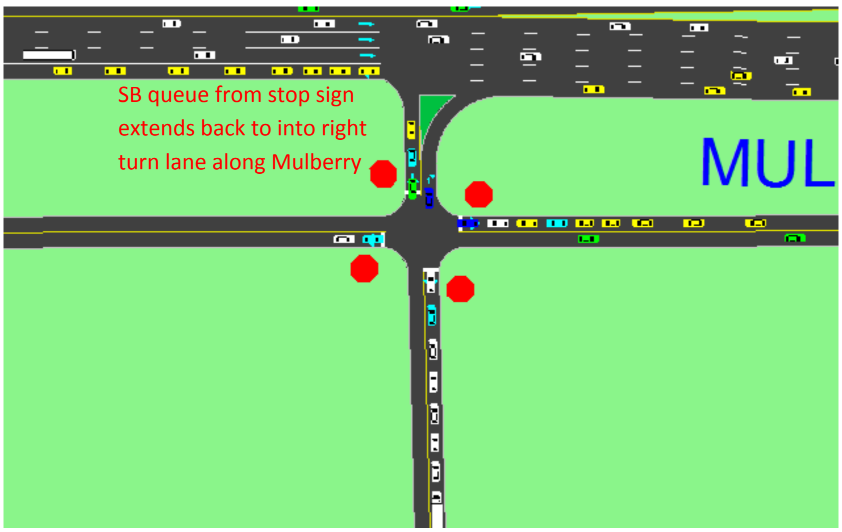
Option: All-Way Stop



LOS F for 3 of the 4 approaches, v/c 1.2 to 1.6

Screen shot of simulation for all-way stop control, south intersection. Inadequate capacity for all-way stop control, and queuing back onto Mulberry

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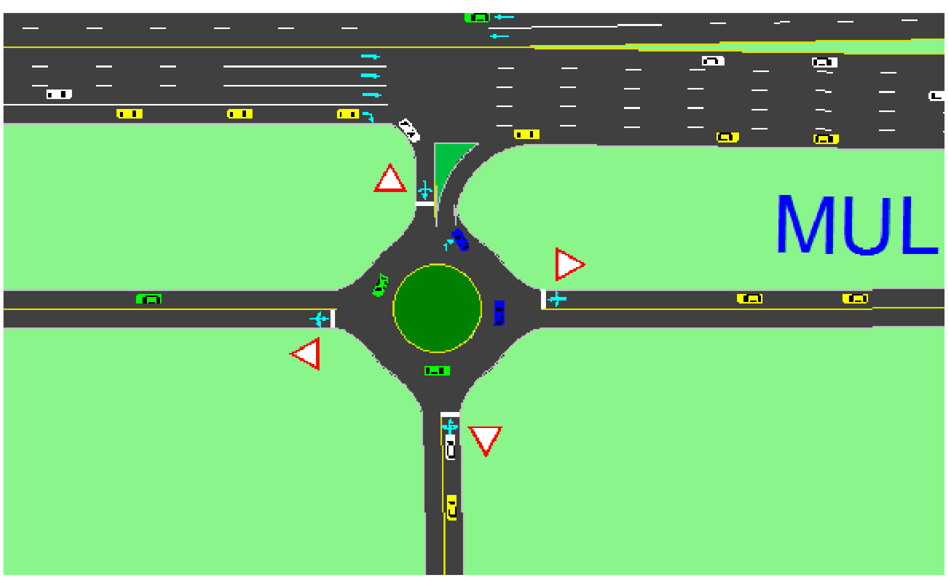
Option: Roundabout



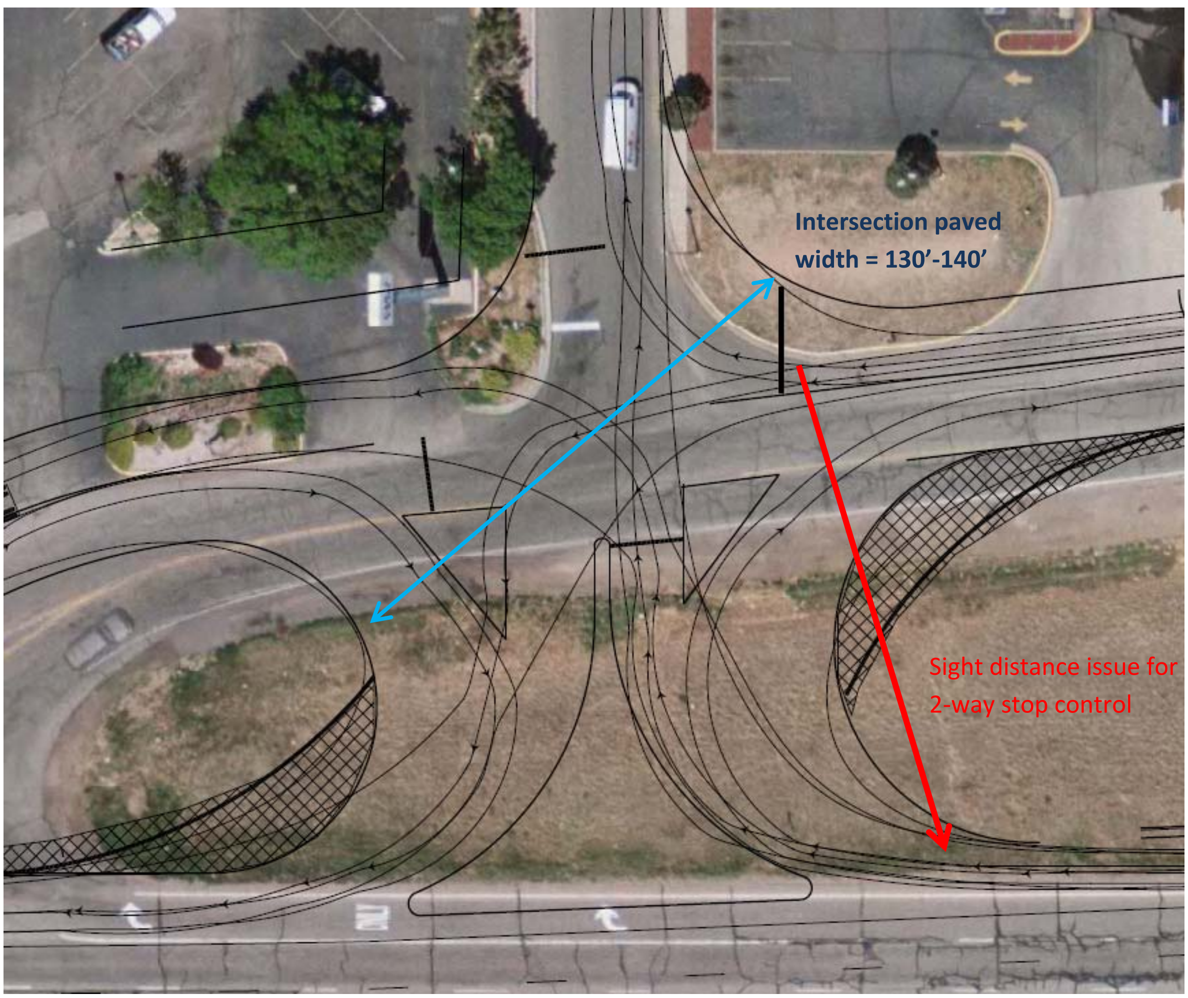
LOS D reported for HCM 2010, v/c of 0.62. Simtraffic simulation did not show any capacity issues

Screen shot of simulation for roundabout control, south intersection
Roundabout provides traffic capacity and does not cause queue back to Mulberry

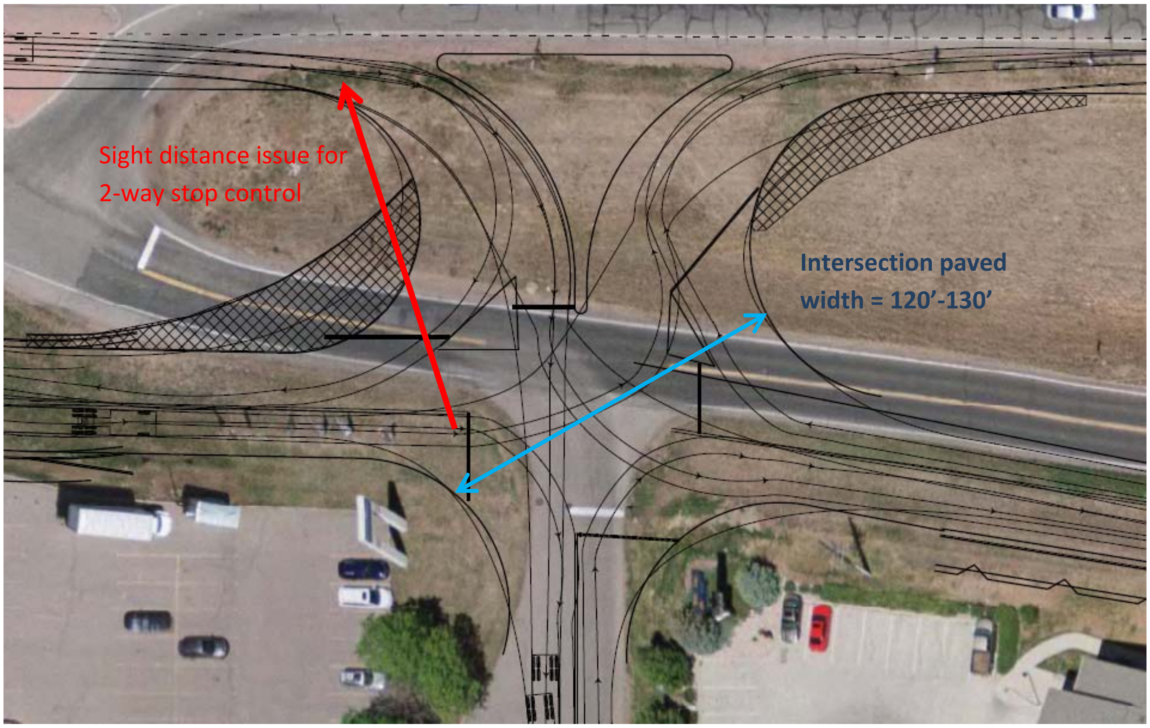
Besides traffic capacity there are several design issues with the frontage road intersections adjacent to mainline Mulberry:

- Sight distance, particularly for the two-way stop scenario shown in the EIS. Traffic on the EB frontage road approach would need to see traffic coming almost from behind (EB Mulberry traffic turning right to Frontage Road) in order to decide if they were clear to enter the intersection. See Figure below.

- Geometry for large trucks. The need to provide truck turning widths for all turning movements results in an intersection that is overly wide for its intended function. A layout of the north and south side frontage road intersections is shown below with the WB 67 truck movements overlaid. Several truck turns cross over the double-yellow line or raised islands for several left or right turn movements using this intersection configuration. The biggest challenge is the close spacing of the frontage road intersections to the Mulberry mainline, creating 180 degree turns for some large vehicles resulting in the expanding of the intersection footprint.



North-side Frontage Road, standard 4-way intersection layout with truck templates. Hatched areas could be raised truck aprons to discourage standard vehicle use.



South-side Frontage Road, standard 4-way intersection layout with truck templates
Hatched areas could be raised truck aprons to discourage standard vehicle use.

Proposed Design Alternative – both interchanges

The above challenges with the EIS concept and the additional need to incorporate bike lanes through the interchange led the project team to test several concepts to meet the unique needs of the west-side access on Mulberry. The following are the primary proposed solutions:

- For both Mulberry and Prospect, incorporate a separated double-right turn lane that develops out of a shared arterial lane. This approach offers better traffic distribution balance on the arterial lanes approaching I-25 and reduces the number of lane changes necessary by drivers.

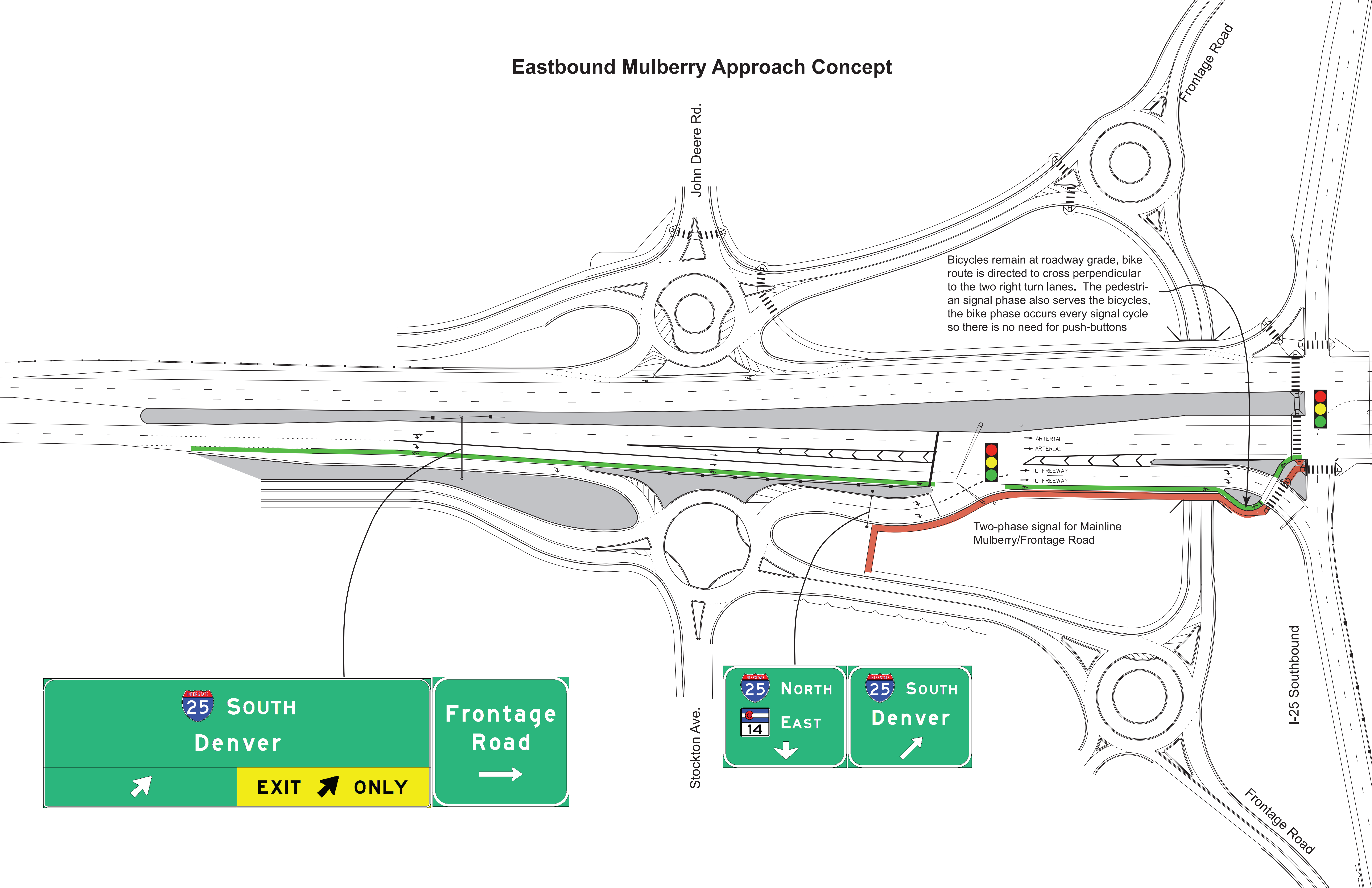
- At Mulberry the use of roundabouts as intersection control for the frontage road intersections. Roundabouts meet the intersection capacity needs and can be modified to accommodate large trucks without compromising the other intended functions of roundabout traffic control.

- Add a signalized access to eastbound Mulberry from the south-side frontage road. This signal eliminates the weave conflict as well as providing one option for a signal-protected on-street bike lane to continue east on Mulberry.

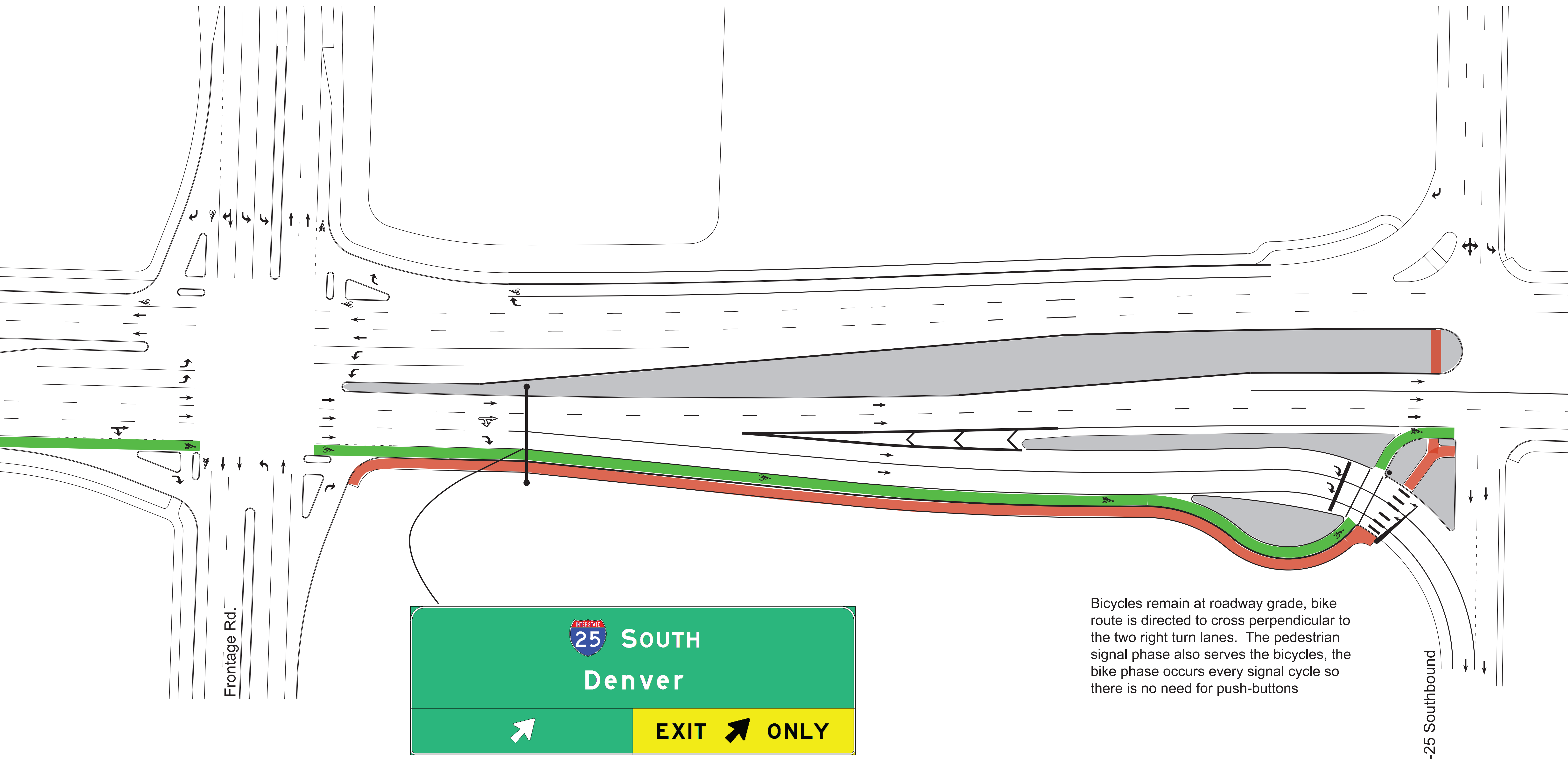
The concepts for each interchange are shown in the 11x17 figures attached. Comparable designs from other locations are shown in the appendix.

Appendix

Eastbound Mulberry Approach Concept



Eastbound Prospect Approach Concept

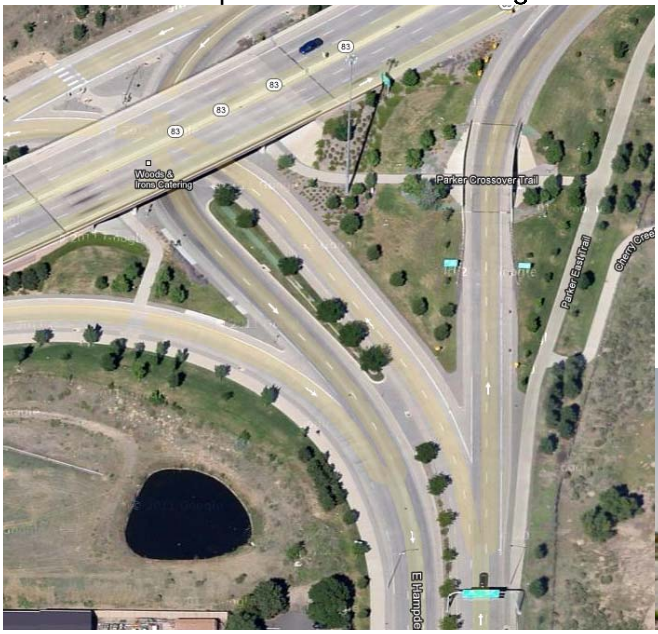


Bicycles remain at roadway grade, bike route is directed to cross perpendicular to the two right turn lanes. The pedestrian signal phase also serves the bicycles, the bike phase occurs every signal cycle so there is no need for push-buttons

Comparable Designs

While the idea of double-right turns at an interchange may seem unusual, there are numerous implementations of similar designs throughout the Denver area. While the traffic volumes are not available for all of these locations, it is likely they are in the same neighborhood as the 1,800 vph forecasted for Mulberry in 2035. It is notable that none of these locations has an on-street bike lane, most of the locations have either an off-street shared-use path or a simple attached sidewalk.

Parker Road (SH 83) and Hampden, Aurora, CO.
#2 lane split similar to Mulberry & Prospect concepts
Ped/bike underpass below double-rights



Arapahoe Road and I-25, Centennial, CO
No bicycle lane, peds cross two-lane ramp with no protection

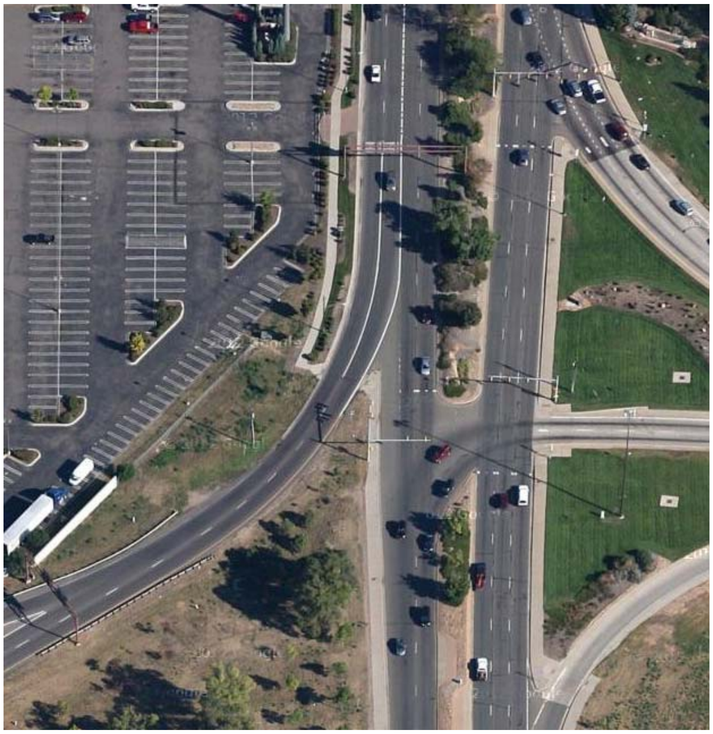
Baseline and US 36, Boulder

Peds/bikes have grade separated crossing on a detached path

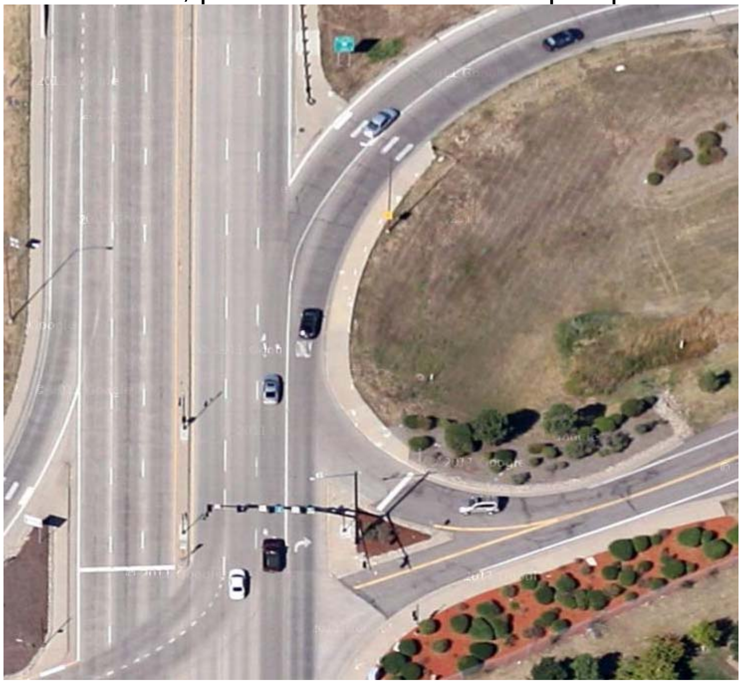
Note also the double-rights signalized at the adjacent intersection, similar to the Mulberry concept.

Wadsworth and I-70/I-76, Arvada, CO

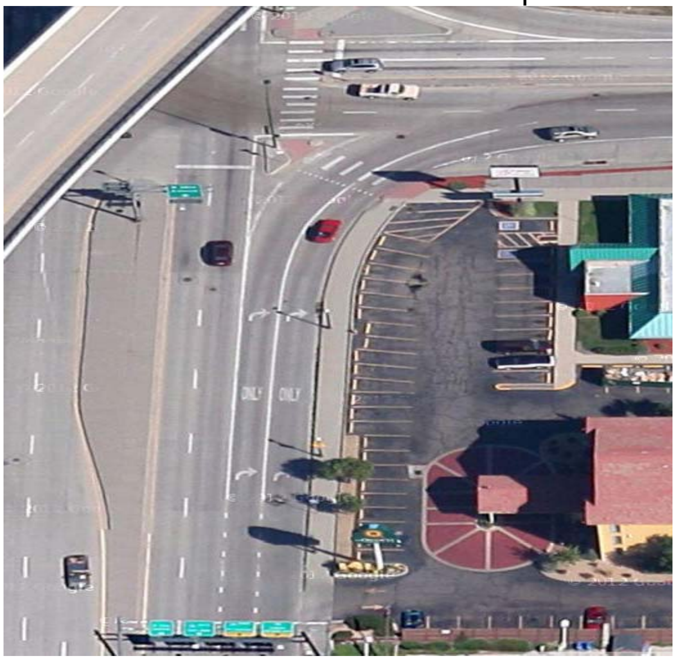
#3 and #4 lanes drop to the ramp. No bike lane, peds cross 2-lane ramp unprotected



Lincoln Blvd. and I-25, Lone Tree, CO
#4 Lane drops at I-25, #3 lane is shared thru-right
No bike lane, peds cross two-lane ramp unprotected



Park Avenue and I-25, Denver, CO
Double-rights yield, originally designed for signalization of double rights but the signals were never installed, perhaps due to low conflicting volume from lefts onto the ramp. Pedestrians cross two lanes with no protection.



Colorado Blvd. and I-25, Denver, CO – pedestrians cross both lanes with no signal protection