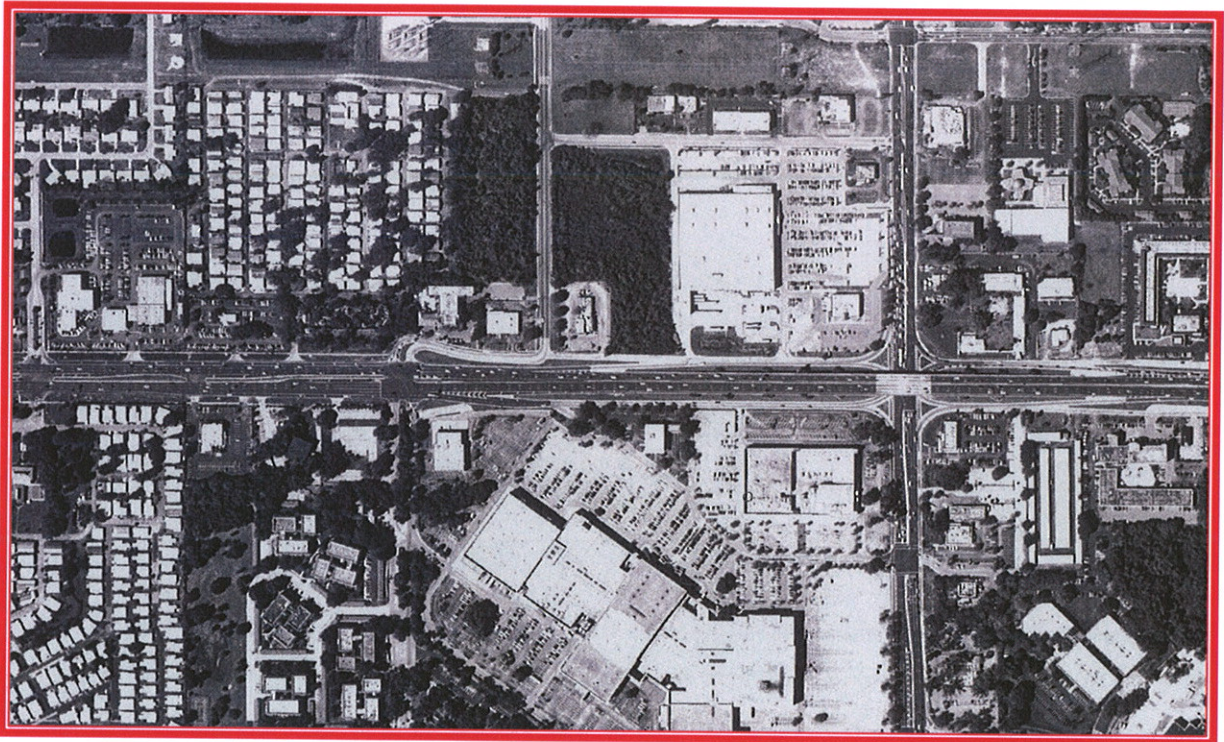


DRAFT SUMMARY REPORT

U.S. 19 SUB CORRIDOR REPORT

**U.S. 19 (Whitney Road to SR 60)
Interchange Feasibility Study
Pinellas County, Florida**



Prepared for



**Florida Department of Transportation
District Seven**

By

LOCHNER

May 2004

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DRAFT SUMMARY REPORT

PURPOSE OF THE REPORT

The purpose of the report is to investigate the feasibility of incorporating new grade separations between Whitney Rd. and SR 60 along US 19 with related facilities as a minimal segment to meet some of the deficiencies outlined in the 2020 Florida Intrastate Highway System (FIHS) Needs Plan for US 19.

BACKGROUND

Upgrading of the US 19 corridor to an enhanced arterial with frontage lanes and grade separations at major intersections has long been a goal of the Pinellas County Metropolitan Planning Organization's (MPO) Long Range Transportation Plan. In 1990, a Project Development and Environment (PD&E) Study was approved by the Federal Highway Administration (FHWA) for implementing the partially controlled access concept from Gandy Blvd. in Pinellas County to Alternate US 19 in Pasco County. Due to financial constraints over the last 10 years, a limited portion of the recommended options have been implemented along the corridor with large portions remaining for future improvements. The necessary improvements and associated costs to improve traffic operations along the remaining segments of US 19 were identified in the 2020 FIHS Needs Plan.

In November 2000, a Cost Estimate Update Study was completed that developed a mainline with frontage lane system concept that minimized right-of-way requirements to improve traffic operations along US 19 from 78th Ave. to Live Oak Street in Pinellas County. Subsequently, sub-corridor reports were done for US 19 through central Pinellas County, including studies at 118th Ave. North, Seville-Druid, Enterprise Rd. and Curlew Rd.. The 118th Ave. (and 110th Ave.) grade separations as well as the Enterprise Rd. grade separations are now under final design. Grade separations had been previously constructed at SR 688 (Ulmerton Rd.), SR 686 (East Bay), Whitney Rd., SR 60 (Gulf-to-Bay) and further north at Countryside Blvd. and SR 580 (Main Street). Once these improvements are completed, the only segment through mid-county not

addressed is the area from north of Whitney Rd. to south of SR 60. Exhibit 1 depicts the status of US 19 improvements from Gandy Blvd. to SR 580 in Pinellas County.

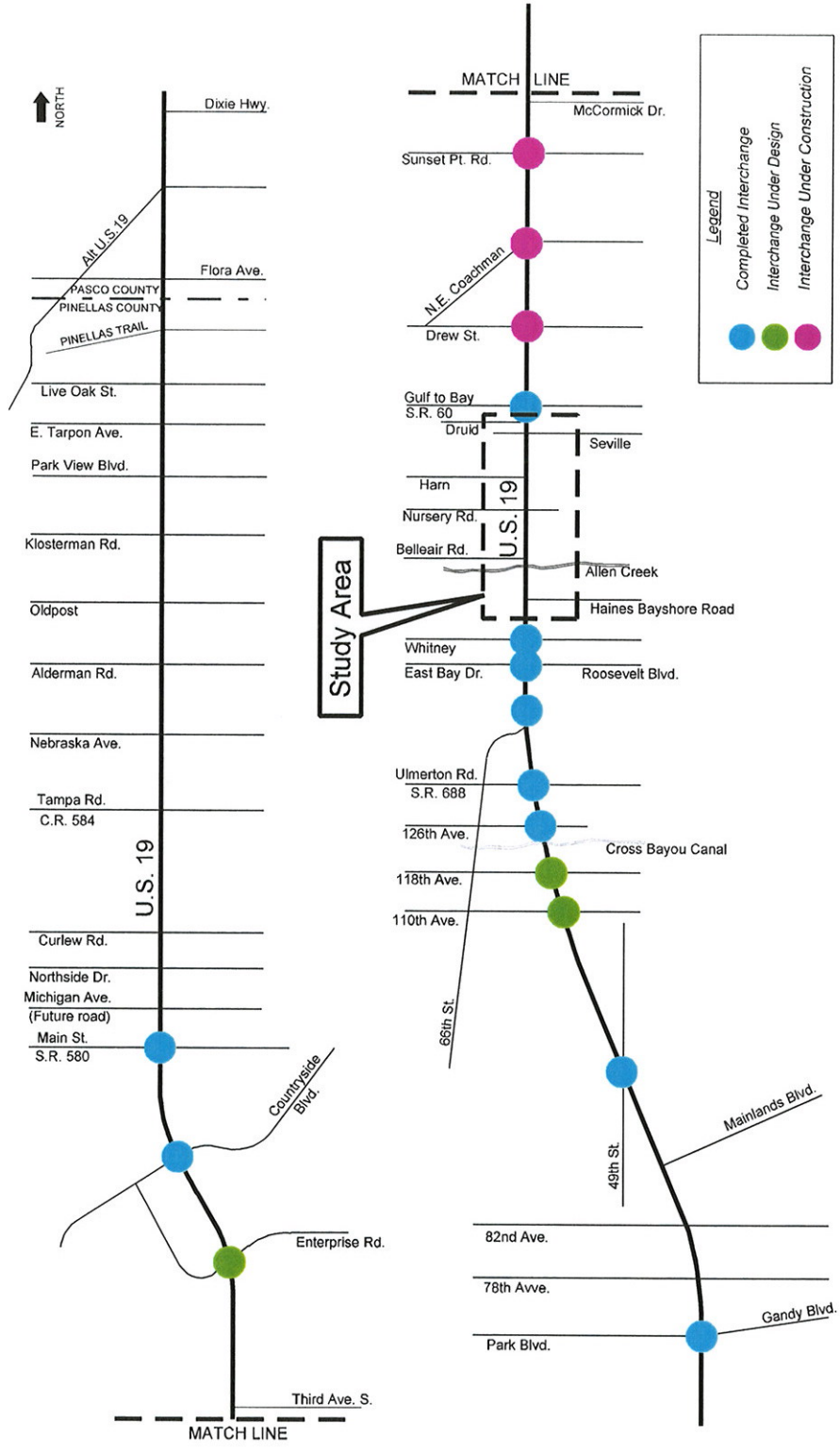
This Whitney Rd. to SR 60 Grade Separation Feasibility Study reviews the construction, operational improvements, and design considerations for several grade separation placement and design configurations as part of the overall improvement plan for US 19. Where appropriate, the grade separation design options are based upon a new design concept that was developed to minimize the right-of-way needs associated with implementing the improvements. This concept minimizes the required right-of-way cross-section by cantilevering mainline lanes and tucking frontage road left turn lanes under the cantilevered section. This concept has been endorsed by the Pinellas County MPO. The new design concept minimizes right-of-way needs by providing all the required improvements within the existing 200-foot wide right-of-way corridor. Therefore, right-of-way needs are minimized for construction and generally to drainage easements, corner clips, and storm water management facility (SWMF) locations.

STUDY METHODOLOGY

Unlike previous studies in the corridor, this study included a full range of activities ranging from modeling and development of updated future year volumes and design options through an operational analysis of the various options identified in the study. Through an iterative process, conceptual design options were developed to a degree necessary to determine ramp touchdown points, geometric constraints, insure constructability and to aid as input to the analysis effort.

The study methodology included the following steps which are presented in subsequent sections of the report.

- Corridor definition
- Data collection
- Optional configurations
- Development of future traffic
- Conceptual design
- Operational analysis



US 19 Interchange Status

U.S. 19 (Whitney-SR 60)
Interchange Feasibility Study
Pinellas County

Corridor Definition

Due to the proximity of adjacent grade separations as well as the influence of the operation of SR 60 at the north end of the study, a network approach was employed in this study, rather than a linear corridor. Through a combination of local knowledge of the corridor, observation of the corridor during peak periods and discussion with District staff, the study area was determined to include US 19 from north of Whitney Rd. to Drew Street, along SR 60 from Old Coachman/Eatonville Rd. on the west to Hampton Rd. on the east and Drew St from US 19 to Hampton Rd.. The study limits for data collection purposes is shown in Exhibit 2. Existing lane configurations at the intersections within the study limits are shown in Exhibit 3 and scaled distances between adjacent intersections are shown in Exhibit 4.

Data Collection

Eight hour turning movement counts were collected at twenty-one intersections within the study boundaries. Intersections counted included:

US 19 at Haines Bayshore	Old Coachman at SR 60
US 19 at Belleair Rd.	Old Coachman at Drew St.
US 19 at Nursery Rd.	Druid Rd at Edenville Ave.
US 19 at Harn Blvd.	SR 60 at Edenville Ave.
US 19 at Tropic Hills Dr.	Drew St at Hampton Rd.
US 19 at Seville Blvd.	SR 60 at Hampton Rd.
US 19 (west frontage road) at Druid Rd.	SR 60 at Sky Harbor Dr. (east side of Clearwater Mall)
US 19 at SR 60	SR 60 at Park Pl.
US 19 at Drew St.	SR 60 at first signal east of US 19
Druid Rd at Bypass Dr.	Drew St at Park Pl.
SR 60 at Bypass Dr.	

Due to the number of count locations and available manpower, counts were taken over a two day period in April 2002, prior to the start of significant construction on the Drew Street grade separation. At this time, the majority of the Clearwater Mall was closed; therefore the counts from the mall exits were recognized as not reflective of actual operating conditions. Fortunately, these exits generally are along SR 60 and thus are only outliers to the operation of US 19 itself.

Seventy-two hour “cordon” counts were taken at six locations during the period of the turning movement counts. These six locations were selected based on their ability to provide a multi-day picture of traffic flow within the study corridor and to function similarly to a cordon count. Since the turning movement counts were segmented geographically, continuous “cordon” counts were taken at the boundary (south of Seville Blvd.) which allowed for a “smoothing” of the count data so as to approximate a scenario as though they were all done on a single day.

During the hours of the turning movement counts, Consultant traffic staff monitored the traffic situation, driving all of the links multiple times and video-taping significant observations. This information was later reviewed during the CORSIM modeling to verify that the CORSIM runs for the existing conditions reflected operations as they were observed in 2002. Existing, seasonally adjusted, peak-hour counts are reflected in Exhibits 5 & 6.

Optional Configurations

Initially, four Build optional configurations in addition to the No-Build option were identified for study. During discussion with MPO and City of Clearwater officials, two additional Build options were identified and added to the list for consideration. All options, Build and No-Build, reflect the construction currently underway on US 19 at Drew Street, planned interim improvements at SR 60 and the grade separation scenario recommended in a prior report for the Druid Rd./Seville Blvd. intersection which included widening of southbound US 19 to six lanes over SR 60.

No-Build Option: For purposes of this study, the No-Build option was assumed to reflect the construction of the interim improvements previously identified for the US 19 / SR 60 grade separation. Those interim improvements included dual left turn lanes east and west bound on SR 60, a dedicated eastbound right turn lane on SR 60 approaching US 19, and provision for the north and south bound through movements along the US 19 frontage roads.

Build Options: All Build options assume a six-lane mainline US 19 with nominal two lane, one-way frontage roads on the east and west sides. The descriptions

presented below are intended to differentiate between the options. Each of the options are shown in generalized form in Exhibits 7 through 11.

Option A: This option consists of a mainline US 19 plus frontage roads with a single-point urban interchange (SPUI) at Nursery Rd. and a partial SPUI at Seville Blvd.

Option B: This option consists of a mainline US 19 plus frontage roads with SPUIs at Belleair Rd. and a partial SPUI at Seville Blvd.

Option C: This option consists of a mainline US 19 plus frontage roads with SPUIs at Nursery Rd. and Belleair Rd. and a partial SPUI at Seville Blvd. The ramps associated with Nursery and Belleair are located north of Nursery and south of Belleair to form a split diamond type configuration relative to US 19. This option was identified to address concerns expressed that Options A and B might each overload their respective cross-streets. Widening of Nursery and/or Belleair was not considered viable due to the nature of these roadways, treed two-lanes through residential areas.

Option D: This option consists of a mainline US 19 plus frontage roads with SPUIs at Harn, Nursery and Belleair (but without ramps associated with these overpasses) and a partial SPUI at Seville Blvd. This allows traffic utilizing these three east-west roads to easily access both directions of US 19 along the frontage roads but forces all local traffic between Whitney Rd. and Seville Blvd. to remain on the frontage roads.

Option E: This option consists of a mainline US 19 with one-way frontage roads on each side but with no overpasses between Whitney Rd. and Seville. Thus, all local traffic between Whitney Rd. and Seville Blvd. remains on the frontage roads and only enters or exits mainline US 19 at the ends of the section.

Option F: This option is similar to Option E except that the frontage roads would be two-way on each side of the mainline between Whitney Rd. and SR 60. This option provides increased accessibility to properties abutting the frontage roads.

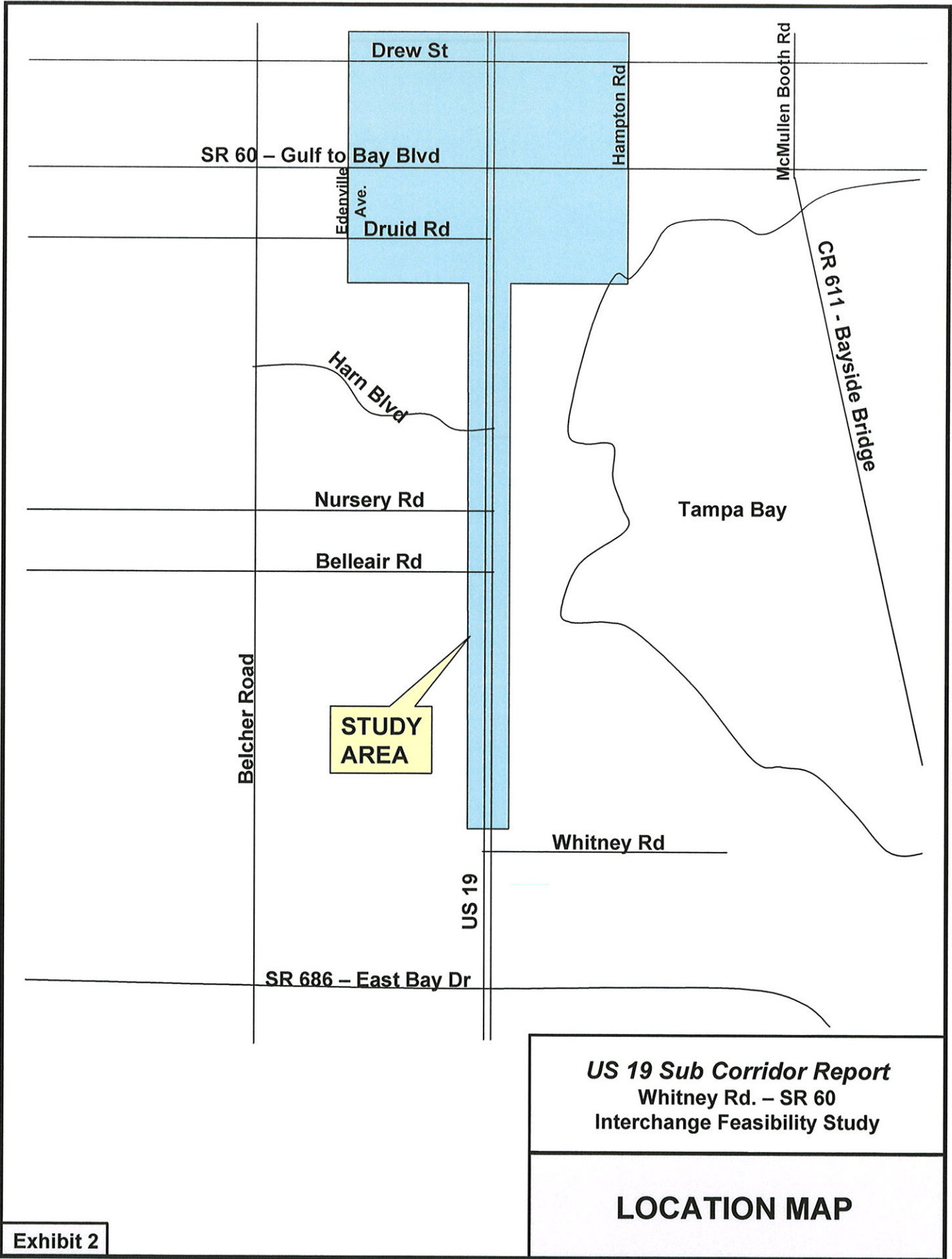
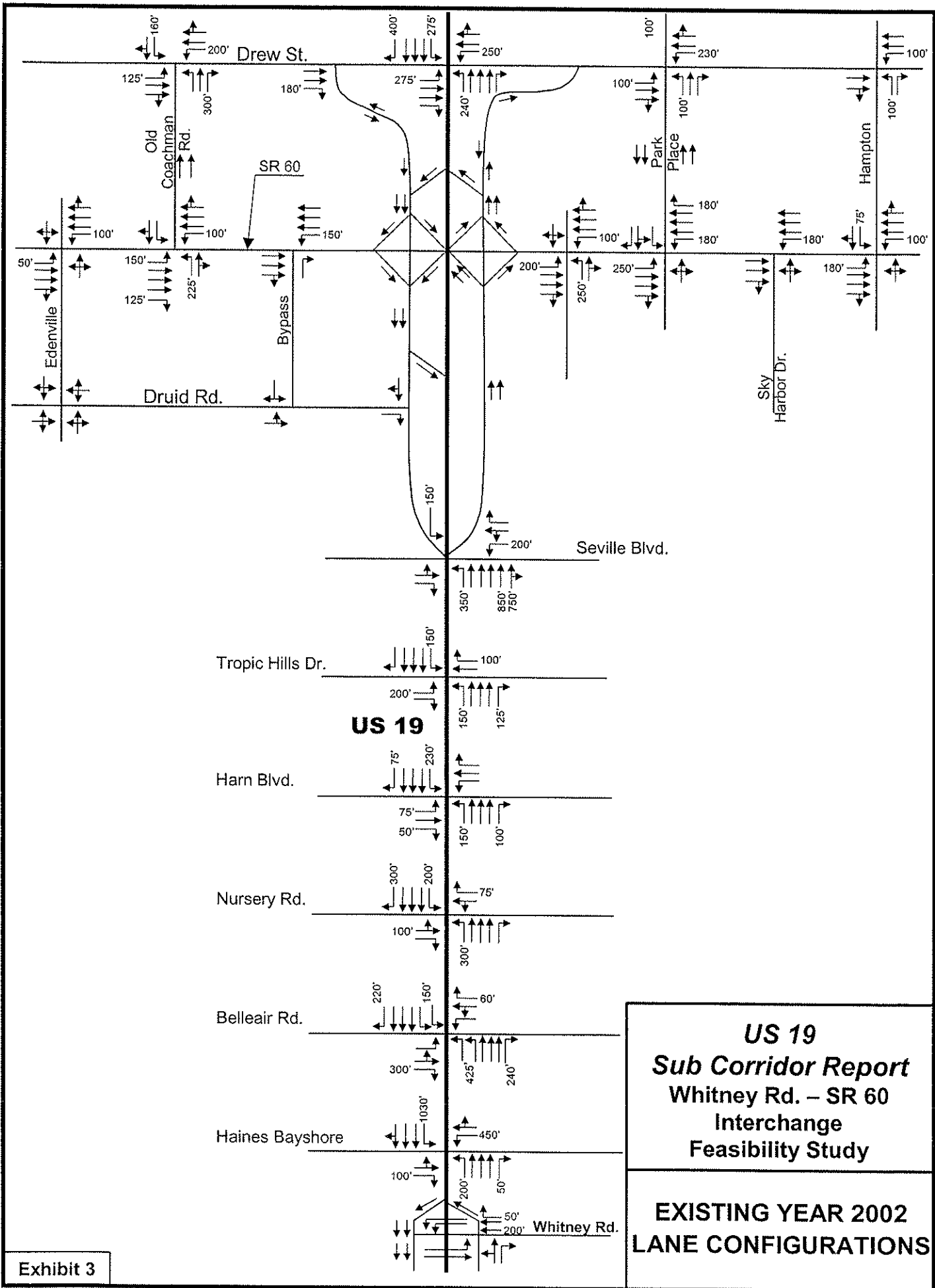


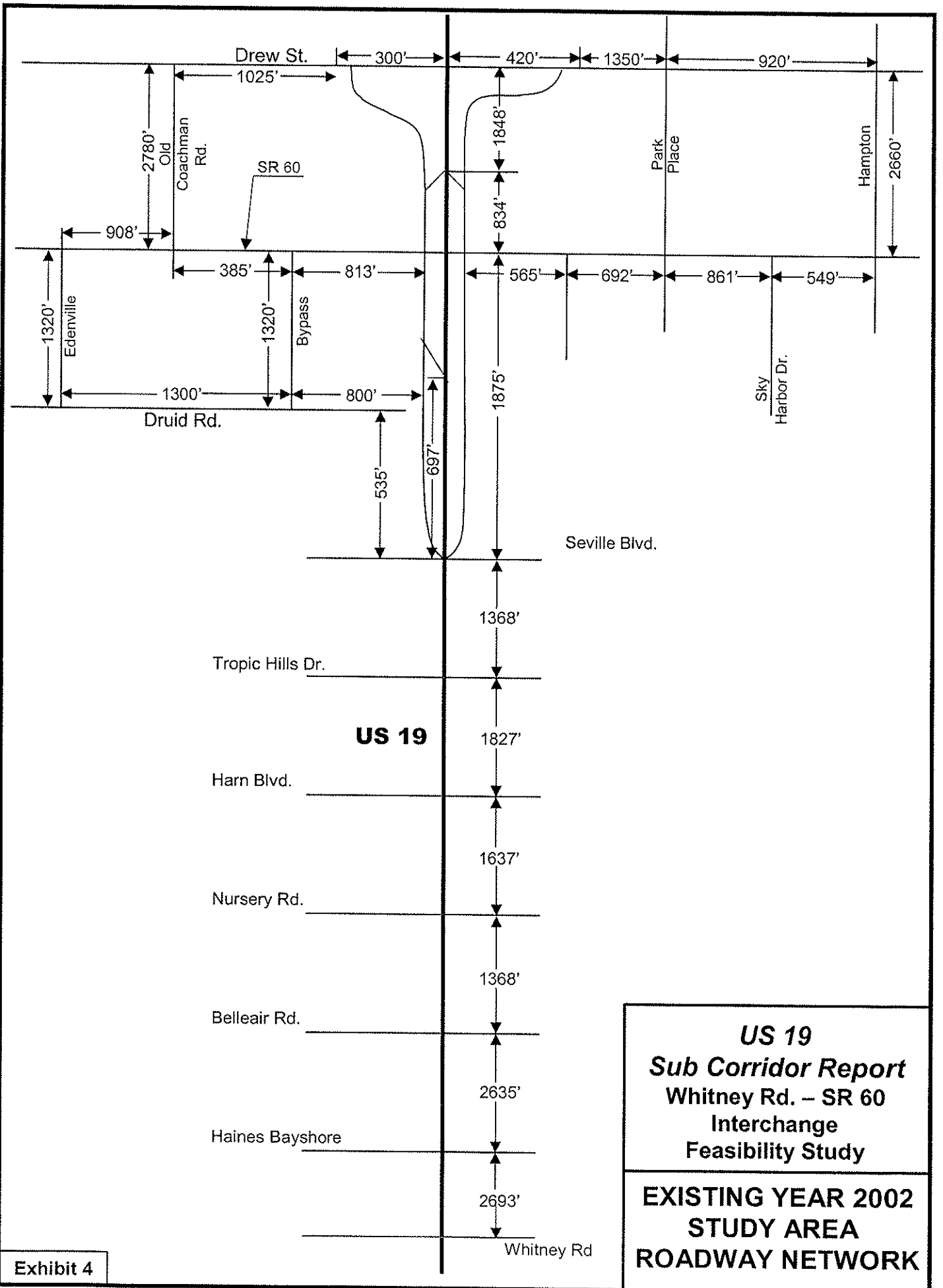
Exhibit 2



US 19
Sub Corridor Report
Whitney Rd. – SR 60
Interchange
Feasibility Study

EXISTING YEAR 2002
LANE CONFIGURATIONS

Exhibit 3



Drew St.

300'

420'

1350'

920'

1025'

2780'

Old Coachman Rd.

SR 60

1848'

834'

Park Place

Hampton

2660'

908'

385'

1320'

Bypass

813'

565'

692'

861'

549'

Sky Harbor Dr.

1320'

Edenville

1300'

Druid Rd.

800'

1875'

Seville Blvd.

1368'

Tropic Hills Dr.

US 19

1827'

Harn Blvd.

1637'

Nursery Rd.

1368'

Belleair Rd.

2635'

Haines Bayshore

2693'

Whitney Rd

US 19

Sub Corridor Report

Whitney Rd. – SR 60

Interchange

Feasibility Study

EXISTING YEAR 2002

STUDY AREA

ROADWAY NETWORK

Exhibit 4

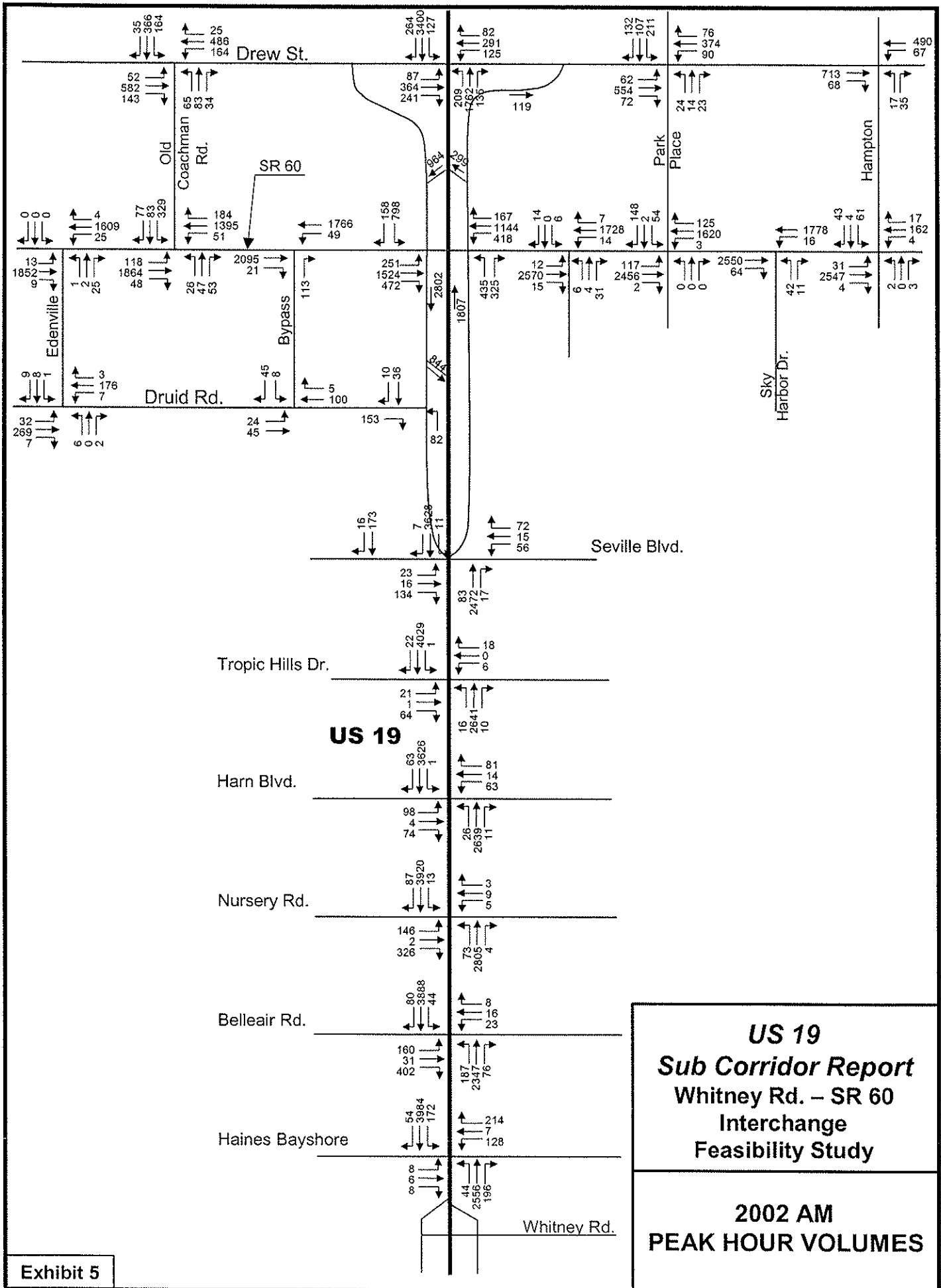


Exhibit 5

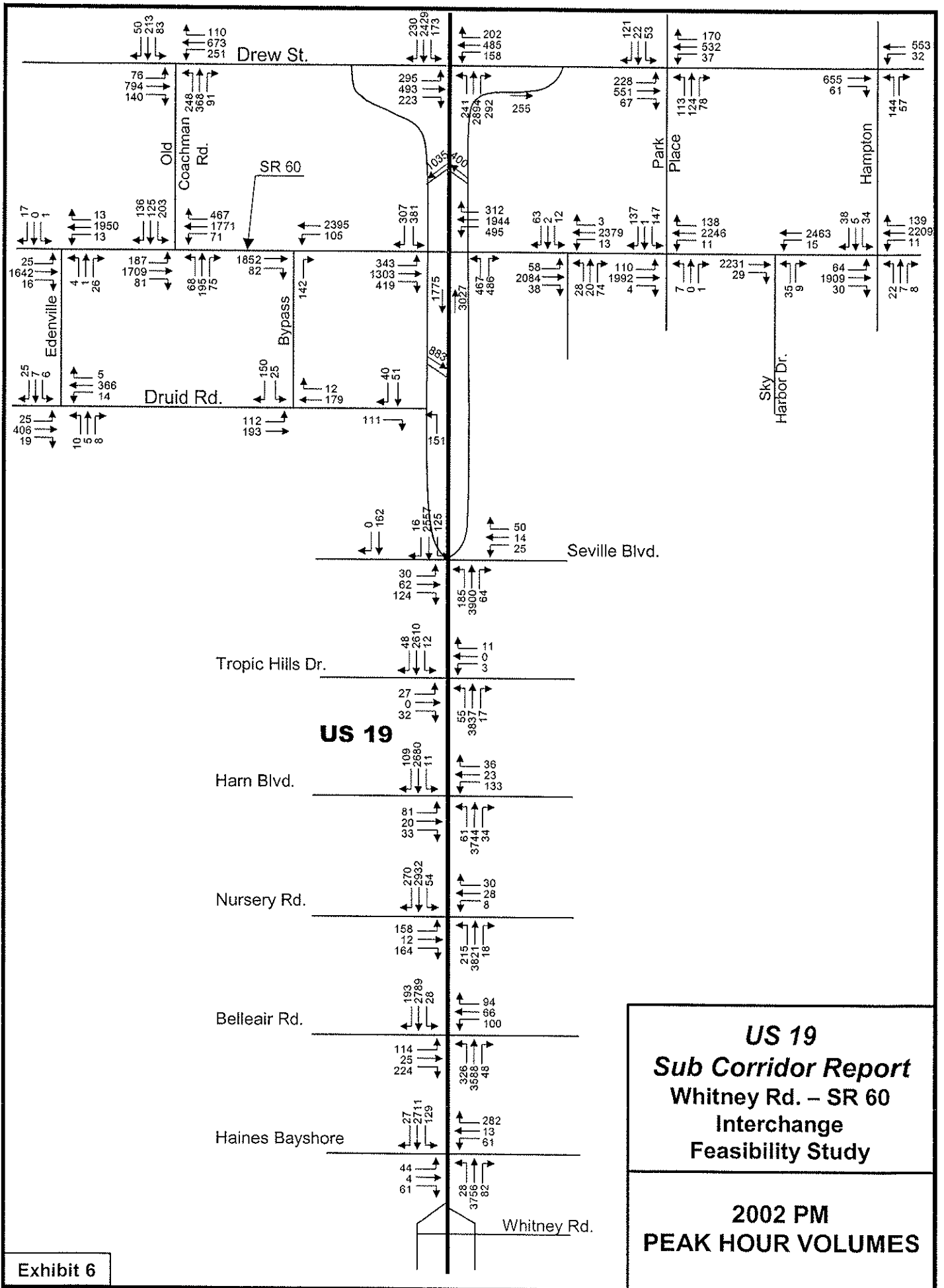
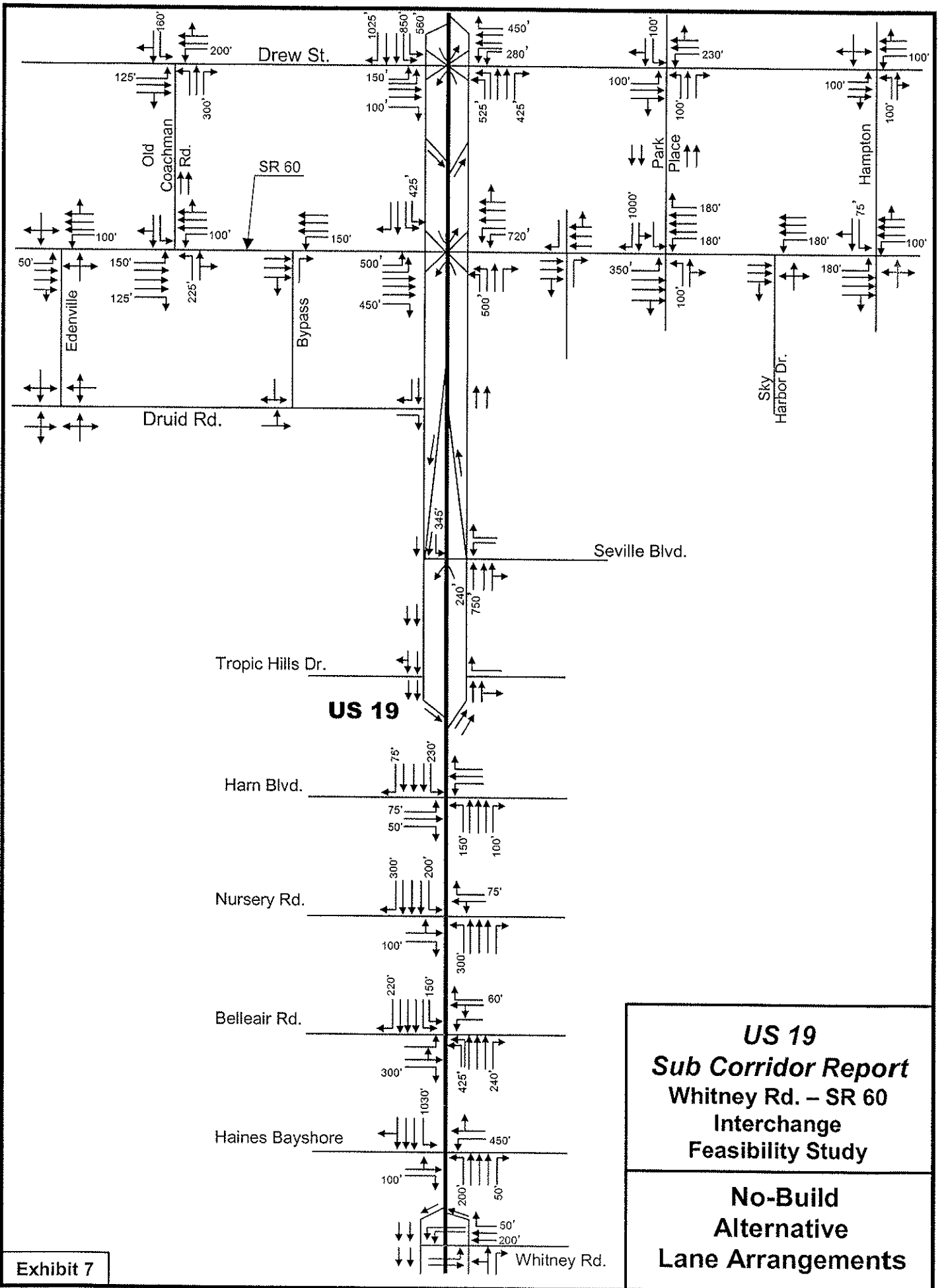


Exhibit 6

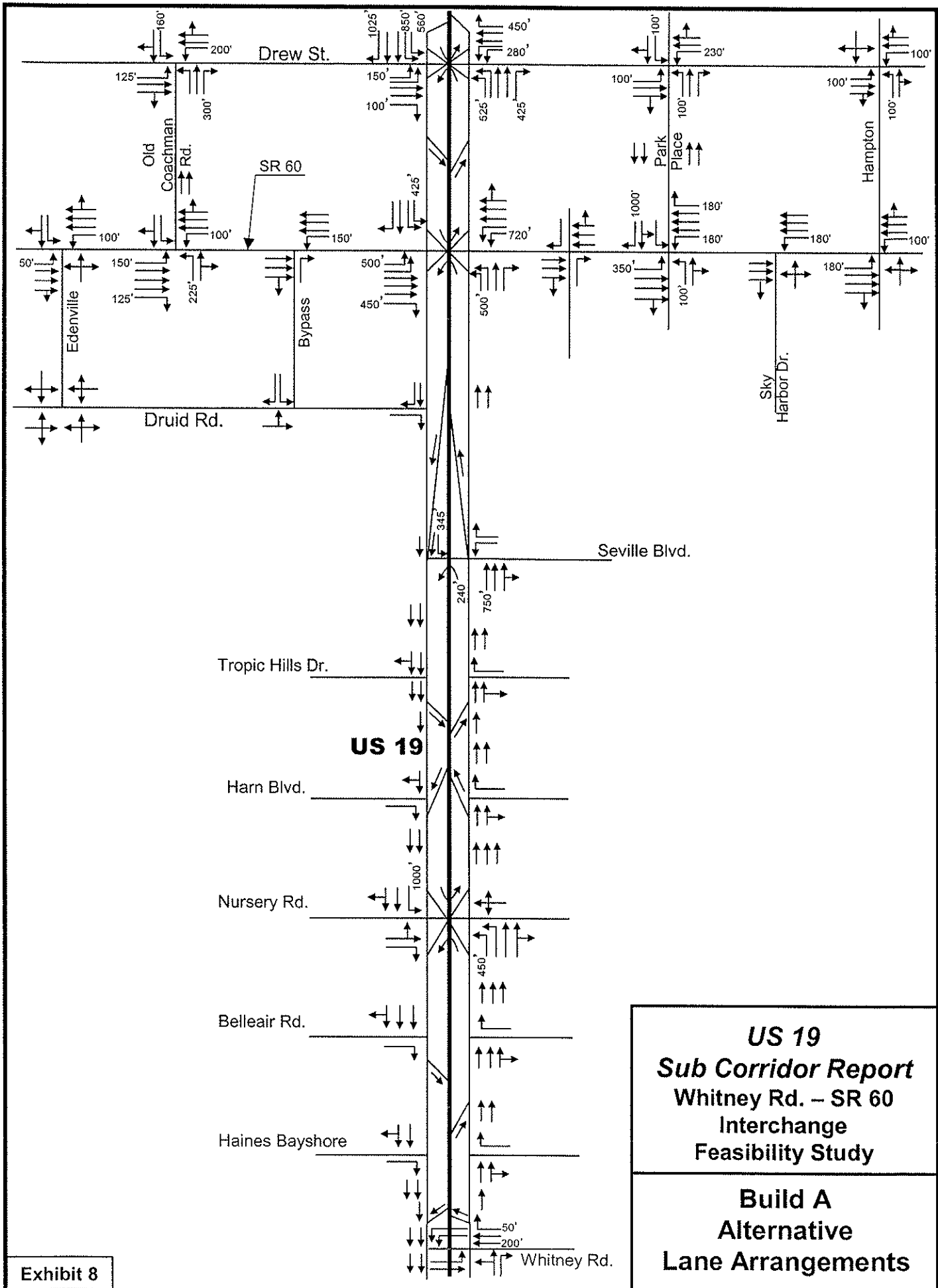
US 19
Sub Corridor Report
Whitney Rd. - SR 60
Interchange
Feasibility Study

2002 PM
PEAK HOUR VOLUMES



US 19
Sub Corridor Report
Whitney Rd. - SR 60
Interchange
Feasibility Study

No-Build
Alternative
Lane Arrangements



US 19
Sub Corridor Report
Whitney Rd. – SR 60
Interchange
Feasibility Study

Build A
Alternative
Lane Arrangements

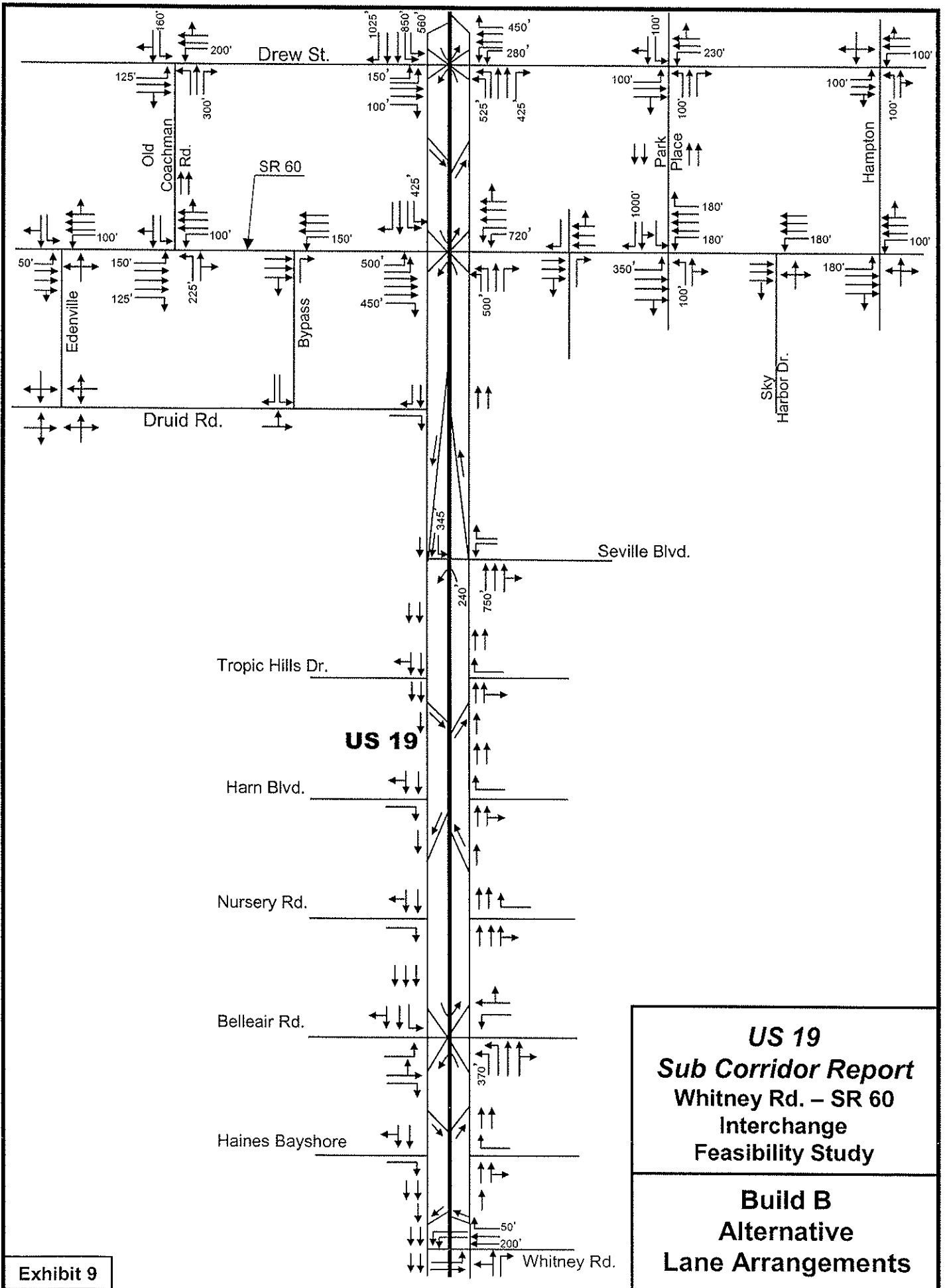
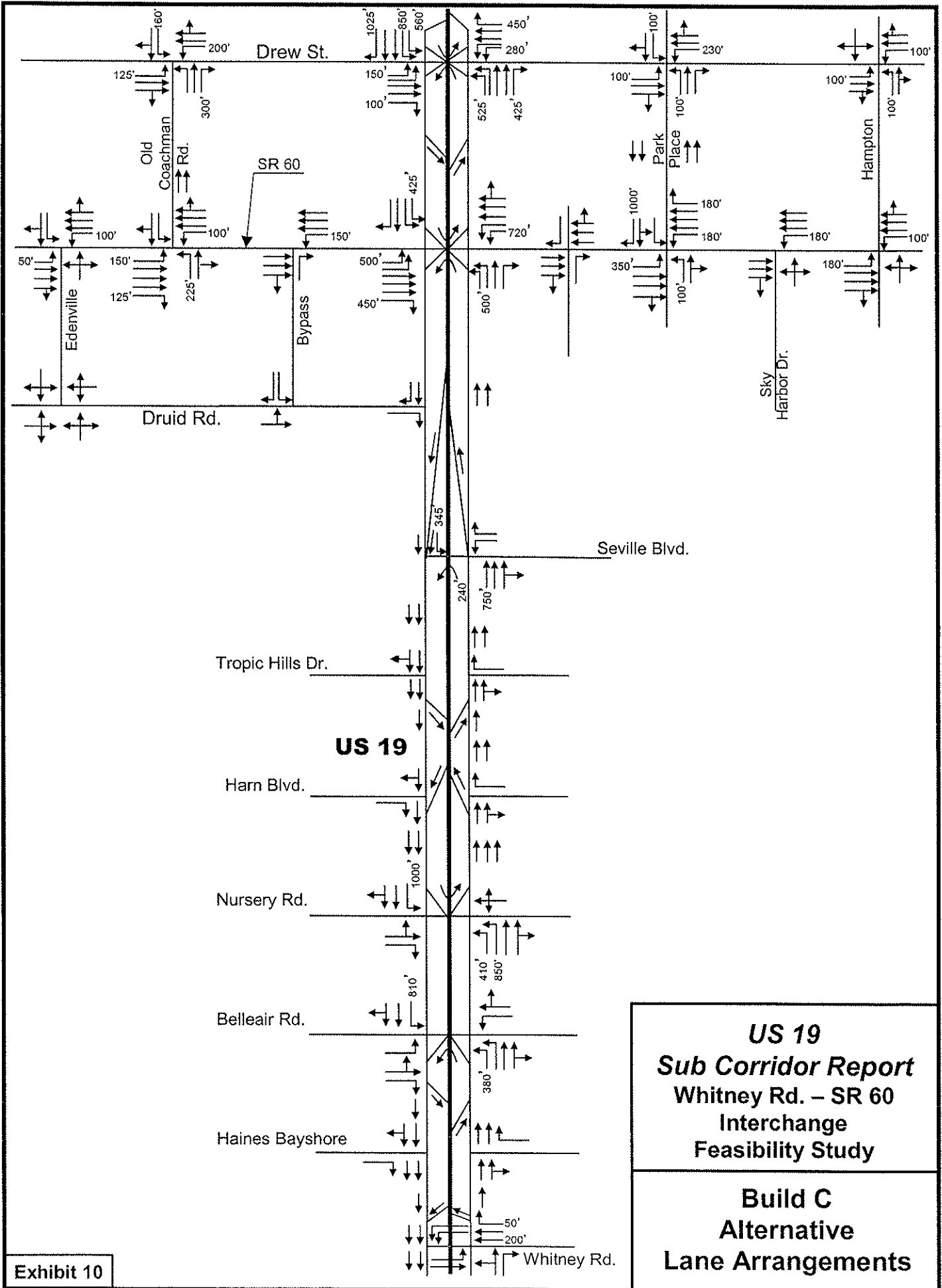
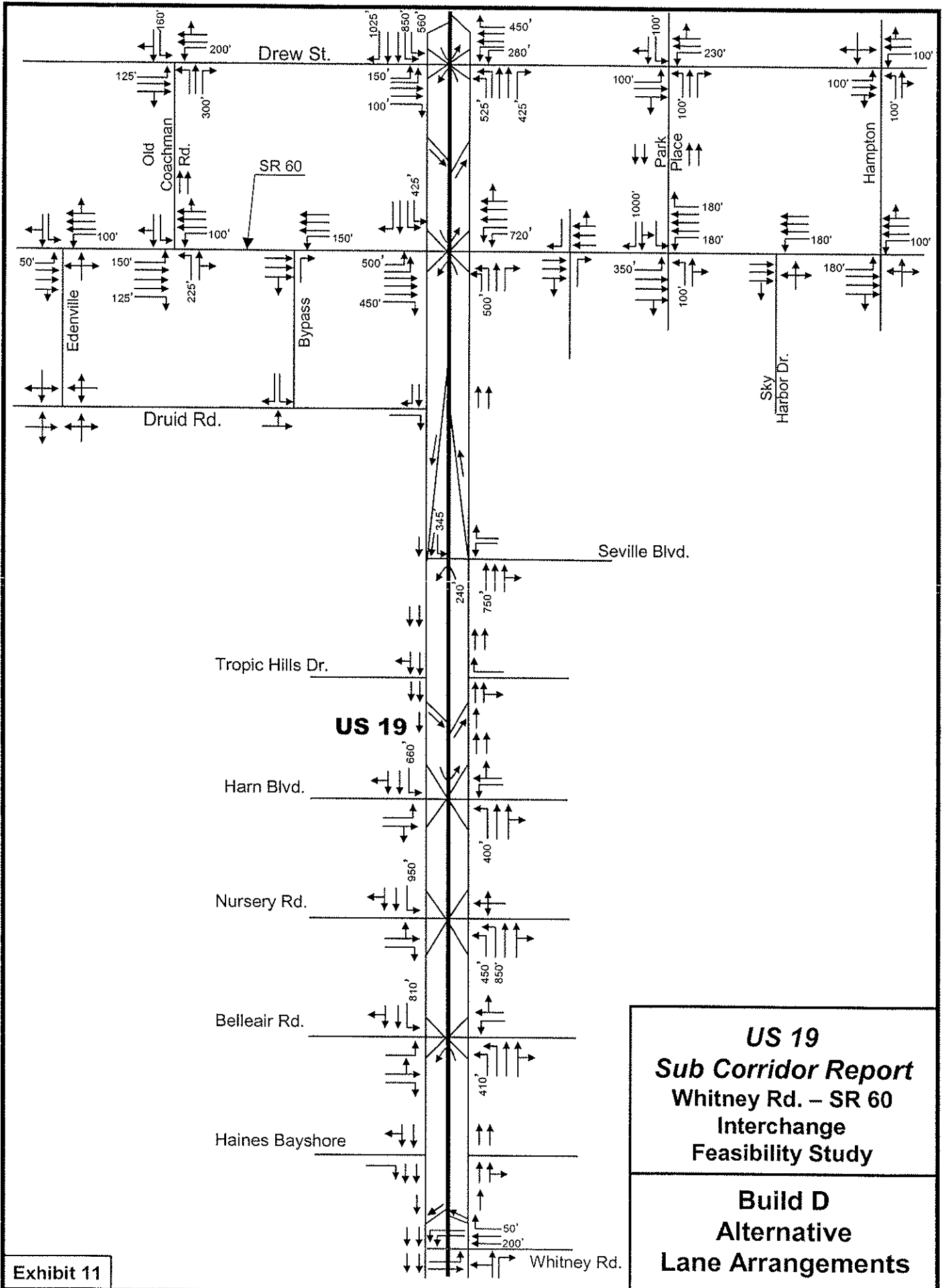


Exhibit 9



US 19
Sub Corridor Report
Whitney Rd. – SR 60
Interchange
Feasibility Study

Build C
Alternative
Lane Arrangements



US 19
Sub Corridor Report
Whitney Rd. - SR 60
Interchange
Feasibility Study

Build D
Alternative
Lane Arrangements

Exhibit 11

Initial Screening of Options

In the initial phase of the study, each potential option was brainstormed for feasibility and any fatal flaws. Each of the options was found to contain positives and negatives; however, options E and F were determined to be sufficiently flawed as to warrant being dropped from further consideration.

Option E was ruled out due to the circuitous requirements for traffic anywhere on either frontage road to access the opposing direction. For example, traffic exiting Congo River Golf on the southbound frontage road would have to travel south to the Whitney Rd. intersection, U-turn and travel along the frontage road to north of Seville Blvd. to access the northbound US 19 mainline. This is a travel distance of approximately four miles. While this is an extreme case, the average distance to effectively accomplish a U-turn is about two miles which is considered to be unreasonable.

Option F was also ruled out due to the operational problems associated with having two-way frontage roads on either side of the SPUI grade separations. The resulting phasing and handling of movements under these conditions was determined to be operationally flawed to the point that the option was deemed not feasible.

DEVELOPMENT OF FUTURE TRAFFIC:

The Tampa Bay Regional Model (TBRPM) Version 4.0 was used to estimate future year traffic volumes. The model generated Year 2025 volumes which were then expanded to project Year 2030 volumes per the study scope. Historical corridor volumes were reviewed and annual growth rates of 1.4% and 0.5% were estimated for the US 19 corridor and for the cross streets respectively. These percentages were then applied to the Year 2025 results to get Year 2030 traffic volumes.

1999 Model Validation

The sub-area model validation for this corridor within the US 19 study area was reviewed to determine model performance in the study area. Year 1999 model traffic

results were adjusted from peak season traffic to annual average daily traffic (AADT) and compared with observed 1999 traffic counts at various FDOT count stations and Pinellas County counts.

A comparison of the 1999 TBRPM model results with the observed 1999 AADT counts is presented in Table 1. From a review of this data, the 1999 TBRPM overestimate traffic at several US 19 count locations. Therefore, network review and adjustments needed to better match existing conditions and centroid loadings were made. Based on the reviews, the following adjustments to the base 1999 model network were made:

1. Missing on and off-ramps were coded at the grade separation of US 19 and East Bay Dr.
2. Recoded the US 19 intersection at Druid Rd. and Seville Blvd. to better represent the existing conditions.
3. Non-existent northbound on-ramp at US 19 south of SR 60 was removed.
4. Missing through movement prohibitors were provided at the intersections of SR 60 and northbound and southbound frontage roads.
5. The number of lanes along US 19 frontage roads south of Drew Street were adjusted to match the existing conditions.
6. Missing turn prohibitors were provided at the intersection of Enterprise Rd. and US 19 southbound frontage road.
7. Non-existent northbound off-ramp south of Countryside Blvd. (in addition to off-ramp north of Enterprise Rd.) was removed.
8. Missing existing on and off-ramps were added at the grade separation of US 19 and Main Street (SR 580).
9. Old Coachman Rd. between SR 60 and Sunset Blvd. was added to the network as an Undivided Arterial. In addition, centroid connectors were added to provide access to Old Coachman Rd. from centroids for traffic analysis zones (TAZs) 895, 896, 908, 910, 932, 933 and 934.

10. Bypass Dr. between Druid Rd. and SR 60 was added to the network as a local collector roadway and a centroid connector was added to provide access from centroid for traffic analysis zone (TAZs) 947.
11. The centroid connector to the US 19 Frontage Rd. from traffic analysis zone (TAZ) 1060, located east of US19 and south of 150th Street, was removed to better reflect existing access to and from TAZ 1060.
12. The centroid connector to the US 19 Frontage Road from traffic analysis zone (TAZ) 1001, located west of US19 and north of East Bay Dr., was moved from Whitney Rd. node to south of Whitney Rd. node to better reflect existing access to US 19 Frontage Road from area businesses and residents.
13. Two new centroid connectors for traffic analysis zone (TAZ) 968, located east of US 19 and north of Nursery Rd., were added and connected to US 19 Frontage Road at the Belleair Rd. intersection and at the north of Nursery Rd. to better represent access from this TAZ.
14. A new centroid connector for traffic analysis zone (TAZ) 958, located east of US 19 and south of Seville Blvd. were added and connected to the US 19 Frontage Road / Seville Blvd. intersection to better represent access from this TAZ.
15. The facility type (FT) of US 19 was changed from FT 22 (Divided Arterial Unsignalized) to FT 23 (Divided Arterial Class 1a) for the following highway segments.
 - From Whitney Rd. to Druid Rd.
 - From Drew Street to Enterprise Rd.
 - From north of Main Street to Curlew Rd.
16. The facility type (FT) of US 19 for the highway segment immediately south of Drew Street was changed from FT 16 (Controlled Access Expressway) to FT 23 (Divided Arterial Class 1a).
17. The facility type (FT) of SR 60 from Old Coachman Rd. to Hampton Rd. was changed from FT 23 (Divided Arterial Class 1a) to FT 24 (Divided Arterial Class 1b).

18. The facility type (FT) of East Bay Dr. from Belcher Rd. to US 19 was changed from FT 23 (Divided Arterial Class 1a) to FT 24 (Divided Arterial Class 1b).
19. The facility type (FT) of Belcher Rd. from East Bay Dr. to Ulmerton Rd. was changed from FT 25 (Divided Arterial Class II/III) to FT 24 (Divided Arterial Class 1b).

By applying these network changes, the revised 1999 TBRPM reasonably matched the 1999 observed traffic counts. A comparison of the revised 1999 model results with the observed 1999 traffic counts is presented in Table 2. This table shows that although some percentage differences seem high, they are within the acceptable National Cooperative Highway Research Program (NCHRP) range. Based on this review, the modified 1999 model appears to reasonably reflect 1999 conditions.

Table 1. 1999 AADT Comparison of TBRPM Forecast Volumes with 1999 Observed Traffic Counts

Count Location	No. of Lanes	1999 Model AADT	1999 Traffic Counts	Volume / Count	Percent Deviation	Within NCHRP Range?
US 19 / SR 55						
South of East Bay Dr.	6	70,150	52,500	1.34	34	No
North of East Bay Dr.	6	101,800	74,000	1.38	38	No
South of Seville Blvd.	6	106,550	83,000	1.28	28	No
South of Drew Street	6	101,650	73,500	1.38	38	No
North of Drew Street ¹	6	90,750	71,500	1.27	27	No
North of Coachman Rd.	6	105,500	82,000	1.29	29	No
North of Sunset Point Rd. ¹	6	94,100	74,800	1.26	26	No
North of Main Street	6	89,450	72,000	1.24	24	No
North of Curlew Rd.	6	75,450	70,500	1.07	7	Yes
Mc Mullen Booth						
Bayside Bridge ¹	6	55,400	62,450	0.89	(11)	Yes
South of Drew Street ¹	6	54,300	60,500	0.90	(10)	Yes
North of Drew Street ¹	6	60,950	69,600	0.88	(12)	Yes
South of Curlew Rd. ¹	6	48,050	56,000	0.86	(14)	Yes
Belcher Rd.						
North of East Bay Dr. ¹	4	30,100	28,250	1.07	7	Yes
North of Nursery Rd. ¹	4	24,450	29,750	0.82	(18)	Yes
South of Drew Street ¹	4	27,200	26,650	1.02	2	Yes
North of Coachman Rd. ¹	4	33,050	27,700	1.19	19	Yes
North of Main Street ¹	4	26,300	28,000	0.94	(6)	Yes
South of Curlew Rd. ¹	4	17,300	22,800	0.76	(24)	Yes
NE Coachman Rd. / SR 590						
East of Belcher Rd.	2	12,850	2,900	1.00	0	Yes
Drew Street / SR 590						
West of Saturn Ave.	4	23,800	25,000	0.95	(5)	Yes
West of NE Coachman Rd.	4	35,150	34,000	1.03	3	Yes
Gulf To Bay Blvd / SR 60						
East of Duncan Ave.	6	52,750	51,000	1.03	3	Yes
West of Old Coachman Rd.	6	64,350	54,500	1.18	18	Yes
East of US 19	6	55,350	49,500	1.12	12	Yes
SR 60 Causeway	4	50,800	53,500	0.95	(5)	Yes
East Bay Dr. / SR 686						
West of US 19	6	68,350	47,000	1.45	45	No
East of US 19	6	45,100	45,500	0.99	(1)	Yes
East of Bay Vista	6	39,700	38,250	1.04	4	Yes

Note: ¹ 1999 County count volumes

NCHRP – National Cooperative Highway Research Program

Table 2. 1999 AADT Comparison of Revised TBRPM Forecast Volumes with 1999 Observed Traffic Counts

Count Location	No. of Lanes	1999 Model AADT	1999 Traffic Counts	Volume / Count	Percent Deviation	Within NCHRP Range?
US 19 / SR 55						
South of East Bay Dr.	6	57,950	52,500	1.10	10	Yes
North of East Bay Dr.	6	83,950	74,000	1.13	13	Yes
South of Seville Blvd.	6	84,900	83,000	1.02	2	Yes
South of Drew Street	6	74,800	73,500	1.02	2	Yes
North of Drew Street ¹	6	74,050	71,500	1.04	4	Yes
North of Coachman Rd.	6	85,350	82,000	1.04	4	Yes
North of Sunset Point Rd ¹	6	81,150	74,800	1.08	8	Yes
North of Main Street	6	78,150	72,000	1.09	9	Yes
North of Curlew Rd.	6	69,300	70,500	0.98	(2)	Yes
Mc Mullen Booth						
Bayside Bridge ¹	6	60,350	62,450	0.97	(3)	Yes
South of Drew Street ¹	6	57,650	60,500	0.95	(5)	Yes
North of Drew Street ¹	6	62,650	69,600	0.90	(10)	Yes
South of Curlew Rd. ¹	6	49,750	56,000	0.89	(11)	Yes
Belcher Rd.						
North of East Bay Dr. ¹	4	31,000	28,250	1.10	10	Yes
North of Nursery Rd. ¹	4	26,150	29,750	0.88	(12)	Yes
South of Drew Street ¹	4	26,300	26,650	0.99	(1)	Yes
North of Coachman Rd. ¹	4	32,150	27,700	1.16	16	Yes
North of Main Street ¹	4	31,950	28,000	1.14	14	Yes
South of Curlew Rd. ¹	4	22,700	22,800	1.00	0	Yes
NE Coachman Rd. / SR 590						
East of Belcher Rd.	2	13,300	12,900	1.03	3	Yes
Drew Street / SR 590						
West of Saturn Ave.	4	25,200	25,000	1.01	1	Yes
West of NE Coachman Rd.	4	34,450	34,000	1.01	1	Yes
Gulf To Bay Blvd / SR 60						
East of Duncan Ave.	6	51,900	51,000	1.02	2	Yes
West of Old Coachman Rd.	6	53,650	54,500	0.98	(2)	Yes
East of US 19	6	49,800	49,500	1.01	1	Yes
SR 60 Causeway	4	50,600	53,500	0.95	(5)	Yes
East Bay Dr. / SR 686						
West of US 19	6	53,700	47,000	1.14	14	Yes
East of US 19	6	42,800	45,500	0.94	(6)	Yes
East of Bay Vista	6	37,700	38,250	0.99	(1)	Yes

Note: ¹ 1999 County count volumes
 NCHRP – National Cooperative Highway Research Program

Future Year 2025 Model Review

The 2025 model network was revised to include the same refinements that were made for the 1999 corridor validation effort. In addition, the following refinements were made to reflect the US 19 No-Build conditions in the project area.

1. Recoded US 19 ramp arrangements and frontage roads between south of Tropic Hills Dr. and north of Drew Street to represent the proposed design conditions.
2. Coded proposed ramps at US 19 and McCormick Dr.
3. Recoded proposed ramps at US 19 and Enterprise Rd. to better represent the proposed design conditions.

With the preceding refinements, the Year 2025 model was reviewed and found to reflect the appropriate conditions for the Year 2025. The model included the proposed grade separation configuration for the US 19 / SR 60 interchange. The model was found to include a reasonable generator for the location of the Clearwater Mall. Review of the traffic report for the rebuilding of the mall indicated that slightly less traffic would be associated with the new, sub-regional mall than with the former regional level mall. Retail square footage, employment numbers and type were checked for both versions of the mall and it was determined that the model, with the higher generation from that TAZ, could be used and would be conservative and allow for traffic beyond that projected by the Mall's traffic report.

Once the Year 2025 model was reviewed and manually adjusted, the options were coded to reflect the proposed design conditions and networks were developed for the different options. Using these developed networks and the revised 1999 TBRPM, Average Annual Daily Traffic (AADT) volumes were estimated for the options. The Pinellas countywide model conversion factor (MOCF) of 0.94 was used to convert the peak season model volumes to AADT.

The comparison of the Year 2025 AADT volumes for the options is shown in Table 3.

Table 3. 2025 AADT Comparison of No-Build and Build Options

Location	No. of Lanes	2025 No-Build AADT	2025 Build A AADT	2025 Build B AADT	2025 Build C AADT	2025 Build D AADT
US 19 / SR 55						
South of East Bay Dr.	6	63,450	89,750	90,300	91,550	83,500
North of East Bay Dr.	6	90,250	117,850	119,100	119,900	111,550
South of Seville Blvd.	6	73,500	103,200	101,850	104,150	96,600
South of Drew Street	6	67,400	92,100	91,500	91,950	90,400
North of Drew Street ¹	6	122,400	133,350	132,600	133,800	132,750
North of Coachman Rd.	6	110,150	114,700	113,950	113,950	113,100
North of Sunset Point	6	110,400	111,450	111,150	110,850	111,300
North of Main Street	6	102,300	99,600	100,100	99,550	98,200
North of Curlew Rd.	6	87,350	87,000	87,300	87,650	87,500
Mc Mullen Booth						
Bayside Bridge	6	79,450	49,550	51,050	49,700	50,650
South of Drew Street	6	62,050	53,150	54,100	53,300	54,100
North of Drew Street	6	58,100	53,100	53,200	53,200	54,050
South of Curlew Rd.	6	49,550	54,100	54,050	54,500	56,250
NE Coachman Rd. / SR 590						
East of Belcher Rd.	2	25,550	23,900	24,150	24,050	24,950
Drew Street / SR 590						
West of Saturn Ave.	4	34,550	35,600	35,750	34,750	36,550
West of NE Coachman Rd.	4	38,700	38,900	37,550	37,550	38,900
Gulf To Bay Blvd / SR 60						
East of Duncan Ave.	6	52,350	52,900	52,400	52,850	52,450
West of Old Coachman Rd.	6	57,050	58,700	60,700	59,100	60,450
East of US 19	6	63,850	50,200	50,750	51,000	51,000
SR 60 Causeway	4	52,650	52,850	53,000	53,150	53,200
East Bay Dr. / SR 686						
West of US 19	6	56,950	64,300	63,250	63,350	64,700
East of US 19	6	47,400	48,750	48,800	48,350	48,500
East of Bay Vista	6	45,300	42,050	40,950	41,600	44,450

Future Year 2020 and 2030 Traffic Projections

Since the study design year is 2030, growth factors were developed to project 2020 and 2030 AADT volumes from the 2025 model projections. Based on the historical traffic growth analysis as shown in Table 4, annual growth rates of 1.4% and 0.5% were proposed for the US 19 corridor and for the cross streets respectively to project 2020 and 2030 AADT from 2025 model AADT estimates.

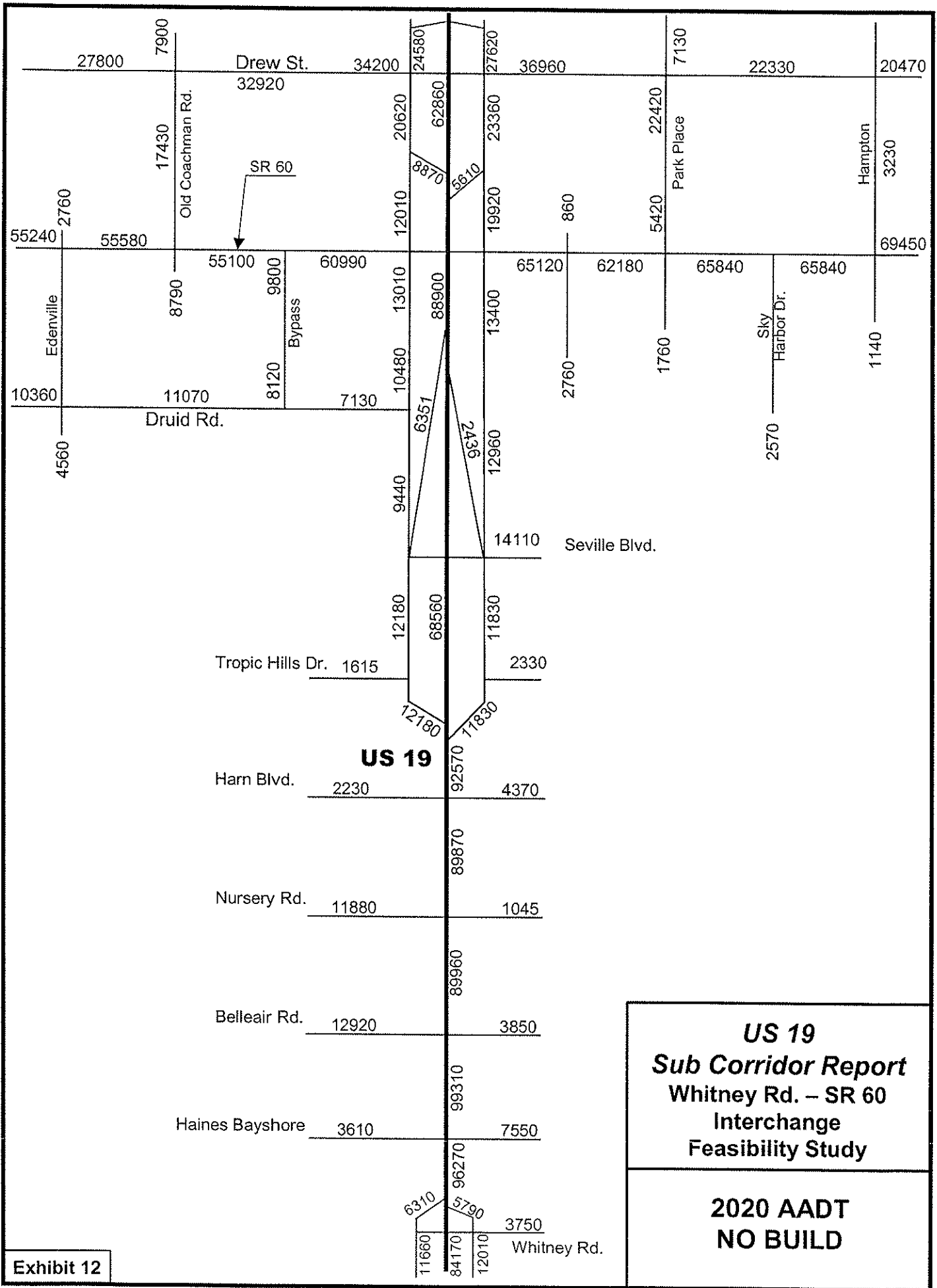
Table 4A. 1984 – 2002 Historical Traffic Growth Rates at the FDOT Count Stations

Traffic Volumes at FDOT Count Stations								
US 19 North of East Bay Dr. (Sta. 0006)					US 19 South of Seville Blvd (Sta. 0016)			
YEAR	ADT	3-year Average	Total Growth	Annual Growth	ADT	3-year Average	Total Growth	Annual Growth
2002	73,000				82,500			
2001	80,500	76,300			80,500	81,300		
2000	75,500				81,000			
1999	74,000				83,000			
1998	76,500				77,000			
1997	69,500		7.5%	1.5%	75,500		7.0%	1.4%
1996	75,500	71,000	1996 - 01	1996 - 01	75,000	76,000	1996 - 01	1996 - 01
1995	68,000		5-year	5-year	77,500		5-year	5-year
1994	64,000				74,000			
1993	96,500				96,000			
1992	94,500				106,000			
1991	90,195				101,029			
1990	NA				NA			
1989	NA				NA			
1988	58,984				73,599			
1987	58,855		28.7%	1.7%	67,803		18.3%	1.1%
1986	59,074	59,300	1986 - 01	1986 - 01	70,217	68,700	1986 - 01	1986 - 01
1985	59,980		15-year	15-year	68,116		15-year	15-year
1984	NA				75,130			
US 19 South of Drew Street (Sta. 0041)					US 19 North of Coachman Rd. (Sta. 0042)			
YEAR	ADT	3-year Average	Total Growth	Annual Growth	ADT	3-year Average	Total Growth	Annual Growth
2002	73,000				82,500			
2001	80,500	73,300			82,000	81,200		
2000	66,500				79,000			
1999	73,500				82,000			
1998	64,000				81,000			
1997	66,500		8.1%	1.6%	74,000		9.3%	1.8%
1996	68,000	67,800	1996 - 01	1996 - 01	76,500	74,300	1996 - 01	1996 - 01
1995	69,000		5-year	5-year	72,500		5-year	5-year
1994	67,000				70,500			
1993	97,500				77,500			
1992	95,500				76,500			
1991	91,418				NA			
1990	NA				NA			
1989	NA				NA			
1988	69,663				91,482			
1987	59,378		20.0%	1.2%	81,237		20.1%	1.2%
1986	62,005	61,100	1986 - 01	1986 - 01	63,061	67,600	1986 - 01	1986 - 01
1985	61,805		15-year	15-year	58,457		15-year	15-year
1984	67,264				NA			

Table 4B. 1984 – 2002 Historical Traffic Growth Rates at the FDOT Count Stations

Traffic Volumes at FDOT Count Stations								
SR 60 West of Old Coachman Rd. (Sta. 0075)					SR 60 East of US 19 (Sta. 0074)			
YEAR	ADT	3-year Average	Total Growth	Annual Growth	ADT	3-year Average	Total Growth	Annual Growth
2002	57,000				56,000			
2001	51,500	55,700			55,000	54,300		
2000	58,500				52,000			
1999	54,500				49,500			
1998	56,500				53,500			
1997	56,000		(0.2%)	0.0%	54,500		3.8%	0.8%
1996	55,000	55,800	1996 - 01	1996 - 01	52,500	52,300	1996 - 01	1996 - 01
1995	56,500		5-year	5-year	50,000		5-year	5-year
1994	56,500				59,000			
1993	27,500				48,000			
1992	55,000				47,000			
1991	52,239				44,573			
1990	NA				NA			
1989	NA				NA			
1988	56,119				NA			
1987	50,811		16.5%	1.0%	NA			
1986	46,028	47,800	1986 - 01	1986 - 01	44,553			
1985	46,421		15-year	15-year	33,618			
1984	47,154				NA			
East Bay Dr. West of US 19 (Sta. 5304)					East Bay Dr. East of US 19 (Sta. 0007)			
YEAR	ADT	3-year Average	Total Growth	Annual Growth	ADT	3-year Average	Total Growth	Annual Growth
2002	55,000				47,000			
2001	57,500	53,500			47,500	47,200		
2000	48,000				47,000			
1999	47,000				45,500			
1998	48,000				47,500			
1997	50,000				42,500			
1996	46,500		18.4%	3.4%	44,500		4.9%	1.0%
1995	39,000	45,200	1996 - 01	1996 - 01	48,000	45,000	1996 - 01	1996 - 01
1994	44,000		5-year	5-year	42,500		5-year	5-year
1993	49,000				58,000			
1992	48,000				48,500			
1991	37,473				45,465			
1990	NA				NA			
1989	NA				NA			
1988	NA				40,076			
1987	NA				40,793		13.2%	0.8%
1986	NA				44,678	41,700	1986 - 01	1986 - 01
1985	NA				39,499		15-year	15-year
1984	NA				NA			

The 2020 AADT volumes for the Build options are shown in Exhibits 12 through 16.
The 2030 AADT volumes for the Build options are shown in Exhibits 17 through 21.



**US 19
Sub Corridor Report
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**2020 AADT
NO BUILD**

Exhibit 12

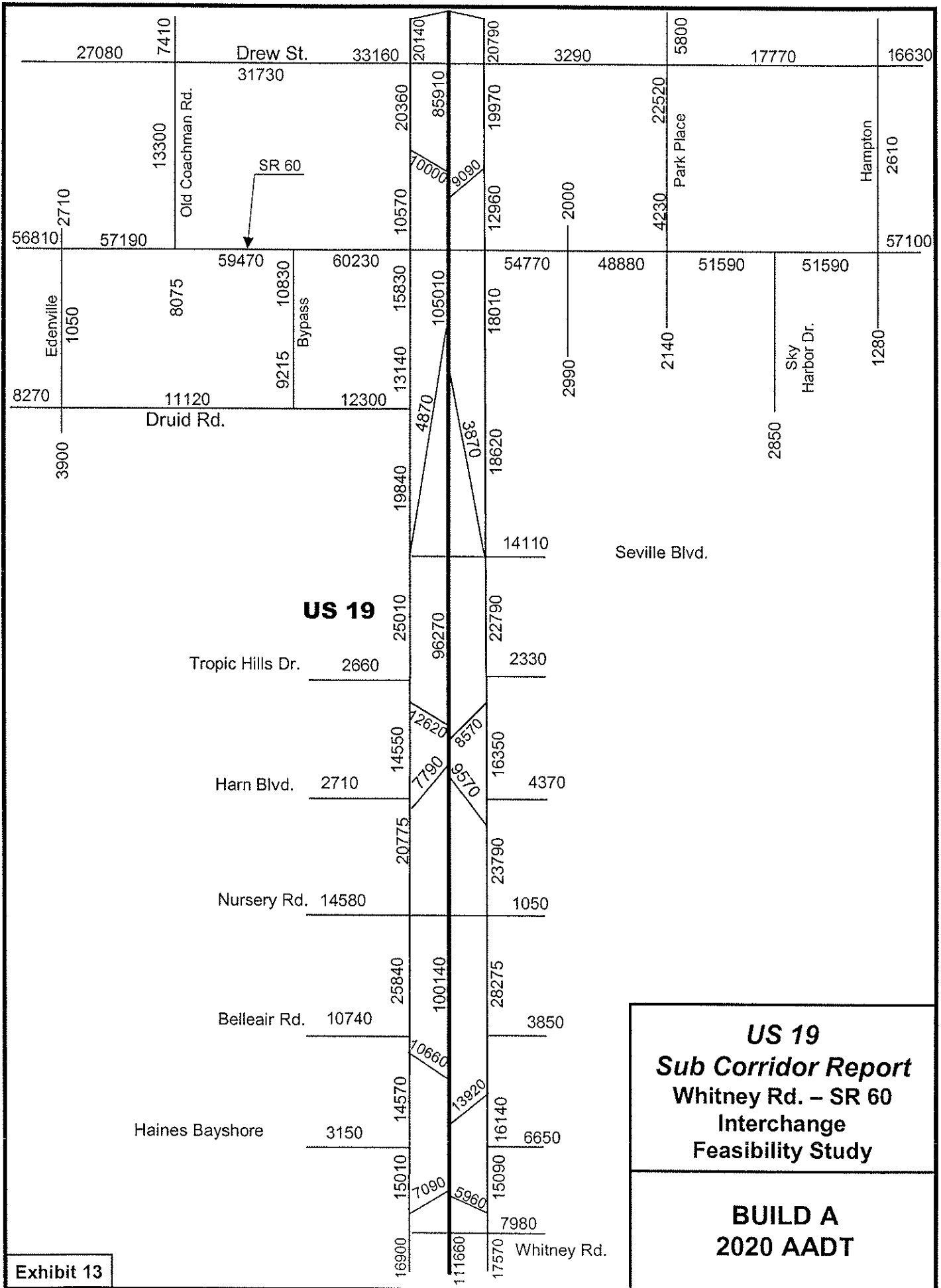


Exhibit 13

**US 19
Sub Corridor Report
Whitney Rd. - SR 60
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**BUILD A
2020 AADT**

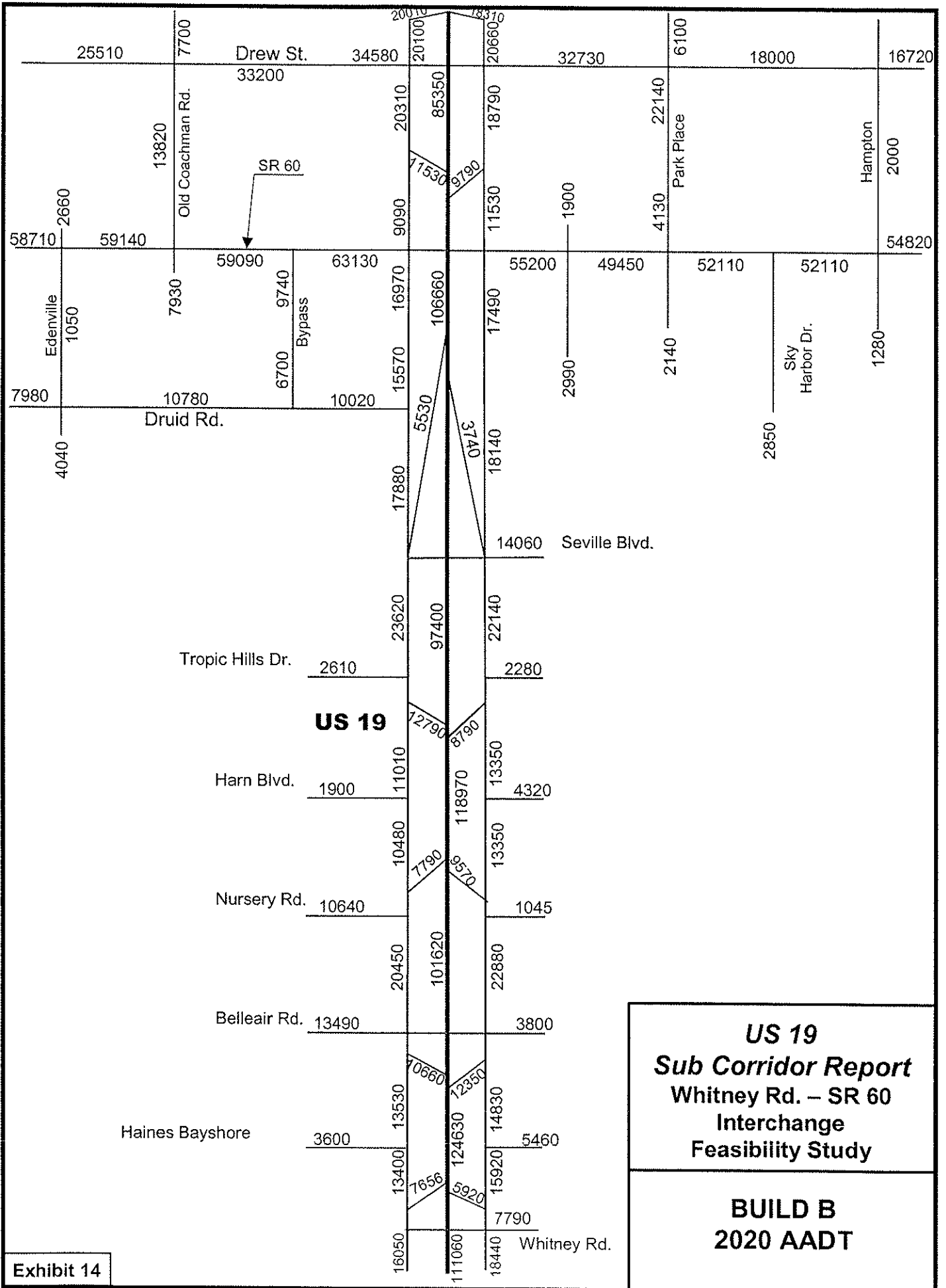


Exhibit 14

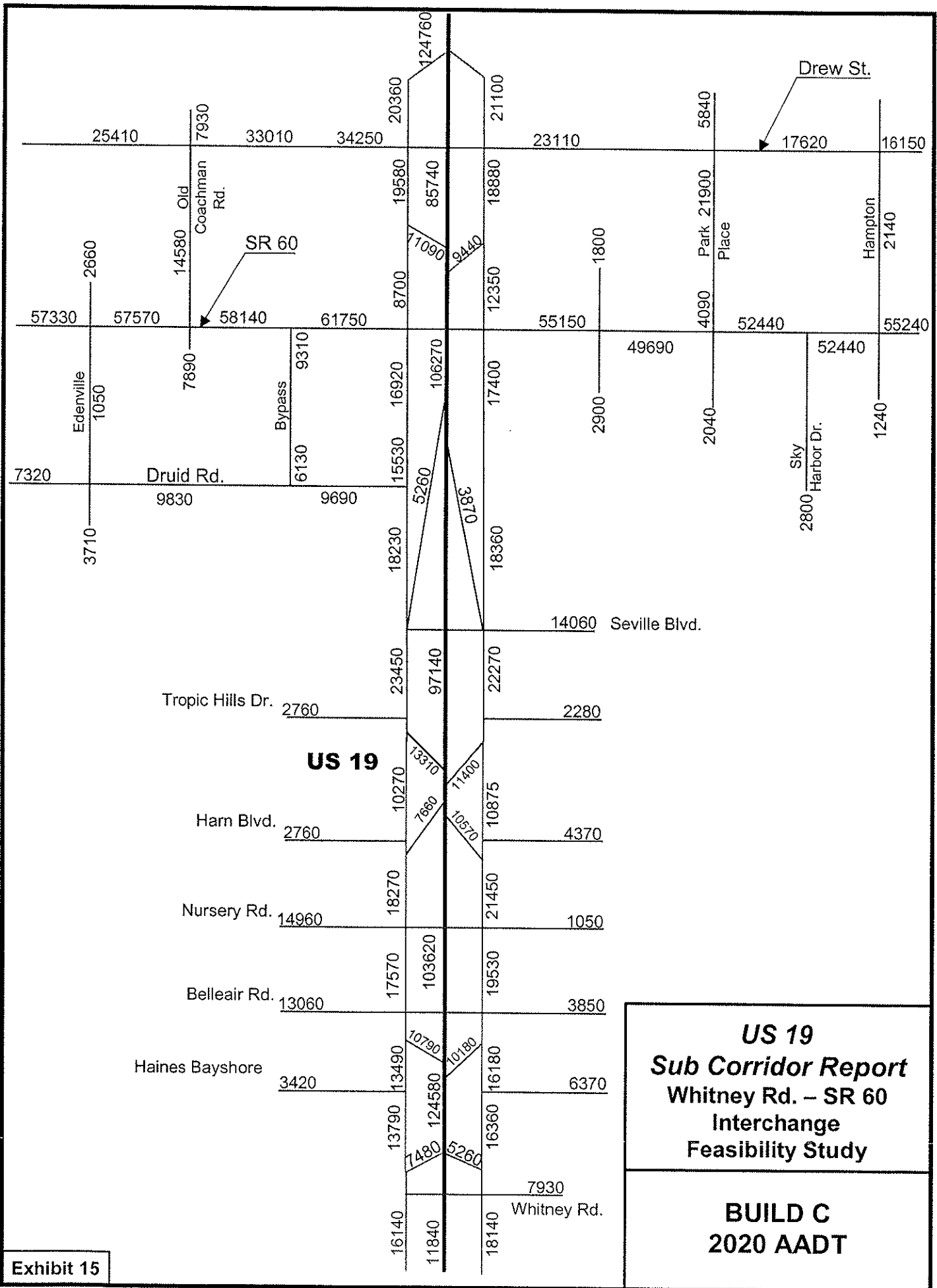


Exhibit 15

US 19
Sub Corridor Report
Whitney Rd. – SR 60
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BUILD C
2020 AADT

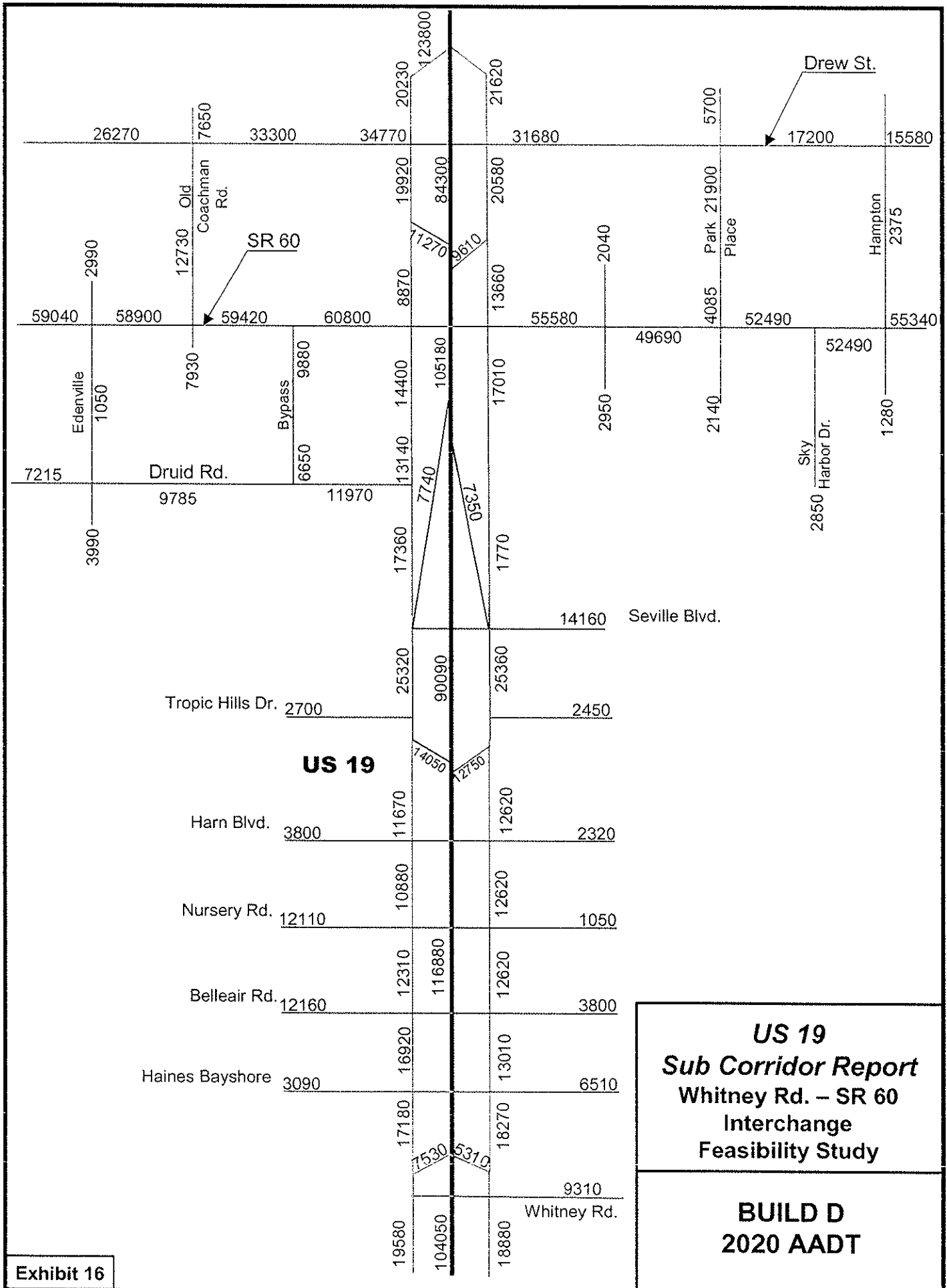
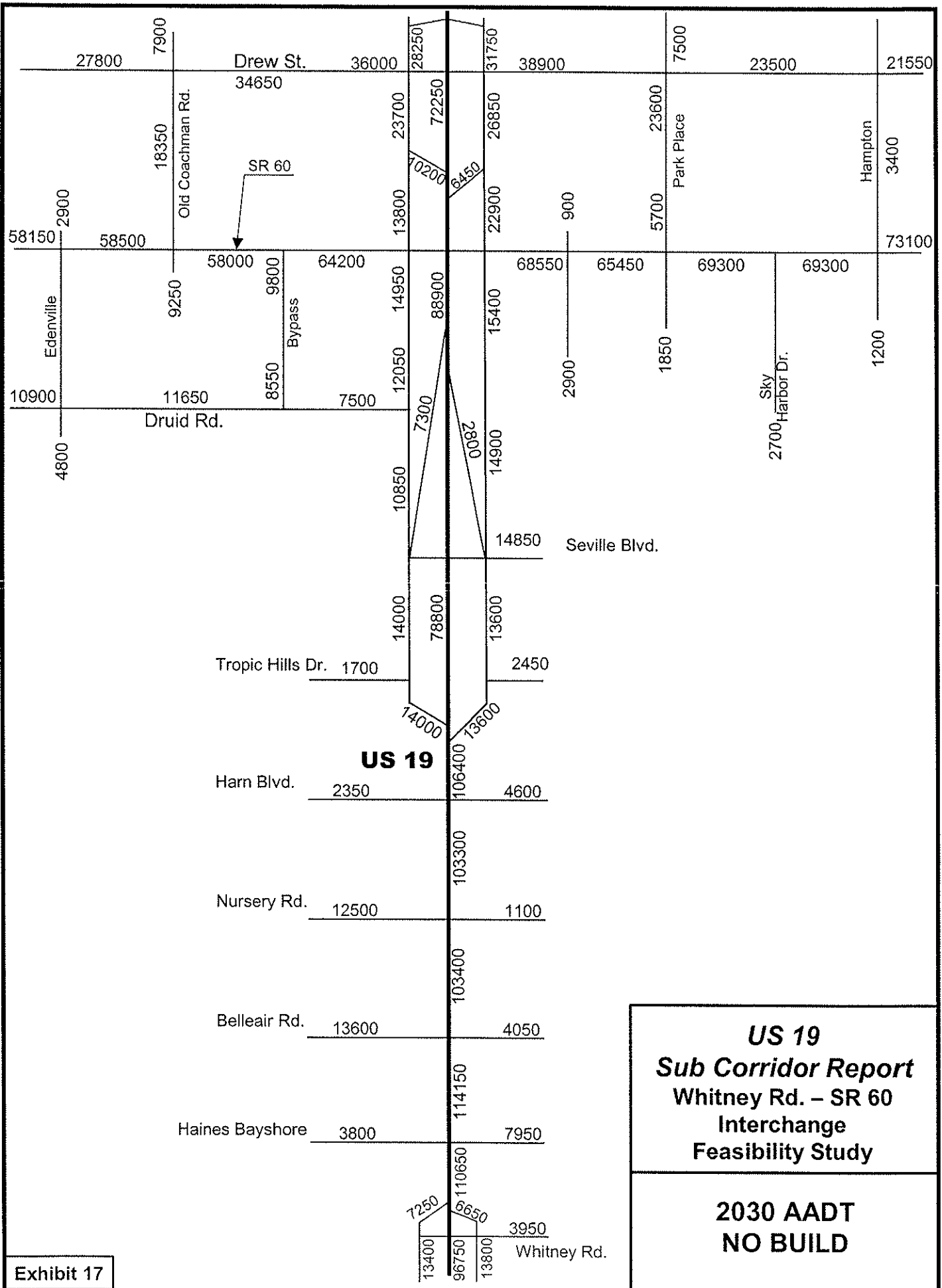


Exhibit 16



US 19
Sub Corridor Report
Whitney Rd. - SR 60
Interchange
Feasibility Study

2030 AADT
NO BUILD

Exhibit 17

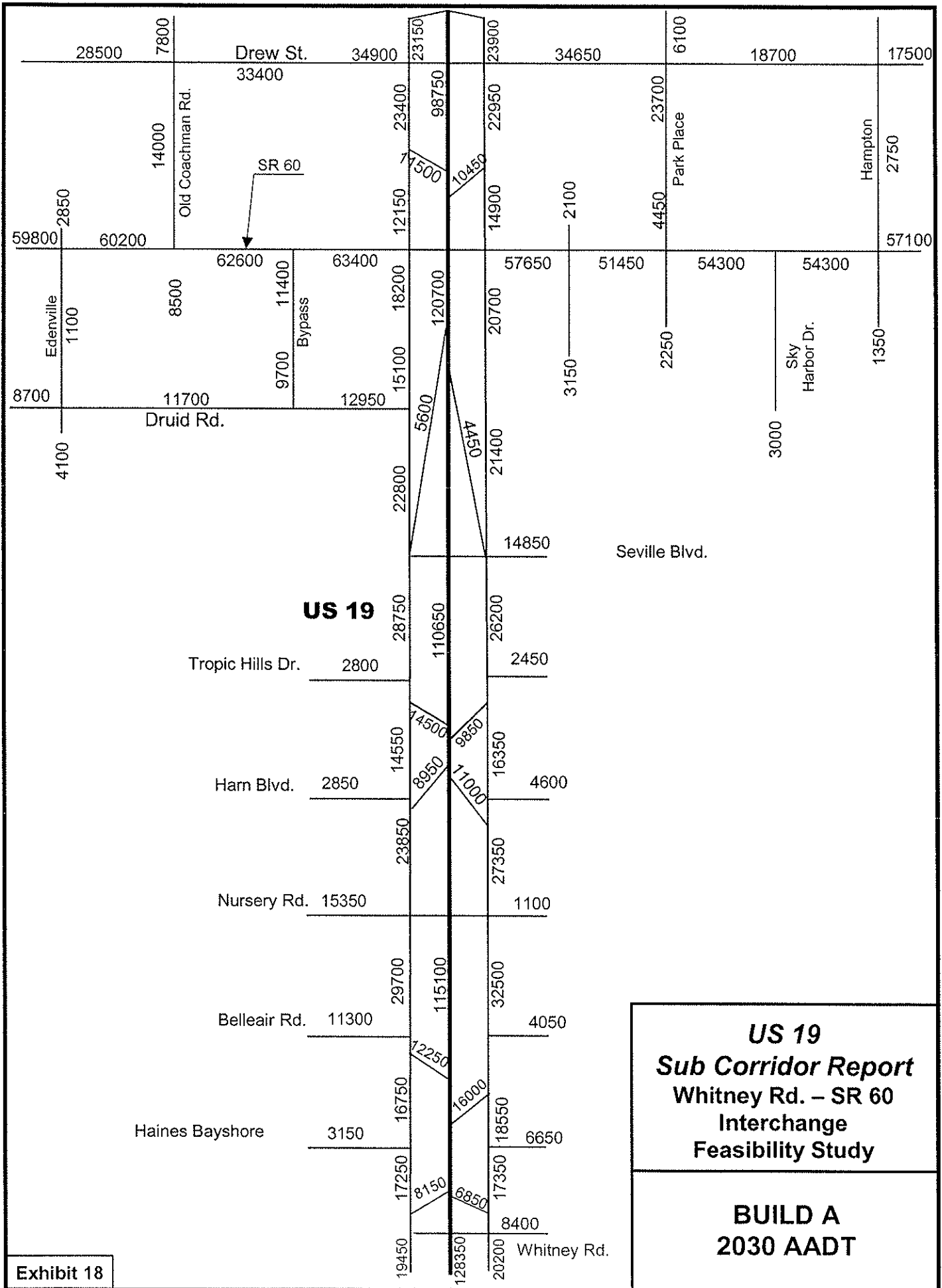
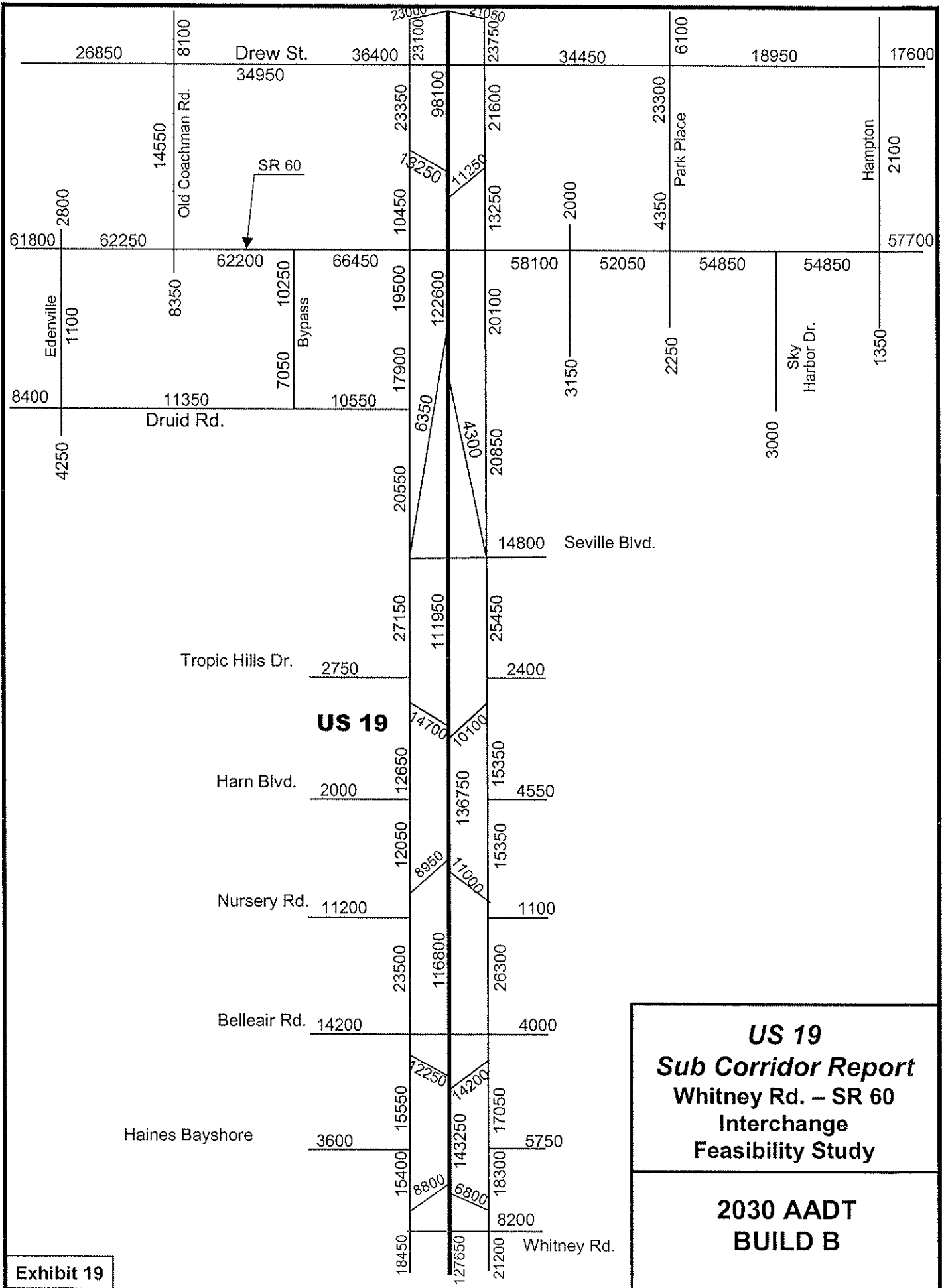


Exhibit 18



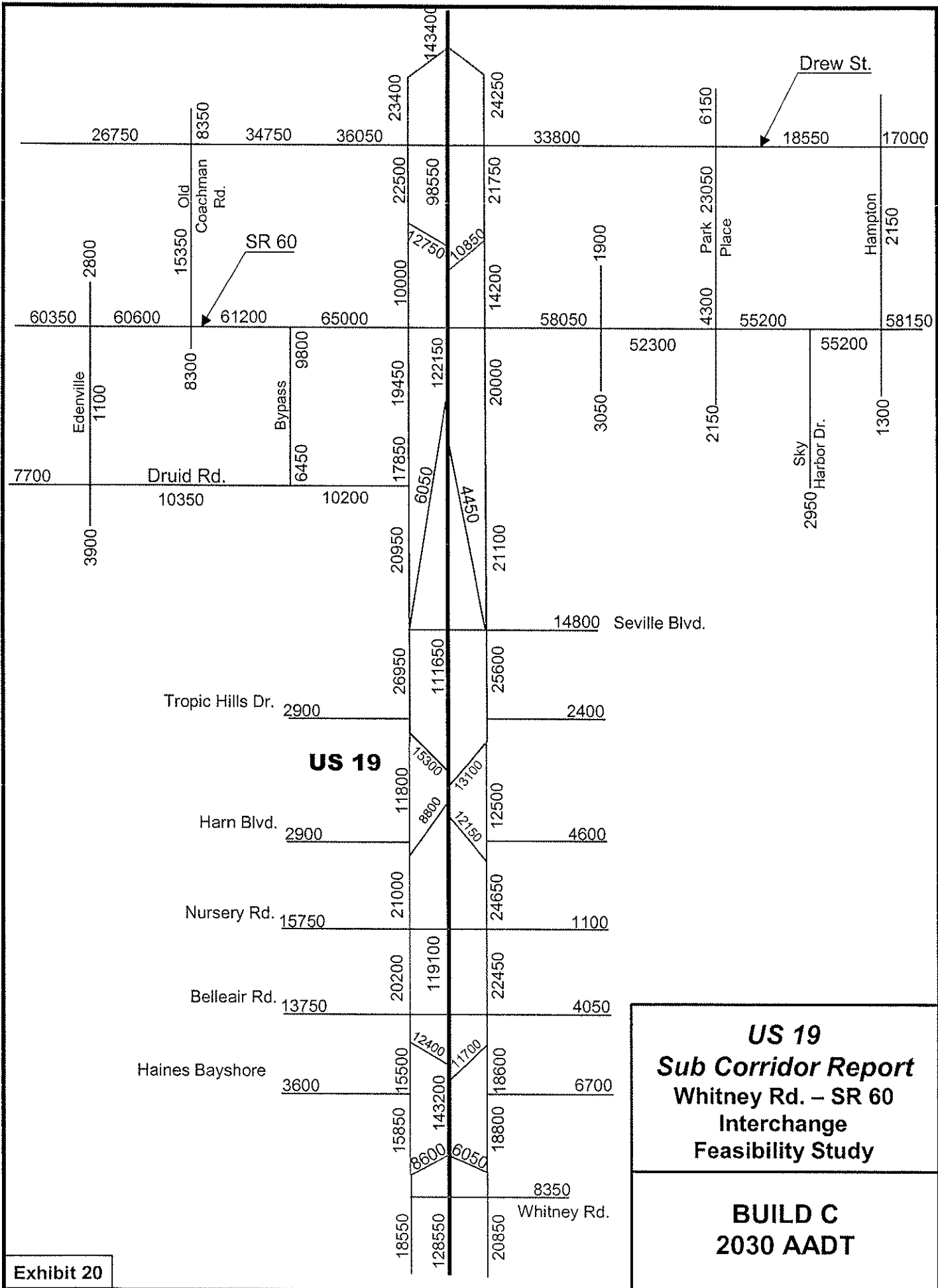


Exhibit 20

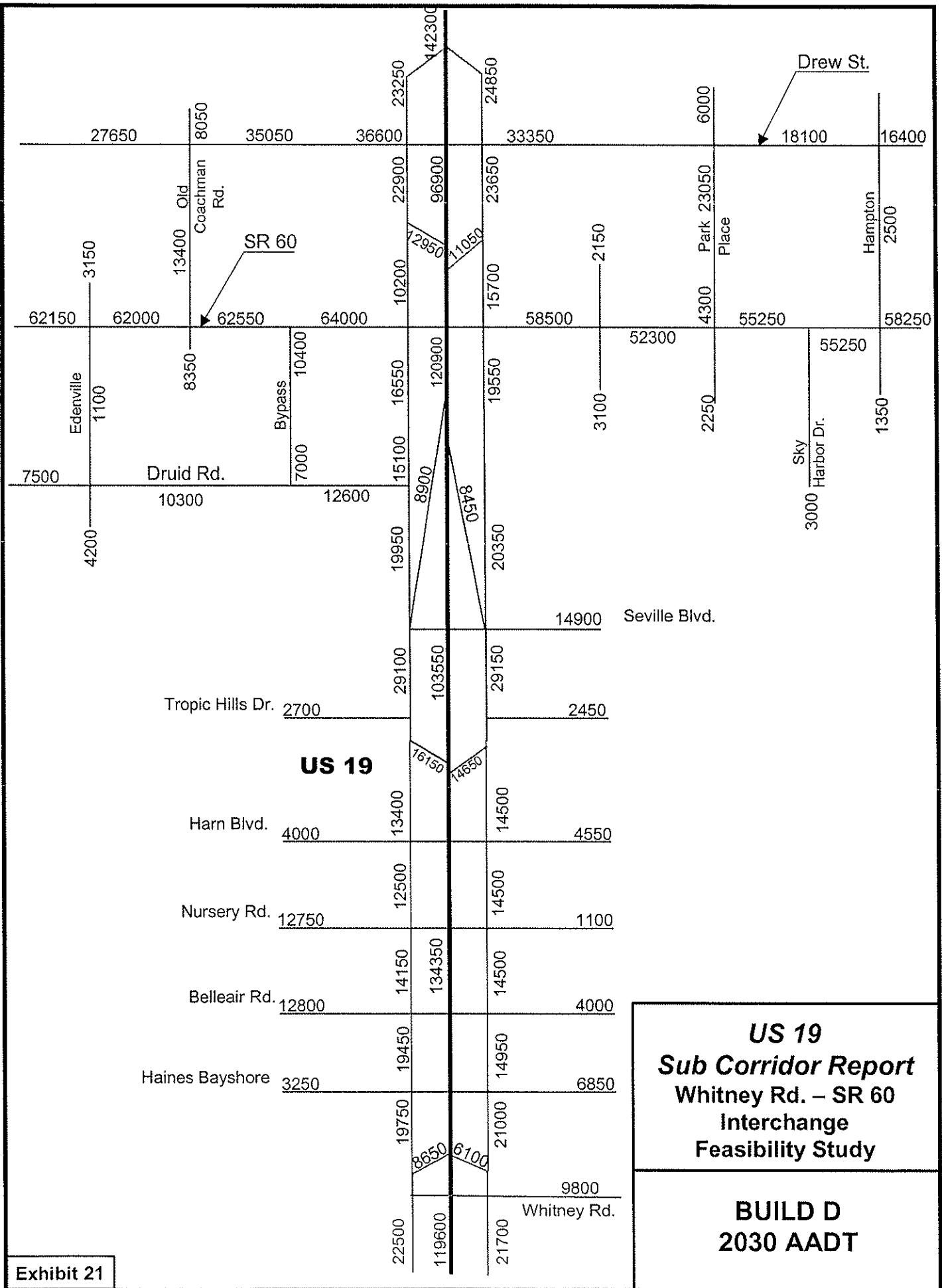


Exhibit 21

Development of Design Hour Volumes

The design year (2030) design hour volumes were estimated using the 30th highest hour K and D (K₃₀ and D₃₀) factors. The K, D, and T factors were estimated based on the procedure outlined in the FDOT Project Traffic Forecasting Handbook, 2002. The K, D, and T traffic factors collected during the past three years from FDOT count stations in the US 19 study area are shown in Table 5. These factors are compared with the State and National data in Table 6.

Table 5. Traffic Characteristics for the US 19 Study Area

Count Station	Location	Facility Type	Year	K30	D30	Daily Truck %
0016	US 19, South of Seville Blvd.	Mainline	2002	9.80	55.66	5.57
			2001	9.98	52.10	5.30
			2000	9.88	59.18	8.46
			Average	9.89	55.65	6.44
0041	US 19, South of Drew Street	Mainline	2002	9.80	55.66	6.31
			2001	9.98	52.10	7.47
			2000	9.88	59.18	12.30
			Average	9.89	55.65	8.69
0075	SR 60, West of Old Coachman Rd.	Urban Arterial	2002	9.80	55.66	7.11
			2001	9.98	52.10	8.61
			2000	9.88	59.18	10.62
			Average	9.89	55.65	8.78
0074	SR 60, East of US 19	Urban Arterial	2002	9.80	55.66	6.60
			2001	9.98	52.10	5.99
			2000	9.88	59.18	10.28
			Average	9.89	55.65	7.62

* Source: FDOT Traffic Information, 1999, 2000 and 2001.

Table 6. Comparison of Site Specific Data with State and National Data For Urban Freeways and Arterials

	FDOT SITE DATA		STATE DATA*		NATIONAL DATA*	
	K ₃₀	D ₃₀	K ₃₀	D ₃₀	K ₃₀	D ₃₀
Urban Freeways						
Observed Minimum	9.80	52.10	9.4	50.4	7.0	50.0
Observed Maximum	9.98	59.18	10.0	61.2	10.0	55.0
Urban Arterials						
Observed Minimum	9.80	52.10	9.2	50.8	9.0	50.0
Observed Maximum	9.98	59.18	11.5	67.1	10.0	55.0

* Source: FDOT Project Traffic Forecasting Handbook, 2002.

Based on the these data the following K and D factors are recommended to develop the design hour traffic characteristics in the level of service analyses for design year (2030).

- K = 9.9 percent
- D = 55 percent
- T – Daily = 8 percent
- T – Design Hour = 4 percent

Design year 2030 AM and PM peak hour volumes were derived by first multiplying the AADT volumes by a K₃₀ factor of 9.9 percent and a D₃₀ factor of 55 percent and then distributing the peak and off-peak direction link volumes. The existing year (2002) intersection turning movement volumes were used to help derive the Year 2030 peak hour intersection turning volumes. The developed turning movement volumes were then balanced using Fratar’s method to capture the future travel patterns. For US 19, the peak travel directions in the year 2030 were assumed to be southbound in the AM peak hour and northbound in the PM peak hour consistent with the existing year (2002) peak hour travel directions. For the cross streets the respective existing year (2002) AM and PM peak hour directions were assumed to be the same as the Year 2030 peak hour directions.

Interim year 2020 AM and PM peak hour volumes were derived by back factoring Year 2030 peak hour volumes using the annual growth rates of 1.4% for the mainline and 0.5% for the cross streets. The Year 2020 peak hour volumes developed for the US 19 corridor No-Build and Build Options are illustrated in Exhibits 22 through 31. The Year 2030 peak hour volumes developed for the US 19 corridor No-Build and Build Options are illustrated in Exhibits 32 through 41.

CONCEPTUAL DESIGN

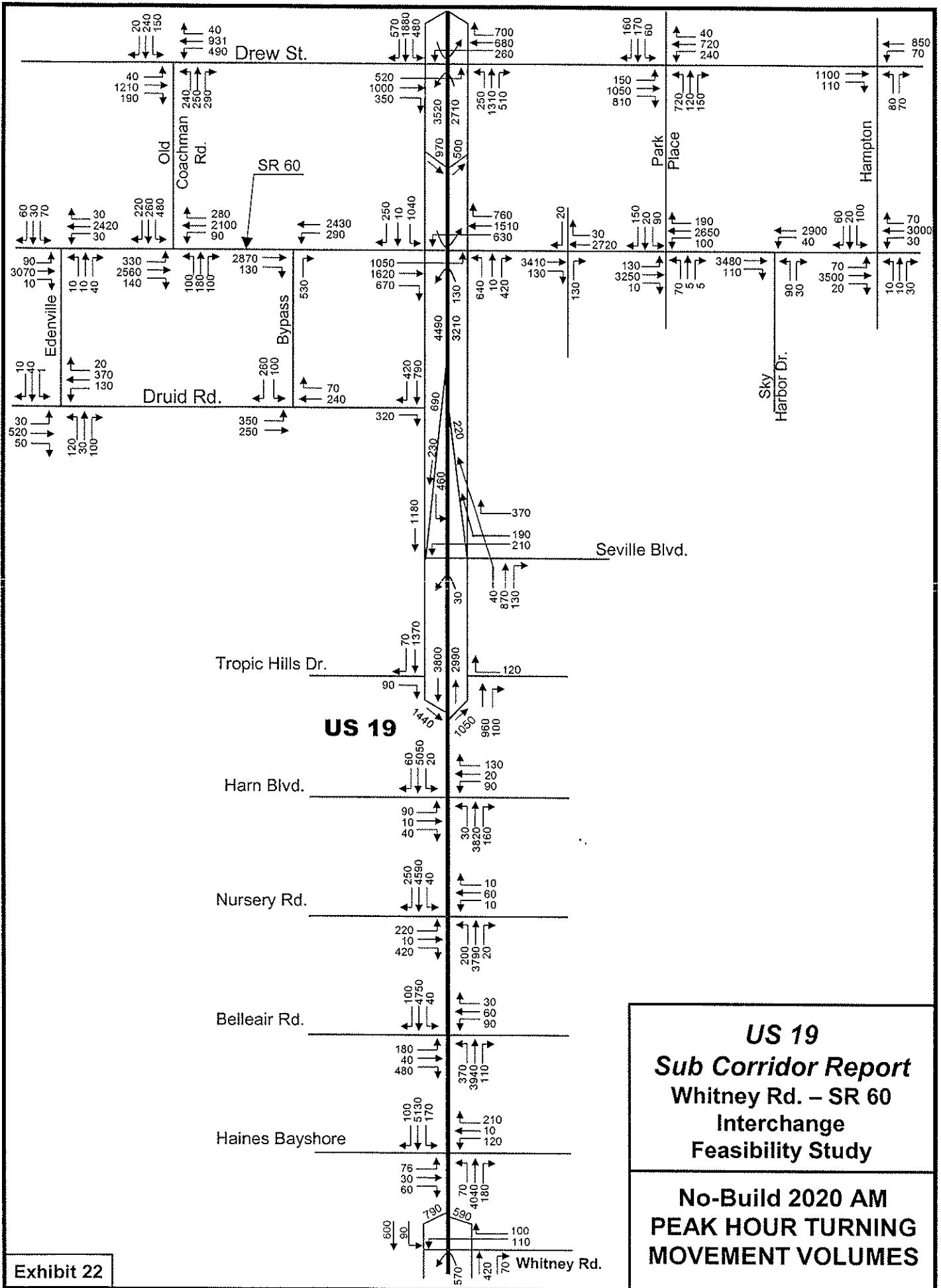
The methodology for investigating the feasibility of new grade separations included the utilization of the design concept developed by H.W. Lochner, Inc. for the US 19 grade separations with Sunset Point Rd. and N.E. Coachman Rd. in Pinellas County.

Utilizing the new concept, a revised typical section was developed for the potential grade separations and related improvements, as shown in Exhibit 42. This typical section includes three lanes in each direction on the US 19 mainline and a one-way, two-lane frontage lane system along each side of the US 19 mainline. Mainline lanes are designed with 11-foot lanes. Frontage lanes are designed with a 14-foot undesignated outside lane to accommodate bicycle usage and a 10-foot inside lane. To accommodate the wider outside frontage lane within the same right-of-way, the inside mainline shoulders are reduced to 8-feet and the inside frontage lanes are reduced to 10-feet. A previously developed typical section, without including undesignated bicycle lanes on the frontage lane system, for the new grade separation at Sunset Point Rd. is also shown in Exhibit 42 for comparison purposes. Due to right-of-way constraints at the intersection locations, a proposed typical section for the grade separation at the overpass areas was developed with a cantilever pile supported slab and is shown in Exhibit 43.

In developing the conceptual plans, the following assumptions were made. These assumptions were based on assumptions used in similar prior studies after discussions with the Florida Department of Transportation (FDOT) District Seven Office.

- Approach turn-lane assignments on the frontage lane system intersections were developed based on the projected traffic movements and volumes. The total number of approach and departure lanes at each intersection was limited to the number that would fit within the existing right-of-way, exclusive of corner clips for signal poles or other street appurtenances.
- The entire concept was developed to minimize right-of-way impacts. This was a fundamental guiding principle in the development of the options.
- All ramp touchdown points were developed to allow for maximum direct access for driveways to US 19 wherever possible.
- A minimum vertical grade separation profile was used.
- The amount of right-of-way required was estimated.
- The estimated quantities were based upon the final design of a new grade separation at Sunset Point Rd. Where applicable, quantities were modified to suit specific location differences from the typical roadway plan and profile.
- Preliminary construction cost estimates were developed. These estimates included construction-related costs and contingencies. They do not include planning, environmental, design, construction administration, inspection, or project administration costs.
- Stormwater Management Facility (SMF) surface area requirements were estimated based on typical conditions; however, exact SMF facility locations will be developed in subsequent project phases. Typical costs for these facilities are included in the construction estimate.

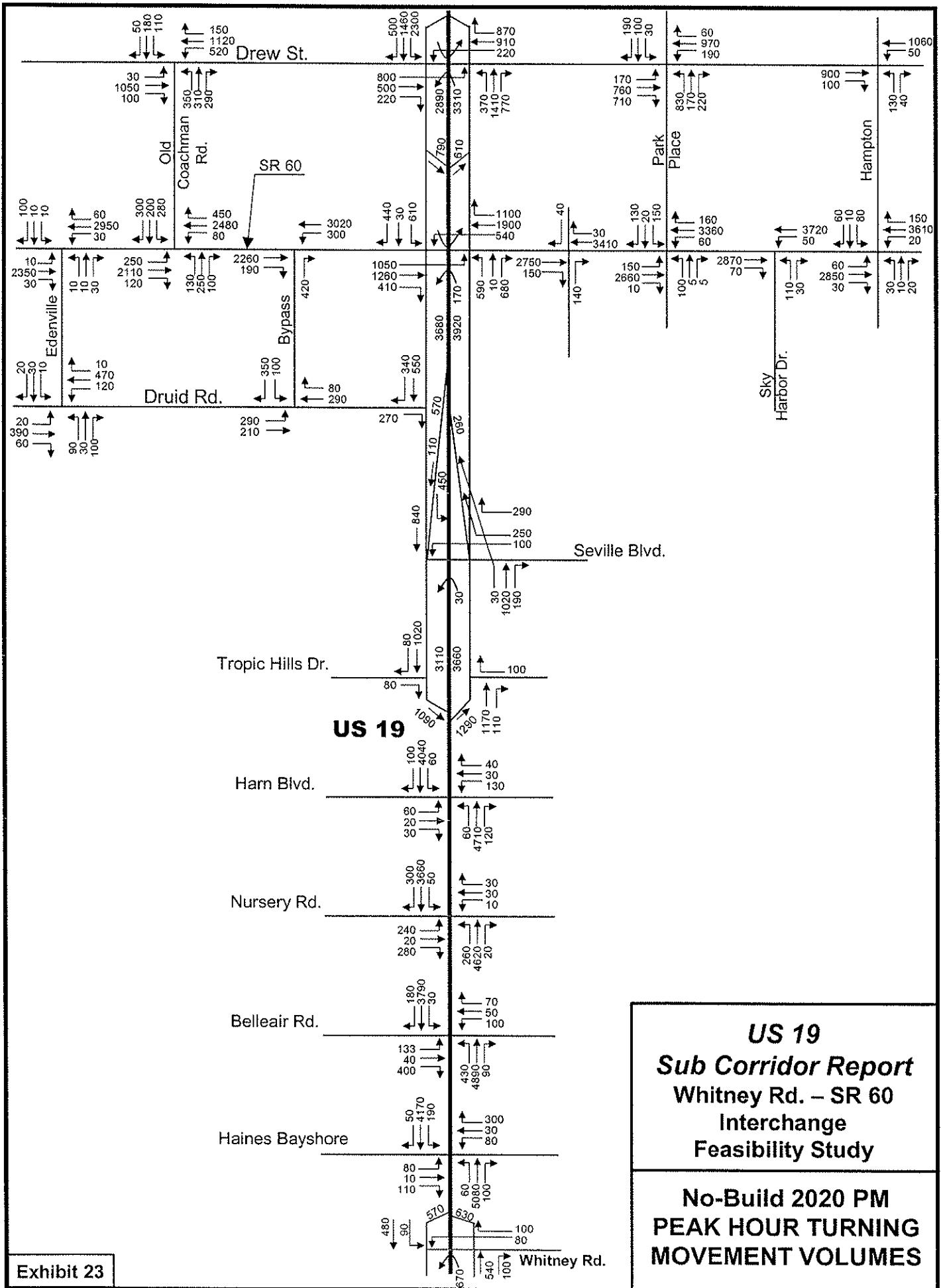
Using an optimal three percent maximum grade and the revised typical section, preliminary conceptual plans for the four options along US 19 were developed. These plans utilized the new minimized right-of-way concept within the framework of the preferred option from the previous PD&E Study. Plan views of each of the four Build options are included in a pocket at the end of this study report.

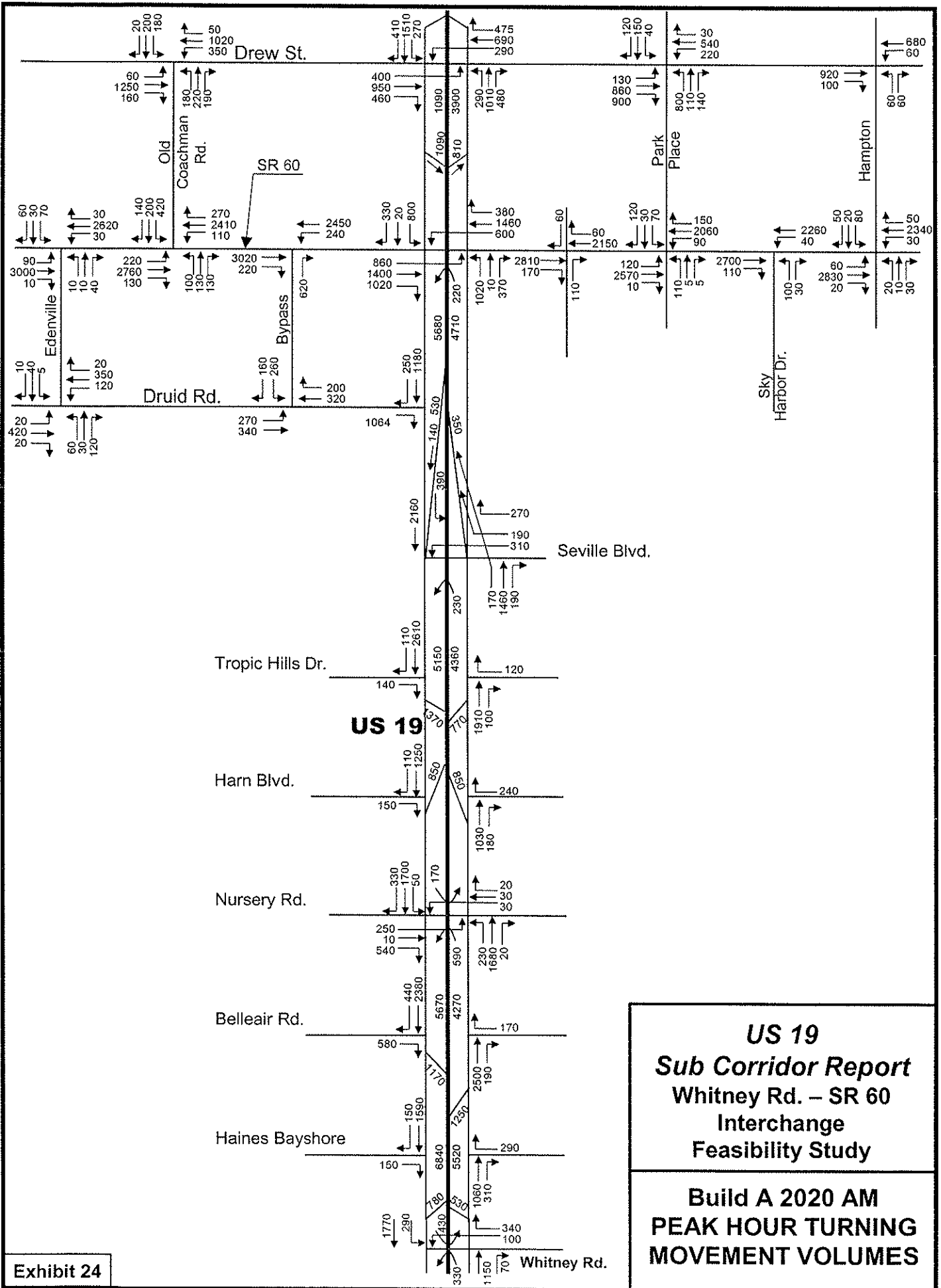


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Whitney Rd. – SR 60
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No-Build 2020 AM
PEAK HOUR TURNING
MOVEMENT VOLUMES

Exhibit 22

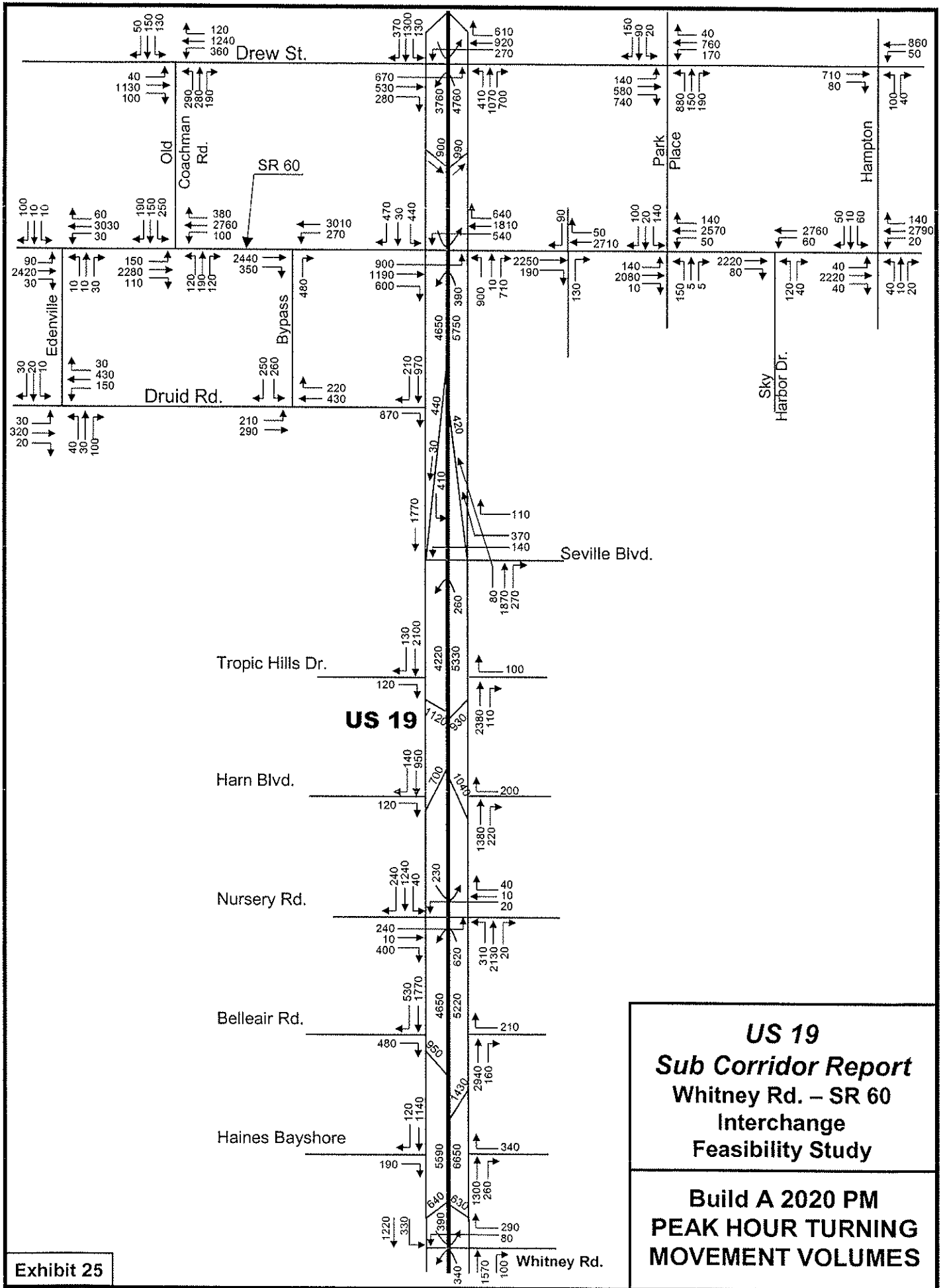


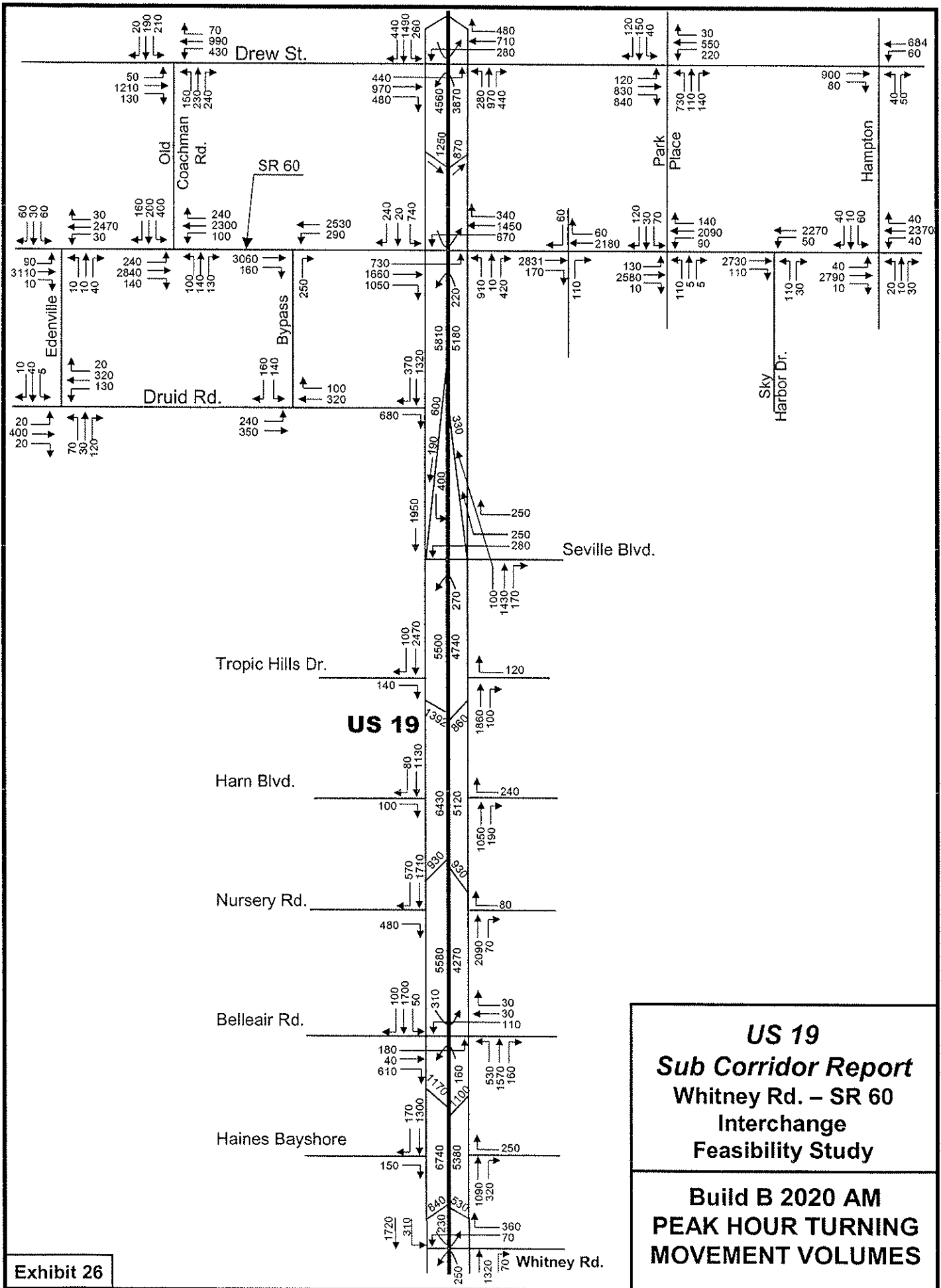


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Build A 2020 AM
PEAK HOUR TURNING
MOVEMENT VOLUMES

Exhibit 24

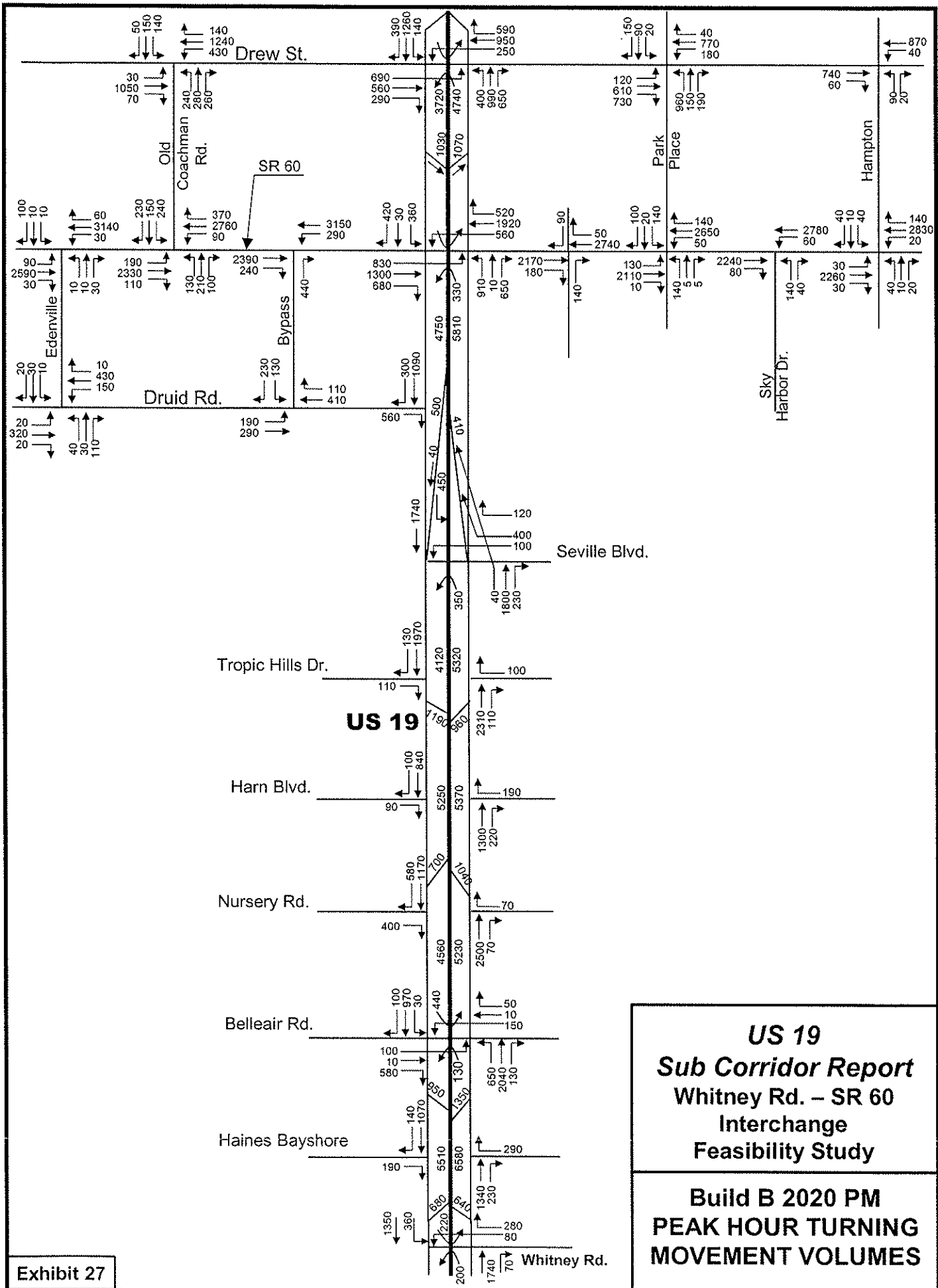




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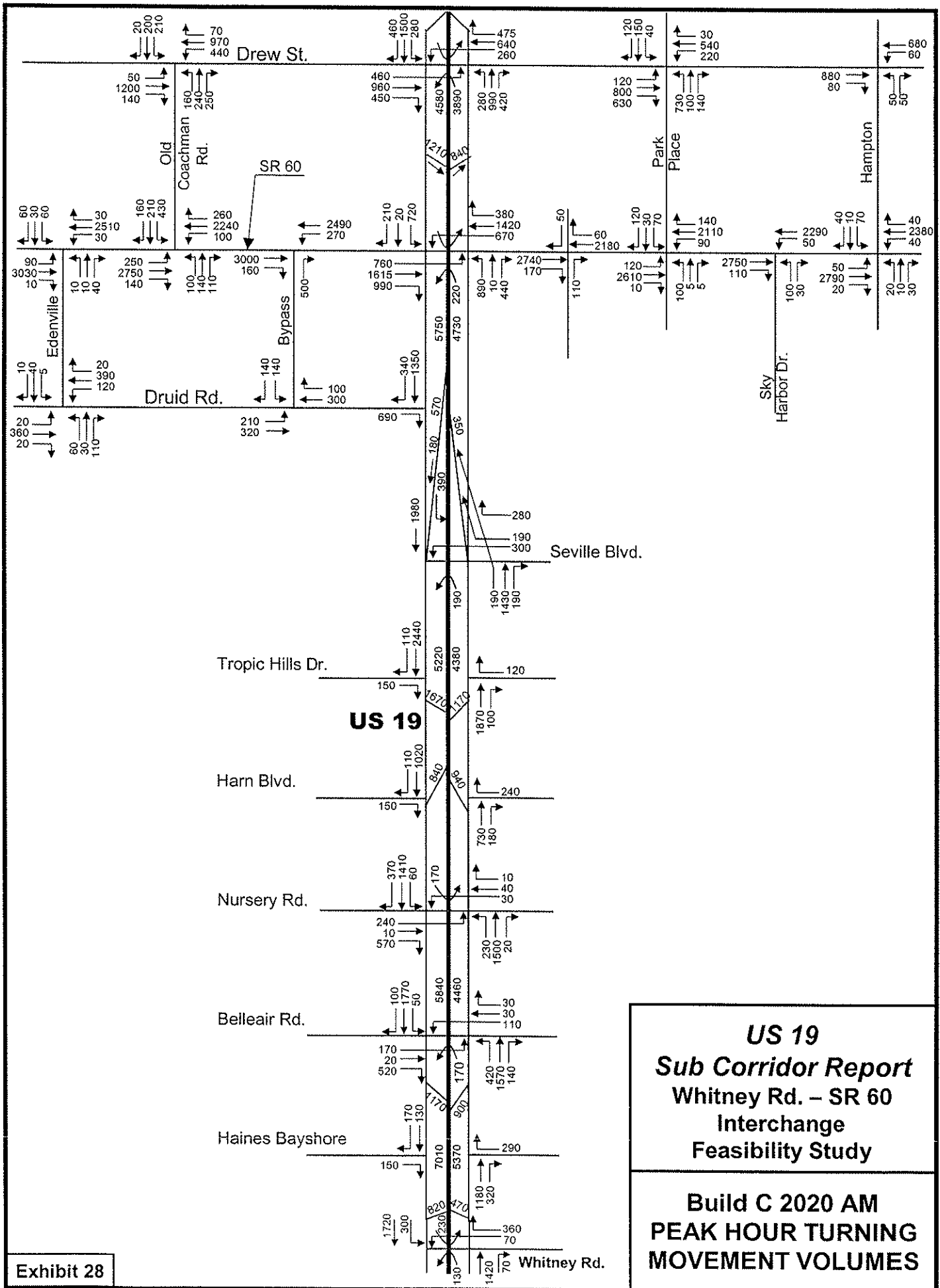
Build B 2020 AM
PEAK HOUR TURNING
MOVEMENT VOLUMES

Exhibit 26



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Build B 2020 PM
PEAK HOUR TURNING
MOVEMENT VOLUMES



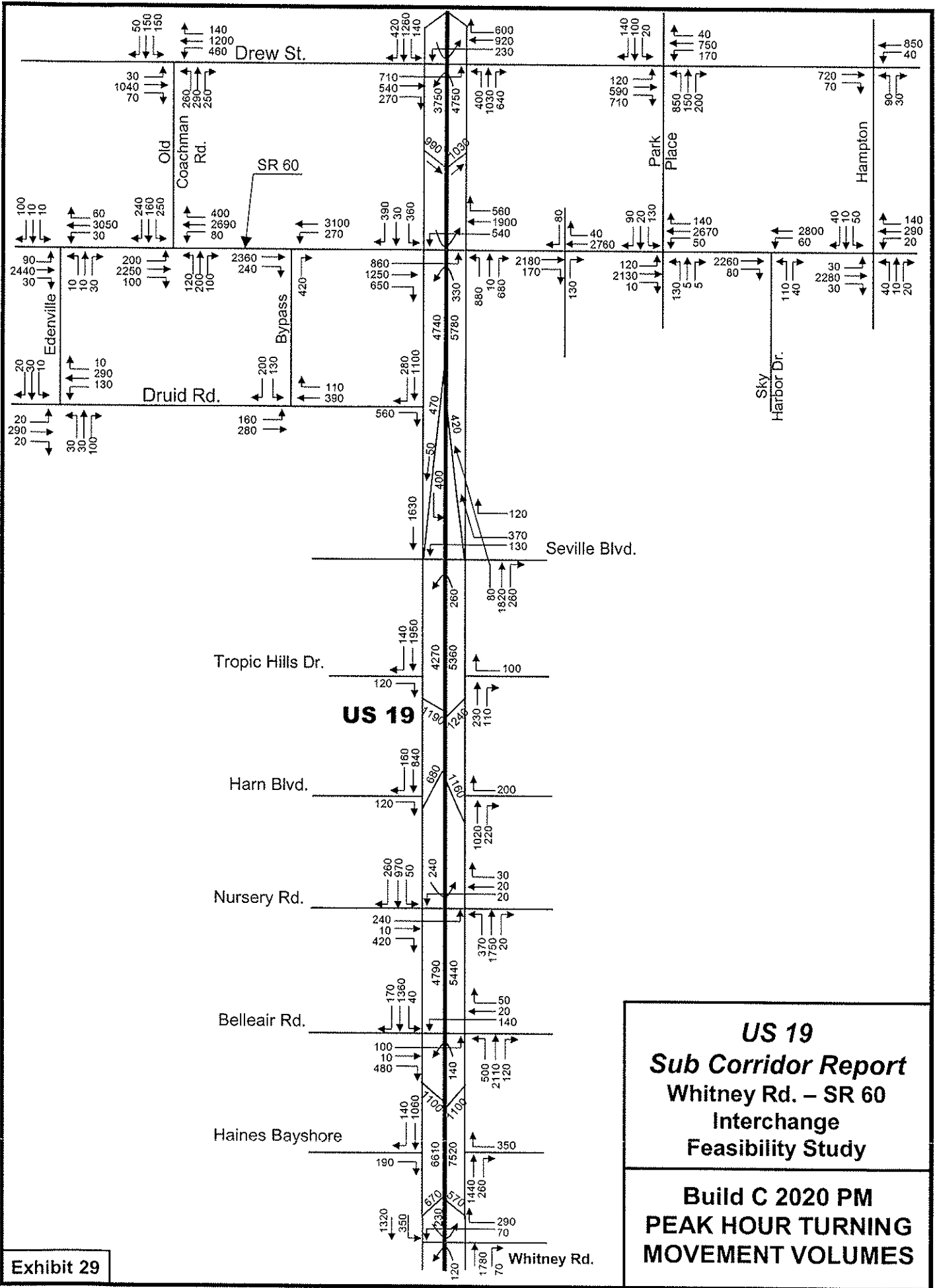
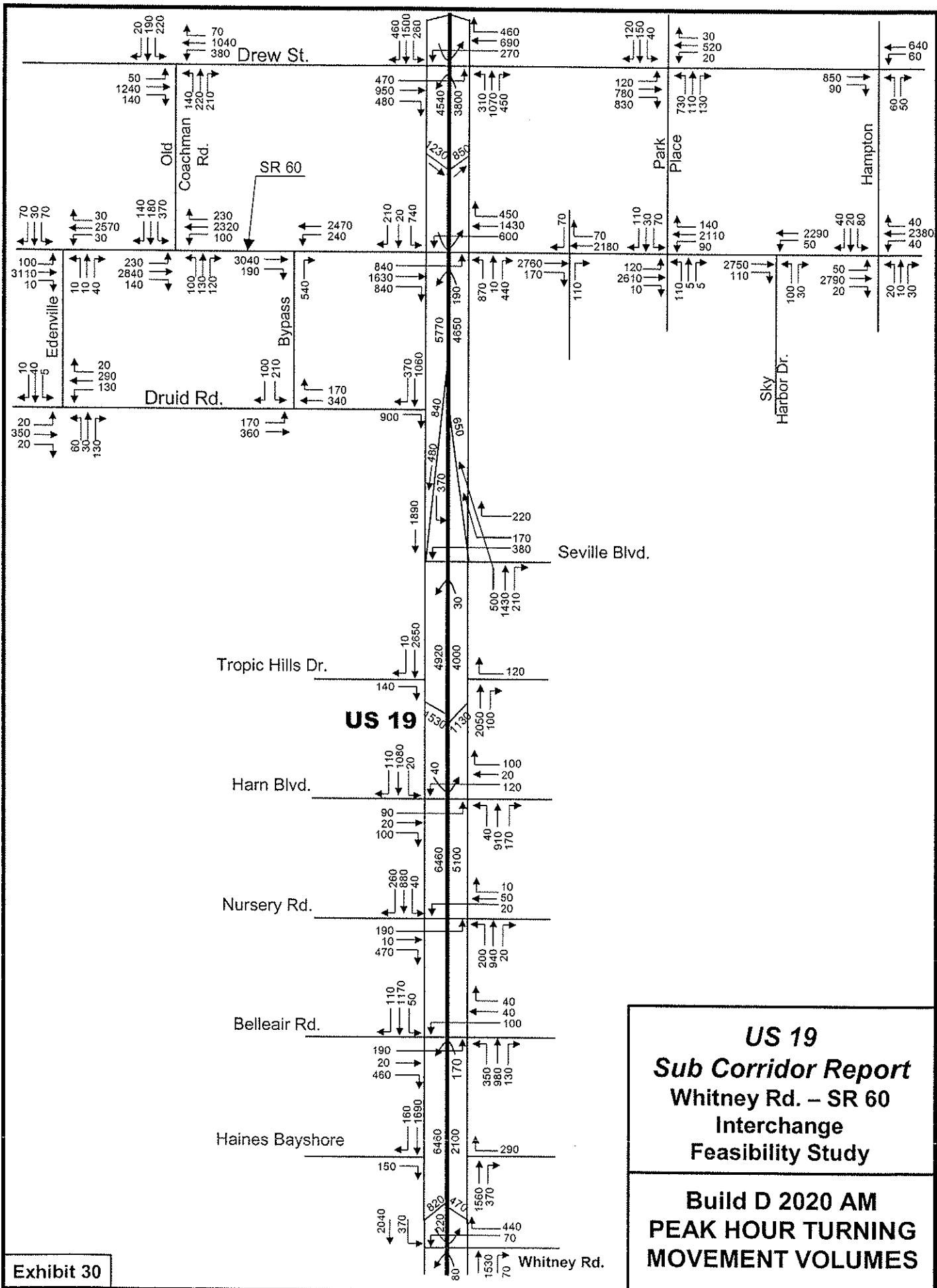


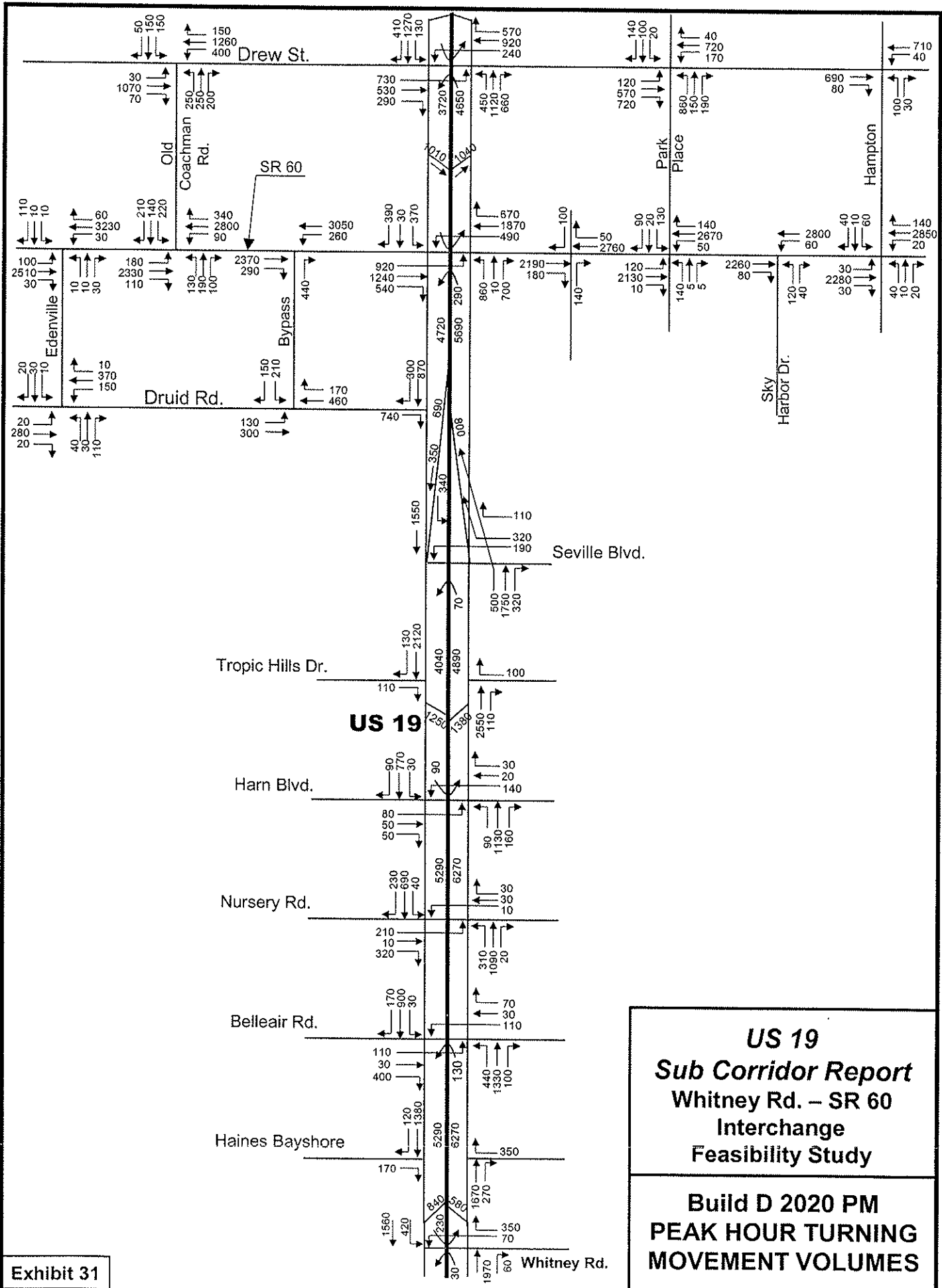
Exhibit 29



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Build D 2020 AM
PEAK HOUR TURNING
MOVEMENT VOLUMES

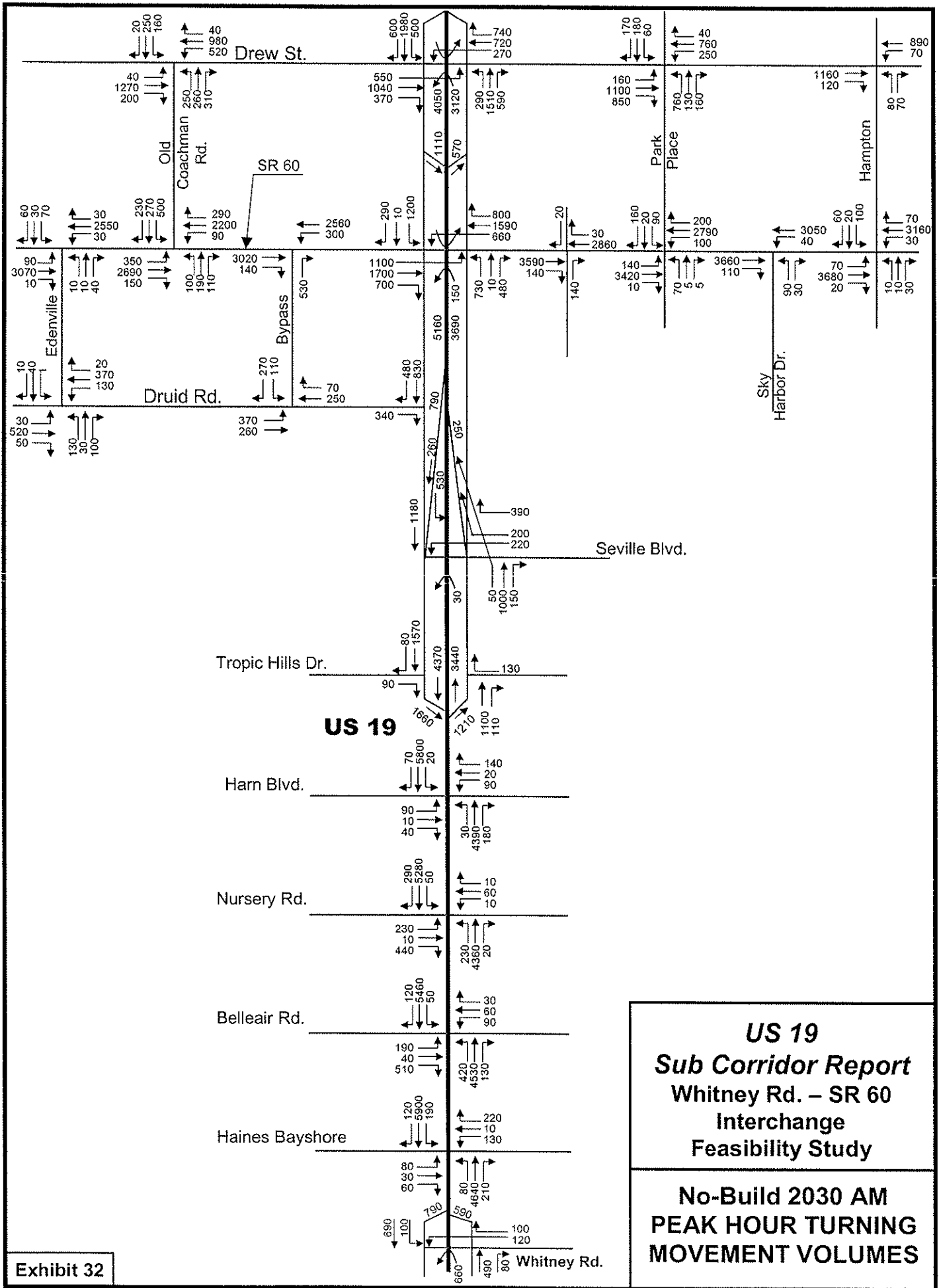
Exhibit 30



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Sub Corridor Report
Whitney Rd. – SR 60
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Build D 2020 PM
PEAK HOUR TURNING
MOVEMENT VOLUMES

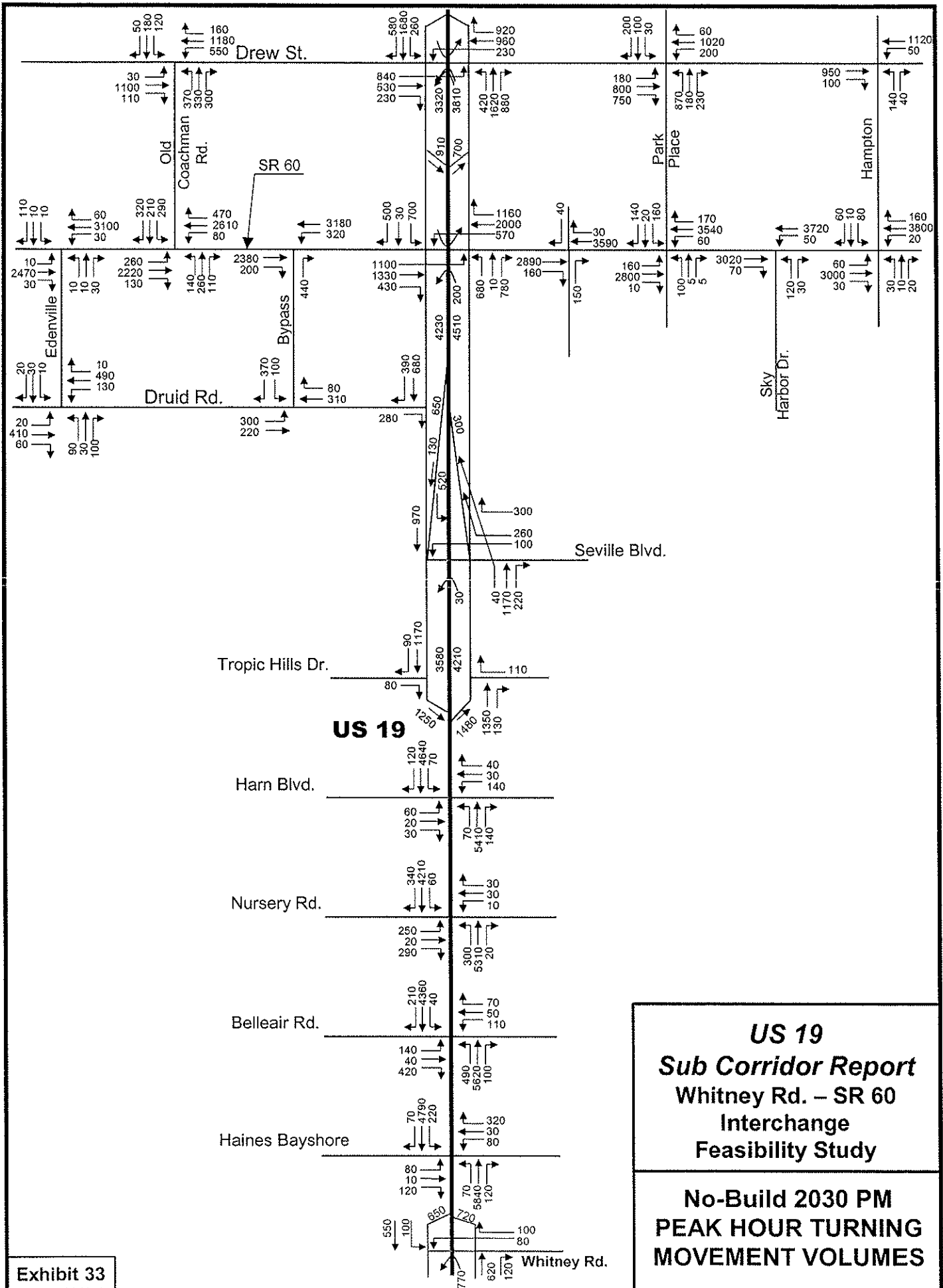
Exhibit 31

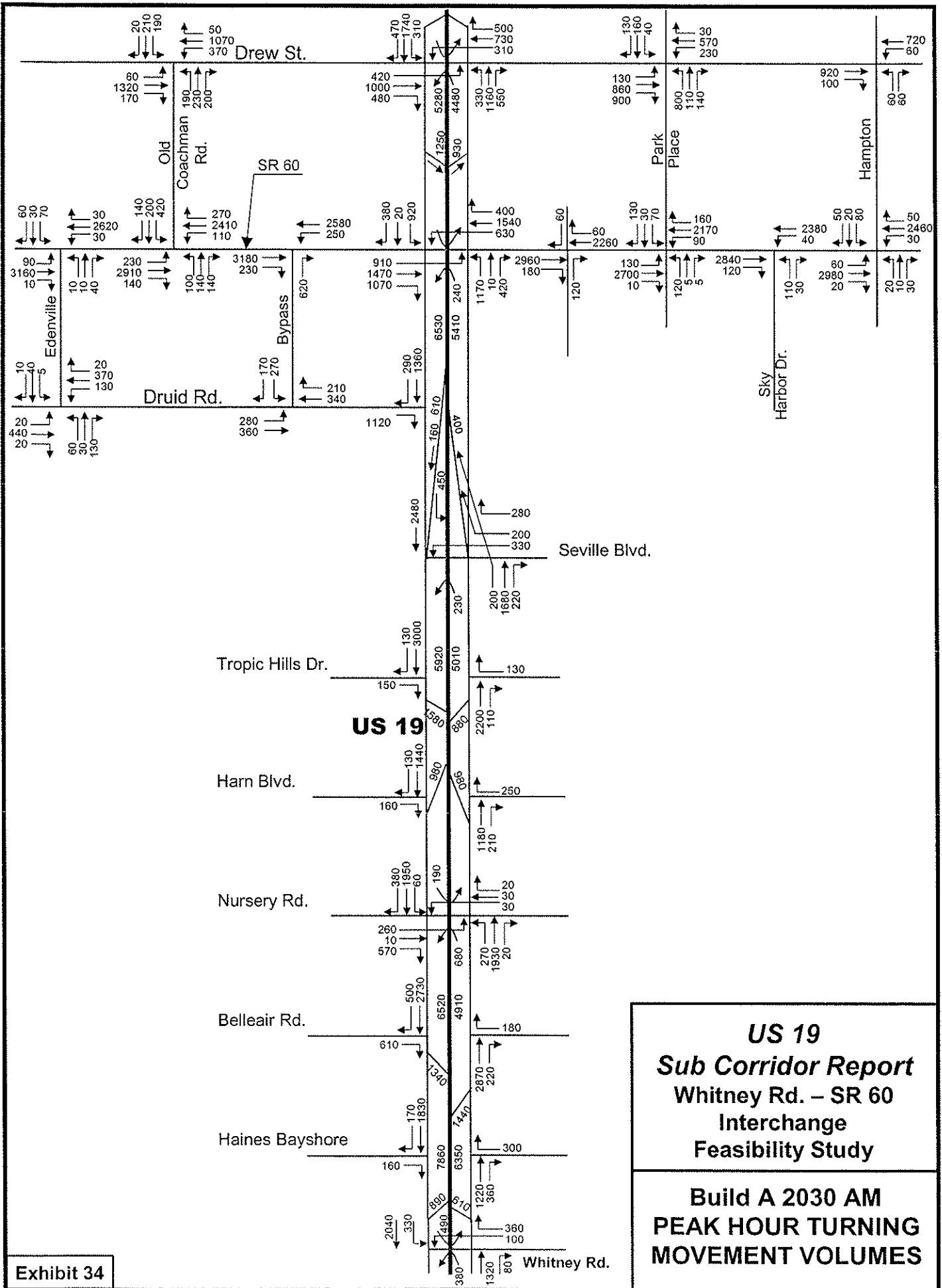


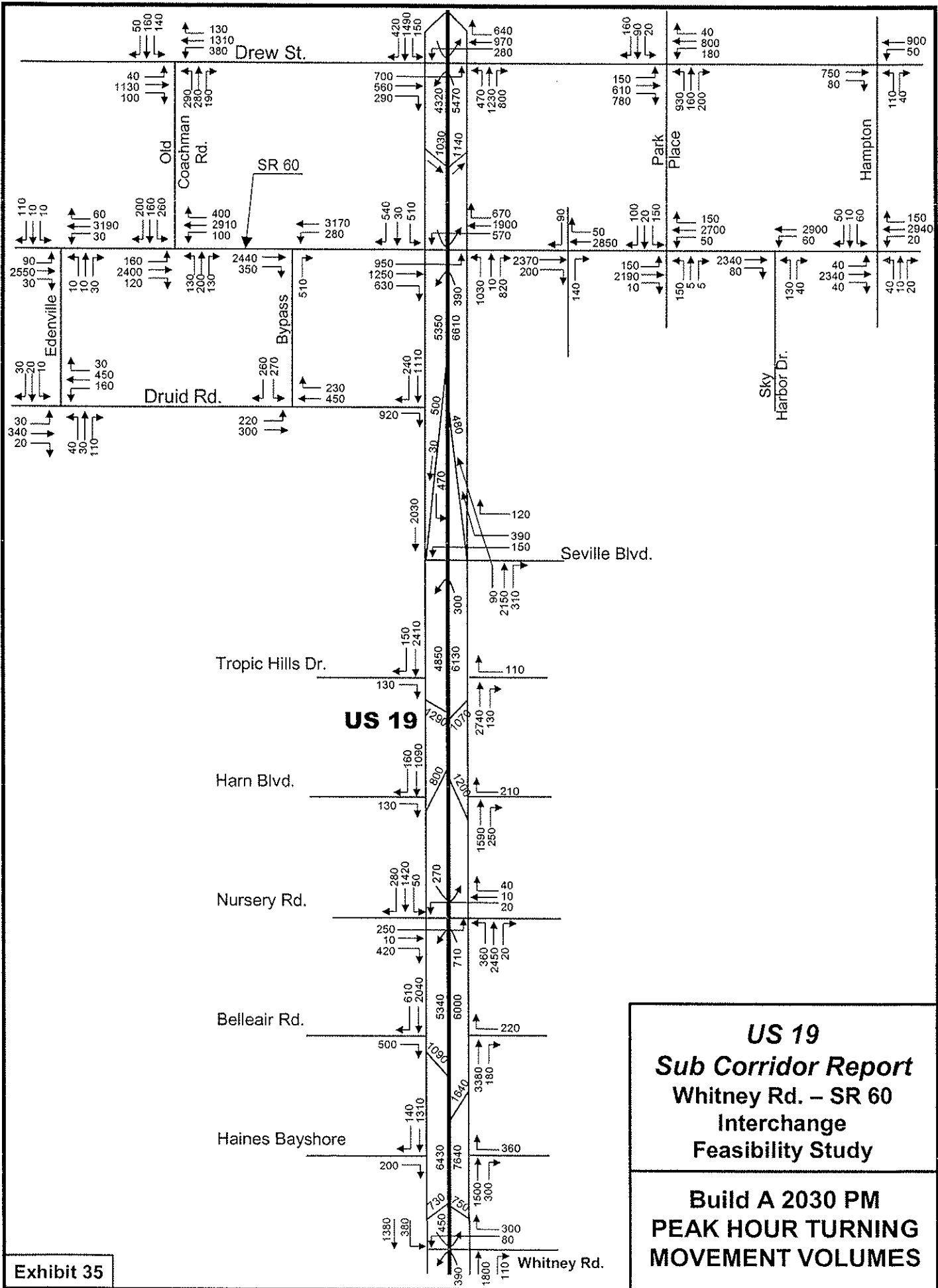
US 19
Sub Corridor Report
Whitney Rd. – SR 60
Interchange
Feasibility Study

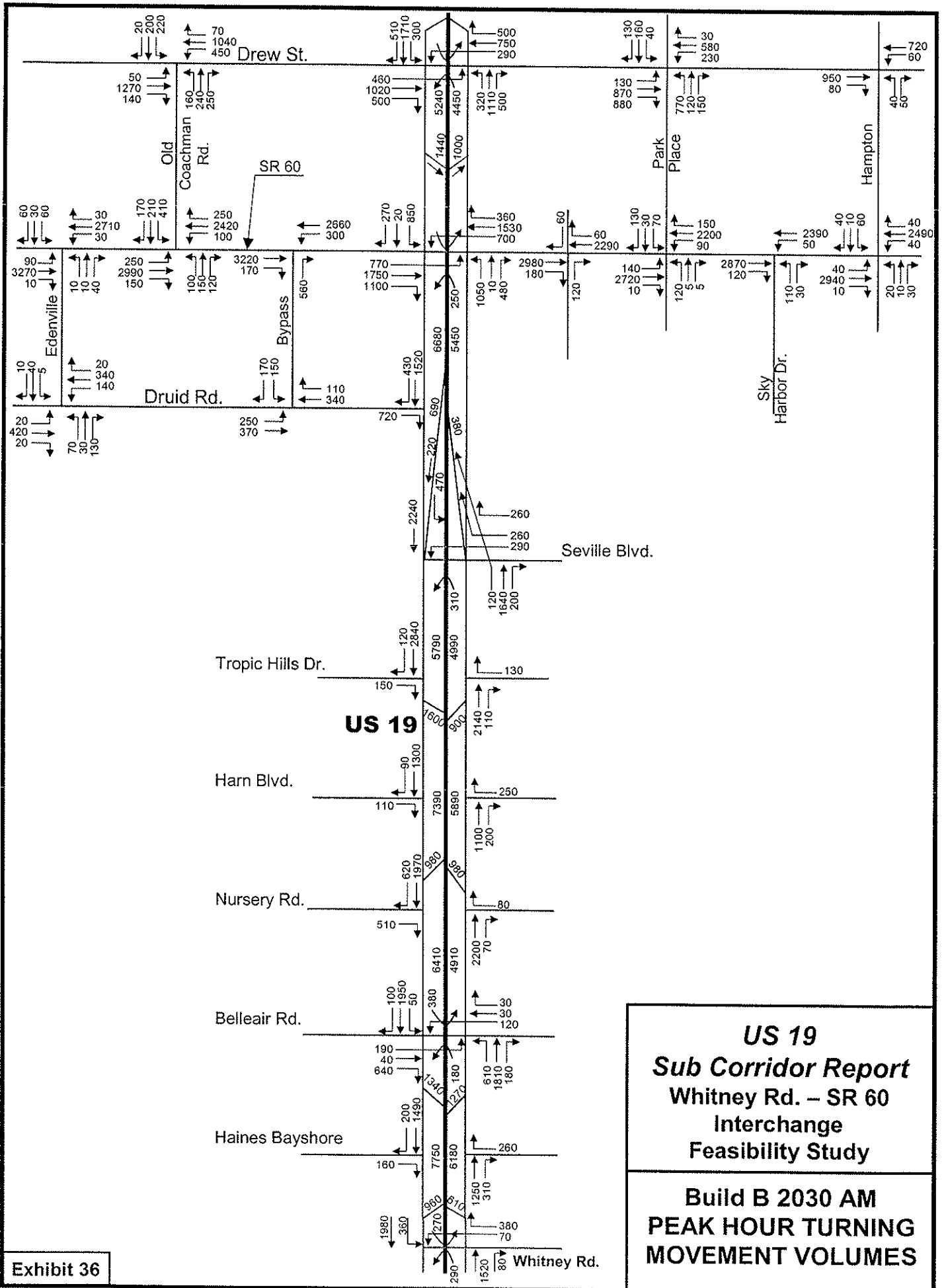
No-Build 2030 AM
PEAK HOUR TURNING
MOVEMENT VOLUMES

Exhibit 32









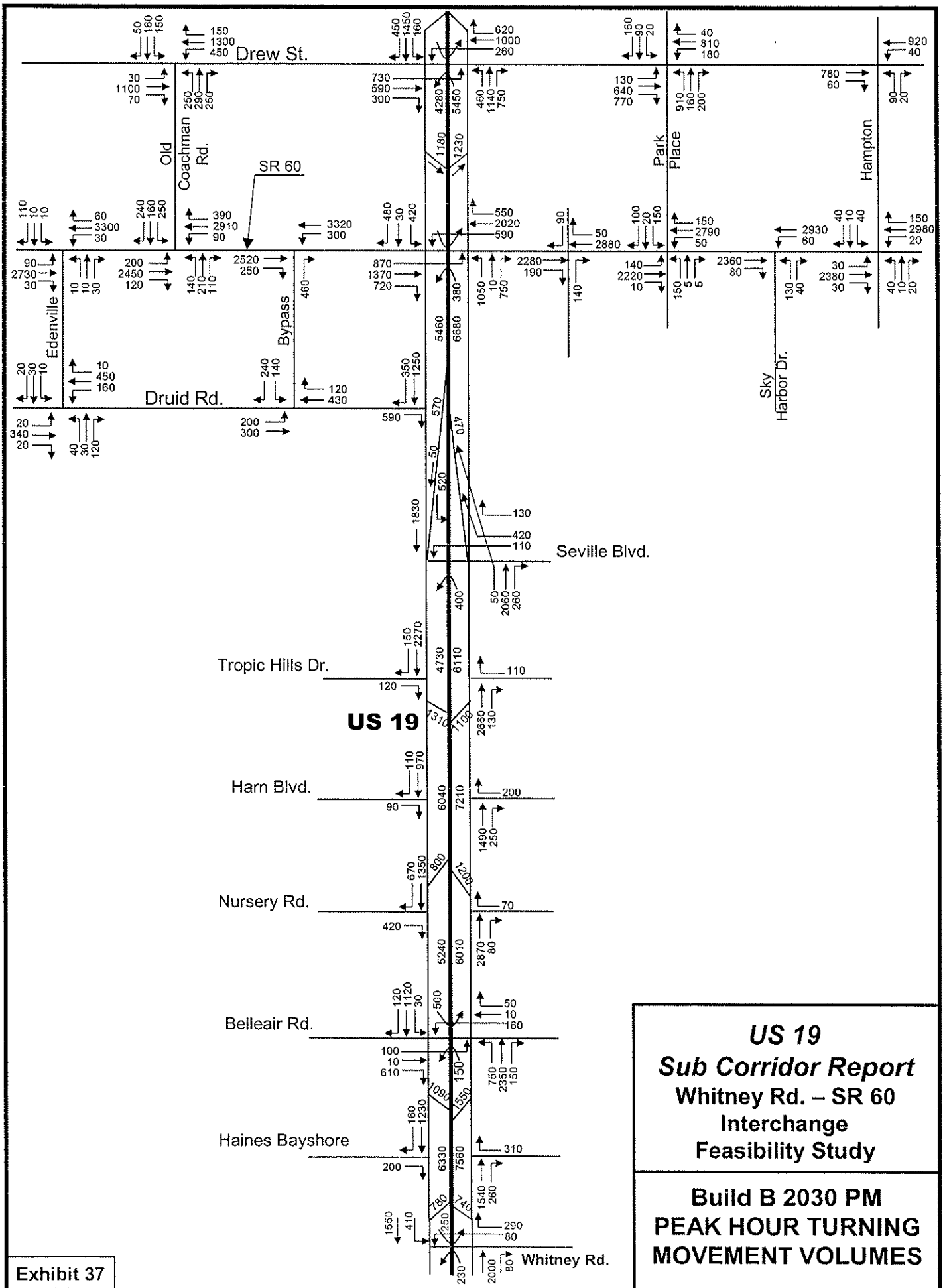


Exhibit 37

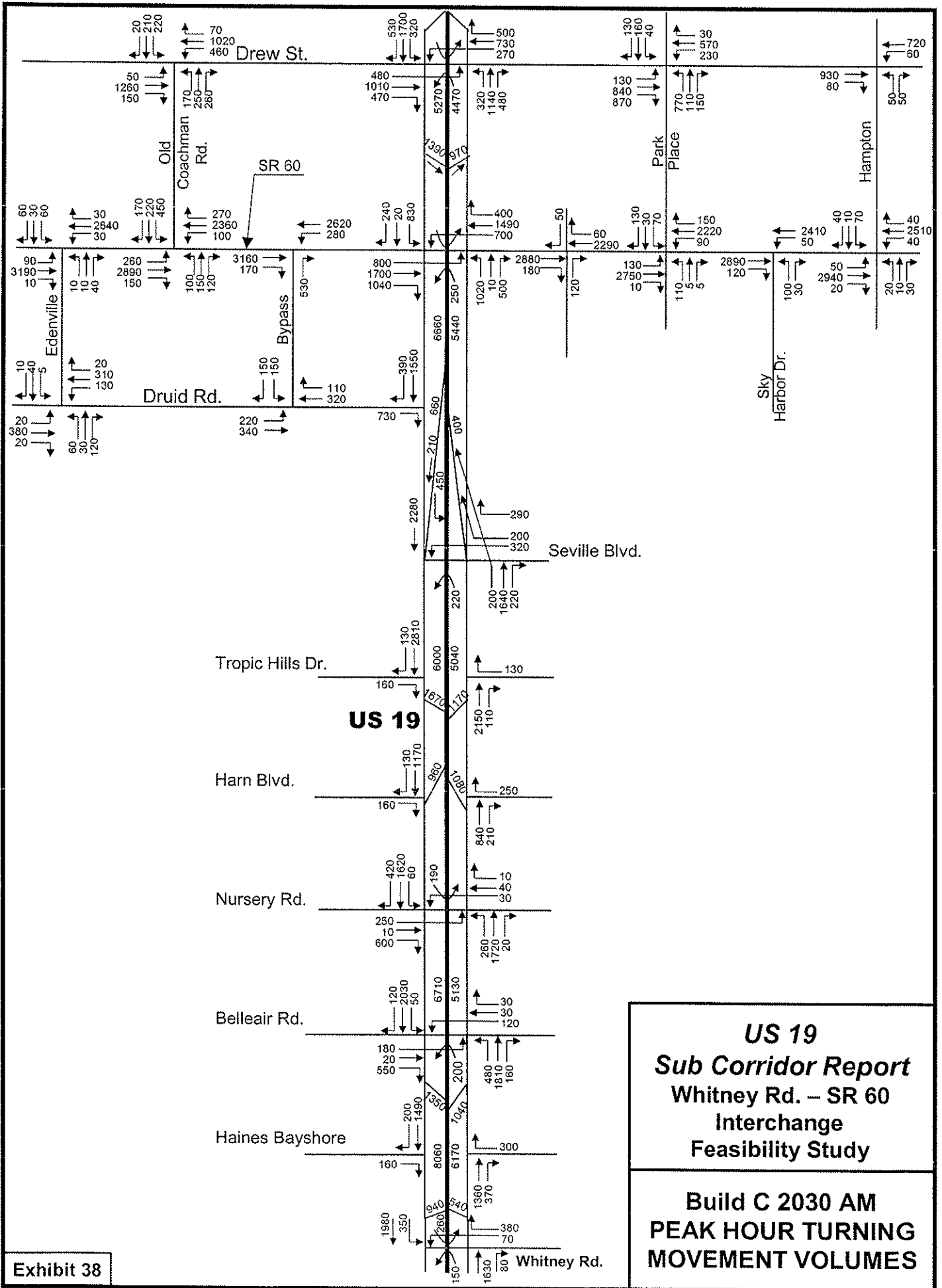
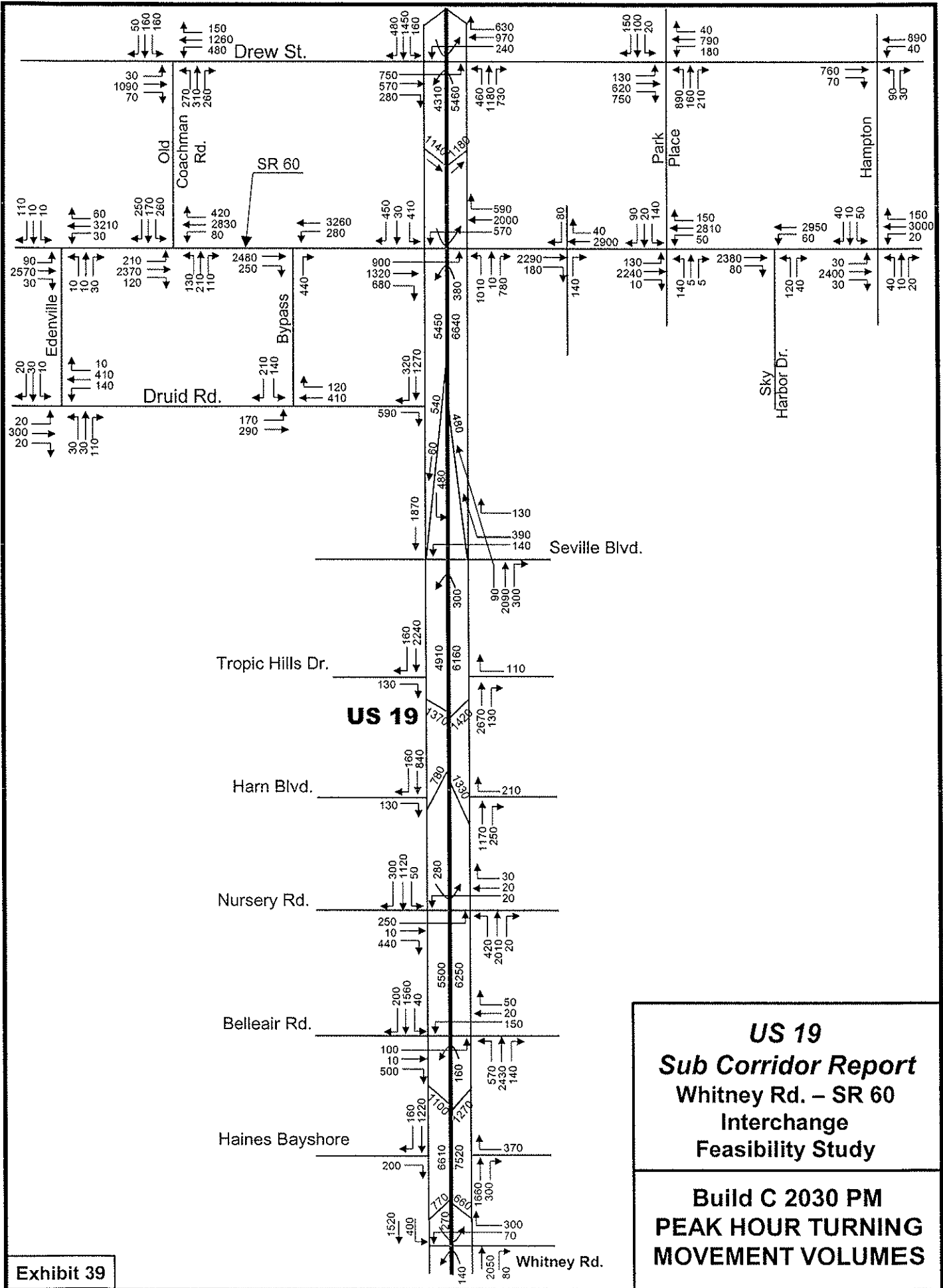
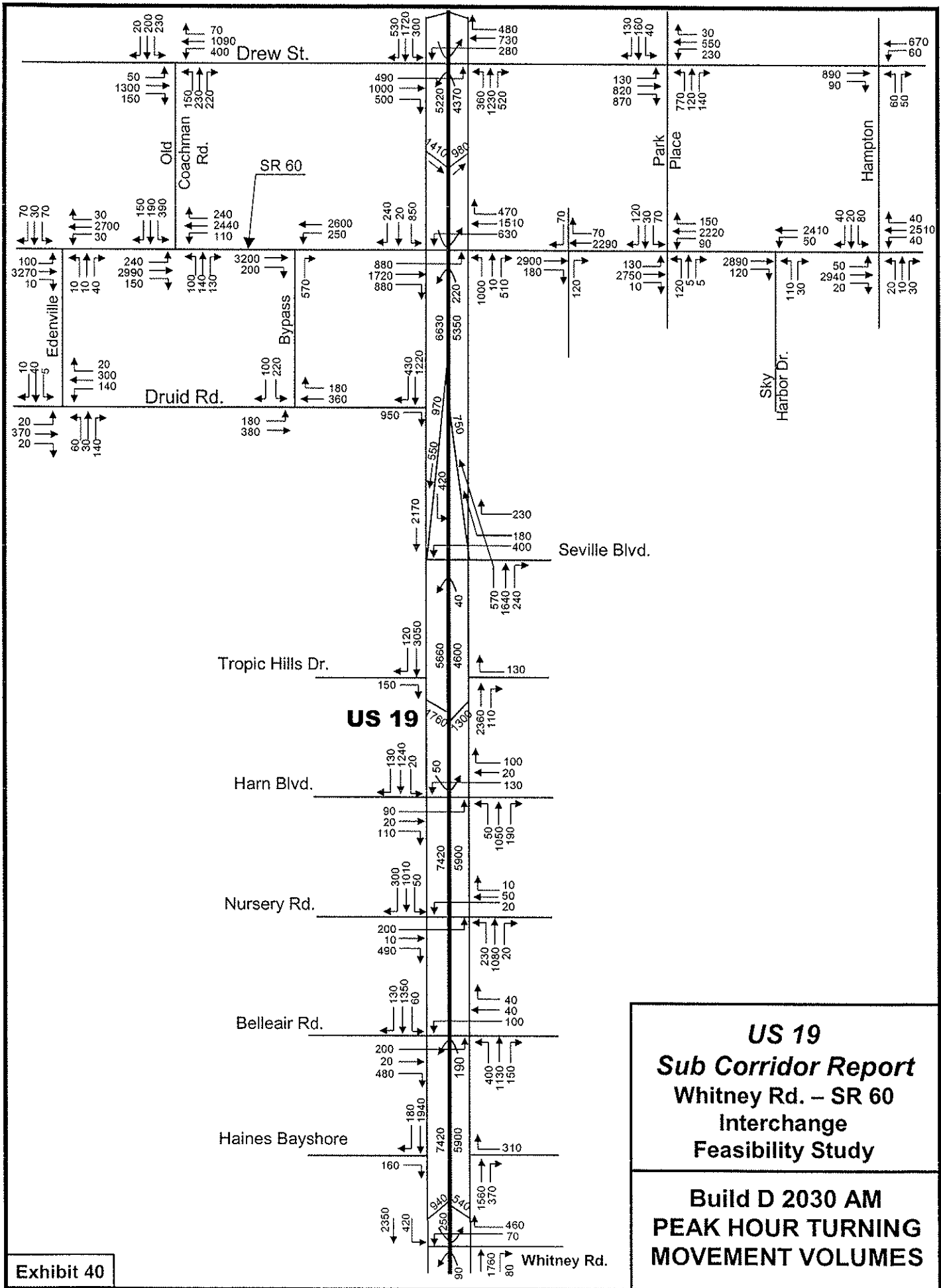


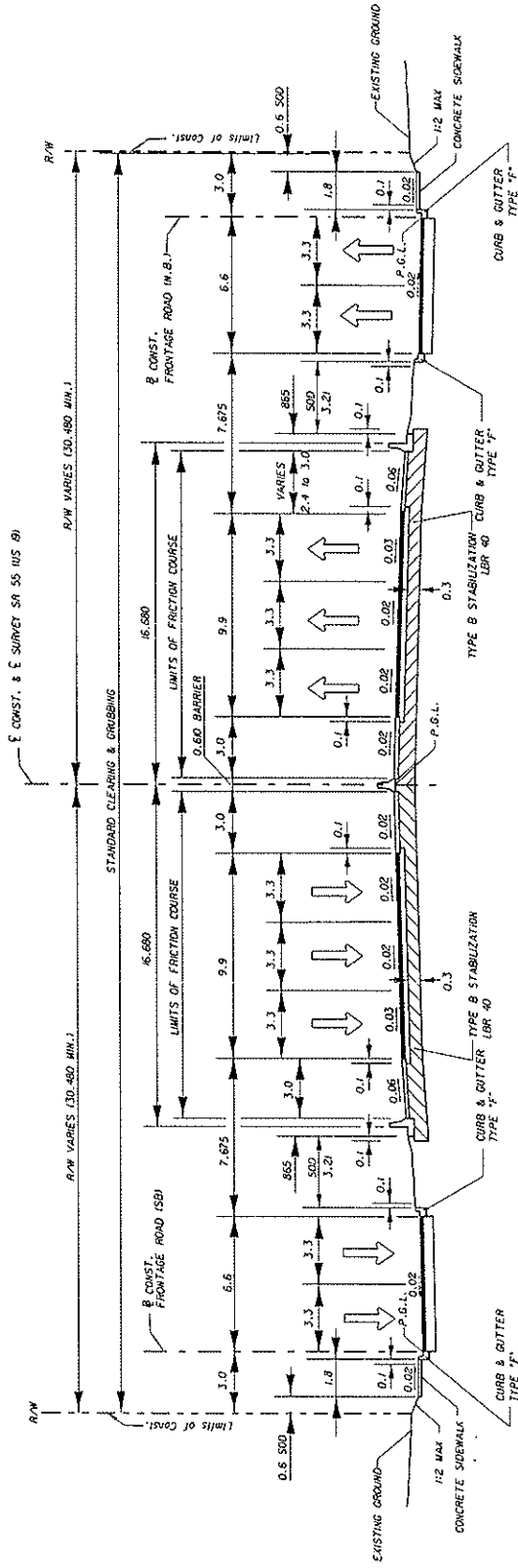
Exhibit 38



US 19
Sub Corridor Report
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Feasibility Study

Build C 2030 PM
PEAK HOUR TURNING
MOVEMENT VOLUMES





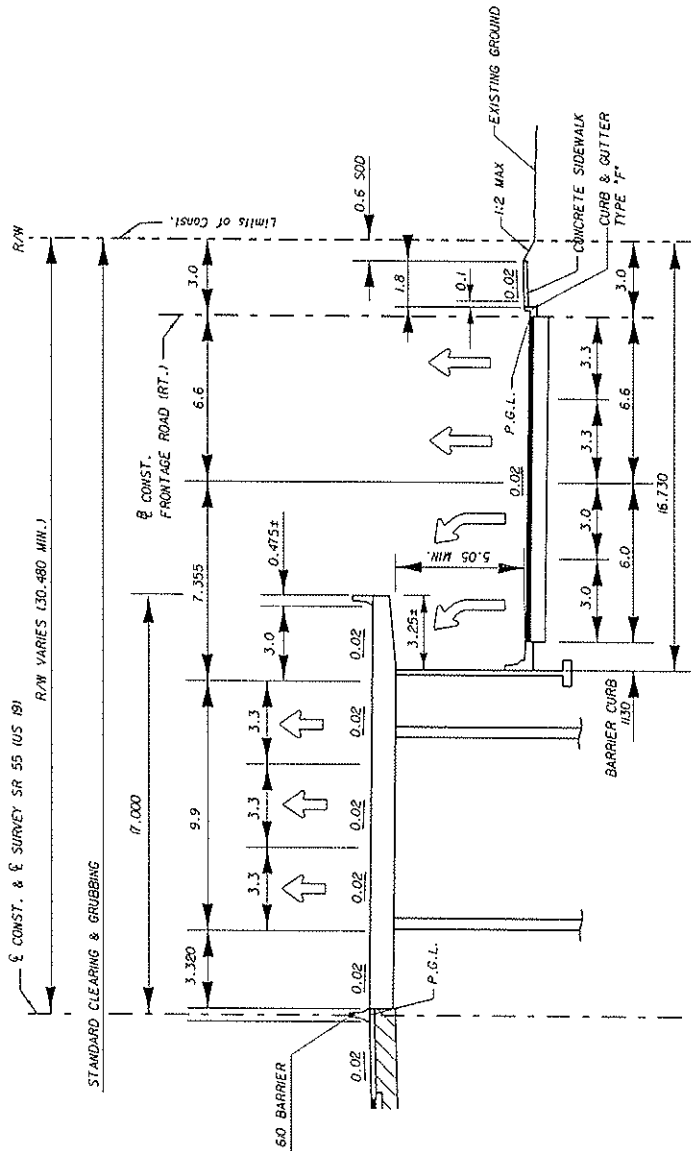
TYPICAL SECTION
S.R. 55 (U.S. 19)

NOTE: All dimensions in metric
SOURCE: U.S. 19 Roadway Design, FPD 256888-1-52-01,
H.W. Lochner, Inc.

U.S. 19 Cost Estimate Update Study
Pinellas County

Typical Section
Minimized ROW Concept

Exhibit
3-1



NOTE: All dimensions in metric

SOURCE: U.S. 19 Roadway Design, FPD 256888-1-52-01, H.W. Lochner, Inc.

TYPICAL DETAIL
 S.R. 55 (U.S. 19)
 (M.T.S.)

U.S. 19 Cost Estimate Update Study
 Pinellas County

Half-Section Cantilever at Frontage Road Left Turn
 Minimized ROW Concept

Exhibit
 3-2

FUTURE TRAFFIC OPERATIONS ANALYSIS

Future traffic conditions were analyzed using the TRAF-CORSIM and Highway Capacity Software (HCS) traffic analysis applications. Analysis was conducted on the No-Build and four (4) Build options of the US 19 Sub Corridor for both the AM and PM peak hours.

TRAF-CORSIM Analysis

TRAF-CORSIM is a software application developed by the USDOT-FHWA for the purpose of determining performance measures for modeled freeway and/or arterial roadway networks. TRAF-CORSIM simulates traffic movement through the modeled roadway network and analyzes its operating characteristics.

The TRAF-CORSIM model is composed of the physical, volume, and traffic control characteristics of a given roadway network. Physical characteristics include number of lanes, lane width, and length of turn, auxiliary, and acceleration / deceleration lanes. Volume characteristics include volume entering the roadway network from all external stations and turning movement volumes at intersections and ramp junctions. Traffic control characteristics include speed limit, type of traffic control, signal phasing, signal timing, and traffic detector locations. As input to the analysis, future signal timing and phasing were determined using Synchro software, which optimizes signal timing based on volume, lane configuration, and coordination with other intersections.

Performance measures determined from the TRAF-CORSIM analysis were average segment speed and density on the US 19 mainline. These performance measures were determined for the north segment (north of Seville Blvd.), the south segment (south of Seville Blvd.) and the overall segment (north of Whitney Rd. to north of SR 60). The north segment utilizes the same geometric configuration for all the options. Traffic in the north segment was analyzed so as to include its' influence on the south segment traffic in the study analysis.

This operational analysis, however, focuses on the south segment because this is the segment where roadway improvements differ between the various options. Thus, an

assessment of different configurations is a choice of south segment options and thus this is the analysis to which most attention should be paid.

Table 7 presents the results of the mainline operations analysis for the entire study area (southbound and northbound segments) of US 19 while Table 8 presents the results for the north segment from south of Seville Blvd. to north of Drew Street and Table 9 presents the results for the south segment from south of Whitney Rd. to south of Seville Blvd. Tables 7 through 9 cover the interim year of 2020. Tables 10 through 12 present the same results but for the design year of 2030.

Levels of Service in these tables are based on ranges of average density given in Chapter 23 – Basic Freeway Segments of the Highway Capacity Manual. For analytical purposes, the procedures of this chapter are most applicable to the configuration being analyzed.

After analyzing the various options, an additional option, Option D_(rev), was also prepared and analyzed for Design Year 2030 conditions. This option modified Option D by adding the northbound on-ramp / southbound off-ramp combination located north of Whitney Rd., which is included in the other options. There is a possibility that, without this ramp combination, additional traffic would be added to the congested intersection of the US 19 Frontage Road at East Bay Dr. A full analysis of this intersection could not be performed as it is located outside the study limits and no data were collected at this site.

Table # 7 - Interim Year (2020) Overall Segment Mainline Performance Measures

		Southbound US 19					
		No Build	A	B	C	D	D (Rev)
AM Peak	Mainline Speed (mph)	40.26	53.24	52.81	47.10	54.47	--
	Level of Service *	F	D	D	E	D	--
PM Peak	Mainline Speed (mph)	44.04	55.75	55.73	55.65	57.10	--
	Level of Service *	E	C	C	C	C	--
		Northbound US 19					
		No Build	A	B	C	D	D (Rev)
AM Peak	Mainline Speed (mph)	46.29	55.99	37.79	55.69	57.76	--
	Level of Service *	D	C	E	D	C	--
PM Peak	Mainline Speed (mph)	43.74	53.21	55.73	49.41	50.10	--
	Level of Service *	F	D	C	E	D	--

* - Level of Service is based on Average Mainline Segment Density

Table # 8 - Interim Year (2020) North Segment Mainline Performance Measures

		Southbound US 19					
		No Build	A	B	C	D	D (Rev)
AM Peak	Mainline Speed (mph)	52.47	53.38	52.90	52.54	53.49	--
	Level of Service *	C	D	D	D	D	--
PM Peak	Mainline Speed (mph)	56.58	56.07	55.86	55.96	55.66	--
	Level of Service *	C	C	C	C	C	--
		Northbound US 19					
		No Build	A	B	C	D	D (Rev)
AM Peak	Mainline Speed (mph)	56.56	56.17	56.07	55.79	56.31	--
	Level of Service *	B	C	C	C	C	--
PM Peak	Mainline Speed (mph)	55.75	55.40	55.05	55.71	55.08	--
	Level of Service *	B	D	D	D	C	--

* - Level of Service is based on Average Mainline Segment Density

Table # 9 - Interim Year (2020) South Segment Mainline Performance Measures

		Southbound US 19					
		No Build	A	B	C	D	D (Rev)
AM Peak	Mainline Speed (mph)	29.79	53.13	52.73	42.36	55.30	--
	Level of Service *	F	D	D	E	D	--
PM Peak	Mainline Speed (mph)	56.58	55.48	55.62	55.39	58.31	--
	Level of Service *	F	D	C	D	D	--

		Northbound US 19					
		No Build	A	B	C	D	D (Rev)
AM Peak	Mainline Speed (mph)	37.89	55.83	55.46	55.61	59.05	--
	Level of Service *	E	D	D	D	D	--
PM Peak	Mainline Speed (mph)	33.93	51.25	38.65	43.77	45.65	--
	Level of Service *	F	E	F	E	E	--

Table # 10 - Design Year (2030) Overall Segment Mainline Performance Measures

		Southbound US 19					
		No Build	A	B	C	D	D (Rev)
AM Peak	Mainline Speed (mph)	31.29	48.63	52.13	45.16	55.62	46.79
	Level of Service *	F	E	D	E	D	E
PM Peak	Mainline Speed (mph)	41.87	52.75	53.85	53.47	56.13	51.40
	Level of Service *	E	D	D	D	D	D

		Northbound US 19					
		No Build	A	B	C	D	D (Rev)
AM Peak	Mainline Speed (mph)	45.92	54.90	54.88	50.69	54.50	57.14
	Level of Service *	D	D	D	D	D	D
PM Peak	Mainline Speed (mph)	38.29	49.55	47.23	48.57	45.40	49.73
	Level of Service *	F	E	E	E	E	E

* - Level of Service is based on Average Mainline Segment Density

Table # 11 - Design Year (2030) North Segment Mainline Performance Measures

		Southbound US 19					
		No Build	A	B	C	D	D _(Rev)
AM Peak	Mainline Speed (mph)	51.63	53.00	52.36	53.24	54.03	50.81
	Level of Service *	C	D	D	D	D	D
PM Peak	Mainline Speed (mph)	52.57	55.05	53.92	53.41	54.35	53.90
	Level of Service *	C	D	D	D	D	D
		Northbound US 19					
		No Build	A	B	C	D	D _(Rev)
AM Peak	Mainline Speed (mph)	57.04	55.52	55.57	55.55	55.82	55.16
	Level of Service *	B	C	D	D	C	D
PM Peak	Mainline Speed (mph)	55.86	55.61	54.99	55.58	55.25	54.85
	Level of Service *	B	D	D	D	D	D

* - Level of Service is based on Average Mainline Segment Density

Table # 12 - Design Year (2030) South Segment Mainline Performance Measures

		Southbound US 19					
		No Build	A	B	C	D	D _(Rev)
AM Peak	Mainline Speed (mph)	13.84	44.88	51.93	38.12	56.96	43.39
	Level of Service *	F	E	D	F	D	E
PM Peak	Mainline Speed (mph)	32.70	50.77	53.79	53.53	57.64	49.29
	Level of Service *	F	D	D	D	D	E
		Northbound US 19					
		No Build	A	B	C	D	D _(Rev)
AM Peak	Mainline Speed (mph)	36.84	54.35	54.25	46.35	53.32	58.91
	Level of Service *	F	D	D	E	D	D
PM Peak	Mainline Speed (mph)	23.94	44.13	40.14	42.31	36.60	45.17
	Level of Service *	F	F	F	F	F	F

* - Level of Service is based on Average Mainline Segment Density

HCS Analysis

TRAF-CORSIM is a network oriented analysis tool where traffic at any point in the network is a function of upstream and downstream operations. Thus, congested conditions at an intersection or bottleneck may act as a valve, limiting traffic flow to downstream intersections. HCS, on the other hand, analyzes an intersection in near isolation without considering upstream or downstream influences. Due to congested conditions in various locations in the modeled roadway network, HCS was used to determine performance measures at selected frontage road intersections.

HCS intersection capacity analysis was performed at the signalized intersections of the US 19 frontage roads at Belleair Rd., Nursery Rd., and Harn Blvd. The four options differ physically only in the segment of US 19 from Whitney Rd. to Seville Blvd., remaining constant from Seville Blvd. north to Drew Street. HCS analysis was performed at all signalized intersections from Belleair Rd. to SR 60.

HCS analyzes intersections based on a number of traffic conditions including turning movement volumes, lane configuration, signal timing, and signal phasing. Level of Service is based on ranges of average approach and intersection delay in Chapter 16 – Signalized Intersections of the Highway Capacity Manual. Although these intersections are split with physically separate northbound and southbound through movements, these intersections were analyzed with HCS as single intersections, since they were designed to be single point urban grade separations. Signal timing and phasing were based on the Synchro analysis.

Table 13 presents the results of the performance measure analysis for signalized intersections on the US 19 frontage road for the interim year of 2020 and Table 14 presents the results for the design year of 2030.

Table # 13 - Interim Year (2020) US 19 Frontage Road Traffic Performance Measures

	Alt A - AM	Alt B - AM	Alt C - AM	Alt D - AM	Alt D _{rev} - AM
Belleair Rd.					
Int. Delay	--	91.4	98.6	53.6	98.0
Int. LOS	--	F	F	D	F
Nursery Rd.					
Int. Delay	62.1	--	89.1	47.7	99.4
Int. LOS	E	--	F	D	F
Harn Blvd.					
Int. Delay	--	--	--	32.9	44.5
Int. LOS	--	--	--	C	D
Seville Blvd.					
Int. Delay	114.7	147.7	115.5	96.2	130.4
Int. LOS	F	F	F	F	F
SR 60 - Gulf to Bay Blvd.					
Int. Delay	316.2	267.1	243.4	274.2	243.4
Int. LOS	F	F	F	F	F
	Alt A - PM	Alt B - PM	Alt C - PM	Alt D - PM	Alt D _{rev} - PM
Belleair Rd.					
Int. Delay	--	71.0	103.6	59.1	160.1
Int. LOS	--	E	F	E	F
Nursery Rd.					
Int. Delay	58.0	--	47.3	47.3	103.3
Int. LOS	E	--	D	D	F
Harn Blvd.					
Int. Delay	--	--	--	41.3	80.5
Int. LOS	--	--	--	D	F
Seville Blvd.					
Int. Delay	175.4	192.1	171.2	138.4	211.2
Int. LOS	F	F	F	F	F
SR 60 - Gulf to Bay Blvd.					
Int. Delay	243.5	269.7	240.5	252.7	240.5
Int. LOS	F	F	F	F	F

Table # 14 - Design Year (2030) US 19 Frontage Road Traffic Performance Measures

	Alt A - AM	Alt B - AM	Alt C - AM	Alt D - AM	Alt D _{rev} - AM
Belleair Rd.					
Int. Delay	--	131.7	151.6	63.6	156.1
Int. LOS	--	F	F	E	F
Nursery Rd.					
Int. Delay	97.1	--	127.2	54.7	162.7
Int. LOS	F	--	F	D	F
Harn Blvd.					
Int. Delay	--	--	--	54.7	35.0 +
Int. LOS	--	--	--	D	D
Seville Blvd.					
Int. Delay	147.3	164.8	140.1	138.5	140.1
Int. LOS	F	F	F	F	F
SR 60 - Gulf to Bay Blvd.					
Int. Delay	394	320.9	292.7	319.7	292.7
Int. LOS	F	F	F	F	F
	Alt A - PM	Alt B - PM	Alt C - PM	Alt D - PM	Alt D _{rev} - PM
Belleair Rd.					
Int. Delay	--	112.5	158.5	73.7	228.7
Int. LOS	--	F	F	E	F
Nursery Rd.					
Int. Delay	90.9	--	61.1	51.9	140
Int. LOS	F	--	E	D	F
Harn Blvd.					
Int. Delay	--	--	--	47.2	44.8
Int. LOS	--	--	--	D	D
Seville Blvd.					
Int. Delay	218.5	237.4	209.9	184.7	209.9
Int. LOS	F	F	F	F	F
SR 60 - Gulf to Bay Blvd.					
Int. Delay	292.3	269.7	287.5	302.6	287.5
Int. LOS	F	F	F	F	F

KEY FINDINGS OF OPERATIONAL ANALYSES

North Segment

The North segment is defined as from south of Seville Blvd. to north of SR 60. North segment operations in both the AM and PM peak periods were found to be LOS D or better; this is true for both the No-Build as well as each of the Build options and for both analysis years.

Exhibits showing all LOS results are to be found on the pages immediately following this discussion of the findings of the operational analyses.

South Segment

The South segment is defined as from north of Whitney Rd. to south of Seville Blvd. Results for the south segment are presented below for the No-Build as well as each of the Build options. For purposes of analysis, the No-Build option assumed that the north segment project was built but that the section from north of Whitney Rd. to south of Seville Blvd. was retained as a six-lane arterial and the three traffic signals (Belleair, Nursery and Harn) were also retained.

No Build Option

The No-Build case retains at-grade signalized intersections at Belleair Rd., Nursery Rd., and Harn Blvd. and, as expected, showed far slower speeds in the south segment of the study area compared to the Build options which have grade-separations. The TRAF-CORSIM analysis showed that grade separating US 19 from Whitney Rd. to Seville Blvd. will significantly improve operations over the No-Build option under both Interim Year (2020) and Design Year (2030) conditions. Exhibits 44 through 51 compare the results of the TRAF-CORSIM analysis for the No-Build with all Build options.

Under Interim Year (2020) conditions, all segments overall operate at Level of Service E or better with many segments working at LOS C. Under Design Year (2030) conditions, however, none of the roadway segment options will work better than LOS D during the peak periods. Reflecting commuting patterns for this part of Pinellas County, northbound (non-peak direction) traffic generally operates better

than southbound (peak direction) traffic in the AM peak period with the reverse holding true in the PM peak period.

Mainline Build Options

Focusing on individual options, A and D operate best under Interim Year (2020) conditions, while options B and D operate best under Design Year (2030) conditions. Reviewing the model operations animator, TRAFVU, it was noticed the northbound on-ramp / off-ramp combination north of Whitney Rd. experiences weaving problems, which disrupts through traffic on mainline US 19. This is due to much higher volume exiting the off-ramp located south of Belleair Rd. in Option C than the other options. Option C does, however, provide improved access to and from US 19 for the areas west of US 19 and distributes the east-west traffic load better than a single grade separation such as in options A or B.

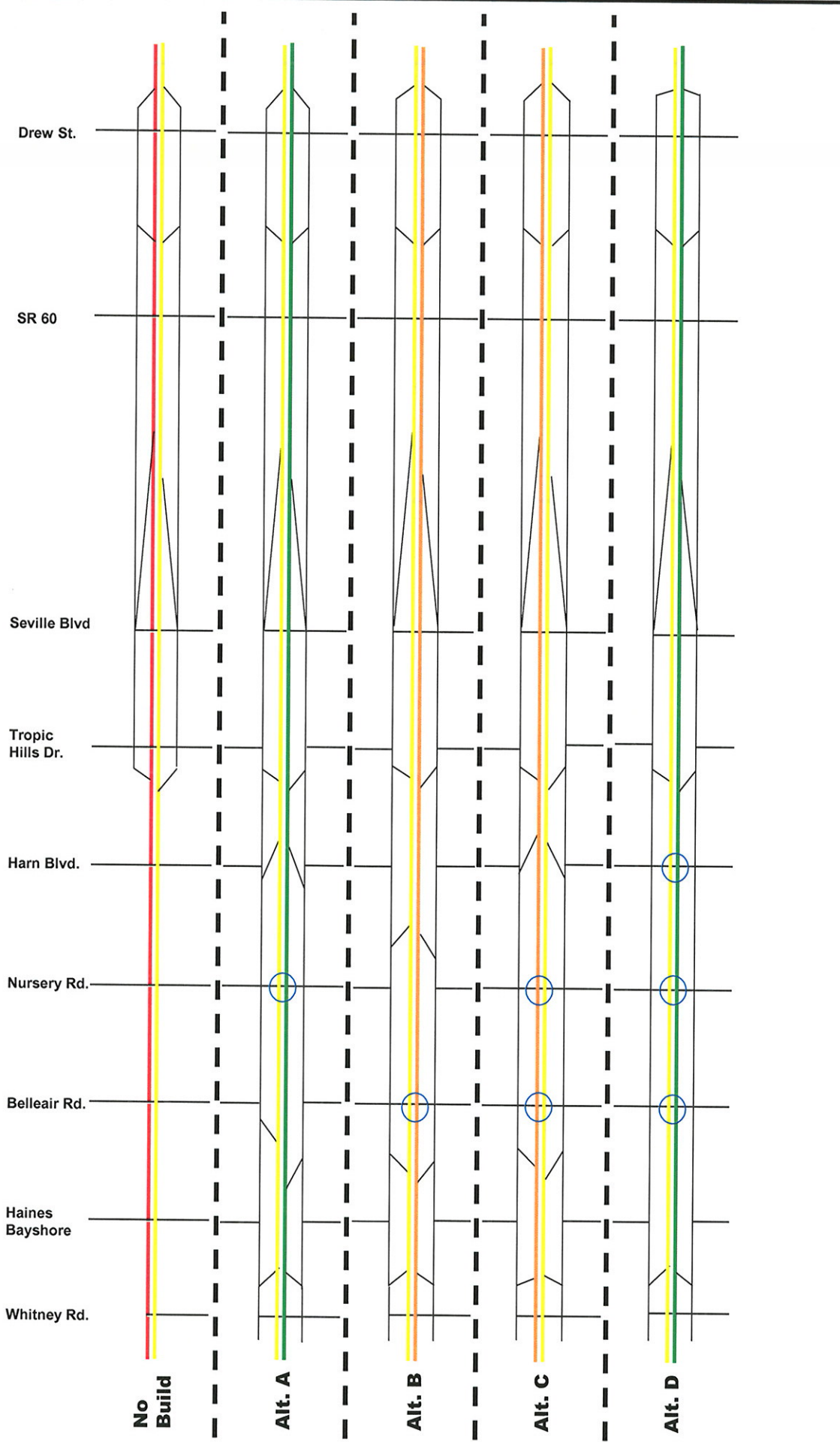
Although option D operates the best of the four options, the lack of an off-ramp south of Seville will require traffic wanting to use Belleair, Nursery, or Harn to exit south of East Bay Dr., which could further worsen conditions at this intersection. Adding an additional ramp combination north of Whitney Rd. (Option D_(rev)) to avoid adding traffic at the East Bay Dr. at-grade crossing will reintroduce the weaving issue present in Option C. Lack of a southbound on-ramp in this area will also traffic bound for mainline US 19 through the East Bay intersection.

Frontage Road Intersections

The HCS analysis showed that Option D will operate the best out of the four options, although this conceals the negative effects that will result on roadways and intersections outside the study area. Without the ramp combination south of Whitney, there will be significantly less traffic on the frontage road which will significantly improve conditions at the single point urban grade separations at Belleair, Nursery, and Harn. Option D appears to operate better because the addition of the Harn Blvd grade separated crossing will distribute traffic among all

addition of the Harn Blvd grade separated crossing will distribute traffic among all three crossings. Exhibits 52 through 55 present the results of the HCS analysis.

As for Option A, B, and C, operations at the signalized intersections at Belleair, Nursery, and Harn operate at mostly capacity (LOS 'E') conditions or worse in the Interim Year (2020) case and mostly failing (LOS 'F') conditions under the Design Year (2030) case. The reason for this is a lack of through lane capacity on the frontage roads to handle the future traffic demand projected in 2030. This is demonstrated with Option D _(rev), which showed failing conditions similar to Options A, B, and C. The addition of the on-ramp / off-ramp combination increases traffic on the frontage road, worsening through capacity at the single point urban grade separations.

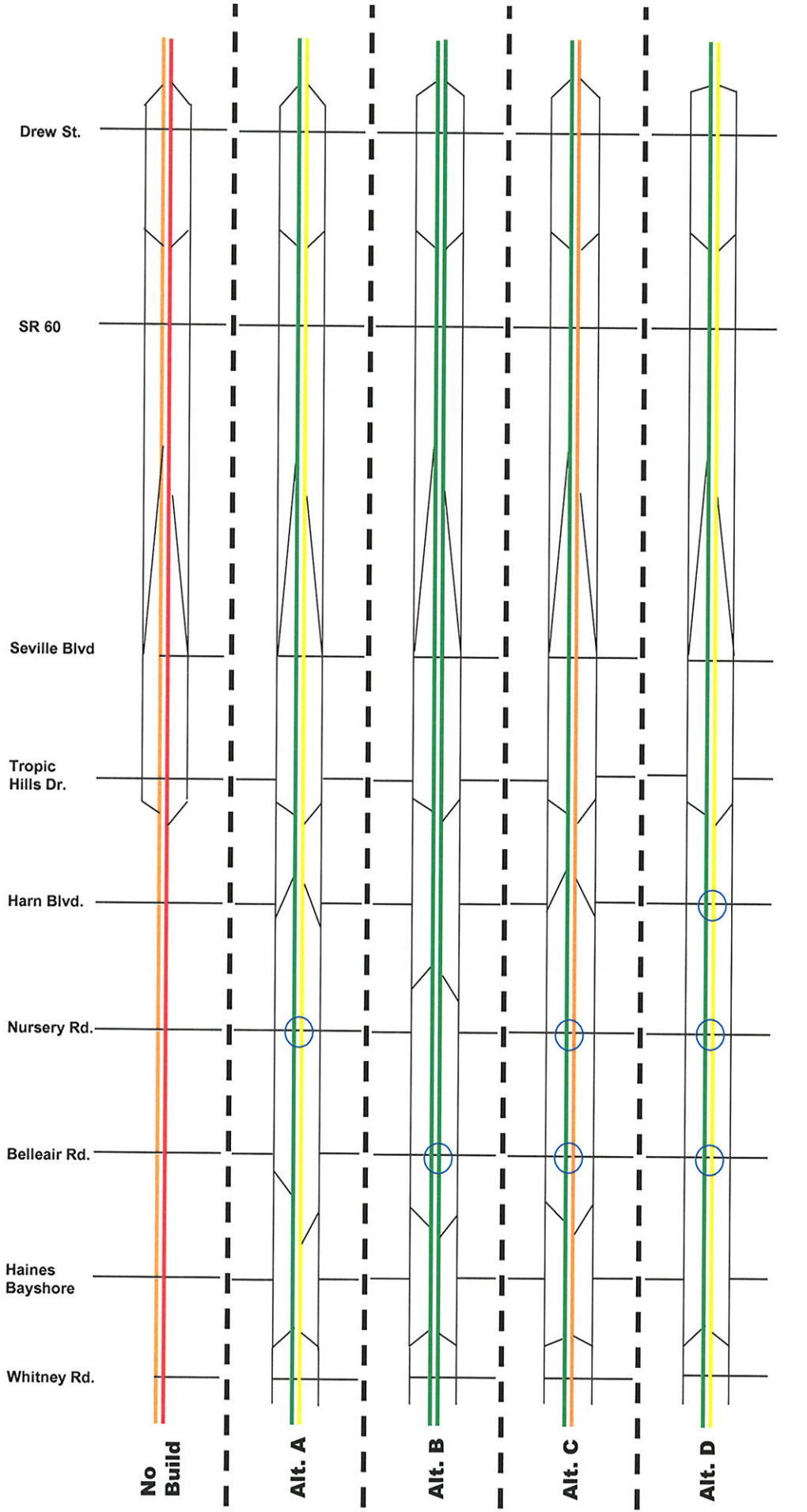


- Interchange Location
- LOS B or C
- LOS D
- LOS E
- LOS F

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US 19
Sub Corridor Report
 Whitney Rd. – SR 60
 Interchange Feasibility Study

2020 TRAFFIC ANALYSIS
 PM PEAK LOS RESULTS
 US 19 MAINLINE OVERALL

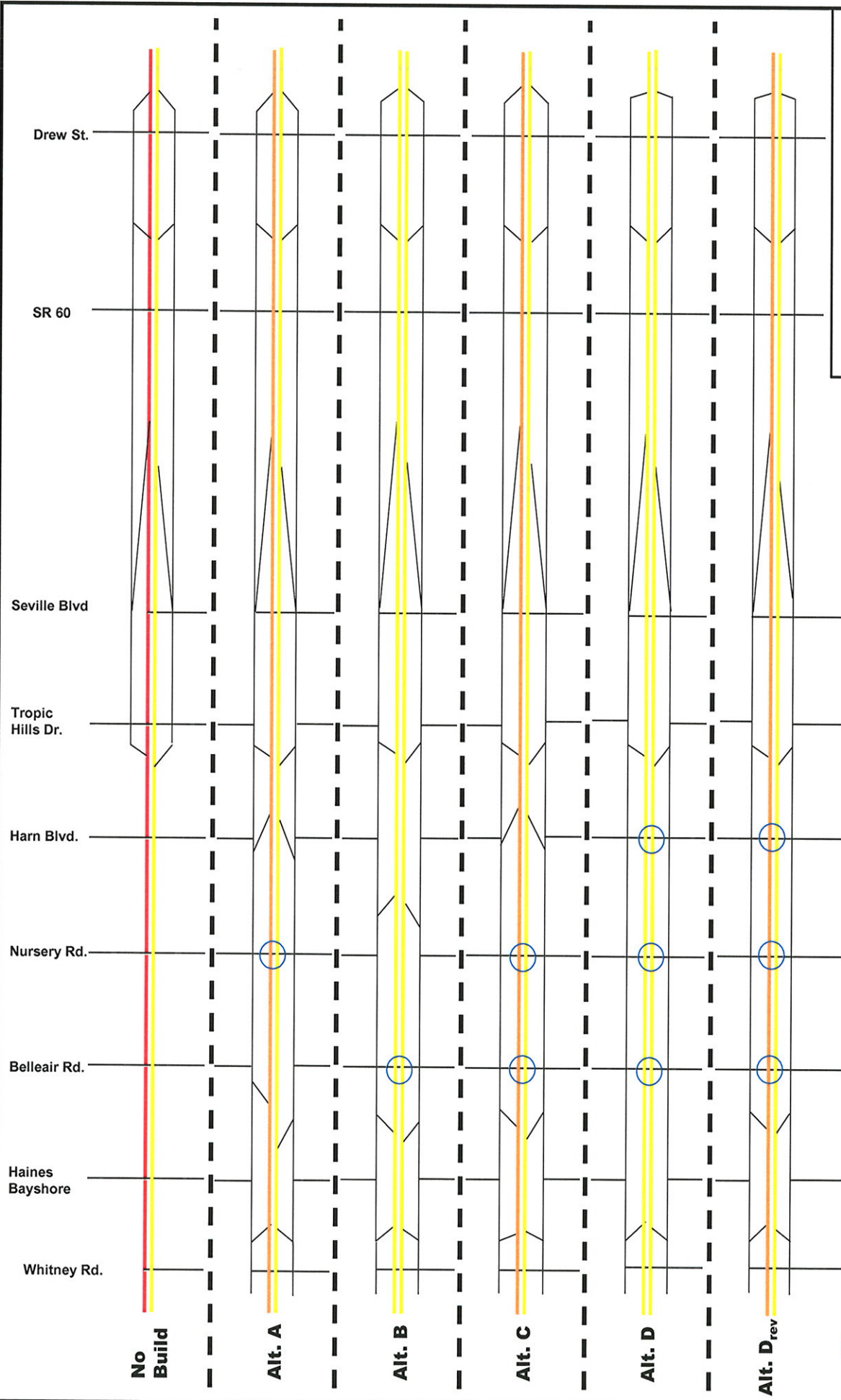


- - Interchange Location
- (Green) - LOS B or C
- (Yellow) - LOS D
- (Orange) - LOS E
- (Red) - LOS F

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 Whitney Rd. - SR 60
 Interchange Feasibility Study

2030 TRAFFIC ANALYSIS
 AM PEAK LOS RESULTS
 US 19 MAINLINE OVERALL

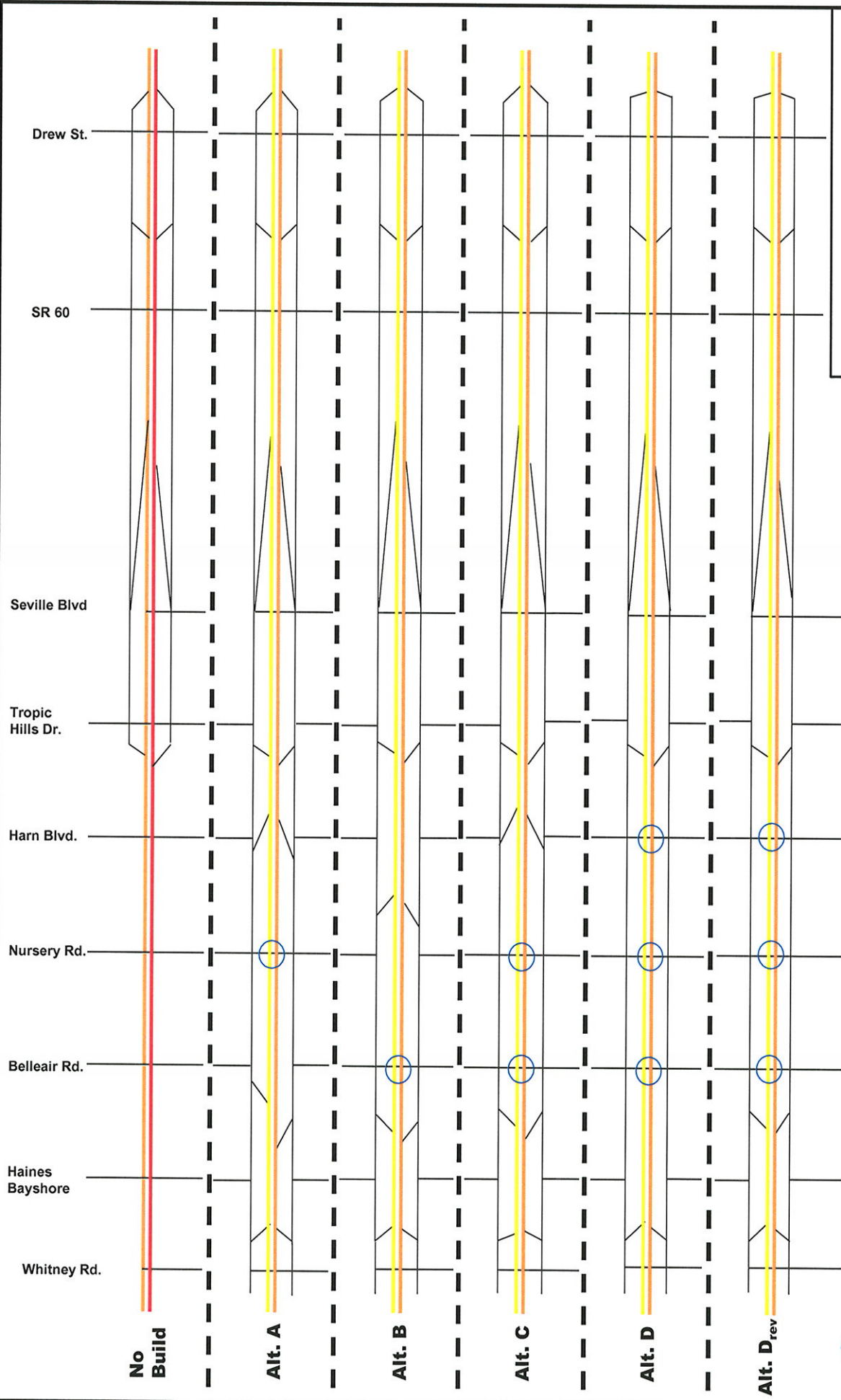


- - Interchange Location
- - LOS B or C
- - LOS D
- - LOS E
- - LOS F

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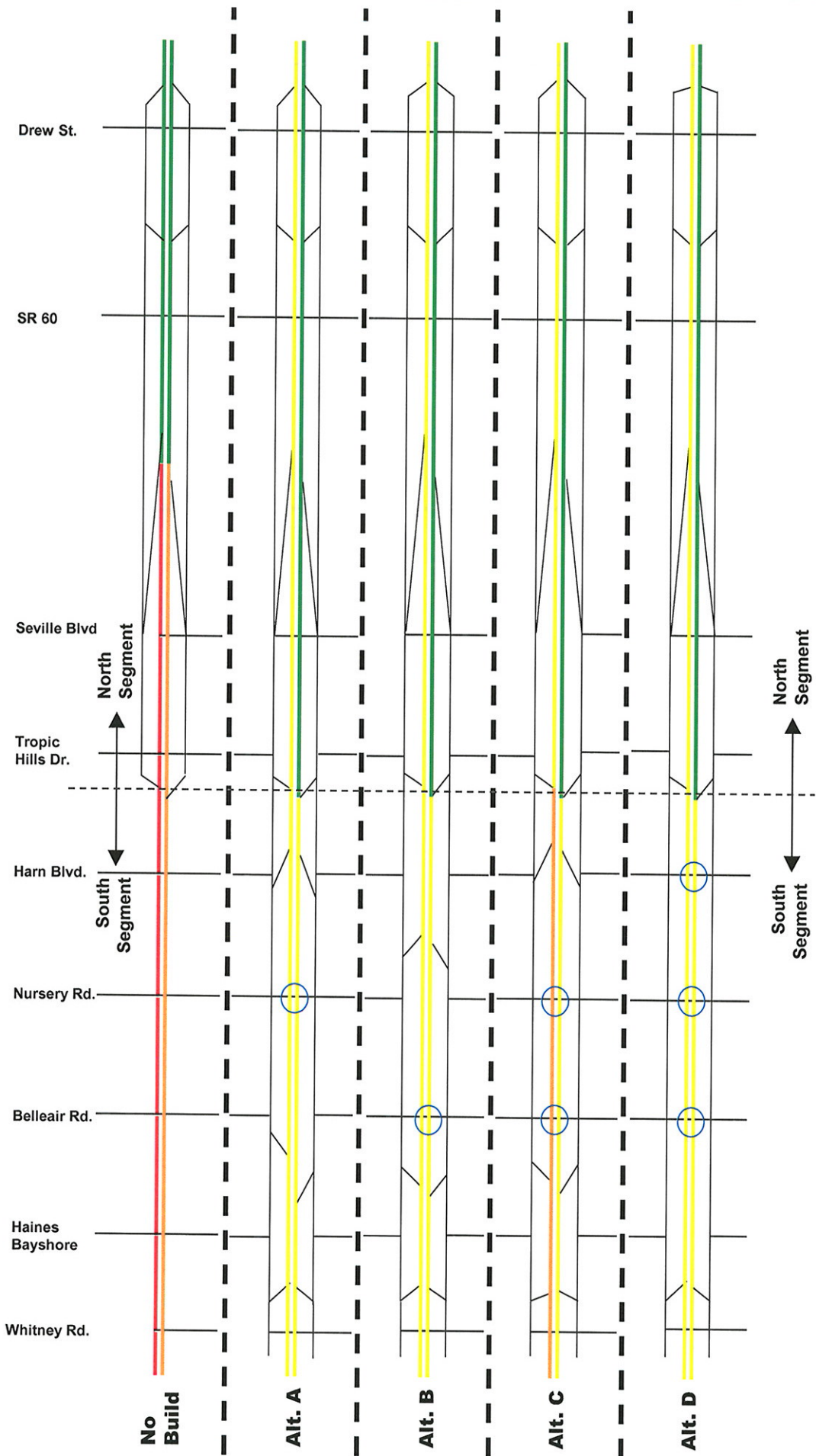
US 19
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 Whitney Rd. - SR 60
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2030 TRAFFIC ANALYSIS
 PM PEAK LOS RESULTS
 US 19 MAINLINE OVERALL



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- - Interchange Location
- (Green) - LOS B or C
- (Yellow) - LOS D
- (Orange) - LOS E
- (Red) - LOS F

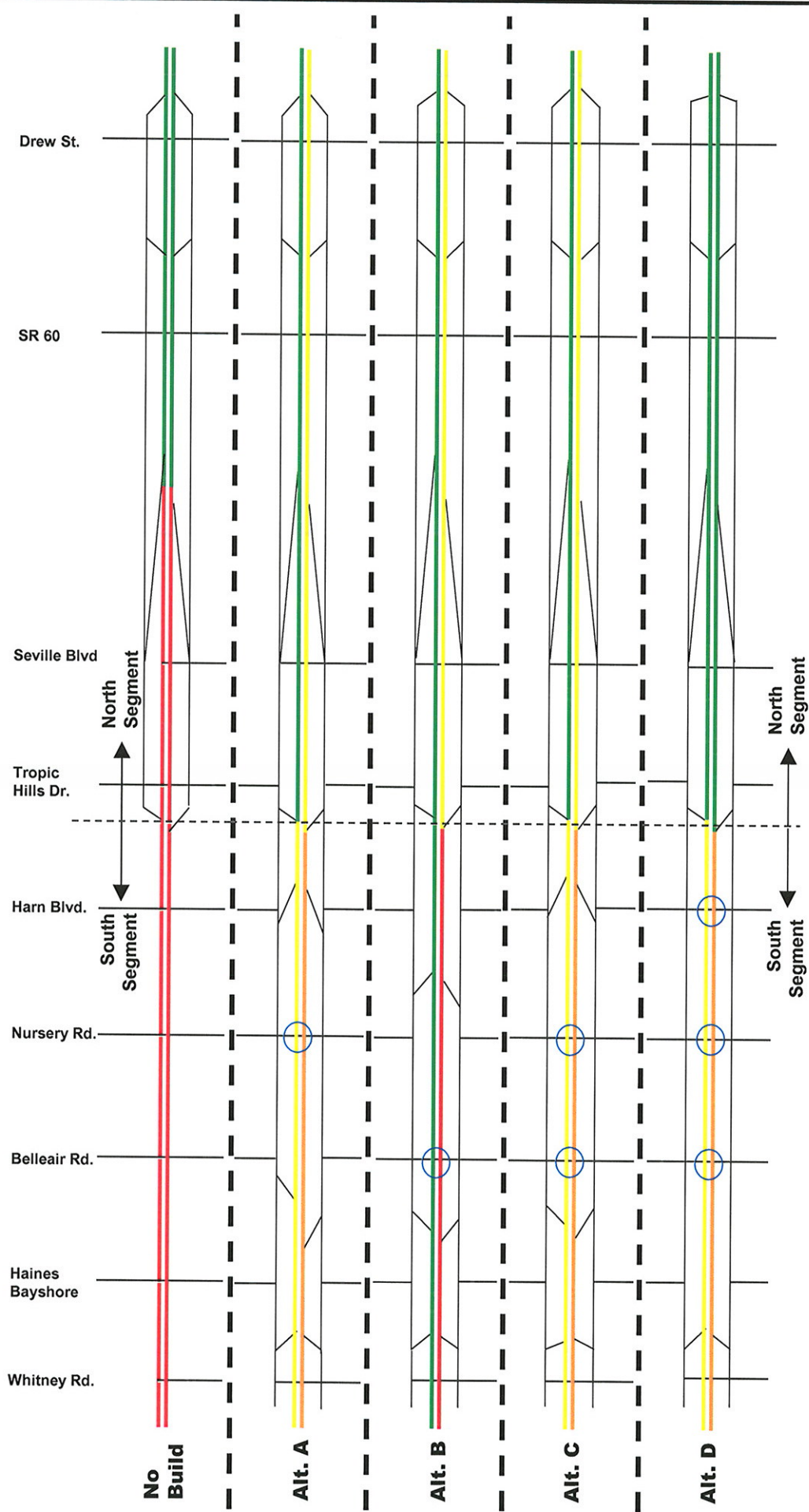


- - Interchange Location
- - LOS B or C
- - LOS D
- - LOS E
- - LOS F

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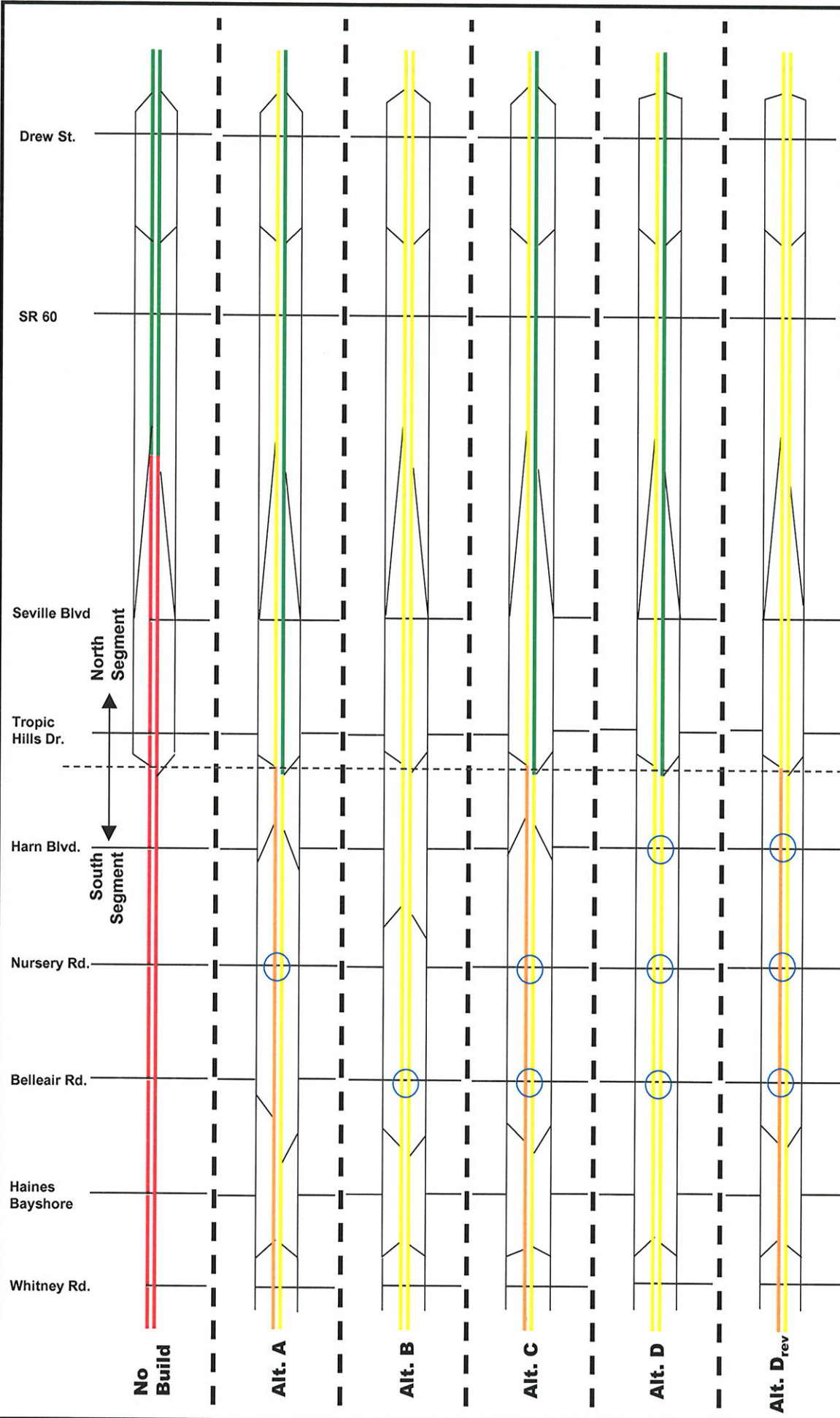
US 19
Sub Corridor Report
 Whitney Rd. – SR 60
 Interchange Feasibility Study

2020 TRAFFIC ANALYSIS
 PM PEAK LOS RESULTS
 US 19 MAINLINE SEGMENTS



- Interchange Location
- LOS B or C
- LOS D
- LOS E
- LOS F

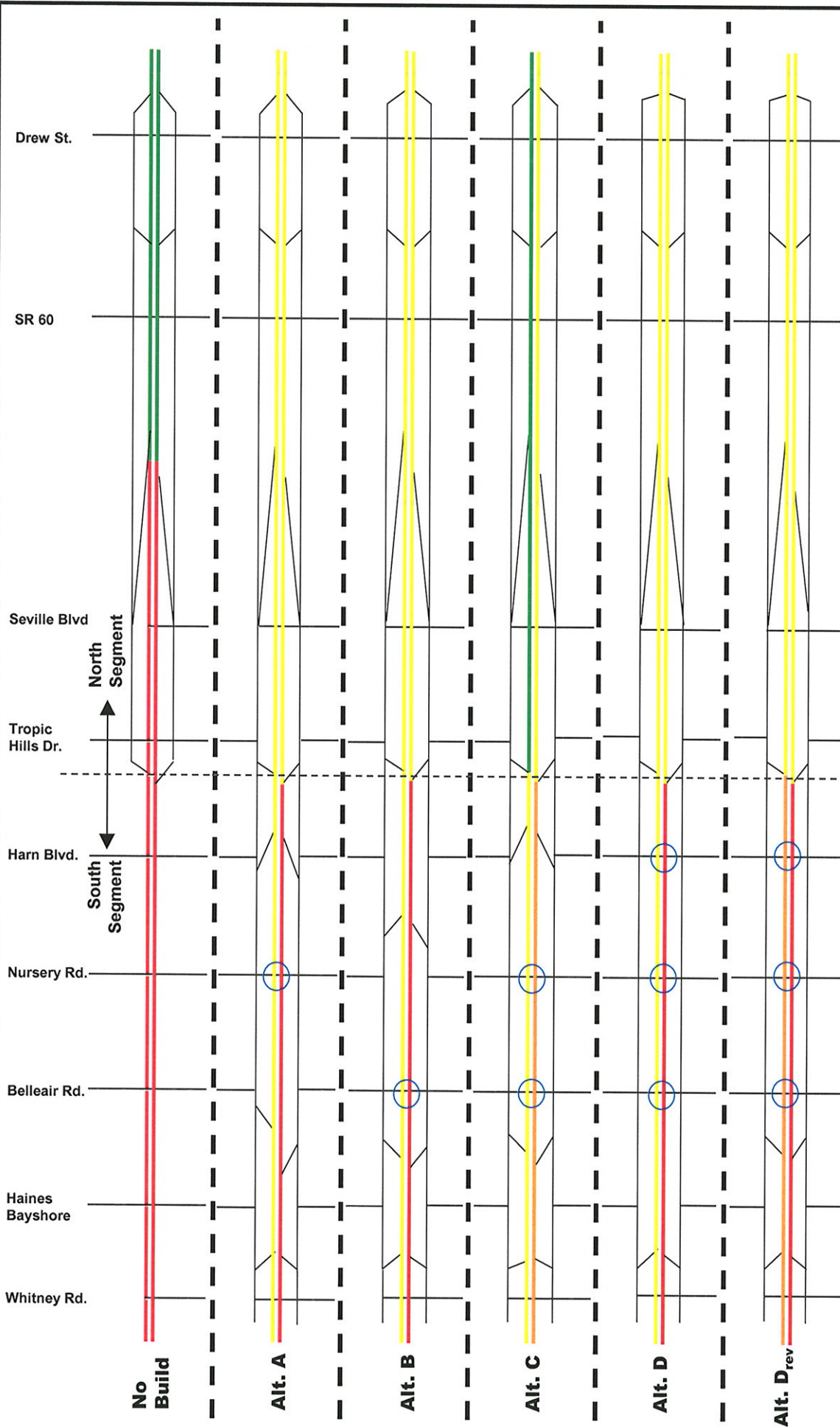
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 Whitney Rd. - SR 60
 Interchange Feasibility Study

2030 TRAFFIC ANALYSIS
 PM PEAK LOS RESULTS
 US 19 MAINLINE SEGMENTS



- Interchange Location
- LOS B or C
- LOS D
- LOS E
- LOS F
- LOS F

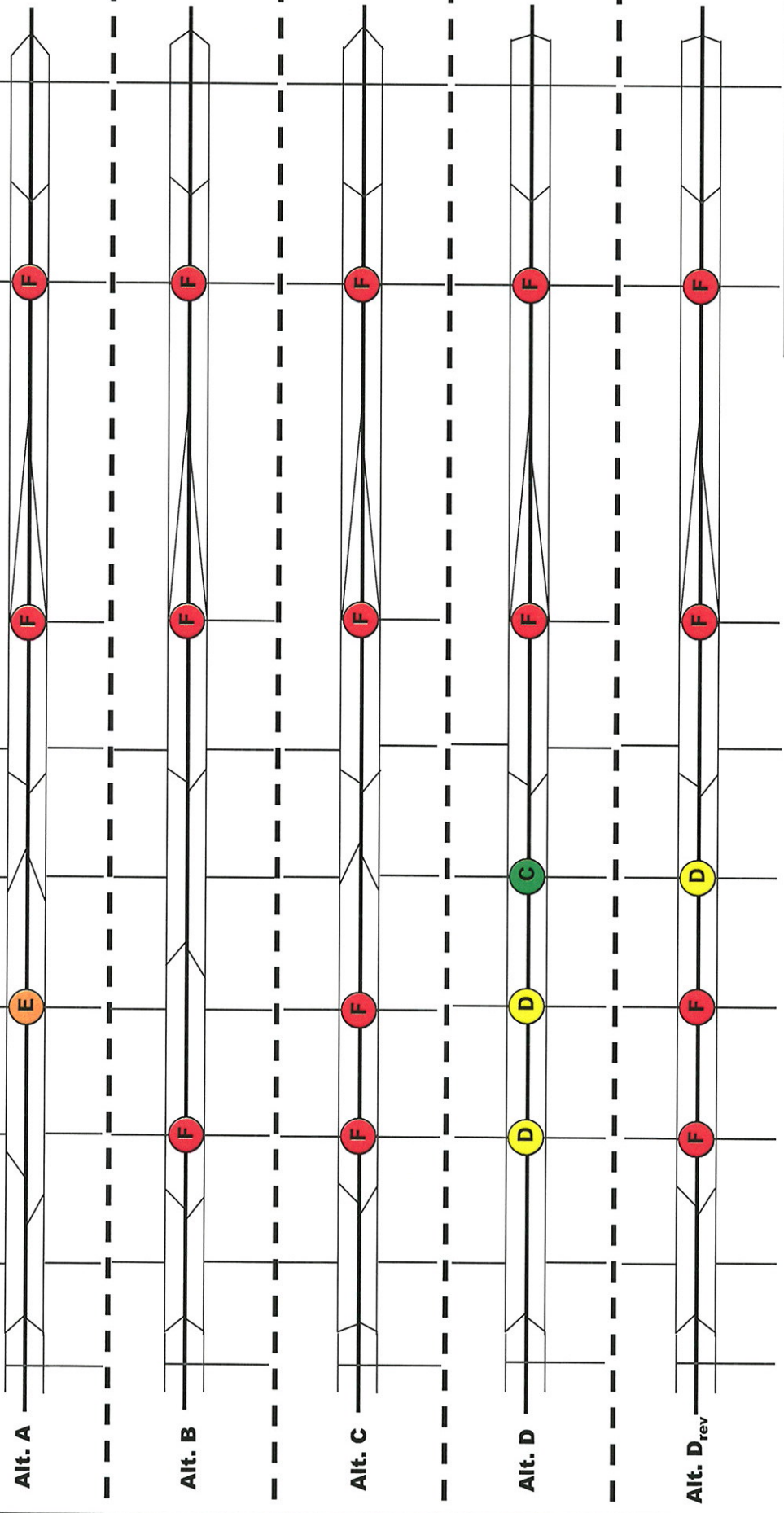
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US 19

Sub Corridor Report
Whitney Rd. - SR 60
Interchange Feasibility Study

2020 TRAFFIC ANALYSIS
AM PEAK LOS RESULTS
FRONTAGE ROAD INTERSECTIONS

Drew St.
SR 60
Seville Blvd
Tropic Hills Dr.
Harn Blvd.
Nursery Rd.
Belleair Rd.
Haines Bayshore
Whitney Rd.

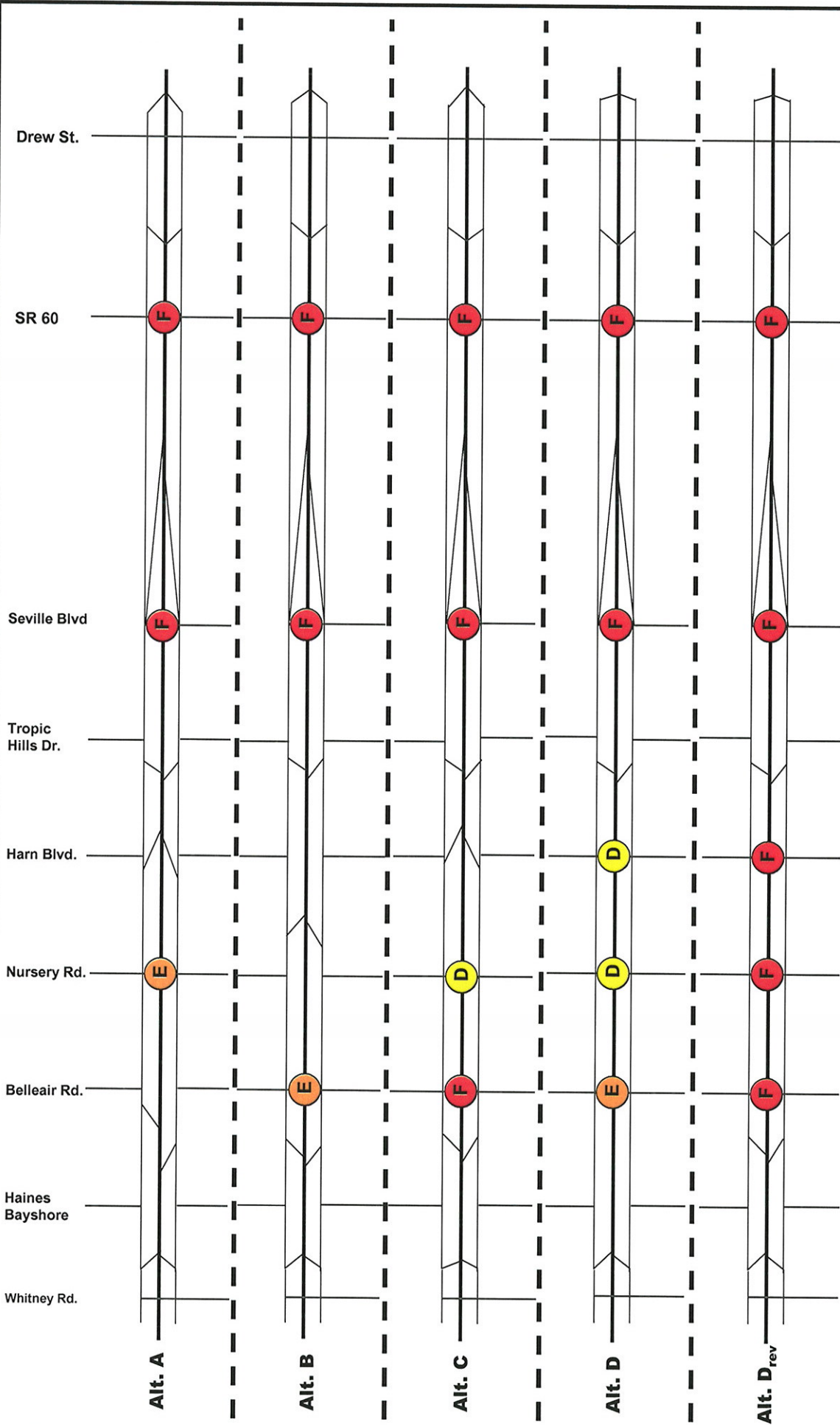


F - Intersection Level of Service

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 Whitney Rd. – SR 60
 Interchange Feasibility Study

2020 TRAFFIC ANALYSIS
 PM PEAK LOS RESULTS
 FRONTAGE ROAD INTERSECTIONS

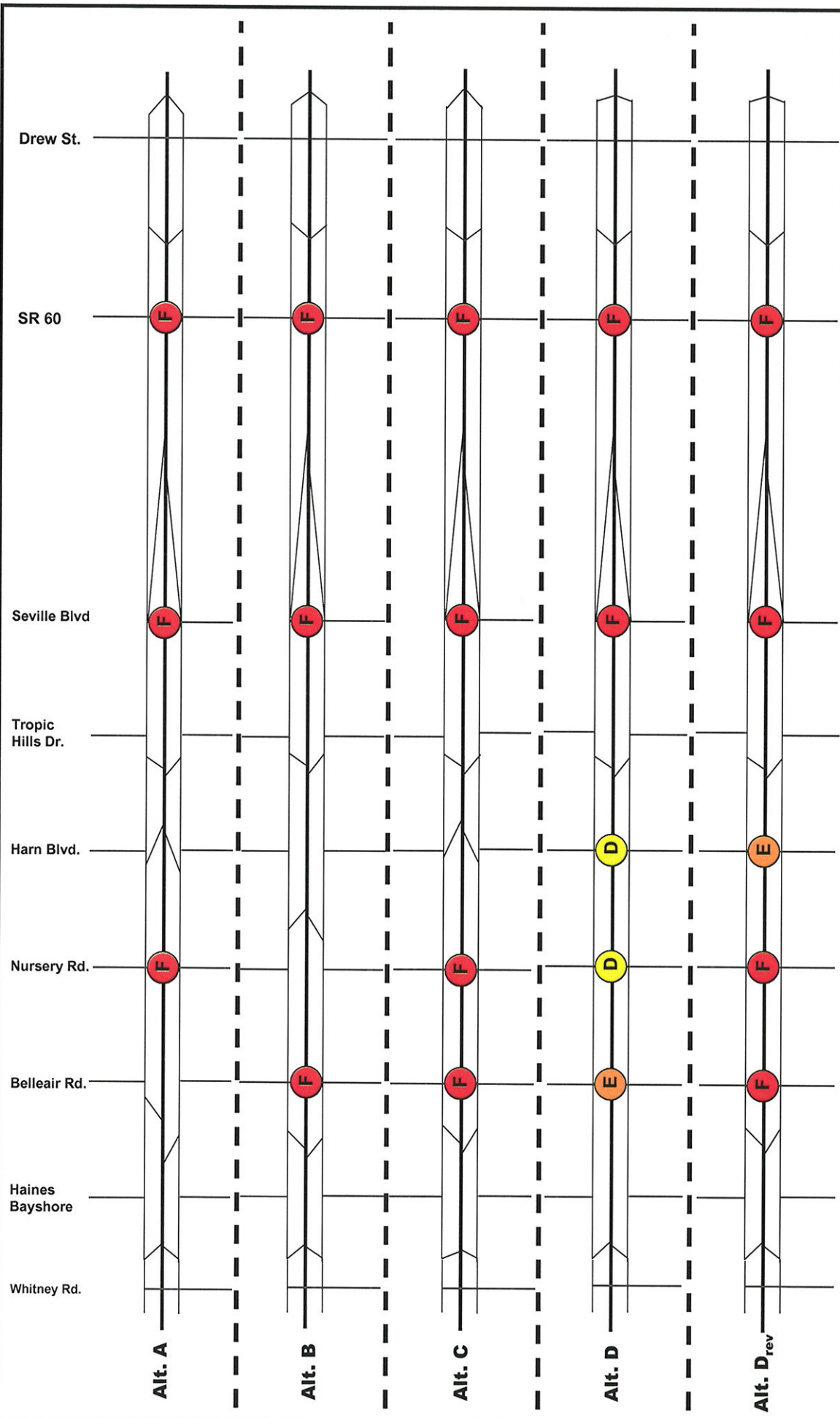


F - Intersection Level of Service

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US 19
Sub Corridor Report
 Whitney Rd. - SR 60
 Interchange Feasibility Study

2030 TRAFFIC ANALYSIS
 AM PEAK LOS RESULTS
 FRONTAGE ROAD INTERSECTIONS

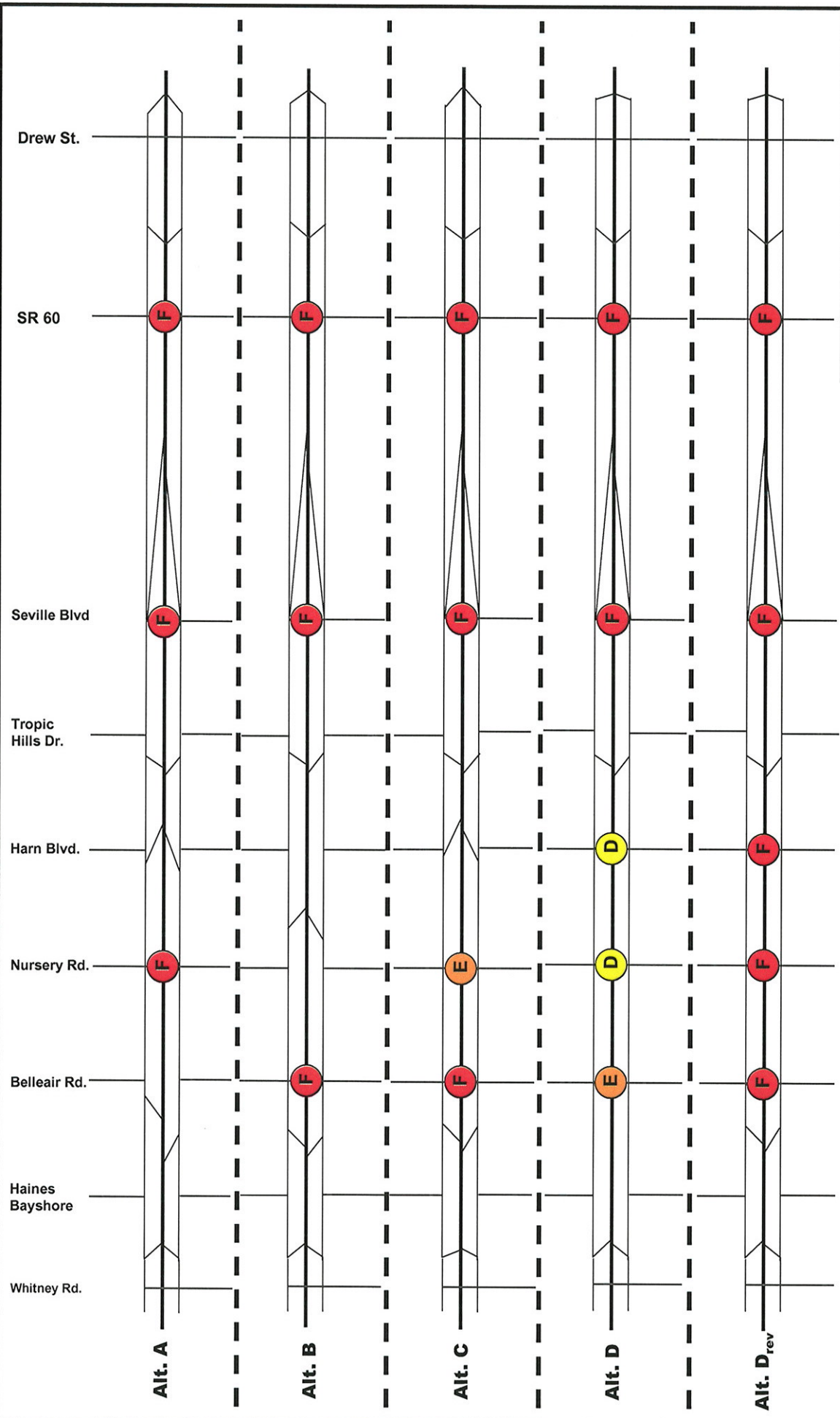


F - Intersection Level of Service

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US 19
Sub Corridor Report
 Whitney Rd. – SR 60
 Interchange Feasibility Study

2030 TRAFFIC ANALYSIS
 PM PEAK LOS RESULTS
 FRONTAGE ROAD INTERSECTIONS



F - Intersection Level of Service

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SUMMARY

The key findings associated with each preliminary conceptual plan along with a brief description and associated construction costs are summarized in Exhibit S-1.

The findings show that:

- Option A has the lowest construction cost with Option B slightly higher and Option D having the highest cost.
- Option C provides slightly better overall traffic circulation than Options A or B while Option D provides improved neighborhood access by providing an additional cross road. Under Option D, mainline access is slightly reduced.
- Options C and D provide the best traffic operations results for mainline and frontage lanes.
- All build options have limited impacts to the Marina sales access (including during frontage lane construction).
- Option B needs minimum cross street approach widening work at Bellair Blvd. whereas Option C needs to widen Nursery Rd. and Option D needs to widen both Nursery Rd. and Harn Blvd.
- Maintenance of Traffic impacts are essentially the same for all Build options.

APPENDICES