Para hacer el análisis operacional de la carretera PR-941, entre la zona urbana y la carretera PR-942 se supuso que esta carretera tendría dos carriles en cada dirección, de 3.65 metros de ancho cada uno, con una isleta o barrera central que separe al tránsito en direcciones contrarias y paseos en ambos lados. Debido a esta geometría y a la utilización que tendrá la intersección de esta carretera con la carretera PR-942, se supuso que a la misma llegarían dos carriles por los tres accesos. Por el oeste llegaría un carril para el viraje a la derecha y un carril para continuar hacia el este. Por el este llegaría un carril para el viraje a la izquierda y otro para continuar hacia el oeste, aunque desde este se permitiría el viraje a la izquierda también. De los dos carriles que llegarían por el sur se supuso que uno será para el viraje a la derecha y el otro para el viraje a la izquierda. El control del tránsito en esta intersección sería mediante un sistema de semáforos mientras que en los carriles para el viraje a la derecha se deberían instalar rótulos de "Ceda". Todos los carriles tendrían un ancho de 3.65 metros excepto los carriles para los movimientos E-S y S-O, los cuales deberían ser de 4.30 metros.

Se supuso que la intersección entre el acceso al proyecto La Aventura y la carretera PR-942 también tendría dos carriles por cada acceso. Desde el proyecto saldría un carril para el viraje a la izquierda y otro para el viraje a la derecha mientras que por el sur y el norte llegaría un carril para virar hacia La Aventura y

otro para continuar el recorrido por la carretera PR-942. Todos los carriles tendrían 3.65 metros de ancho y el tránsito sería controlado por un sistema de semáforos. El tránsito en el carril para el viraje a la derecha desde el norte estaría controlado por un rótulo de "Ceda". Debido a la cercanía entre esta y la anterior intersección y dada la geometría supuesta, se realizó el análisis operacional considerando que la carretera PR-942 entre estas dos intersecciones tendría una geometría similar a la carretera PR-941 entre la zona urbana de Gurabo y la intersección anterior. También se supuso que los sistemas de semáforos de las dos intersecciones estarían coordinados.

El otro cambio en la geometría y el control del tránsito supuesto en este análisis fue en la intersección de las carreteras PR-942 y PR-9942. Aquí se supuso que se llegarían dos carriles por el sur, uno para el viraje a la derecha y otro para continuar el recorrido por la carretera PR-942. Por los otros dos accesos se supuso que continuaría llegando un carril. También se supuso que los carriles que lleguen por el sur tendrían un ancho de 3.65 metros mientras que los otros tendrían un ancho de 4.00 metros. El control del tránsito en la intersección sería mediante un sistema de semáforos, excepto en el carril para el movimiento S-E, el cual se proveería mediante un rótulo de "Ceda".

Estos supuestos se hicieron para poder analizar facilidades que tuviesen suficiente capacidad. Algunos de estos supuestos se

hicieron tomando en consideración la capacidad necesaria a diez años más adelante que el momento de este análisis operacional, o sea, tal vez no todas las características supuestas serán necesarias para el momento en el que se ocupe completamente La Aventura pero sí para el año 2030.

Luego de observar el tránsito vehicular que utilizará la red vial analizada, en el año 2020, se evaluaron los dispositivos de control del tránsito que deben existir en la intersección de la carretera PR-941 y el camino Félix Díaz para asegurar el mejor funcionamiento de la misma. En las otras intersecciones no se hizo esta evaluación debido a que, o el control de tránsito actual es suficiente para manejar adecuadamente la intersección en el futuro o se contempló un cambio en el control del tránsito para que pudiera operar a un Nivel de Servicio adecuado. Para recomendar los dispositivos de control del tránsito a utilizar en la intersección mencionada, antes de realizar el análisis operacional de la intersección, se evaluó utilizando como guías las justificaciones[3] que aparecen en el Manual de Dispositivos Uniformes para el Control del Tránsito (MUTCD, por sus siglas en inglés) (**ATSSA/ITE/AASHTO, 2003**). Estas justificaciones se evaluaron utilizando el programa HCS+ (**University of Florida, 2005**). Al aplicar estas justificaciones a la situación que se

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[3]En este informe se utiliza el término "justificaciones" para definir la palabra en inglés "warrants"

experimentará en la intersección se puede formular una recomendación relacionada a la necesidad de un sistema de semáforos en el lugar. El detalle de las justificaciones aparece en el Apéndice 8 de este informe. Al aplicar la justificación relacionada al tránsito durante la hora pico, que es la única que se puede evaluar en este caso, se obtuvo que no será necesaria la instalación de un sistema de semáforos en la intersección indicada. El nombre de los archivos de esta evaluación aparece en el Apéndice 4 de este informe y los cómputos en el Apéndice 5 y en el discompacto que se acompaña.

En la tabla 6 se resumen los resultados del análisis operacional del sistema vial para las horas pico durante un día laborable típico, para el año 2020. El detalle de estos resultados se resume en la figura 20. El nombre de los archivos donde se almacenan los cómputos para el análisis operacional se resume en el Apéndice 4. Los cómputos aparecen en el Apéndice 5 y en el discompacto que se incluye con el informe.

Se obtuvo que, al momento en el que La Aventura se ocupe, la red vial analizada debe operar satisfactoriamente. Esto, tomando en consideración los cambios en la geometría y control de tránsito supuestos, descritos anteriormente. Por lo tanto, estos cambios son necesarios para el buen funcionamiento del sistema vial analizado. El análisis de la carretera PR-942, al sur de la carretera PR-9942, se realizó suponiendo la geometría actual. De

Tabla 6. Nivel de Servicio al que operarán, en el año 2020, durante las horas pico, en un día laborable típico, las facilidades analizadas en este estudio

Intersección	Acceso Oeste	Acceso Este	Acceso Sur	Acceso Norte	Total
Hora Pico AM					
PR-941/ PR-9941		B(13.9)		A(8.2)	
PR-941/Félix Díaz	B(14.8)		A(8.5)		
PR-941, sur de PR-942	Hacia Gurabo: B(9.4), hacia Jaguas: A(1.5)				
PR-941/PR-942	D(52.2)	D(37.9)	D(53.8)		D(40.0)
PR-942/La Aventura	A(8.4)		D(45.8)	D(37.3)	B(18.7)
PR-942, sur de PR-9942	C(58.6)				
PR-942/PR-9942		A(7.1)	B(16.5)	B(18.2)	B(10.3)
PR-942/PR-9943	A(8.7)		A(7.4)		
PR-942/PR-9941		A(7.4)	A(8.7)		
Hora Pico PM					
PR-941/PR-9941		C(17.9)		A(8.3)	
PR-941/Félix Díaz	C(17.3)		A(9.2)		
PR-941, sur de PR-942	Hacia Gurabo: A(5.0), hacia Jaguas: A(6.6)				
PR-941/PR-942	E(64.4)	C(29.8)	D(36.6)		D(35.9)
PR-942/La Aventura	C(27.3)		B(11.1)	A(5.6)	B(16.4)
PR-942, sur de PR-9942	D(77.9)				
PR-942/PR-9942		C(20.1)	D(39.2)	E(63.8)	C(26.1)
PR-942/PR-9943	A(8.8)		A(7.5)		
PR-941/PR-9941		A(7.3)	A(8.6)		

Notas:

1. Los valores de las variables se ilustran entre paréntesis. En intersecciones, demoras por control, en segundos por vehículo. En segmentos de carretera de dos carriles, el porciento del tiempo del recorrido en el que un vehículo sigue a otro. En carreteras de carriles múltiples, densidad, en vehículos de pasajeros por kilómetro por carril.
2. Según el procedimiento utilizado, en las intersecciones sin semáforo no se determina el LOS para la intersección completa.

acuerdo a los resultados se obtuvo que la misma se puede mantener como una vía con un carril en cada dirección y funcionará adecuadamente.

Al efectuar el análisis operacional para el escenario correspondiente al año 2030 se obtuvieron los resultados resumidos en la tabla 7. Este análisis es requerido por las *Guías para la Preparación de Estudios Operacionales de Accesos y de Tránsito* (**ELA, 2004**) para proyectos de gran magnitud, para tener una idea de como se comportará la red vial analizada a largo plazo. Los datos de tránsito utilizados para este análisis se obtuvieron proyectando los viajes en la red vial esperados durante las horas pico para el año 2020 hasta el año 2030 utilizando la ley de interés compuesto, tal y como se explica en el Apéndice 7. Los datos de tránsito utilizados se ilustran en la figura 21. Según los resultados obtenidos, las facilidades viales analizadas podrán manejar el flujo vehicular que las utiliza ya que las demoras que sufrirán los usuarios serán tolerables. Se observa que los casos más críticos serán las intersecciones de la carretera PR-942 con las carreteras PR-941 y PR-9942 durante la hora pico de la tarde. En esos momentos estas facilidades operarán a capacidad. Durante el resto del día los usuarios experimentarán demoras menores. Los resultados obtenidos para este escenario se resumen en la figura 22.

Tabla 7. Nivel de Servicio al que operarán, en el año 2030, durante las horas pico, en un día laborable típico, las facilidades analizadas en este estudio

Intersección	Acceso Oeste	Acceso Este	Acceso Sur	Acceso Norte	Total
Hora Pico AM					
PR-941/ PR-9941		C(16.3)		A(8.5)	
PR-941/Félix Díaz	C(19.6)		A(8.9)		
PR-941, sur de PR-942	Hacia Gurabo: C(11.5), hacia Jaguas: A(1.8)				
PR-941/PR-942	E(69.2)	C(23.3)	E(71.8)		C(29.4)
PR-942/La Aventura	B(15.3)		D(49.0)	D(38.2)	C(25.4)
PR-942, sur de PR-9942	C(63.6)				
PR-942/PR-9942		A(6.8)	B(18.6)	C(21.3)	B(10.9)
PR-942/PR-9943	A(8.8)		A(7.4)		
PR-942/PR-9941		A(7.4)	A(8.8)		
Hora Pico PM					
PR-941/PR-9941		C(23.4)		A(8.6)	
PR-941/Félix Díaz	C(22.7)		B(10.2)		
PR-941, sur de PR-942	Hacia Gurabo: A(6.1), hacia Jaguas: B(8.1)				
PR-941/PR-942	A(9.8)	E(59.0)	E(79.9)		E(61.9)
PR-942/La Aventura	D(49.1)		D(43.9)	A(6.7)	D(41.7)
PR-942, sur de PR-9942	D(84.1)				
PR-942/PR-9942		E(78.5)	D(48.7)	E(77.7)	E(75.5)
PR-942/PR-9943	A(8.9)		A(7.6)		
PR-941/PR-9941		A(7.4)	A(8.7)		

Notas:

1. Los valores de las variables se ilustran entre paréntesis. En intersecciones, demoras por control, en segundos por vehículo. En segmentos de carretera de dos carriles, el porciento del tiempo del recorrido en el que un vehículo sigue a otro. En carreteras de carriles múltiples, densidad, en vehículos de pasajeros por kilómetro por carril.
2. Según el procedimiento utilizado, en las intersecciones sin semáforo no se determina el LOS para la intersección completa.

De acuerdo a los resultados obtenidos, las facilidades viales, con los ajustes en geometría y control sugeridos, podrán manejar el tránsito que utilizará las vías impactadas por La Aventura tanto desde el momento en el que se ocupe el proyecto como diez años después. Los cómputos para efectuar los análisis operacionales aparecen en el Apéndice 5 y en el discompacto que se acompaña con el informe.

Conclusiones

El proyecto La Aventura proveerá al municipio de Gurabo con 2000 nuevas unidades de vivienda, un centro comercial y una nueva escuela. Existiendo una creciente demanda de vivienda en el área, procedente del área metropolitana de San Juan, el proyecto ayudará a que se densifique la región, produciendo beneficios para la economía de Gurabo. Las residencias se unirán a otros proyectos residenciales propuestos para el área, tales como Alborada del Río, Valle Encantado, La Esmeralda, Vista Bella de Gurabo y Palacios de Gurabo, densificando el uso de los terrenos en el sector. La red de carreteras del sector, compuesta por las vías PR-941, PR-942, PR-9941, PR-9942, PR-9943 y el camino Félix Díaz podrán manejar el tránsito que las utilizará, tal y como lo hacen actualmente, si se construyen mejoras geométricas en algunos de sus componentes y se cambia el control del tránsito en otros.

De acuerdo con los resultados obtenidos, la geometría de la carretera PR-941 entre la zona urbana de Gurabo y la carretera PR-942, la geometría de esta última vía entre la primer vía y el acceso a La Aventura, y la geometría de las intersecciones de la carretera PR-941 con el camino Félix Díaz y con la carretera PR-942 así como la geometría de la intersección de esta última carretera con la carretera PR-9942 no tendrán suficiente capacidad para manejar el tránsito futuro, una vez se ocupen los proyectos propuestos para el área. Los resultados indican que estos

componentes del sistema vial analizado no tienen problemas de capacidad actualmente pero si los tendrán a mediano y largo plazo si no se hacen los ajustes pertinentes.

De acuerdo a los resultados obtenidos se observó que las vías analizadas registran un patrón de flujo vehicular durante un día laborable típico de acuerdo a su función y al lugar donde se encuentran. El proyecto La Aventura está cónsono con el uso principal del área, el residencial, por lo que su construcción no debe cambiar los patrones vehiculares que se registran. Esta consistencia en el comportamiento del tránsito será en beneficio de la operación de la carretera ya que los niveles del flujo vehicular se mantendrán dentro de los niveles aceptables, permitiendo que el sistema vial aquí analizado opere de forma satisfactoria por los próximos años, una vez se le hagan los ajustes pertinentes. Los cambios serán suficientes para manejar sin problemas el tránsito que utilizará la red vial analizada a mediano y largo plazo. Debido a que se proponen otros proyectos de gran impacto en el sector, los desarrolladores de La Aventura deberán compartir con los dueños de los otros proyectos la inversión que haya que realizar en las mejoras detalladas, mientras que deberán costear completamente la intersección de acceso a su proyecto.

Recomendaciones

El proyecto La Aventura traerá una cantidad adicional de vehículos al área analizada que será manejada adecuadamente por las carreteras PR-941, PR-942, PR-9941, PR-9942, PR-9943 y el camino Félix Díaz si se hacen los ajustes necesarios en la geometría y control del tránsito de algunos de los componentes. Entre las modificaciones al sistema vial que se tienen que realizar está construir una isleta central en la carretera PR-941, en su intersección con el camino Félix Díaz, que separe el tránsito en ambas direcciones, antes y después del camino. Se deberá proveer un carril para el viraje a la izquierda para los vehículos que llegan a la intersección por el sur, además de mantener el carril para continuar por la carretera PR-941, así como proveer dos carriles, uno para el viraje a la izquierda y el otro para el viraje a la derecha, en la llegada a la intersección por el camino Félix Díaz. El control del tránsito en esta intersección continuará siendo un rótulo de "Pare" para los vehículos que llegan a la misma por el camino Félix Díaz.

La carretera PR-941, entre la zona urbana y la carretera PR-942 deberá tener dos carriles en cada dirección, de 3.65 metros de ancho cada uno, con una isleta o barrera central que separe al tránsito en direcciones contrarias y paseos en ambos lados. Esta mejora se había requerido al proyecto Valle Encantado luego de su evaluación de tránsito. Se recomienda la misma geometría para el

segmento de la carretera PR-942, entre la carretera PR-941 y el acceso al proyecto La Aventura.

La intersección las carreteras PR-941 y PR-942 deberá tener dos carriles por los tres accesos. Por el oeste deberá tener un carril para el viraje a la derecha y un carril para continuar hacia el este. Por el este se debe proveer un carril para el viraje a la izquierda y otro para continuar hacia el oeste, aunque desde este se deberá permitir el viraje a la izquierda también. De los dos carriles que deberán llegar por el sur, uno deberá ser para el viraje a la derecha y el otro para el viraje a la izquierda. Se recomienda que el control del tránsito en esta intersección sea mediante un sistema de semáforos actuado, con detectores en todos los accesos, excepto en los carriles para los movimientos O-S y S-E, donde se deberán instalar rótulos de "Ceda". Todos los carriles deberán ser de 3.65 metros de ancho excepto los carriles para los movimientos E-S y S-O, los cuales deberán tener 4.30 metros de ancho.

También se recomienda que la intersección entre el acceso al proyecto La Aventura y la carretera PR-942 tenga dos carriles por cada acceso. Desde el proyecto deberá salir un carril para el viraje a la izquierda y otro para el viraje a la derecha mientras que por el sur y por el norte deberá llegar un carril para virar hacia La Aventura y otro para continuar el recorrido por la carretera PR-942. Se recomienda que todos los carriles tengan 3.65

metros de ancho y que el tránsito sea controlado por un sistema de semáforos actuado, con detectores en todos los accesos, excepto el carril para el movimiento N-O. El tránsito en este carril deberá estar controlado por un rótulo de "Ceda". Debido a la cercanía entre esta y la anterior intersección y dada la geometría recomendada, los sistemas de semáforos de las dos intersecciones deberán estar coordinados.

El otro cambio recomendado para la geometría y el control del tránsito es en la intersección de las carreteras PR-942 y PR-9942. Se recomienda que lleguen dos carriles por el sur, uno para el viraje a la derecha y otro para continuar el recorrido por la carretera PR-942. Por los otros dos accesos se puede mantener un carril. Se recomienda que los carriles que lleguen por el sur tengan un ancho de 3.65 metros mientras que los otros deberán tener un ancho de 4.00 metros cada uno. El control del tránsito en la intersección deberá ser mediante un sistema de semáforos actuado, con detectores en todos los accesos, excepto en el carril para el movimiento S-E. En este último carril se deberá controlar el tránsito mediante un rótulo de "Ceda".

Los demás componentes del sistema vial analizado pueden mantenerse con las características actuales, ya que tienen la capacidad suficiente para poder manejar el tránsito que las utilizará en el futuro. Los criterios que se deben tomar en consideración al efectuar el diseño geométrico y de los

dispositivos de control del tránsito, de acuerdo a lo aquí recomendado, aparecen en el Reglamento para el Control de Accesos a las Vías Públicas de Puerto Rico (**ELA, 1976**) y demás guías vigentes (**AASHTO, 2004**) (**AATSA/ITE/AASHTO, 2003**) (**ACT, 1996**). En la figura 23 se ilustran, de forma esquemática, las mejoras geométricas aquí propuestas. Las fases y tiempos propuestos para los semáforos se ilustran en la figura 24. Los desarrolladores de todos los proyectos propuestos para el sector analizado deberán aportar para poder construir las mejoras recomendadas. Además, el desarrollador del proyecto La Aventura deberá hacerse cargo de la intersección del acceso a su proyecto con la carretera PR-942. Con las recomendaciones aquí resumidas, las facilidades viales aquí analizadas deberán operar bien a mediano y largo plazo, tomando en consideración la construcción del proyecto residencial, comercial e institucional La Aventura. A corto plazo la red vial analizada funciona adecuadamente.

Varias entidades, incluyendo el Gobierno Municipal de Gurabo, han propuesto la extensión de la carretera PR-203 desde su intersección con la carretera PR-30 (Expreso Cruz Ortiz Stella) hasta la intersección de las carreteras PR-941 y PR-942. De hecho, esta vía se está incluyendo como parte del programa del Plan de Ordenamiento Territorial del municipio. La justificación para la programación de esta vía es que el área aquí analizada se desarrollará grandemente en los próximos años. Como la mayoría de

las personas que residirán en estos nuevos proyectos vendrán de otros municipios y tendrán su rutina de vida formada en otros lugares, esta carretera les permitiría continuar con la misma sin mayores dificultades ya que podrían comunicarse desde sus residencias con su entorno diario de forma rápida y directa por medio de las carreteras PR-203 nueva y PR-30. En el Apéndice 9 se incluye copia de algunas de las cartas que se han cursado como parte de las gestiones que se han hecho para lograr incluir la extensión de la carretera PR-203 en el Plan Vial de la ACT, lo que aumentará las posibilidades de construcción de la misma.

De construirse la extensión de la carretera PR-203 no será necesaria la ampliación a cuatro carriles de la carretera PR-941 desde la zona urbana de Gurabo hasta su intersección con la carretera PR-942 ya que el flujo vehicular que hace esta ampliación necesaria utilizará, en su gran mayoría, la extensión de la carretera PR-203. Además, la geometría actual de la carretera PR-203 es similar a la que aquí se recomienda para la ampliación de la carretera PR-941, por lo que la extensión de la primer carretera deberá tener la misma geometría, la que será suficiente para manejar el tránsito que la utilizará, que será el que se genere en el sector aquí analizado.

La extensión de la carretera PR-203 hasta el área analizada traerá una serie de beneficios al compararlo con la expansión de la carretera PR-941 a cuatro carriles. Por ejemplo, la distancia a

recorrerse desde el expreso Cruz Ortiz Stella hasta el área estudiada disminuirá significativamente. Actualmente, desde la intersección de las carreteras PR-30 y PR-203 hasta la intersección de las carreteras PR-941 y PR-942 el automovilista tiene que recorrer unos 5.8 kilómetros, los cuales no disminuirán con la ampliación a cuatro carriles de la vía PR-941. Esta distancia disminuiría a 2.7 kilómetros si se construye la extensión de la carretera PR-203. Esto representará una disminución en tiempo de un 115%, tomando en cuenta los límites de velocidad existente, los cuales apenas deben variar. En este estimado tampoco se tomó en cuenta las demoras que continuarían experimentando los usuarios en las intersecciones de la carretera PR-9944 con las carreteras PR-30, PR-189 y PR-941, las cuales el conductor no tendría que utilizar si se extiende la carretera PR-203 hasta el área analizada. Al analizar esto se debe tener en cuenta que se espera que gran parte de los residentes y visitantes de los proyectos que se proponen para el lugar analizado realicen su vida cotidiana al oeste de la intersección de las carreteras PR-30 y PR-203. Por lo tanto, la extensión de esta última vía representará una economía significativa en tiempo, gasolina, lubricantes y mantenimiento de los vehículos para los futuros residentes y visitantes del sector. También significará una menor utilización de las vías ubicadas al oeste de la zona urbana de Gurabo.

También se debe considerar que prácticamente todo el recorrido de la carretera PR-941, entre la zona urbana de Gurabo y el sector analizado está ubicado en un área inundable, por lo que ampliar esta carretera a cuatro carriles también conllevaría realizar las obras necesarias para que la misma no se inunde, incluyendo aumentar su elevación. Esto también traería otras situaciones como por ejemplo, se tendrían que expropiar estructuras aledañas a la carretera PR-941 y realizar las mejoras de forma que no afecte negativamente a los vecinos y estructuras existentes a los lados de la carretera. También su reconstrucción se deberá efectuar con la carretera en operaciones ya que es el acceso a un gran número de comunidades. Estos factores harán el proyecto uno sumamente costoso. Parte de la extensión de la carretera PR-203 cruzaría el valle inundable del Río Gurabo, pero por ser construcción nueva y no existir desarrollos en lo que serían sus orillas, sería más sencillo tratar la situación, haciendo este proyecto uno menos costoso que la ampliación a cuatro carriles de la carretera PR-941.

La construcción de la extensión de la carretera PR-203 tiene múltiples beneficios sobre la expansión de la carretera PR-941 a cuatro carriles. Además incentivará el desarrollo propuesto para el sector ya que los proponentes tendrán una vía más eficiente, tanto desde el punto de vista práctico como desde el punto de vista económico, para accesar al lugar, lo que hará sus proyectos más atractivos.

Certificación

Este informe fue preparado basado en las condiciones existentes al momento del trabajo de campo y otras condiciones citadas en las referencias aquí indicadas e información provista por el proponente. Los diseñadores del proyecto deben verificar que ninguna condición haya sido pasada por alto, requiriéndose mayor clarificación y/o comentarios adicionales. Este documento fue preparado por Pérez Berenguer y Asociados y aplica únicamente al proyecto La Aventura (Caso 2001-47-0705-JPU en la Junta de Planificación de Puerto Rico), propuesto por Integral Development Strategists, para la intersección de las carreteras PR-941 y PR-942, barrios Jaguas y Celada, Gurabo, Puerto Rico. Los datos, análisis, discusiones, conclusiones y recomendaciones aquí formuladas no aplican a ningún otro proyecto.

Certifico Correcto

José L. Pérez Berenguer, MSCE, PE, PTOE, SIT

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Apéndices

Apéndice 1. Figuras

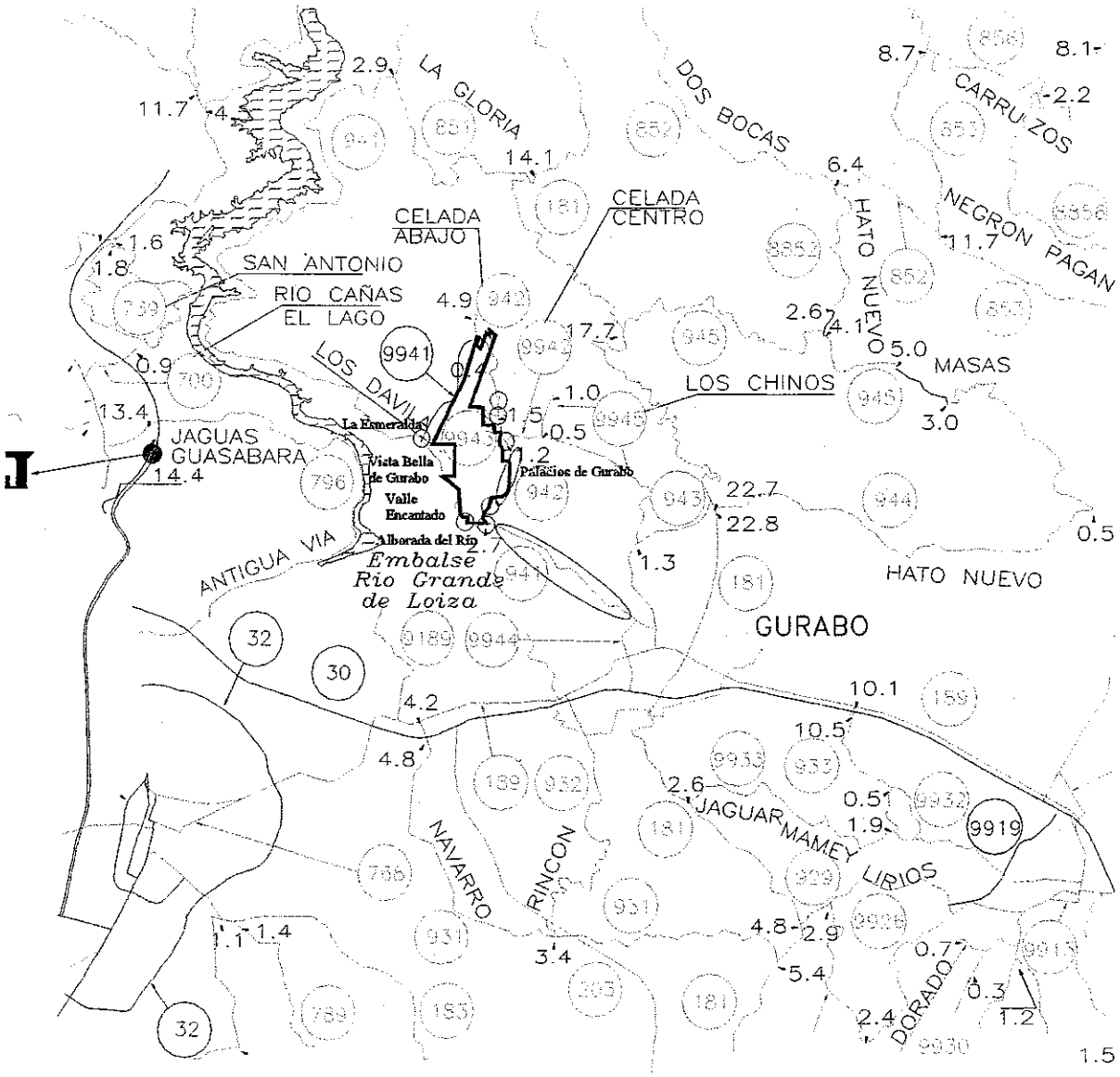

Figura 1. Mapa de carreteras del área. La finca se ilustra en rojo mientras que los componentes de la red vial evaluados se circulan en azul. Se indica la localización aproximada de los otros proyectos propuestos para el área. No está a escala

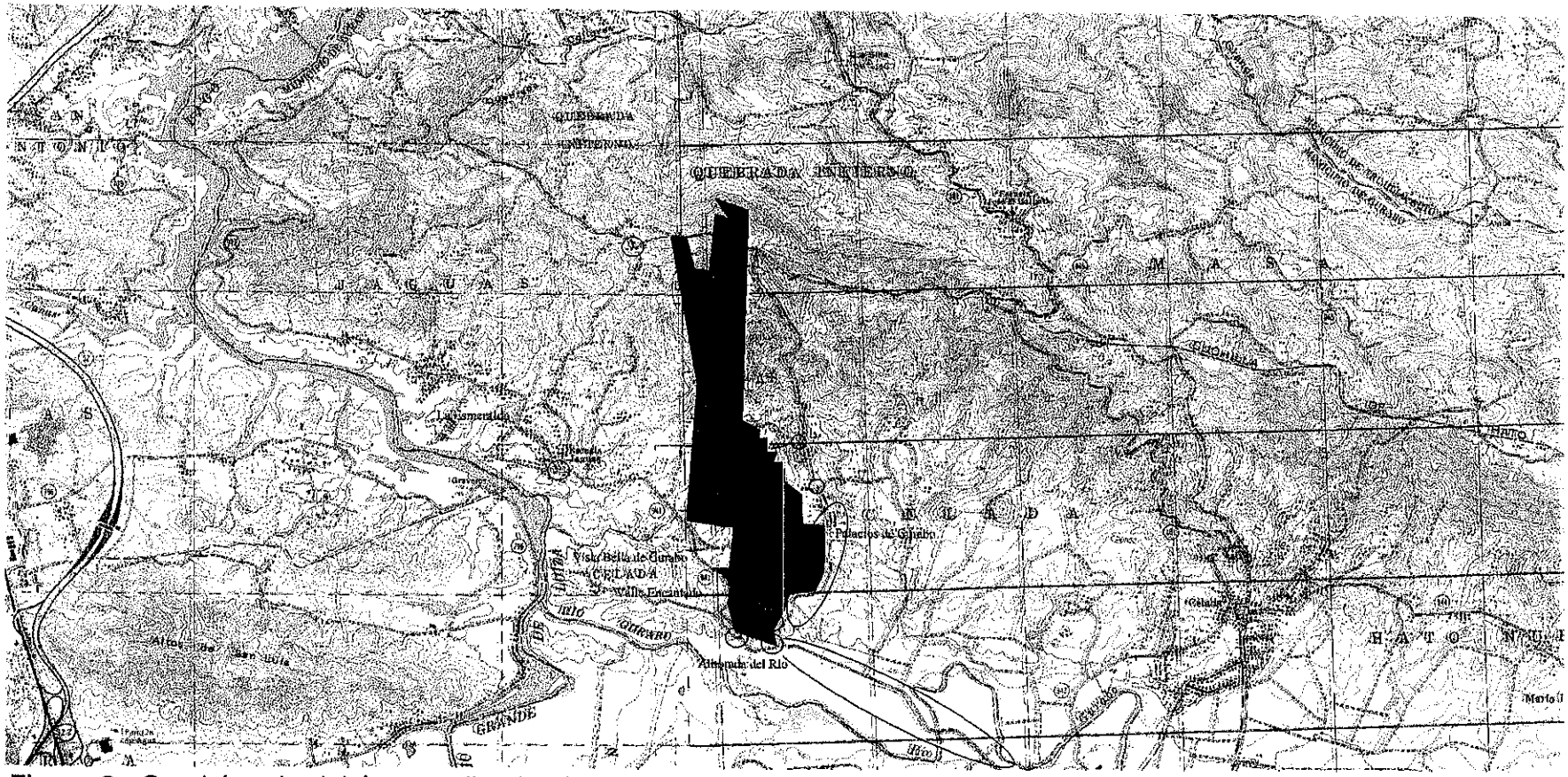

Figura 2. Cuadrángulo del área analizada. Los componentes analizados del sistema vial se ilustran en azul. La finca donde se propone el proyecto se ilustra en rojo. También se indica la localización aproximada de los otros proyectos propuestos en el sector

Figura 3. Intersección de las carreteras PR-941 y PR-9941, vista desde el este

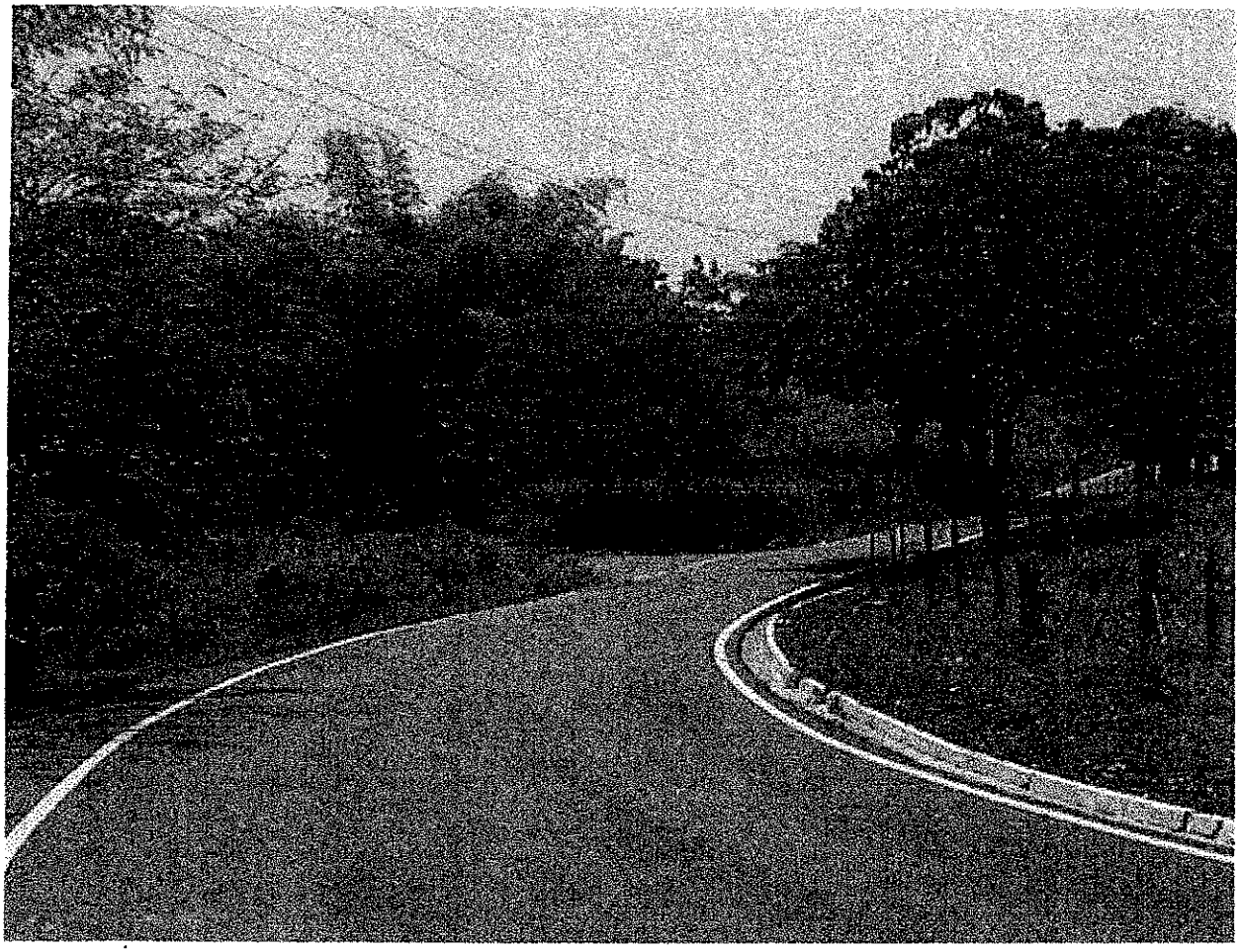

Figura 4. Intersección de la carretera PR-941 y el camino Félix Díaz, vista desde el sur

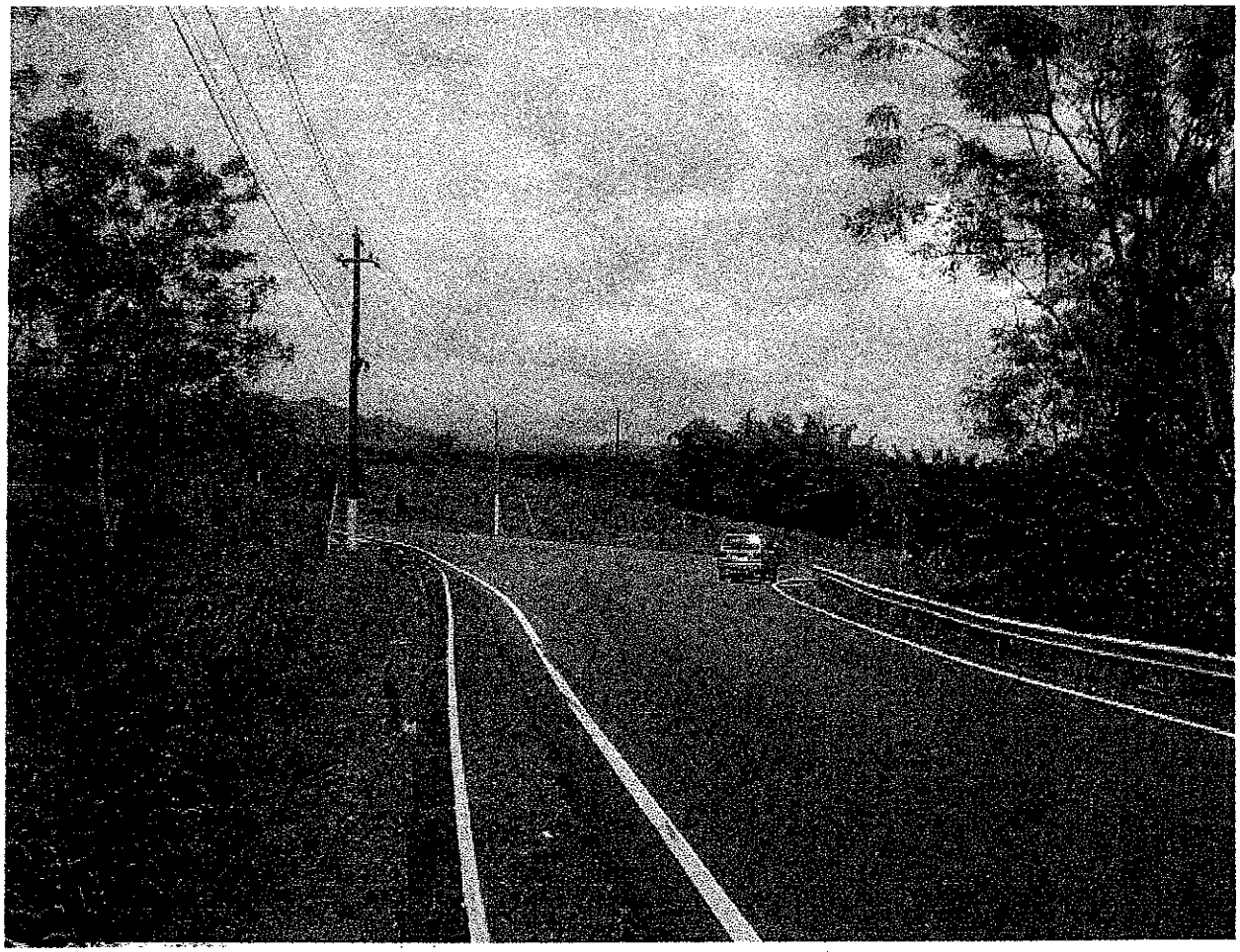

Figura 5. Intersección de las carreteras PR-941 y PR-942, vista desde el oeste

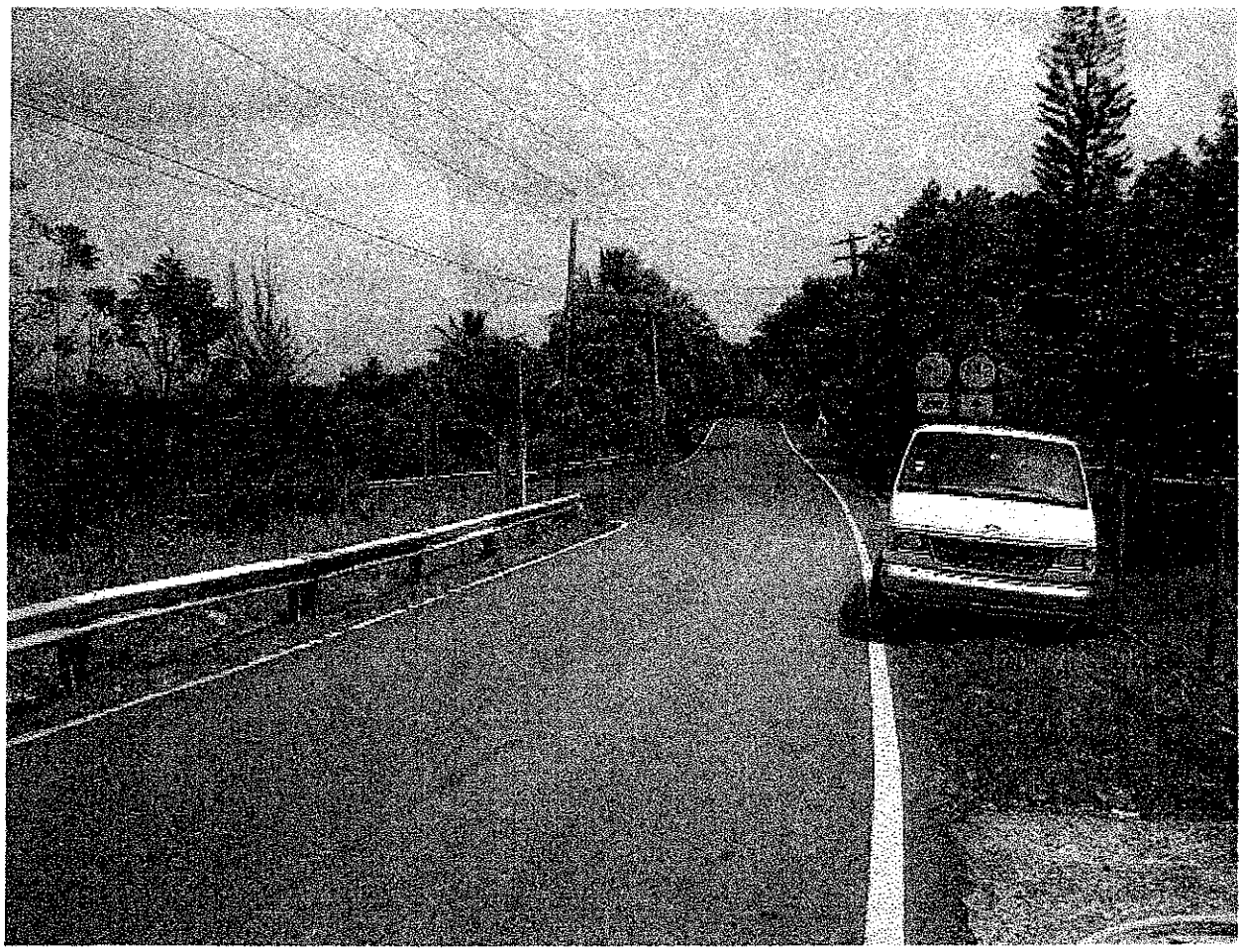

Figura 6. Intersección de las carreteras PR-942 y PR-9942, vista desde el norte

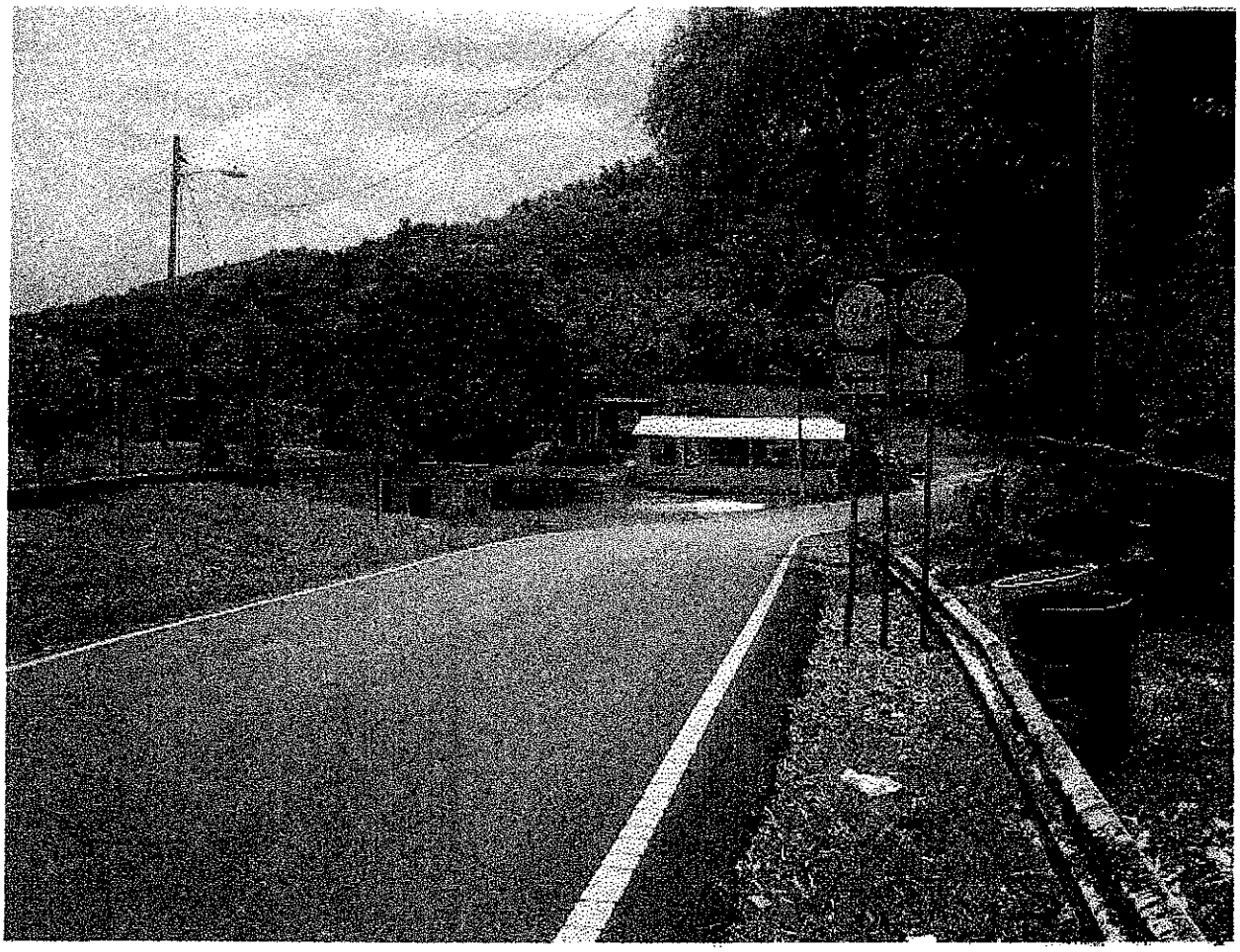

Figura 7. Intersección de las carreteras PR-942 y PR-9943, vista desde el sur

Figura 8. Intersección de las carreteras PR-942 y PR-9941, vista desde el oeste

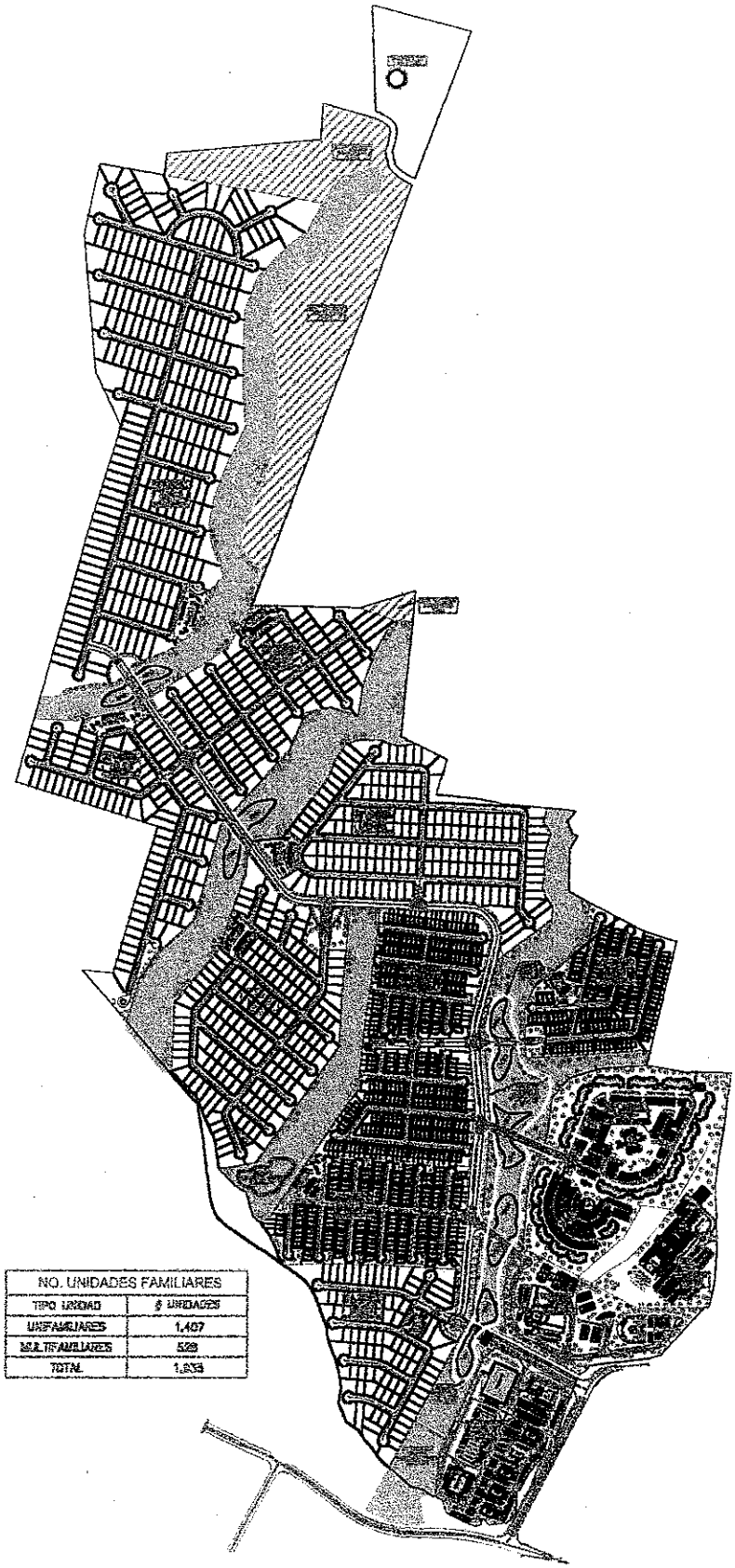

Figura 9. Planta del Proyecto Propuesto

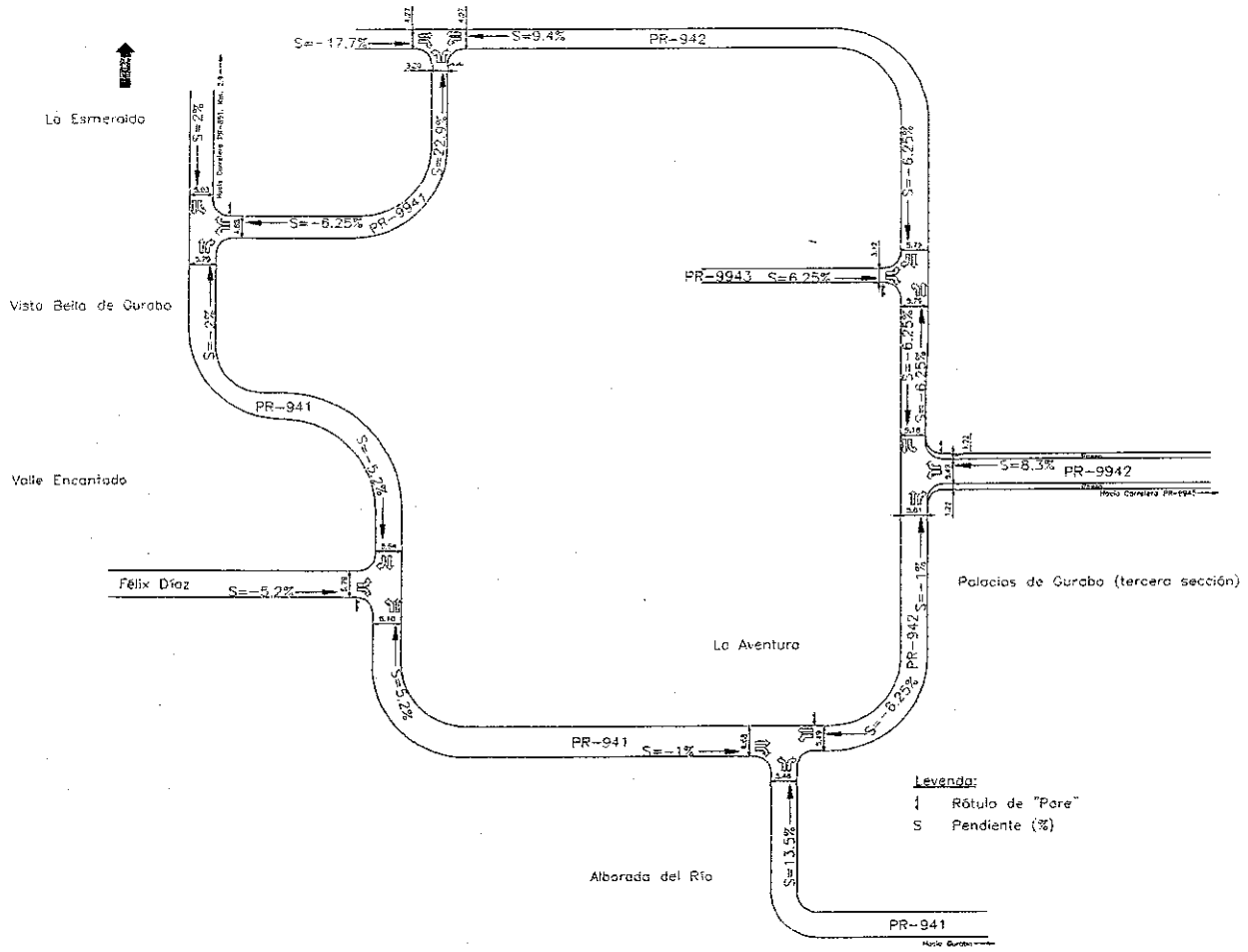

Figura 10. Esquema ilustrando la geometría de la red vial y los movimientos analizados en este estudio. El límite de velocidad en las carreteras analizadas es 25 mph excepto en la carretera PR-941, donde es 40 mph. No existen paradas de autobuses ni se permite el estacionamiento en el área ilustrada. Las dimensiones se expresan en metros. Dibujo no está a escala

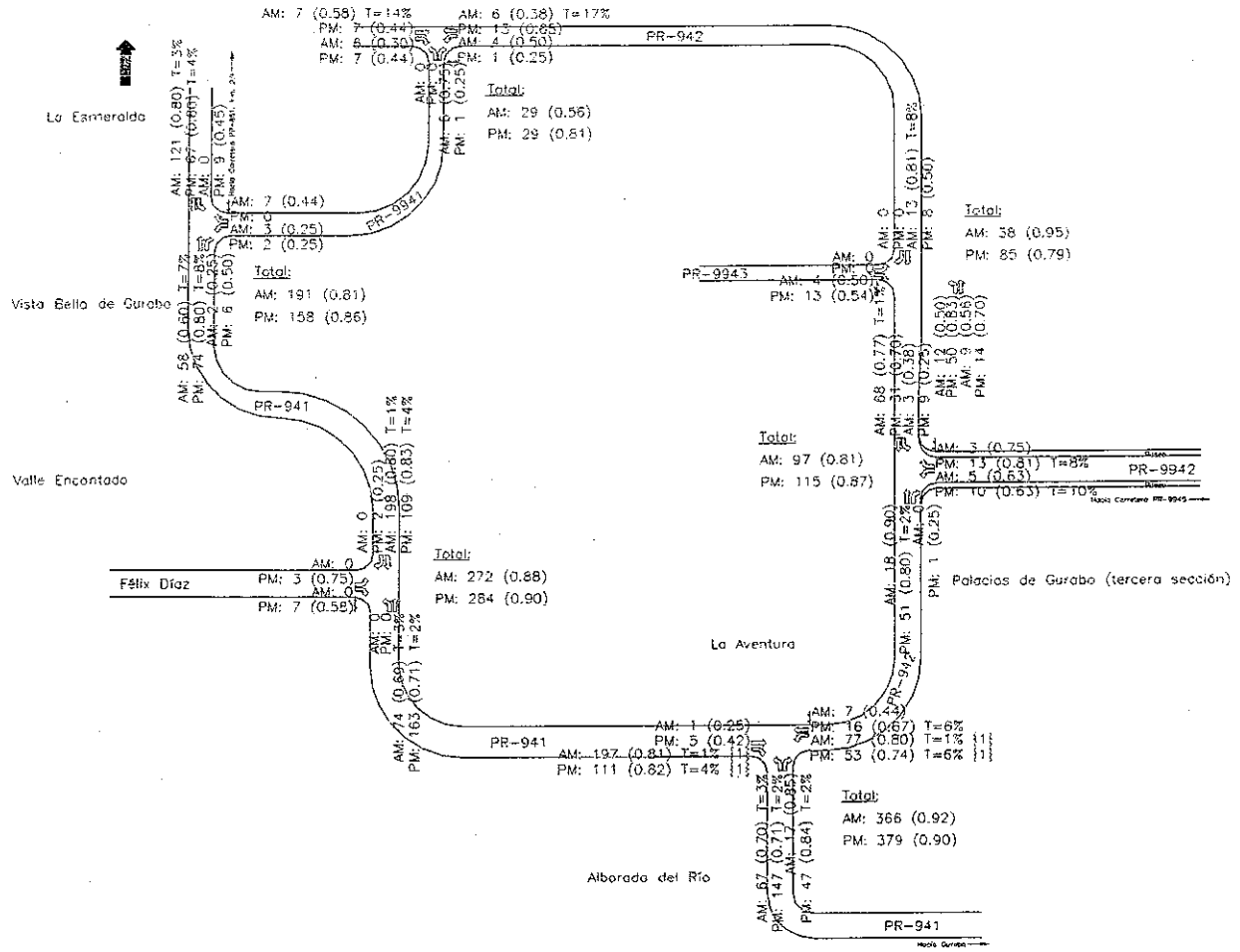

Figura 11. Flujo vehicular y Factor de la Hora Pico actual, por movimiento y total, durante las horas críticas de un día laborable típico, en las intersecciones analizadas en este estudio. También se ilustra el porciento de camiones y la longitud de cola, por movimiento, para los mismos períodos. El flujo vehicular se expresa en vehículos por hora. El Factor de la Hora Pico se ilustra entre paréntesis. La longitud de cola se expresa en vehículos y se indica entre corchetes. En los movimientos donde no se indica el valor de alguna de estas variables, su valor es cero

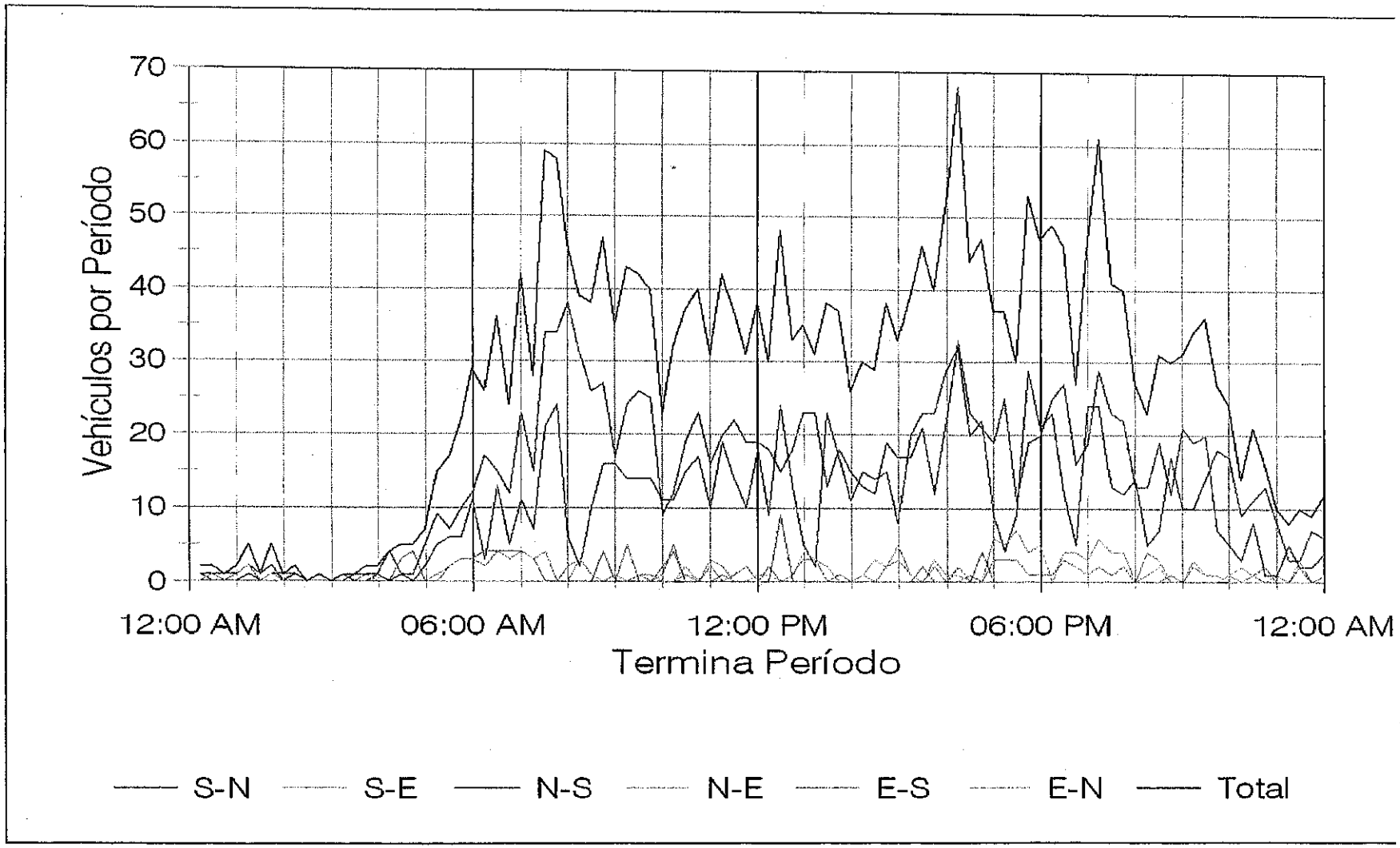

Figura 12. Patrón actual del flujo vehicular, por movimiento y total, en la intersección de las carreteras PR-941 y PR-9941, durante un día laborable típico

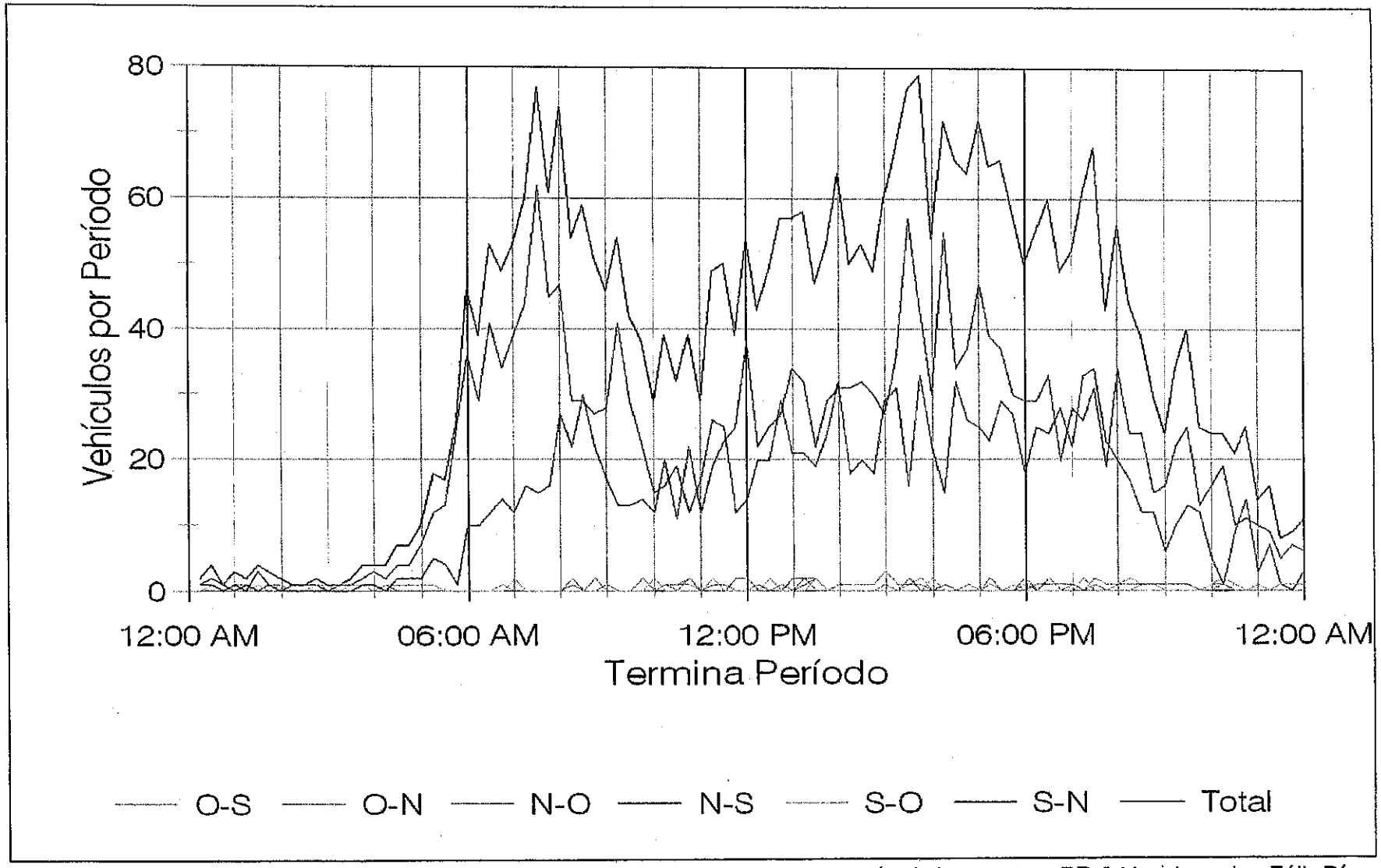

Figura 13. Patrón actual del flujo vehicular, por movimiento y total, en la intersección de la carretera PR-941 y el camino Félix Díaz, durante un día laborable típico

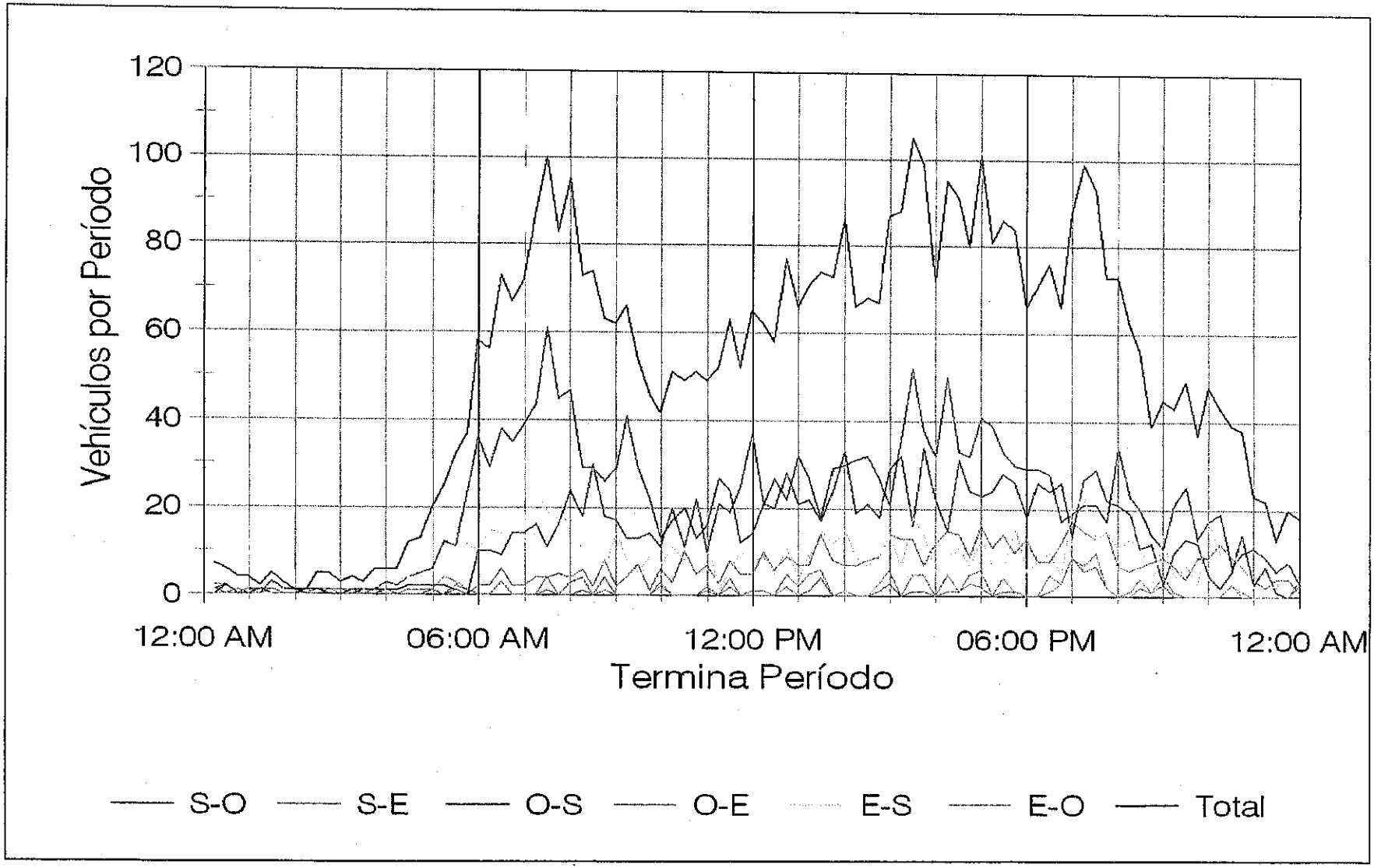

Figura 14. Patrón actual del flujo vehicular, por movimiento y total, en la intersección de las carreteras PR-941 y PR-942, durante un día laborable típico

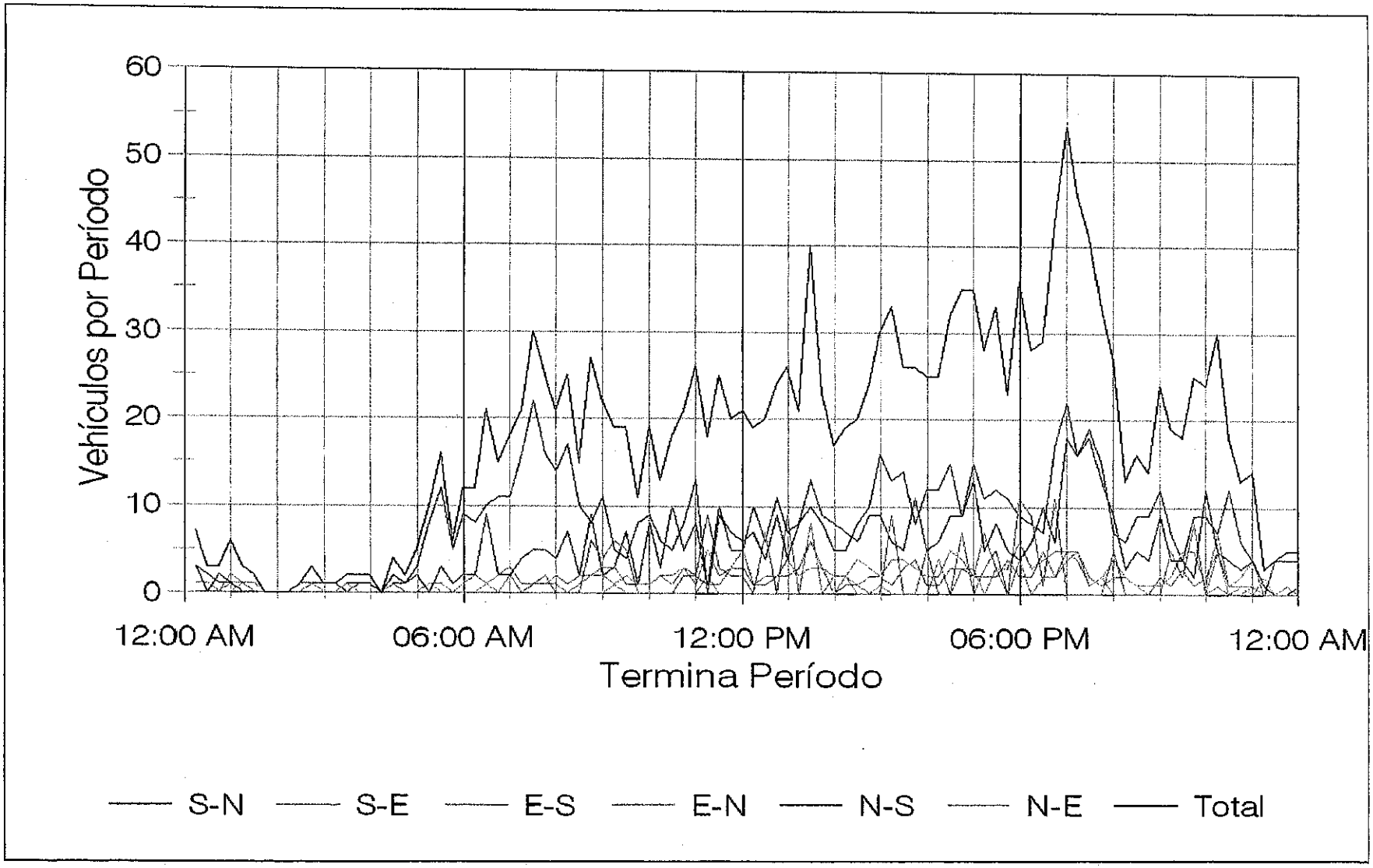

Figura 15. Patrón actual del flujo vehicular, por movimiento y total, en la intersección de las carreteras PR-942 y PR-9942, durante un día laborable típico

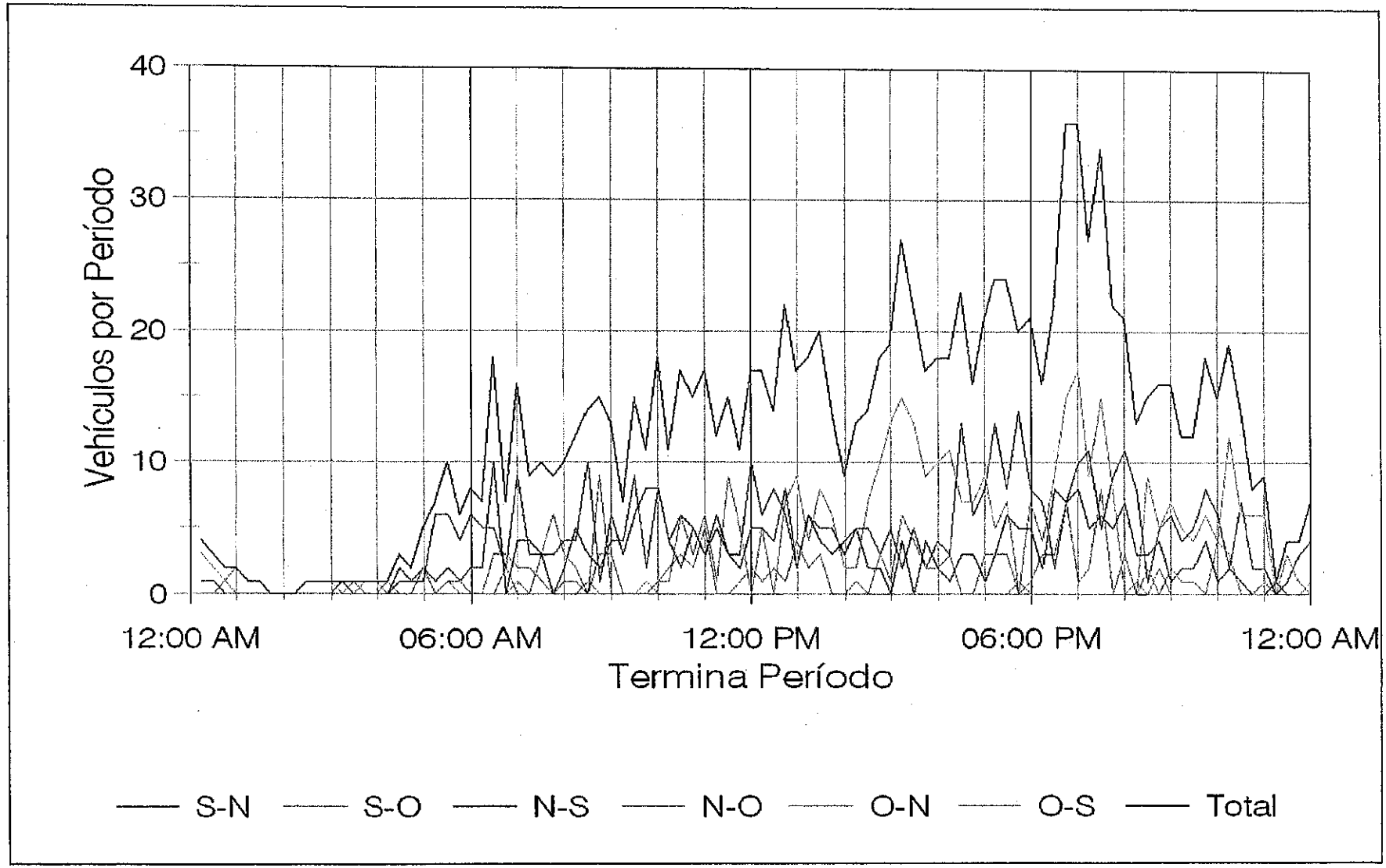

Figura 16. Patrón actual del flujo vehicular, por movimiento y total, en la intersección de las carreteras PR-942 y PR-9943, durante un día laborable típico

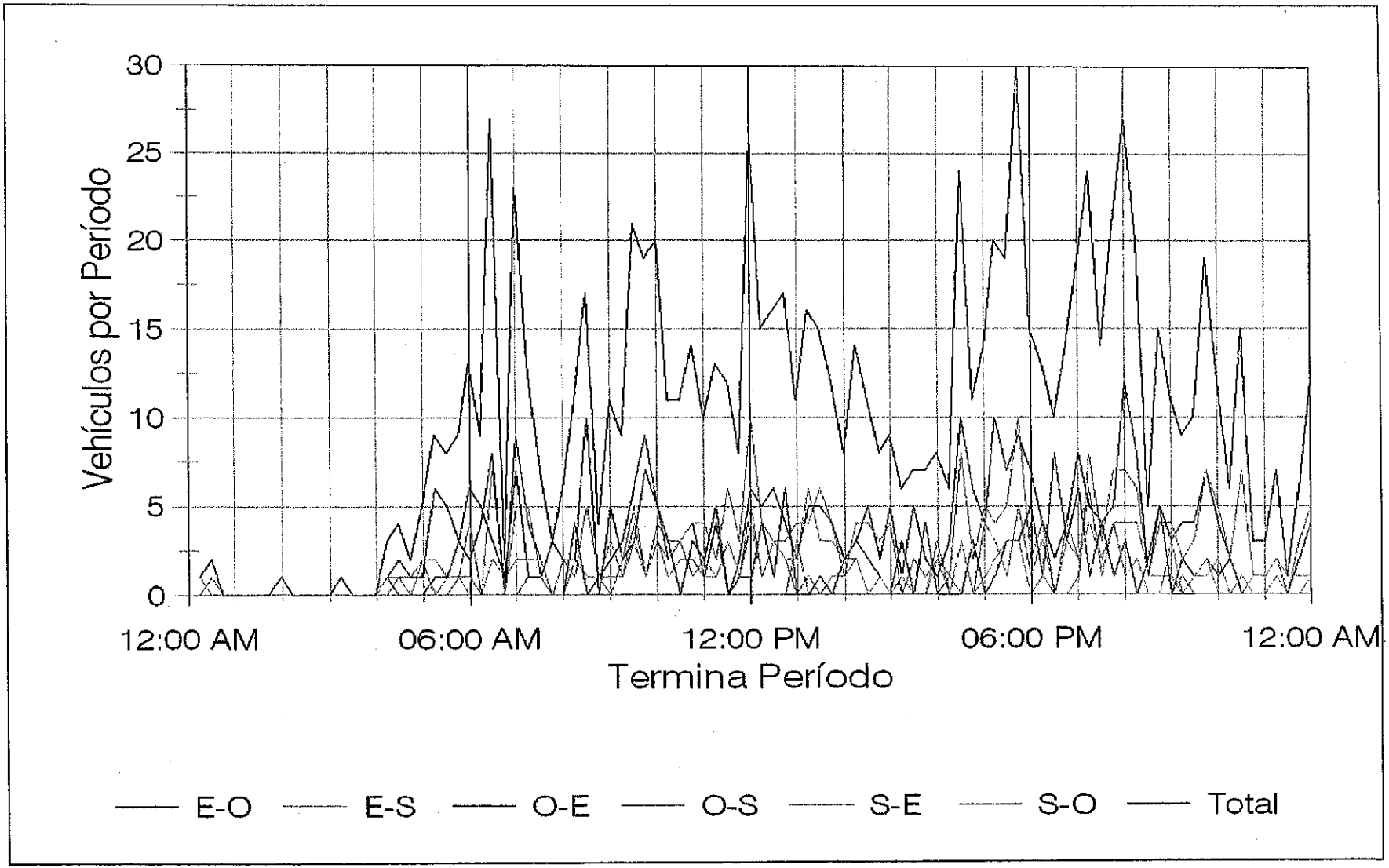

Figura 17. Patrón actual del flujo vehicular, por movimiento y total, en la intersección de las carreteras PR-942 y PR-9941, durante un día laborable típico

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Unit Serial No: 157695 Version: 06.27A XB

Network Address: 00:80:91:22:67:ff

Network Topology: Ethernet Connector: RJ45

Network Speed: 100 Megabits

Novell Network Information enabled
    Print Server Name: XRX_0080912267FF
    Password Defined: No
    Preferred File Server not defined
    Directory Services Context not defined
    Frame Type: 802.2 On 802.3

Peer-to-Peer Information enabled
    Frame Type: 802.2 On 802.3
    Network ID: 0 h

TCP/IP Network Information enabled
    Frame Type: Ethernet II Protocol Address: 192.168.0.154
    Subnet Mask: 255.255.255.0 Default Gateway: 192.168.1.254
    DNS Address: 192.168.1.254
    WINS Name: Not Registered -

IPP Network Information enabled
    Internet Printing Protocol http://192.168.0.154:631/Print

AppleTalk Network Information disabled

Novell inactive

Peer-to-Peer Connection Information
    Printer Name: XRX_0080912267FF

AppleTalk Inactive

TCP/IP Connection Information
    Port Number : 9100

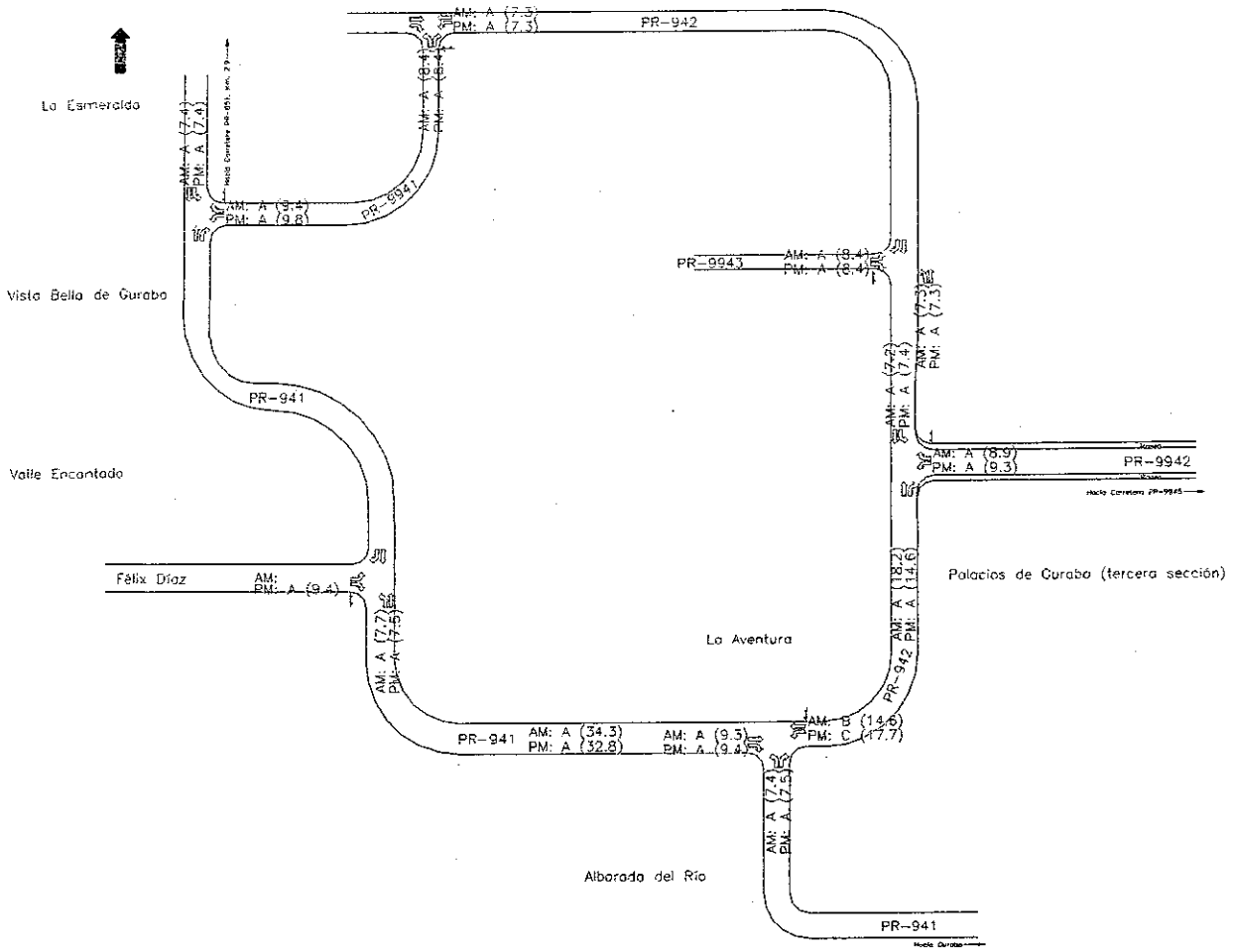


Figura 18. Nivel de Servicio al que operan actualmente, durante las horas pico de un día laborable típico, por movimiento y total, los componentes de la red vial analizada en este estudio. Para las intersecciones, la demora por control aparece entre paréntesis, en segundos por vehículo. En los segmentos de carretera de dos carriles, el porciento del tiempo de recorrido de un vehículo siguiendo a otro se ilustra entre paréntesis

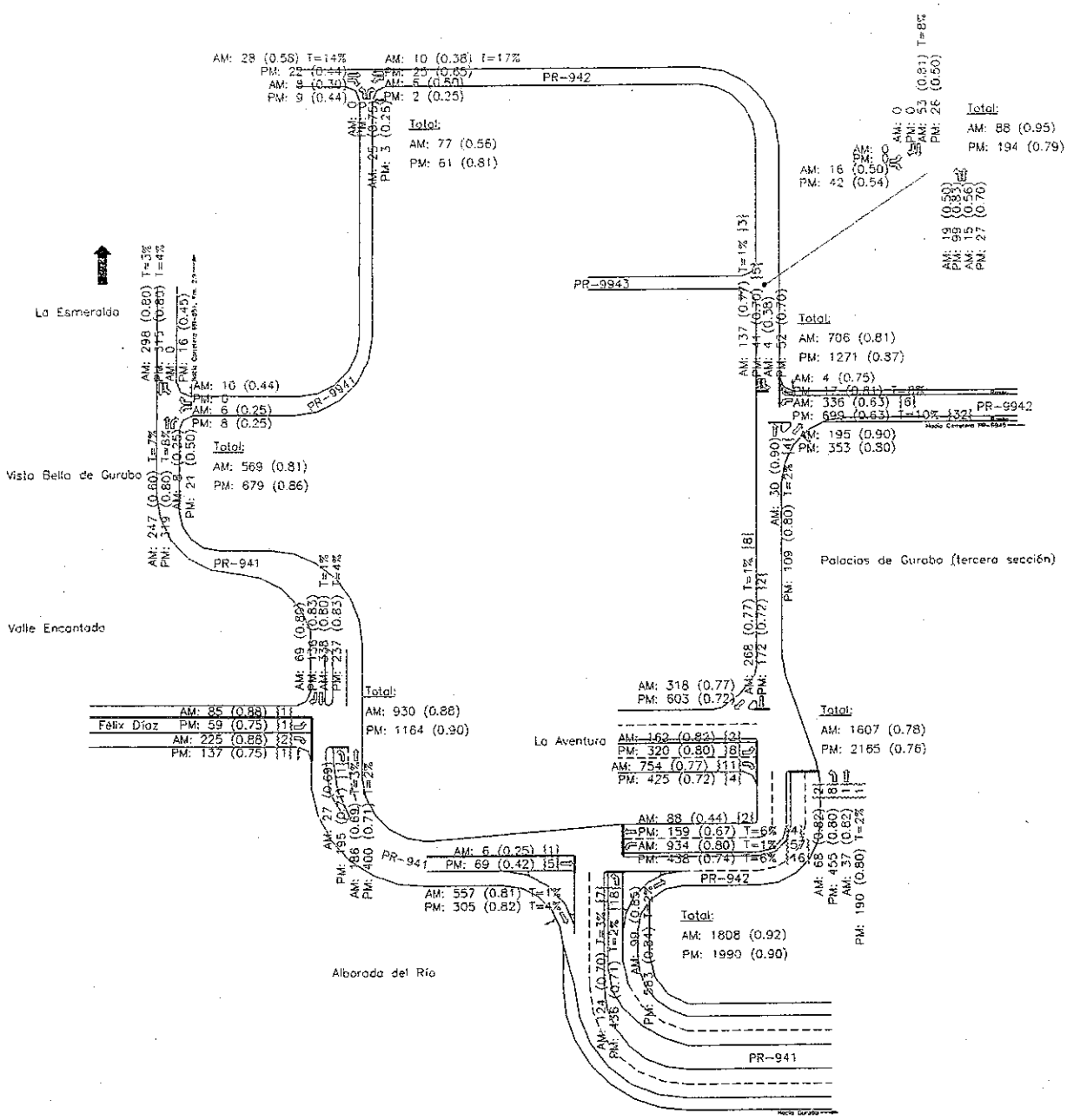


Figura 19. Flujo vehicular y Factor de la Hora Pico, por movimiento y total, durante las horas críticas de un día laborable típico, en las intersecciones analizadas en este estudio, para el momento en el que La Aventura se ocupe completamente. También se indica el porciento de camiones y la longitud de cola, por movimiento, para los mismos períodos. El flujo vehicular se expresa en vehículos por hora. El Factor de la Hora Pico se ilustra entre paréntesis. La longitud de cola se expresa en vehículos y se ilustra entre corchetes. En los movimientos donde no se indica el valor de alguna variable, su valor es cero

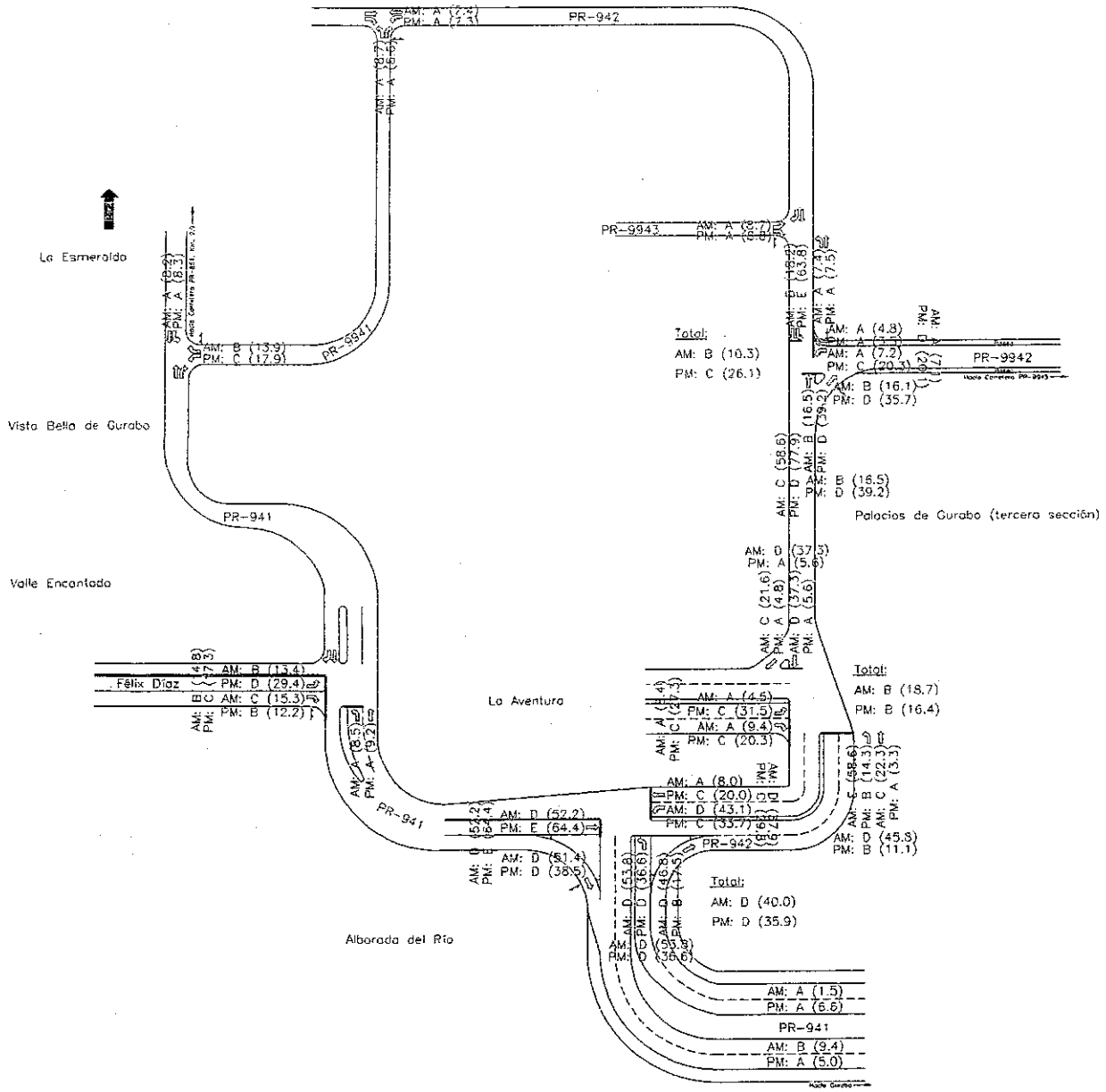


Figura 20. Nivel de Servicio al que operarán en el año 2020, durante las horas pico de un día laborable típico, por movimiento y total, los componentes de la red vial analizada en este estudio. Para las intersecciones, la demora por control aparece entre paréntesis, en segundos por vehículo. En los segmentos de carretera de dos carriles, el porciento del tiempo de recorrido de un vehículo siguiendo a otro se ilustra entre paréntesis. En carreteras de carriles múltiples, la densidad, por dirección, se indica entre paréntesis

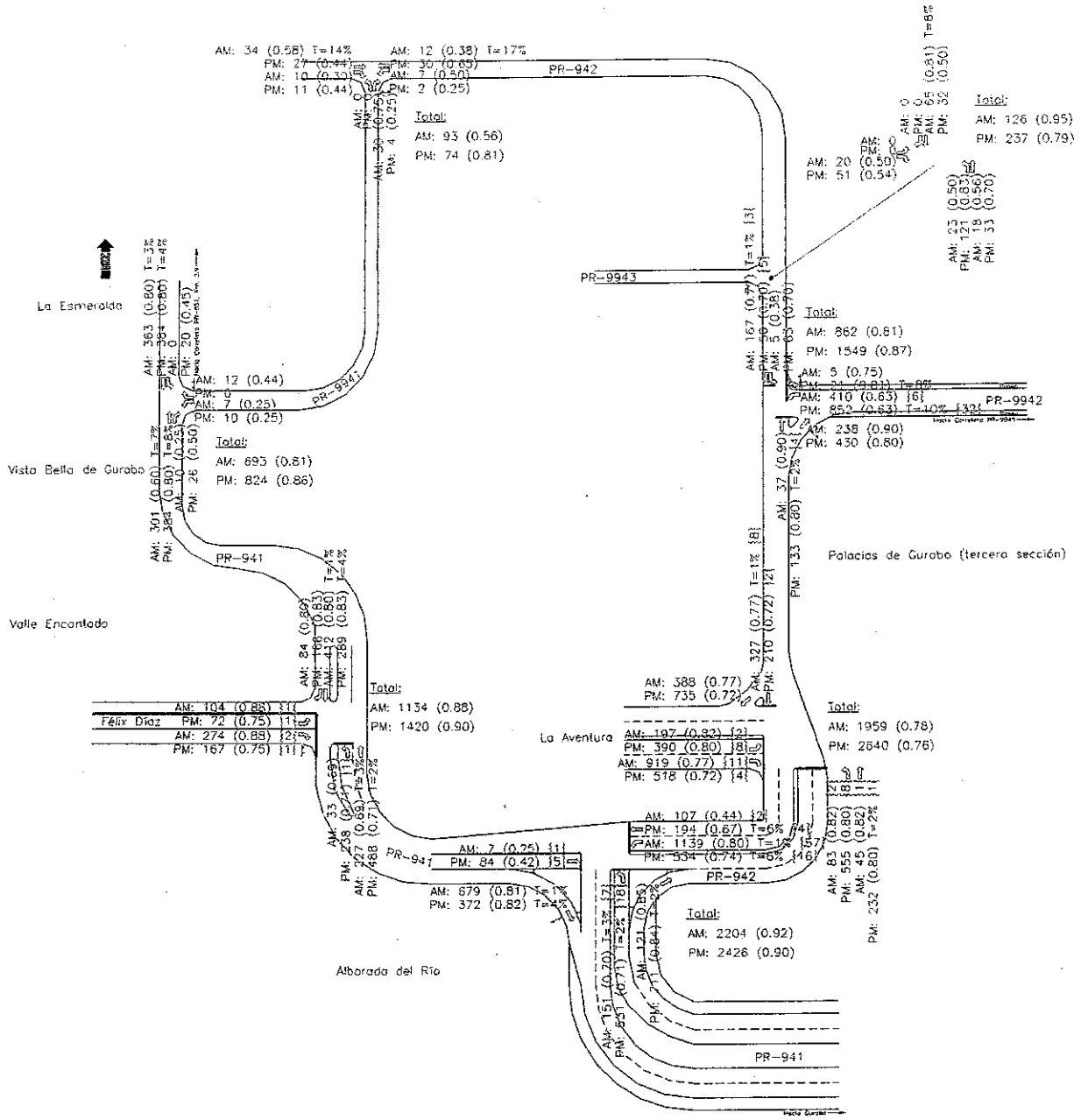


Figura 21. Flujo vehicular y Factor de la Hora Pico, por movimiento y total, durante las horas críticas de un día laborable típico, en las intersecciones analizadas en este estudio, para el año 2030. También se indica el porciento de camiones y la longitud de cola, por movimiento, para los mismos períodos. El flujo vehicular se expresa en vehículos por hora. El Factor de la Hora Pico se ilustra entre paréntesis. La longitud de cola se expresa en vehículos y se ilustra entre corchetes. En los movimientos donde no se indica el valor de alguna variable, su valor es cero

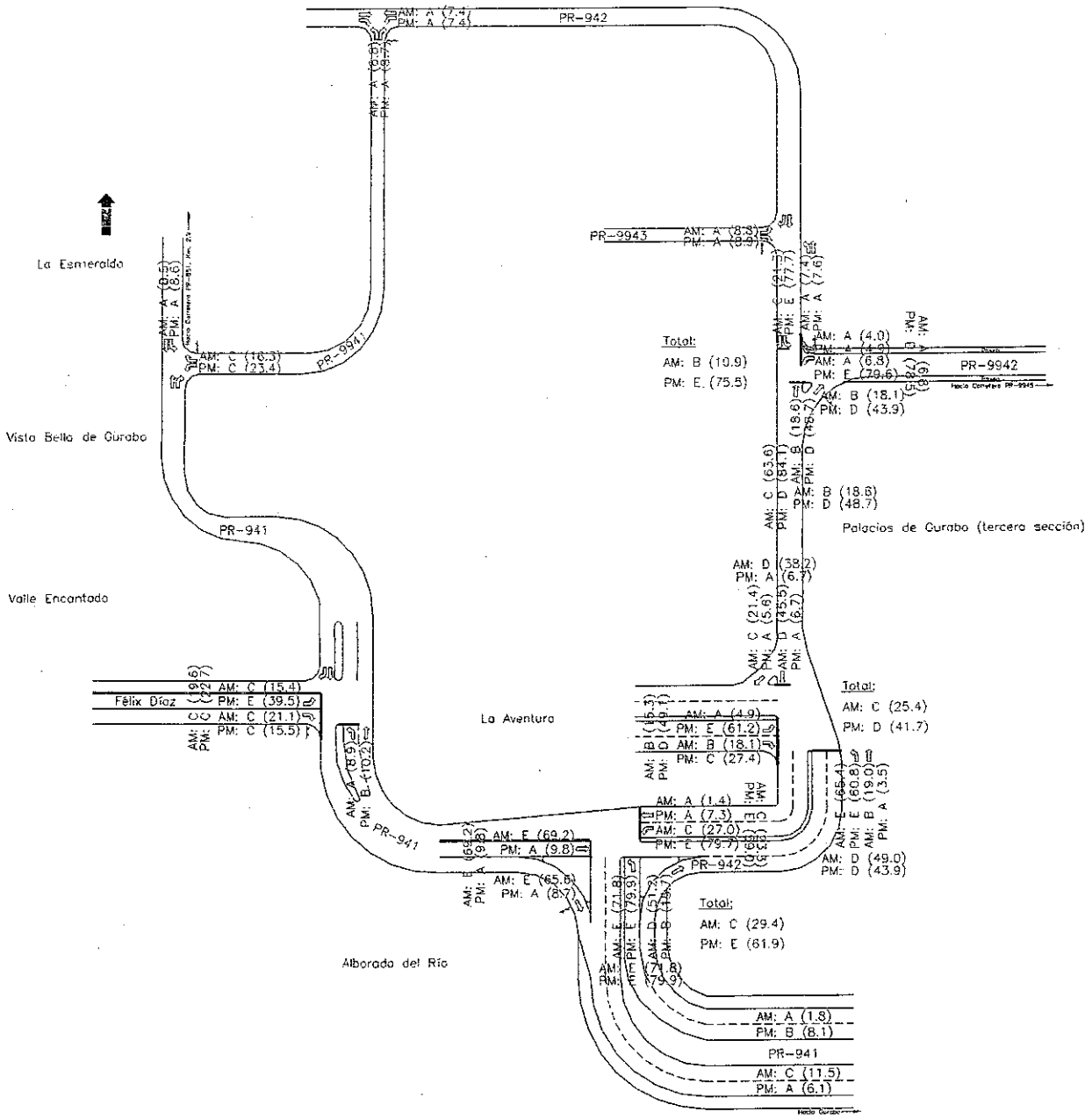


Figura 22. Nivel de Servicio al que operarán en el año 2030, durante las horas pico de un día laborable típico, por movimiento y total, los componentes de la red vial analizada en este estudio. Para las intersecciones, la demora por control aparece entre paréntesis, en segundos por vehículo. En los segmentos de carretera de dos carriles, el porciento del tiempo de recorrido de un vehículo siguiendo a otro se ilustra entre paréntesis. En carreteras de carriles múltiples, la densidad, por dirección, se indica entre paréntesis

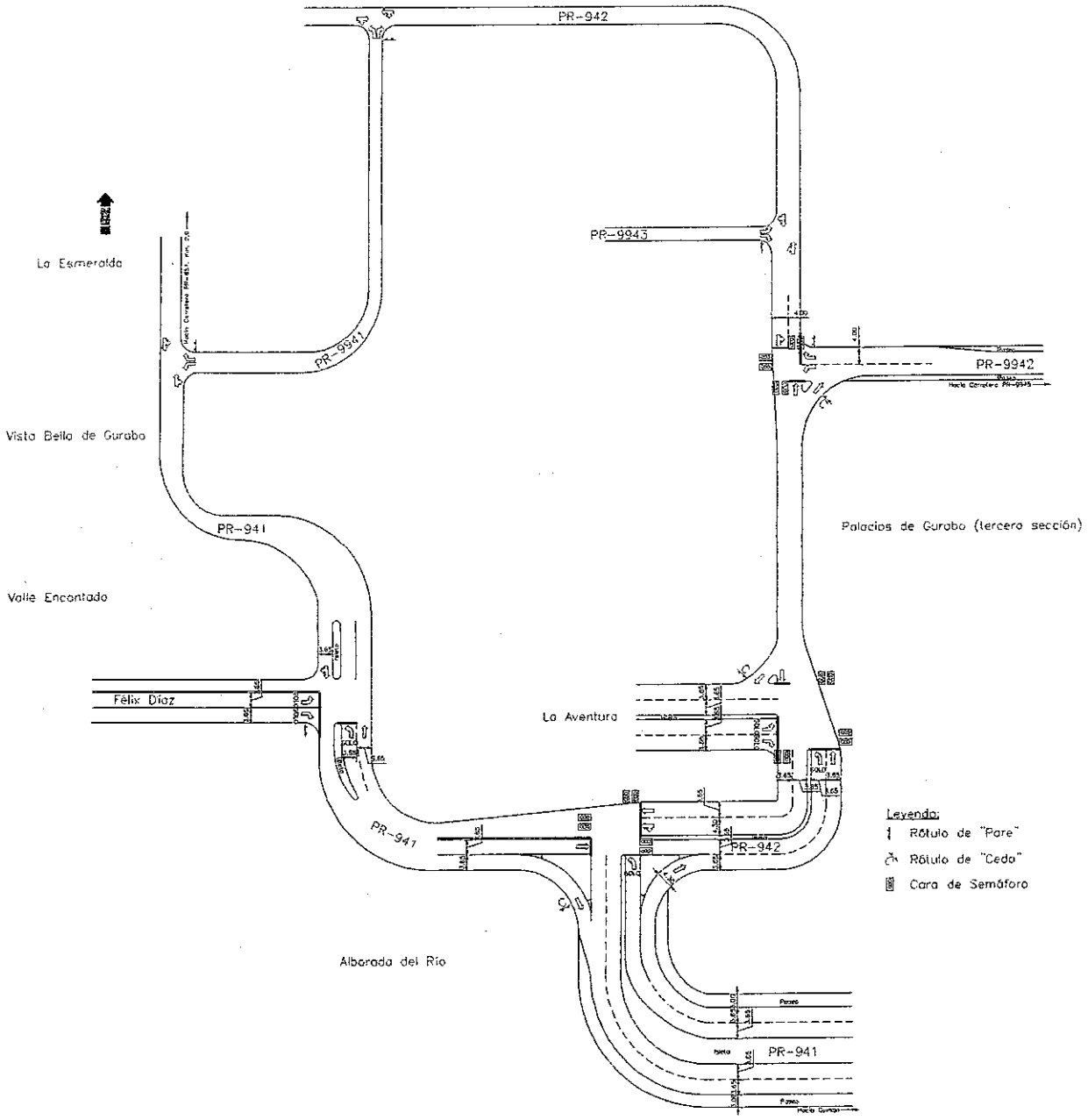


Figura 23. Esquema ilustrando la geometría y controles del tránsito en la red vial analizada, una vez se hayan incorporado los cambios recomendados en este estudio. Dibujo no está a escala

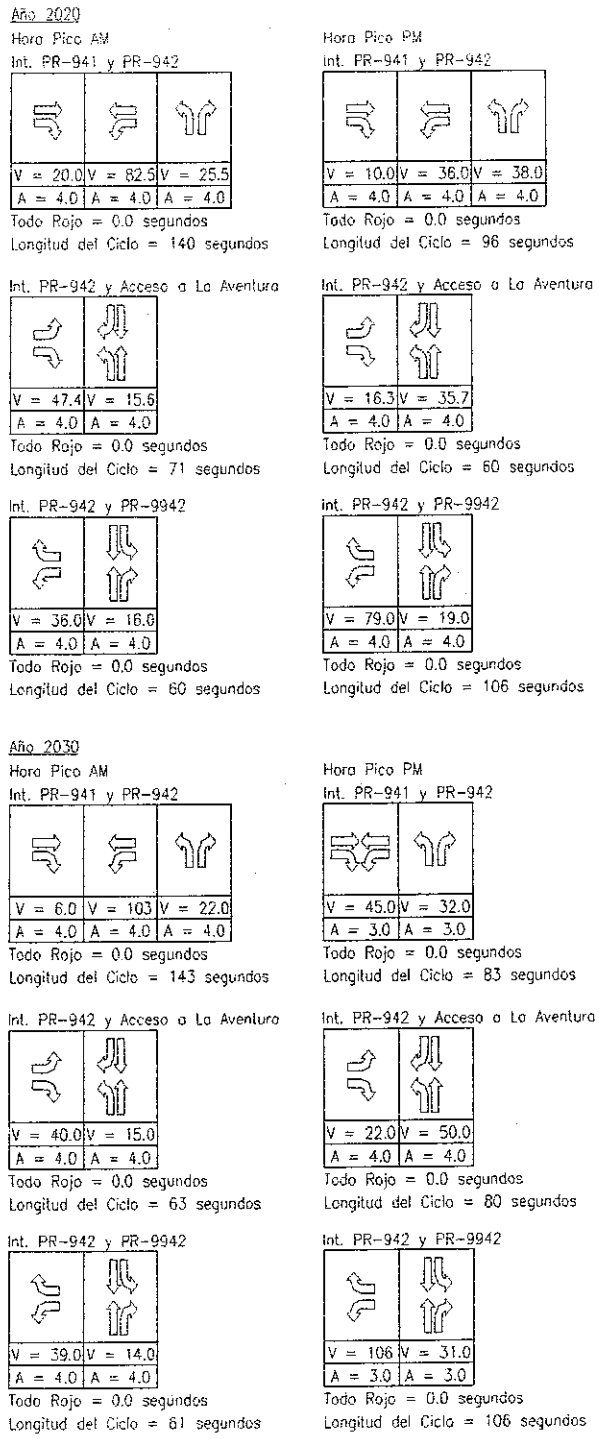


Figura 24. Fases y tiempos propuestos para los semáforos recomendados para las intersecciones de las carreteras PR-941 y PR-942, la carretera PR-942 y el acceso a La Aventura y las carreteras PR-942 y PR-9942. Los sistemas de semáforos deben ser actuados, con los detectores ubicados en todos los accesos, excepto en las rampas para el viraje a la derecha. También, los semáforos de las primeras dos intersecciones deben estar interconectados. Los tiempos ilustrados se expresan en segundos

If a multiple-period analysis is selected, the analyst must carry over the residual queue from one period to the next as discussed in Chapter 16, Appendix F (Extension of Signal Delay Models to Incorporate the Effect of an Initial Queue). The analyst will have to modify or adapt these procedures in the case of unsignalized intersections. Speed, delay, and LOS can then be computed for each analysis period. The analyst must determine how to report these results, since averaging LOS across multiple analysis periods may obscure some of the results.

If a single longer analysis period is selected (such as 1 h), the analyst should use caution in performing the analysis and interpreting the results. The peak-hour factor (which normally is used to compute the peak 15-min flow rate from a 1-h volume) may have to be modified to provide the appropriate flow rate for the longer analysis period. The analyst must also recognize that LOS criteria for urban streets, signalized intersections, and unsignalized intersections were developed for a 15-min analysis period. Conditions that persist for longer periods (presumably with worse peak conditions within those periods) may no longer meet the 15-min LOS criteria provided in this manual.

SERVICE VOLUME TABLE

Exhibit 10-7 is an example service volume table for the four urban street classes. This table is useful for estimates of how many vehicles an urban street can carry at a given level of service, for a particular class and number of lanes (per direction). It is most accurate when the defaults shown in Exhibit 10-7 are applicable. If conditions on a given street vary considerably from those used to create this table, the tabular values are not appropriate.

III. SIGNALIZED INTERSECTIONS

The capacity of an urban street is related primarily to the signal timing and the geometric characteristics of the facility as well as to the composition of traffic on the facility. Geometrics are a fixed characteristic of a facility. Thus, while traffic composition may vary somewhat over time, the capacity of a facility is generally a stable value that can be significantly improved only by initiating geometric improvements.

At signalized intersections, the additional element of time allocation is introduced into the concept of capacity. A traffic signal essentially allocates time among conflicting traffic movements that seek to use the same space. The way in which time is allocated significantly affects the operation and the capacity of the intersection and its approaches.

In analyzing a signalized intersection, the physical unit of analysis is the lane group. A lane group consists of one or more lanes on an intersection approach. The outputs from application of the method in this manual are reported on the basis of each lane group.

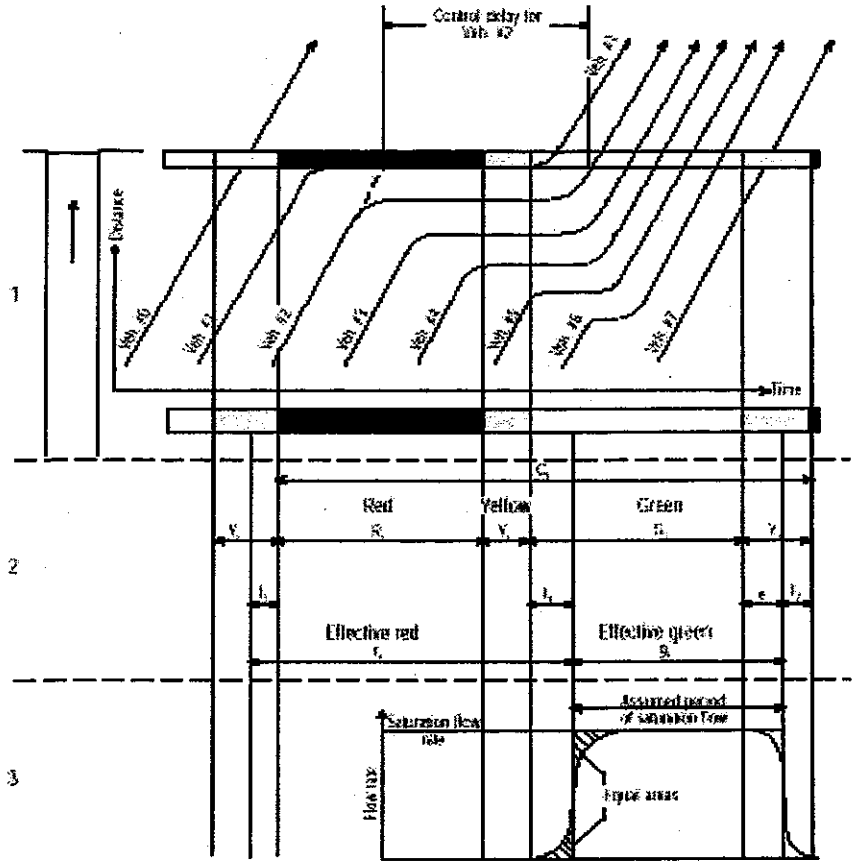
Lane group defined

SIGNALIZED INTERSECTION FLOW CHARACTERISTICS

For a given lane group at a signalized intersection, three signal indications are displayed: green, yellow, and red. The red indication may include a short period during which all indications are red, referred to as an all-red interval, which with the yellow indication forms the change and clearance interval between two green phases.

Exhibit 10-8 provides a reference for much of the discussion in this section. It presents some fundamental attributes of flow at signalized intersections. The diagram represents a simple situation of a one-way approach to a signalized intersection having two phases in the cycle.

EXHIBIT 10-8. FUNDAMENTAL ATTRIBUTES OF FLOW AT SIGNALIZED INTERSECTIONS



The exhibit is divided into three parts. The first part shows a time-space plot of vehicles on the northbound approach to the intersection. The intervals for the signal cycle are indicated in the diagram. The second part repeats the timing intervals and labels the various time intervals of interest with the symbols used throughout this chapter. The third part is an idealized plot of flow rate passing the stop line, indicating how saturation flow is derived. Further definitions of these variables and other basic terms are provided in Exhibit 10-9.

Effective green defined
Effective red defined

The signal cycle for a given lane group has two simplified components: effective green and effective red. Effective green time is the time that may be used by vehicles on the subject lane group at the saturation flow rate. Effective red time is defined as the cycle length minus the effective green time.

It is important that the relationship between the actual green, yellow, and red times shown on signal faces and the effective green and red times be understood. Each time a movement is started and stopped, two lost times are experienced. At the beginning of movement, the first several vehicles in the queue experience start-up losses that result in movement at less than the saturation flow rate (Exhibit 10-8). At the end of a movement, a portion of the change and clearance interval (yellow and all-red) is not used for vehicular movement.

EXHIBIT 10-9. SYMBOLS, DEFINITIONS, AND UNITS FOR FUNDAMENTAL VARIABLES OF TRAFFIC FLOW AT SIGNALIZED INTERSECTIONS

Name	Symbol	Definition	Unit
Change and clearance interval	Y_i	The yellow plus all-red interval that occurs between phases of a traffic signal to provide for clearance of the intersection before conflicting movements are released	s
Clearance lost time	t_2	The time between signal phases during which an intersection is not used by any traffic	s
Control delay	d_i	The component of delay that results when a control signal causes a lane group to reduce speed or to stop; it is measured by comparison with the uncontrolled condition	s
Cycle		A complete sequence of signal indications	
Cycle length	C_i	The total time for a signal to complete one cycle	s
Effective green time	g_i	The time during which a given traffic movement or set of movements may proceed; it is equal to the cycle length minus the effective red time	s
Effective red time	r_i	The time during which a given traffic movement or set of movements is directed to stop; it is equal to the cycle length minus the effective green time	s
Extension of effective green time	e	The amount of the change and clearance interval, at the end of the phase for a lane group, that is usable for movement of its vehicles	s
Green time	G_i	The duration of the green indication for a given movement at a signalized intersection	s
Interval		A period of time in which all traffic signal indications remain constant	
Lost time	t_L	The time during which an intersection is not used effectively by any movement; it is the sum of clearance lost time plus start-up lost time	s
Phase		The part of the signal cycle allocated to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals	
Red time	R_i	The period in the signal cycle during which, for a given phase or lane group, the signal is red	s
Saturation flow rate	s_i	The equivalent hourly rate at which previously queued vehicles can traverse an intersection approach under prevailing conditions, assuming that the green signal is available at all times and no lost times are experienced	veh/h
Start-up lost time	l_1	The additional time consumed by the first few vehicles in a queue at a signalized intersection above and beyond the saturation headway, because of the need to react to the initiation of the green phase and to accelerate	s
Total lost time	L	The total lost time per cycle during which the intersection is effectively not used by any movement, which occurs during the change and clearance intervals and at the beginning of most phases	s

Start-up and clearance lost times are combined and considered to occur at the start of a lane group movement.

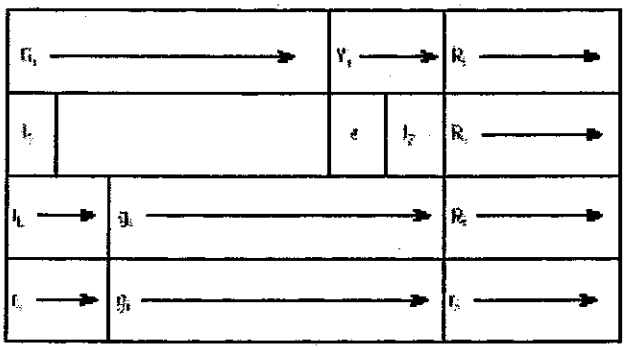
At the beginning of green, the start-up losses are called start-up lost time (l_1). At the beginning of the yellow, when vehicles tend to continue to enter the intersection for a short period of time, an extension of effective green (e) is experienced. When this extension of green has been exhausted, the remainder of the change and clearance interval is considered to be clearance lost time (l_2). The lost time for a lane group, t_L, is the sum of the start-up and clearance lost times.

Research (2) has shown that start-up lost time (l_1) is about 2 s and that the extension of effective green (e) is about 2 s (sometimes longer under congested conditions). Thus, the relationship shown in Equation 10-1 exists for typical conditions, and the relationship

among actual green, lost time, extension of effective green, and effective green is shown in Exhibit 10-10. When $l_1 = 2$ and $e = 2$ (typical), then $t_L = Y_i$.

$$t_L = l_1 + l_2 = l_1 + Y_i - e \qquad (10\text{-}1)$$

EXHIBIT 10-10. RELATIONSHIP AMONG ACTUAL GREEN, LOST-TIME ELEMENTS, EXTENSION OF EFFECTIVE GREEN, AND EFFECTIVE GREEN



As shown in Exhibit 10-10, the lost time for the movement is deducted from the beginning of the actual green phase. Thus, a small portion of G_i becomes part of the effective red, r_i. This portion is equal to the lost time for the movement, t_L. Because all of the lost time for the movement is deducted at the beginning of the green, effective green can be assumed to run through the end of the yellow-plus-all-red change and clearance interval, Y_i. Thus, for any given movement, effective green time is computed by Equation 10-2 and effective red time by Equation 10-3.

$$g_i = G_i + Y_i - t_L \qquad (10\text{-}2)$$

$$r_i = R_c + t_L \qquad (10\text{-}3)$$

A lost time is applied each time a movement is started

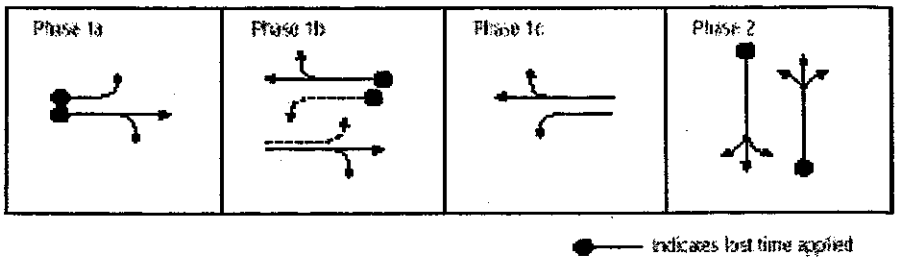
The simplified concept of applying all of the lost time at the beginning of a movement makes it easier to analyze more complex signalization involving protected-plus-permitted left-turn phasing. As a general rule, a lost time, t_L, is applied each time a movement is started. Thus, where a given movement starts in a protected phase and continues through a permitted phase (or vice versa), only one lost time is deducted. No lost time is assumed to occur at the boundary between the permitted and protected phases for continuing movements.

Exhibit 10-11 diagrams a more complex situation involving a protected-plus-permitted and permitted-plus-protected compound left-turn phasing, a classic lead-lag phasing scheme in which left turns are protected in Phase 1a [eastbound (EB)] and Phase 1c [westbound (WB)] and permitted during the common Phase 1b. The question of how many lost times are included in such a phase sequence is important. Using the general rule that the entire lost time for a movement is applied at the time the movement begins, the following may be determined:

- In Phase 1a, the EB through and left-turn movements begin. Thus, a lost time is applied to both movements.
- In Phase 1b, the EB through and left-turn movements continue. No lost times are assigned to the continuing movements in this phase. The WB through and left-turn movements begin in this phase, and a lost time is applied to these movements.
- In Phase 1c, only the WB through and left turns continue. Because these movements did not start in this phase, no lost time is applied here. Further, because no movements begin in Phase 1c, no lost time is applied to any movement in Phase 1c.

• In Phase 2, northbound (NB) and southbound (SB) auto effects begin, and a lost time is applied.

EXHIBIT 10-11. LOST TIME APPLICATION FOR COMPOUND LEFT-TURN PHASING



Total lost time is the sum of lost time for the path through the critical movements

The total lost time in the signal cycle, L, is also important. This is the total lost time involved in the critical path through the signal cycle. Determining the critical path and finding L are discussed in Chapter 16.

TRAFFIC SIGNAL CHARACTERISTICS

Modern traffic signals allocate time in a variety of ways, from the simplest two-phase pretimed mode to the most complex multiphase actuated mode.

There are three types of traffic signal controllers:

• Pretimed, in which a sequence of phases is displayed in repetitive order. Each phase has a fixed green time and change and clearance interval that are repeated in each cycle to produce a constant cycle length.

• Fully actuated, in which the timing on all of the approaches to an intersection is influenced by vehicle detectors. Each phase is subject to a minimum and maximum green time, and some phases may be skipped if no demand is detected. The cycle length for fully actuated control varies from cycle to cycle.

• Semiactuated, in which some approaches (typically on the minor street) have detectors and some of the approaches (typically on the major street) have no detectors.

While these equipment-based definitions have persisted in traffic engineering terminology, the evolution of traffic control technology has complicated their function from the analyst's perspective. For purposes of capacity and level-of-service analysis, it is no longer sufficient to use the controller type as a global descriptor of the intersection operation. Instead, an expanded set of these definitions must be applied individually to each lane group.

Each traffic movement may be served by a phase that is either actuated or nonactuated. Signal phases may be coordinated with neighboring signals on the same route, or they may function in an isolated mode without influence from other signals. Nonactuated phases generally operate with fixed minimum green times, which may be extended by reassigning unused green time from actuated phases with low demand, if such phases exist.

Actuated phases are subject to being shortened on cycles with low demand. On cycles with no demand, they may be skipped entirely, or they may be displayed for their minimum duration. With systems in which the nonactuated phases are coordinated, the actuated phases are also subject to early termination (force off) to accommodate the progression design for the system.

Not only the allocation of green time but also the manner in which turning movements are accommodated within the phase sequence significantly affects capacity and operations at a signalized intersection. Signal phasing can provide for protected, permitted, or not opposed turning movements.

A permitted turning movement is made through a conflicting pedestrian or bicycle flow or opposing vehicle flow. Thus, a left-turn movement concurrent with the opposing through movement is considered to be permitted, as is a right-turn movement concurrent with pedestrian crossings in a conflicting crosswalk. Protected turns are those made without these conflicts, such as turns made during an exclusive left-turn phase or a right-turn phase during which conflicting pedestrian movements are prohibited. Permitted turns experience the friction of selecting and passing through gaps in a conflicting vehicle or pedestrian flow. Thus, a single permitted turn often consumes more of the available green time than a single protected turn. Either permitted or protected turning phases may be more efficient in a given situation, depending on the turning and opposing volumes, intersection geometry, and other factors.

Permitted turning movement

Protected turning movement

Turning movements that are not opposed do not receive a dedicated left-turn phase (i.e., a green arrow), but because of the nature of the intersection, they are never in conflict with through traffic. This condition occurs on one-way streets, at T-intersections, and with signal phasing plans that provide complete separation between all movements in opposite directions (i.e., split-phase operation). Such movements must be treated differently in some cases because they can be accommodated in shared lanes without impeding the through traffic. Left turns that are not opposed at any time should be distinguished from those that may be unopposed during part of the signal cycle and opposed during another part. Left turns that are opposed during any part of the sequence will impede through traffic in shared lanes.

SATURATION FLOW RATE

Saturation flow rate is a basic parameter used to derive capacity. It is defined in Exhibits 10-8 and 10-9. It is essentially determined on the basis of the minimum headway that the lane group can sustain across the stop line as the vehicles depart the intersection. Saturation flow rate is computed for each of the lane groups established for the analysis. A saturation flow rate for prevailing conditions can be determined directly from field measurement and can be used as the rate for the site without adjustment. If a default value is selected for base saturation flow rate, it must be adjusted for a variety of factors that reflect geometric, traffic, and environmental conditions specific to the site under study.

SIGNALIZED INTERSECTION CAPACITY

Lane group capacity defined

Capacity at intersections is defined for each lane group. The lane group capacity is the maximum hourly rate at which vehicles can reasonably be expected to pass through the intersection under prevailing traffic, roadway, and signalization conditions. The flow rate is generally measured or projected for a 15-min period, and capacity is stated in vehicles per hour (veh/h).

Traffic conditions include volumes on each approach, the distribution of vehicles by movement (left, through, and right), the vehicle type distribution within each movement, the location and use of bus stops within the intersection area, pedestrian crossing flows, and parking movements on approaches to the intersection. Roadway conditions include the basic geometrics of the intersection, including the number and width of lanes, grades, and lane use allocations (including parking lanes). Signalization conditions include a full description of the signal phasing, timing, and type of control, and an evaluation of signal progression for each lane group. The analysis of capacity at signalized intersections (Chapter 16) focuses on the computation of saturation flow rates, capacities, v/c ratios, and level of service for lane groups.

LEVEL OF SERVICE

Control delay is the sole measure that defines LOS

Level of service for signalized intersections is defined in terms of control delay, which is a measure of driver discomfort, frustration, fuel consumption, and increased travel time. The delay experienced by a motorist is made up of a number of factors that

relate to control, geometrics, traffic, and incidents. Total delay is the difference between the travel time actually experienced and the reference travel time that would result during base conditions: in the absence of traffic control, geometric delay, any incidents, and any other vehicles. Specifically, LOS criteria for traffic signals are stated in terms of the average control delay per vehicle, typically for a 15-min analysis period. Delay is a complex measure and depends on a number of variables, including the quality of progression, the cycle length, the green ratio, and the v/c ratio for the lane group.

The critical v/c ratio is an approximate indicator of the overall sufficiency of an intersection. The critical v/c ratio depends on the conflicting critical lane flow rates and the signal phasing. The computation of the critical v/c ratio is described in detail in Appendix A and in Chapter 16.

Back of queue defined

The average back of queue is another performance measure that is used to analyze a signalized intersection. The back of queue is the number of vehicles that are queued depending on arrival patterns of vehicles and vehicles that do not clear the intersection during a given green phase. The computation of average back of queue is explained in Appendix G of Chapter 16.

Levels of service are defined to represent reasonable ranges in control delay.

LOS A describes operations with low control delay, up to 10 s/veh. This LOS occurs when progression is extremely favorable and most vehicles arrive during the green phase. Many vehicles do not stop at all. Short cycle lengths may tend to contribute to low delay values.

LOS B describes operations with control delay greater than 10 and up to 20 s/veh. This level generally occurs with good progression, short cycle lengths, or both. More vehicles stop than with LOS A, causing higher levels of delay.

Cycle failure occurs when a given green phase does not serve queued vehicles, and overflows occur

LOS C describes operations with control delay greater than 20 and up to 35 s/veh. These higher delays may result from only fair progression, longer cycle lengths, or both. Individual cycle failures may begin to appear at this level. Cycle failure occurs when a given green phase does not serve queued vehicles, and overflows occur. The number of vehicles stopping is significant at this level, though many still pass through the intersection without stopping.

LOS D describes operations with control delay greater than 35 and up to 55 s/veh. At LOS D, the influence of congestion becomes more noticeable. Longer delays may result from some combination of unfavorable progression, long cycle lengths, and high v/c ratios. Many vehicles stop, and the proportion of vehicles not stopping declines. Individual cycle failures are noticeable.

LOS E describes operations with control delay greater than 55 and up to 80 s/veh. These high delay values generally indicate poor progression, long cycle lengths, and high v/c ratios. Individual cycle failures are frequent.

LOS F describes operations with control delay in excess of 80 s/veh. This level, considered unacceptable to most drivers, often occurs with oversaturation, that is, when arrival flow rates exceed the capacity of lane groups. It may also occur at high v/c ratios with many individual cycle failures. Poor progression and long cycle lengths may also contribute significantly to high delay levels.

Delays in the range of LOS F (unacceptable) can occur while the v/c ratio is below 1.0. Very high delays can occur at such v/c ratios when some combination of the following conditions exists: the cycle length is long, the lane group in question is disadvantaged by the signal timing (has a long red time), and the signal progression for the subject movements is poor. The reverse is also possible (for a limited duration): a saturated lane group (i.e., v/c ratio greater than 1.0) may have low delays if the cycle length is short or the signal progression is favorable, or both.

Thus, the designation LOS F does not automatically imply that the intersection, approach, or lane group is over capacity, nor does an LOS better than F automatically imply that unused capacity is available.

I. INTRODUCTION

SCOPE OF THE METHODOLOGY

This chapter contains a methodology for analyzing the capacity and level of service (LOS) of signalized intersections. The analysis must consider a wide variety of prevailing conditions, including the amount and distribution of traffic movements, traffic composition, geometric characteristics, and details of intersection signalization. The methodology focuses on the determination of LOS for known or projected conditions.

Background and underlying concepts for this chapter are in Chapter 10.

The methodology addresses the capacity, LOS, and other performance measures for lane groups and intersection approaches and the LOS for the intersection as a whole. Capacity is evaluated in terms of the ratio of demand flow rate to capacity (v/c ratio), whereas LOS is evaluated on the basis of control delay per vehicle (in seconds per vehicle). Control delay is the portion of the total delay attributed to traffic signal operation for signalized intersections. Control delay includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. Appendix A presents a method for observing intersection control delay in the field. Exhibit 10-9 provides definitions of the basic terms used in this chapter.

Each lane group is analyzed separately. Equations in this chapter use the subscript i to indicate each lane group. The capacity of the intersection as a whole is not addressed because both the design and the signalization of intersections focus on the accommodation of traffic movement on approaches to the intersection.

A lane group is indicated in formulas by the subscript i.

The capacity analysis methodology for signalized intersections is based on known or projected signalization plans. Two procedures are available to assist the analyst in establishing signalization plans. The first is the quick estimation method, which produces estimates of the cycle length and green times that can be considered to constitute a reasonable and effective signal timing plan. The quick estimation method requires minimal field data and relies instead on default values for the required traffic and control parameters. It is described and documented in Chapter 10.

See Chapter 10 for description of quick estimation method.

A more detailed procedure is provided in Appendix B of this chapter for estimating the timing plan at both pretimed and traffic-actuated signals. The procedure for pretimed signals provides the basis for the design of signal timing plans that equalize the degree of saturation on the critical approaches for each phase of the signal sequence. This procedure does not, however, provide for optimal operation.

The methodology in this chapter is based in part on the results of a National Cooperative Highway Research Program (NCHRP) study (*1, 2*). Critical movement capacity analysis techniques have been developed in the United States (*3–5*), Australia (*6*), Great Britain (*7*), and Sweden (*8*). Background for delay estimation procedures was developed in Great Britain (*7*), Australia (*9, 10*), and the United States (*11*). Updates to the original methodology were developed subsequently (*12–24*).

LIMITATIONS TO THE METHODOLOGY

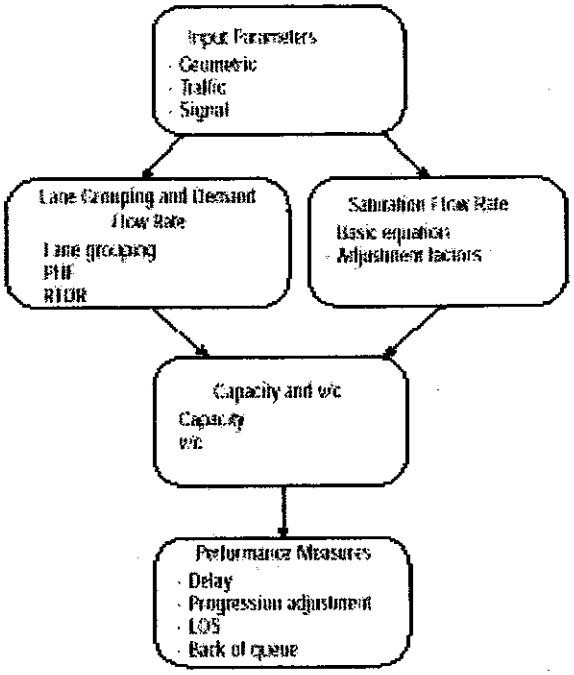
The methodology does not take into account the potential impact of downstream congestion on intersection operation. Nor does the methodology detect and adjust for the impacts of turn-pocket overflows on through traffic and intersection operation.

II. METHODOLOGY

Exhibit 16-1 shows the input and the basic computational order for the method. The primary output of the method is level of service (LOS). This methodology covers a wide range of operational configurations, including combinations of phase plans, lane

illumination, and left-turn treatment alternatives. It is important to note that some of these configurations may be considered unacceptable by some operating agencies from a traffic safety point of view. The safety aspect of signalized intersections cannot be ignored, and the provision in this chapter of a capacity and LOS analysis methodology for a specific operational configuration does not imply an endorsement of the suitability for application of such a configuration.

EXHIBIT 16-1. SIGNALIZED INTERSECTION METHODOLOGY



LOS

The average control delay per vehicle is estimated for each lane group and aggregated for each approach and for the intersection as a whole. LOS is directly related to the control delay value. The criteria are listed in Exhibit 16-2.

LOS criteria

EXHIBIT 16-2. LOS CRITERIA FOR SIGNALIZED INTERSECTIONS

LOS	Control Delay per Vehicle (s/veh)
A	≤ 10
B	> 10-20
C	> 20-35
D	> 35-55
E	> 55-80
F	> 80

EXHIBIT 10-24. EXAMPLE SERVICE VOLUMES FOR SIGNALIZED INTERSECTION (SEE FOOTNOTE FOR ASSUMED VALUES)

Through Lanes	LOS A	LOS B	LOS C	LOS D	LOS E
	Service Volumes (veh/h)				
1	N/A	150	350	530	590
2	N/A	200	860	1090	1220
3	N/A	N/A	1230	1510	1680

Notes
N/A - not achievable given assumptions listed below.
This table is derived from the following assumptions.
a. Entries are total hourly volumes for subject approach including turns.
b. All approaches of intersection have the same demand as the subject approach.
c. Left turns equal to 10% of approach demand. Right turns equal to 10% of approach demand.
d. Phasing is protected lefts with exclusive left turn lane in addition to through lanes.
e. All approaches are two-way streets.
f. Peak hour factor = 0.92.
g. Saturation flow for each approach is computed assuming the following defaults: lane width = 3.6 m, percent heavy vehicles = 2%, grades = 0%, parking = 0/h, bus = 2/h, pedestrians = 50/h, area type = CBD, lane utilization = 1.05 for 2 lane, 1.10 for 3 lanes, base saturation flow rate = 1900 pcphpln. These assumptions result in adjusted saturation flow rates of 1770 for the left turn lanes and 1560 veh/h/ln for the through lanes.
h. Lost time is 16 s.
i. No upstream signal (I = 1.0).
j. Pretimed signal (k = 0.50).
k. Arrival type = 3.
l. Analysis period (T) = 0.25 h.
m. Initial queue = 0.
n. Signal timing set to maximize service volumes subject to pedestrian clearance time requirements, minimum phase lengths, and maximum signal length of 150 s.
o. Cycle lengths are as follows.

Through Lanes	LOS A	LOS B	LOS C	LOS D	LOS E
1	N/A	60 s	60 s	110 s	150 s
2	N/A	70 s	80 s	110 s	150 s
3	N/A	N/A	80 s	100 s	150 s

Note that minimum cycle lengths to serve pedestrians are 60 s for single-lane approach intersection, 70 s for two-lane approach intersection, and 80 s for a three-lane approach intersection. Minimum pedestrian clearance times are 3 s per through lane plus 3 s for left-turn lane plus 6 s walk display. Minimum left turn phase was set at 6 s plus phase change and clearance interval of 4 s.

IV. UNSIGNALIZED INTERSECTIONS

Three types of unsignalized intersections are addressed in this manual: two-way stop-controlled (TWSC), all-way stop-controlled (AWSC), and roundabouts.

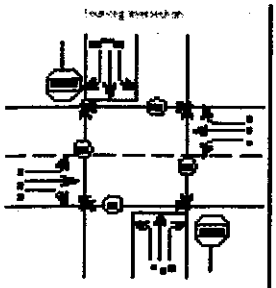
CHARACTERISTICS OF TWSC INTERSECTIONS

TWSC intersections are common in the United States and abroad. Stop signs are used to control vehicle movements at such intersections. At TWSC intersections, the stop-controlled approaches are referred to as the minor street approaches; they can be either public streets or private driveways. The intersection approaches that are not controlled by stop signs are referred to as the major street approaches.

A three-leg intersection is considered to be a standard type of TWSC intersection if the single minor street approach (i.e., the stem of the T configuration) is controlled by a stop sign. Three-leg intersections where two of the three approaches are controlled by stop signs are a special form of unsignalized intersection control.

FLOW AT TWSC INTERSECTIONS

TWSC intersections assign the right-of-way among conflicting traffic streams according to the following hierarchy:



Rank	Traffic stream
1	2, 5, 7, 8, 11, 14, 16
2	1, 3, 4, 10, 13, 15, 17
3	9, 12
4	6, 18

Rank 1 All conflicting movements yield the right-of-way to any through or right-turning vehicle on the major street approaches. The major street through and right-turning movements are the highest-priority movements at a TWSC intersection.

Rank 2 Vehicles turning left from the major street onto the minor street yield only to conflicting major street through and right-turning vehicles. All other conflicting movements at a TWSC intersection yield to these major street left-turning movements. Vehicles turning right from the minor street onto the major street yield only to conflicting major street through movements.

Rank 3 Minor street through vehicles yield to all conflicting major street through, right-turning, and left-turning movements.

Rank 4 Minor street left-turning vehicles yield to all conflicting major street through, right-turning, and left-turning vehicles and to all conflicting minor street through and right-turning vehicles.

Even though the hierarchy described above suggests that the highest-priority movements experience no delay as they travel through a TWSC intersection, experience shows that their right-of-way is sometimes preempted by other conflicting movements. Such preemptions most often occur during periods of congestion when vehicles in the conflicting movements are experiencing long delays and queues (or when separate left-turn bays are not provided on the major street).

GAP ACCEPTANCE MODELS

Gap acceptance models begin with the recognition that TWSC intersections give no positive indication or control to the driver on the minor street as to when it is safe to leave the stop line and enter the major traffic stream. The driver must determine both when a gap in the major stream is large enough to permit safe entry and when it is the driver's turn to enter on the basis of the relative priority of the competing traffic streams. This decision-making process has been formalized into what is commonly known as gap acceptance theory. Gap acceptance theory includes three basic elements: the size and distribution (availability) of gaps in the major traffic stream, the usefulness of these gaps to the minor stream drivers, and the relative priority of the various traffic streams at the intersection.

Availability of Gaps

The first element to consider is the proportion of gaps of a particular size in the major traffic stream offered to the driver entering from the minor stream, as well as the pattern of arrival times of vehicles. The distribution of gaps between the vehicles in the different streams has a major effect on the performance of the intersection.

Usefulness of Gaps

Critical gap is the minimum time between successive major street vehicles where minor street vehicles make a maneuver

Follow-up time is the time span between the departure of one vehicle from the minor street and the departure of the next vehicle using the same gap

The second element to consider is the extent to which drivers find gaps of a particular size useful when attempting to enter the intersection. It is generally assumed in gap acceptance theory that drivers are both consistent and homogeneous. In reality, this assumption is not entirely correct. Studies have demonstrated that different drivers have different gap acceptance thresholds and even that the gap acceptance threshold of an individual driver often changes over time (9). In this manual the critical gap and follow-up times are considered representative of a statistical average of the driver population in the United States.

Relative Priority of Various Streams at the Intersection

Different streams have different ranking in a priority hierarchy. The gap acceptance process evaluates them with impedance terms through the order of departures. Typically, gap acceptance processes assume that drivers on the major road or stream are unaffected

by the minor stream drivers. If this is not the case, the gap acceptance process has to be modified.

CAPACITY OF TWSC INTERSECTIONS

At TWSC intersections, drivers on the controlled approaches are required to select gaps in the major street flow through which to execute crossing or turning maneuvers on the basis of judgment. In the presence of a queue, each driver on the controlled approach must also use some time to move into the front-of-queue position and prepare to evaluate gaps in the major street flow. Thus, the capacity of the controlled legs is based on three factors: the distribution of gaps in the major street traffic stream, driver judgment in selecting gaps through which to execute the desired maneuvers, and the follow-up time required by each driver in a queue.

The basic capacity model assumes that gaps in the conflicting stream are randomly distributed. When traffic signals on the major street are within 0.4 km of the subject intersection, flows may not be random but will likely have some platoon structure.

Pedestrians crossing an intersection impede lower-ranked minor street vehicles, but only one lane at a time. This is because vehicles performing a given through or turning movement tend to pass in front of or behind pedestrians once a driver's target lane is clear. Pedestrian flows are counted somewhat differently than are vehicle flows. If the typical pattern is for pedestrians to walk individually, each pedestrian is counted individually in the pedestrian flow. However, if pedestrians tend to cross in groups, the number of groups is counted. The important factor is to determine the number of blockages. In most cases, this will be a combination of individual pedestrians and groups of pedestrians. Thus, as defined for the purpose of determining the pedestrian impedance, the pedestrian volume is the sum of individual pedestrians crossing individually and groups of pedestrians crossing together during the analysis time period.

The existence of a raised or striped median or a two-way left-turn lane (TWLTL) on the major street often causes some degree of a gap acceptance phenomenon known as "two-stage gap acceptance." For example, the existence of a raised or striped median causes a significant proportion of the minor street drivers to first cross part of the major street approach and then pause in the middle of the road to wait for another gap in the other approach. If a TWLTL exists on the major street, the minor street left-turn vehicle usually merges into the TWLTL first, then seeks a usable gap on the other approach while slowly moving some distance along the TWLTL. Both of these behaviors can increase capacity.

The geometric elements near the stop line on the stop-controlled approaches of many intersections may result in a higher capacity than the shared-lane capacity equation may predict. This is because, at such approaches, two vehicles may occupy or depart from the stop line simultaneously as a result of a large curb radius, a tapered curb, or a parking prohibition. The magnitude of this effect will depend in part on the turning movement volumes and the resultant probability of two vehicles being simultaneously at the stop line and on the storage length available to feed the second position at the stop line.

Often, two or three movements share a single lane on the minor approach. With this lane sharing, vehicles from different movements do not have simultaneous access to gaps, nor can more than one vehicle from the sharing movements use the same gap, which influences capacity.

The existence of nearby signalized intersections (i.e., traffic signals on the major street within 0.4 km of the subject intersection) typically causes vehicles to arrive at the intersection in platoons. This influences the size and distribution of available gaps and may cause an increase in the minor street capacity. The greater the number of vehicles traveling in platoons, the higher the minor street capacity for a given opposing volume. This is due to the greater proportion of large gaps that more than one minor street vehicle can use. If signalized intersections exist upstream of the subject intersection in both directions, the effect is much more complex.

PREFACE

OVERVIEW

The procedures in this chapter can be used to analyze the capacity and level of service, lane requirements, and effects of traffic and design features of two-way stop-controlled (TWSC) and all-way stop-controlled (AWSC) intersections. In addition, a procedure for estimating capacity of roundabouts is presented.

Each type of unsignalized intersection (TWSC, AWSC, and roundabout) is addressed in a separate part of this chapter. TWSC intersections are covered in Part A, AWSC intersections are covered in Part B, and information on roundabouts is provided in Part C. References for all parts are found in Part D. Example problems that demonstrate the calculations and results achieved by applying the procedures are also found in Part D.

Background and concepts for TWSC intersections are in Chapter 10

LIMITATIONS OF THE METHODOLOGY

This chapter does not include a detailed method for estimating delay for yield sign-controlled intersections. However, with appropriate changes in the values of key parameters, the analyst could apply the TWSC method to yield-controlled intersections.

All of the methods are for steady-state conditions (i.e., the demand and capacity conditions are constant during the analysis period); the methods are not designed to evaluate how fast or how often the facility transitions from one demand/capacity state to another. Analysts interested in that kind of information should consider applying simulation models.

PART A. TWO-WAY STOP-CONTROLLED INTERSECTIONS

I. INTRODUCTION - PART A

In this section a methodology for analyzing capacity and level of service of two-way stop-controlled (TWSC) intersections is presented.

II. METHODOLOGY - PART A

Capacity analysis at TWSC intersections depends on a clear description and understanding of the interaction of drivers on the minor or stop-controlled approach with drivers on the major street. Both gap acceptance and empirical models have been developed to describe this interaction. Procedures described in this chapter rely on a gap acceptance model developed and refined in Germany [1]. The concepts from this model are described in Chapter 10. Exhibit 17-1 illustrates input to and the basic computation center of the method described in this chapter.

Both theoretical and empirical approaches have been used to arrive at a methodology

LEVEL-OF-SERVICE CRITERIA

Level of service (LOS) for a TWSC intersection is determined by the computed or measured control delay and is defined for each minor movement. LOS is not defined for the intersection as a whole. LOS criteria are given in Exhibit 17-2.

LOS is not defined for the overall intersection

EXHIBIT 17-1. TWSC UNSIGNALIZED INTERSECTION METHODOLOGY

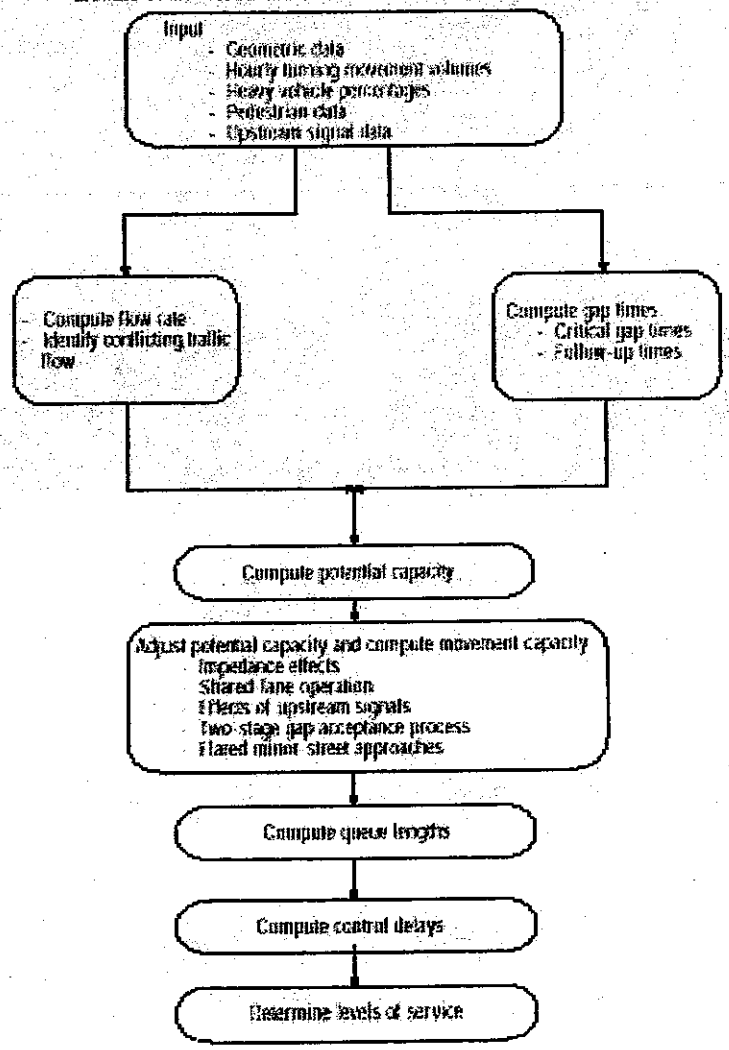


EXHIBIT 17-2. LEVEL-OF-SERVICE CRITERIA FOR TWSC INTERSECTIONS

Level of Service	Average Control Delay (s/veh)
A	0–10
B	> 10–15
C	> 15–25
D	> 25–35
E	> 35–50
F	> 50

The LOS criteria for TWSC intersections are somewhat different from the criteria used in Chapter 16 for signalized intersections primarily because different transportation facilities create different driver perceptions. The expectation is that a signalized intersection is designed to carry higher traffic volumes and experience greater delay than an unsignalized intersection.

LOS thresholds differ from those for signalized intersections to reflect different driver expectations.

INPUT DATA REQUIREMENTS

Data requirements for the TWSC intersection methodology are similar to those for other capacity analysis techniques. Detailed descriptions of the geometrics, control, and volumes at the intersection are needed.

Key geometric factors include number and use of lanes, channelization, two-way left-turn lane (TWLTL) or raised or striped median storage (or both), approach grade, and existence of flared approaches on the minor street.

The number and use of lanes are critical factors. Vehicles in adjacent lanes can use the same gap in the traffic stream simultaneously (unless impeded by a conflicting user of the gap). When movements share lanes, only one vehicle from those movements can use each gap. A TWLTL or a raised or striped median (or both) allows a minor-stream vehicle to cross one major traffic stream at a time. The grade of the approach has a direct and measurable effect on the capacity of each minor movement. Compared with a level approach, downgrades increase capacity and upgrades decrease capacity. A flared approach on the minor street increases the capacity by allowing more vehicles to be served simultaneously.

Volumes must be specified by movement. For the analysis to reflect conditions during the peak 15 min, the analyst must divide the full hour volumes by the peak-hour factor (PHF) before beginning computations. If the analyst has peak 15-min flow rates, they can be entered directly with the PHF set to 1.0. The adjusted flow rate for movement x is designated as v_x in this chapter.

By convention, subscripts 1 to 6 define vehicle movements on the major street, and subscripts 7 to 12 define movements on the minor street. Pedestrian flows impede all minor-street movements. Pedestrian volumes must be specified by movement. Subscripts 13 to 16 define the pedestrian movements.

The presence of traffic signals upstream from the intersection on the major street will produce platoon flows and affect the capacity of the minor-street approaches if the signal is within 0.4 km of the intersection. The basic capacity model assumes that the headways on the major street are exponentially distributed. To assess the effect on capacity, a separate analysis is provided that requires the signalized intersection data (cycle length, green times, the saturation flow rate, and information on platooned flow).

PRIORITY OF STREAMS

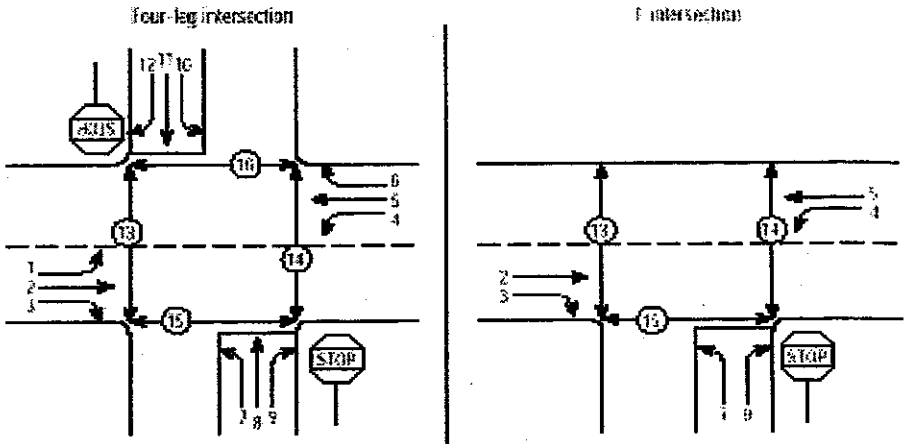
In using the methodology, the priority of right-of-way given to each traffic stream must be identified. Some streams have absolute priority, whereas others have to give way or yield to higher-order streams. Exhibit 17-3 shows the relative priority of streams at both T- and four-leg intersections.

Movements of Rank 1 (denoted by the subscript i) include through traffic on the major street and right-turning traffic from the major street. Movements of Rank 2 (subordinate to 1 and denoted by the subscript j) include left-turning traffic from the major street and right-turning traffic onto the major street.

Movements of Rank 3 (subordinate to 1 and 2 and denoted by the subscript k) include through traffic on the minor street (in the case of a four-leg intersection) and left-turning traffic from the minor street (in the case of a T-intersection). Movements of Rank 4 (subordinate to all others and denoted by the subscript l) include left-turning traffic from the minor street. Rank 4 movements only occur at four-leg intersections.

Rank	Subscript
1	i
2	j
3	k
4	l

EXHIBIT 17-3. TRAFFIC STREAMS AT A TWSC INTERSECTION



Rank	Traffic stream
1	2, 3, 5, 6, 15, 16
2	1, 4, 13, 14, 9, 12
3	8, 11
4	7, 10

Rank	Traffic stream
1	2, 3, 5, 15
2	4, 13, 14, 9
3	7

For example, if a left-turning vehicle on the major street and a through vehicle from the minor street are waiting to cross the major traffic stream, the first available gap of acceptable size would be taken by the left-turning vehicle. The minor-street through vehicle must wait for the second available gap. In aggregate terms, a large number of such left-turning vehicles could use up so many of the available gaps that minor-street through vehicles would be severely impeded or unable to make safe crossing movements.

Because right-turning vehicles from the minor street merely merge into gaps in the right-hand lane of the stream into which they turn, they require only a gap in that lane, not in the entire major-street traffic flow (this may not be true for some trucks and vans with long wheelbases that encroach on more than one lane in making their turns). Furthermore, a gap in the overall major-street traffic could be used simultaneously by another vehicle. For this reason, the method assumes that right turns from the minor street do not impede any of the other flows using major-street gaps.

Pedestrians are treated as movements

Pedestrian movements also have priorities with respect to vehicular movements. While this may be a policy issue varying by jurisdiction, both the American Association of State Highway and Transportation Officials (AASHTO) (2) and the *Manual on Uniform Traffic Control Devices* (MUTCD) (3) infer that pedestrians must use acceptable gaps in major-street (Rank 1) traffic streams and that pedestrians have priority over all minor-street traffic at a TWSC intersection. Specific rankings are shown in Exhibit 17-3.

CONFLICTING TRAFFIC

Each movement at a TWSC intersection faces a different set of conflicts that are directly related to the nature of the movement. These conflicts are shown in Exhibit 17-4, which illustrates the computation of the parameter $v_{c,x}$, the conflicting flow rate for movement x, that is, the total flow rate that conflicts with movement x (veh/h).

The right-turn movement from the minor street, for example, is in conflict with only the major-street through movement in the right-hand lane into which right-turners will merge. Exhibit 17-4 includes one-half of the right-turn movement from the major street, because only some of these turns tend to impede the subject movement.

Left turns from the major street are in conflict with the total opposing through and right-turn flows, because they must cross the through flow and merge with the right-turn flow. The method does not differentiate between crossing and merging conflicts. Left turns from the major street and the opposing right turns from the major street are considered to merge, regardless of the number of lanes provided in the exit roadway.

Minor-street through movements have a direct crossing or merging conflict with all movements on the major street, as indicated in Exhibit 17-4, except the right turn into the subject approach. Only one-half of this movement is included in the computation, for the reasons discussed above. In addition, field research (*4*) has shown that the effect of left-turn vehicles is twice their actual number. This effect is reflected in Exhibit 17-4.

The left turn from the minor street is the most difficult maneuver to execute at a TWSC intersection, and it faces the most complex set of conflicting flows, which include all major-street flows, in addition to the opposing right-turn and through movements on the minor street. Only one-half of the opposing right-turn and through movement flow rate is included as conflicting flow rate because both movements are stop-controlled and their effect on left turns is diminished. The additional capacity impedance effects of the opposing right-turn and through movement flow rates are taken into account elsewhere in the procedure.

Pedestrians may also conflict with vehicular traffic streams. Pedestrian flow rates, also defined as v_x, with x noting the leg of the intersection being crossed, should be included as part of the conflicting flow rates, since they, like vehicular flows, define the beginning or ending of a gap that may be used by a minor-stream vehicle. Although it recognizes some peculiarities associated with pedestrian flows, this method takes a uniform approach to vehicular and pedestrian movements.

In using Exhibit 17-4 to compute conflicting flow rates, the analyst should carefully consult the footnotes, which allow modifications to the equations in special cases.

While regulations or practices may vary between jurisdictions, this methodology assumes that pedestrians crossing the subject or opposing approaches have Rank 1 status and that pedestrians crossing the two conflicting approaches to the left or right of the subject minor-street approach have Rank 2 status. The conflicting pedestrian flow rates are identified in Exhibit 17-4.

Exhibit 17-4 also identifies the conflicting flow rates for each stage of a two-stage gap acceptance process that takes place at some intersections where vehicles store in the median area. If a two-stage gap acceptance process is not present, the conflicting flow rates shown in the rows labeled Stage I and Stage II should be added together and considered as one conflicting flow rate for the movement in question.

CRITICAL GAP AND FOLLOW-UP TIME

Critical gap defined

The critical gap, t_c, is defined as the minimum time interval in the major-street traffic stream that allows intersection entry for one minor-street vehicle (*5*). Thus, the driver's critical gap is the minimum gap that would be acceptable. A particular driver would reject any gaps less than the critical gap and would accept gaps greater than or equal to the critical gap. Estimates of critical gap can be made on the basis of observations of the largest rejected and smallest accepted gap for a given intersection.

Follow-up time defined

The time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major-street gap, under a condition of continuous queuing on the minor street, is called the follow-up time, t_f. Thus, t_f is the headway that defines the saturation flow rate for the approach if there were no conflicting vehicles or movements of higher rank.

EXHIBIT 12-5. EXAMPLE SERVICE VOLUMES FOR MULTILANE HIGHWAYS (SEE FOOTNOTE FOR ASSUMED VALUES)

FFS (km/h)	Number of Lanes	Terrain	Service Volumes (veh/h)				
			A	B	C	D	E
100	2	Level	1200	1880	2700	3450	4060
		Rolling	1140	1800	2570	3290	3870
		Mountainous	1040	1640	2350	3010	3540
	3	Level	1800	2830	4050	5180	6100
		Rolling	1710	2700	3860	4940	5810
		Mountainous	1570	2470	3530	4520	5320
80	2	Level	960	1510	2190	2920	3520
		Rolling	910	1440	2090	2790	3360
		Mountainous	830	1310	1910	2550	3070
	3	Level	1440	2260	3290	4390	5290
		Rolling	1370	2160	3140	4190	5040
		Mountainous	1250	1970	2870	3830	4610

Notes:
Assumptions: highway with 100-km/h FFS has 5 access points/km; highway with 80-km/h FFS has 15 access points/km; lane width = 3.6 m; shoulder width = 1.8 m; divided highway; PHF = 0.88; 5 percent trucks; and regular commuters.

This table contains approximate values. It is meant for illustrative purposes only. The values are highly dependent on the assumptions and should not be used for operations analyses or final design. This table was derived from the assumed values listed in the footnote.

III. TWO-LANE HIGHWAYS

A two-lane highway is an undivided roadway with two lanes, one for use by traffic in each direction. Passing a slower vehicle requires use of the opposing lane as sight distance and gaps in the opposing traffic stream permit. As volumes and geometric restrictions increase, the ability to pass decreases and platoons form. Motorists in platoons are subject to delay because they are unable to pass. Illustration 12-9 shows a typical view of a two-lane, two-way rural highway.

ILLUSTRATION 12-9. Typical two-lane, two-way highway in a rural environment.

Two-lane highways are a key element in the highway systems of most countries. They perform a variety of functions, are located in all geographic areas, and serve a wide range of traffic. Any consideration of operating quality must account for these disparate functions.

Traffic operations on two-lane, two-way highways differ from those on other uninterrupted-flow facilities. Lane changing and passing are possible only in the face of oncoming traffic in the opposing lane. Passing demand increases rapidly as traffic volumes increase, and passing capacity in the opposing lane declines as volumes increase. Therefore, on two-lane highways, unlike other types of uninterrupted-flow facilities, normal traffic flow in one direction influences flow in the other direction. Motorists must adjust their travel speeds as volume increases and the ability to pass declines.

Efficient mobility is the principal function of major two-lane highways that connect major traffic generators or that serve as primary links in state and national highway networks. These routes tend to serve long-distance commercial and recreational travelers, and long sections may pass through rural areas without traffic-control interruptions. Consistent high-speed operations and infrequent passing delays are desirable for these facilities.

Other paved, two-lane rural highways serve for accessibility. They provide all-weather access to an area, often for relatively low traffic volumes. Cost-effective access is the dominant consideration. Although beneficial, high speed is not the principal concern. Delay—as indicated by the formation of platoons—is more relevant as a measure of service quality.

Two-lane roads also serve scenic and recreational areas in which the vista and environment are meant to be experienced and enjoyed without traffic interruption or delay. A safe roadway is desired, but high-speed operation is neither expected nor desired. For these reasons, there are two performance measures to describe service quality for two-lane highways: percent time-spent-following and average travel speed.

Percent time-spent-following defined

Car-following criterion is 3 s

Average travel speed defined

Percent time-spent-following represents the freedom to maneuver and the comfort and convenience of travel. It is the average percentage of travel time that vehicles must travel in platoons behind slower vehicles due to the inability to pass. Percent time-spent-following is difficult to measure in the field. However, the percentage of vehicles traveling with headways of less than 3 s at a representative location can be used as a surrogate measure.

Average travel speed reflects the mobility on a two-lane highway: it is the length of the highway segment divided by the average travel time of all vehicles traversing the segment in both directions during a designated interval.

LOS criteria use both these performance measures. On major two-lane highways, for which efficient mobility is paramount, both percent time-spent-following and average travel speed define LOS. However, roadway alignments with reduced design speeds will limit the LOS that can be achieved. On highways for which accessibility is paramount and mobility less critical, LOS is defined only in terms of percent time-spent-following, without consideration of average travel speed.

CLASSIFICATION OF TWO-LANE HIGHWAYS

Two-lane highways are categorized into two classes for analysis:

Class I highways

- Class I—These are two-lane highways on which motorists expect to travel at relatively high speeds. Two-lane highways that are major intercity routes, primary arterials connecting major traffic generators, daily commuter routes, or primary links in state or national highway networks generally are assigned to Class I. Class I facilities most often serve long-distance trips or provide connecting links between facilities that serve long-distance trips.

Class II highways

- Class II—These are two-lane highways on which motorists do not necessarily expect to travel at high speeds. Two-lane highways that function as access routes to Class I facilities, serve as scenic or recreational routes that are not primary arterials, or pass through rugged terrain generally are assigned to Class II. Class II facilities most often serve relatively short trips, the beginning and ending portions of longer trips, or trips for which sightseeing plays a significant role.

The classes of two-lane roads closely relate to their functions—most arterials are considered Class I, and most collectors and local roads are considered Class II. However, the primary determinant of a facility's classification in an operational analysis is the motorist's expectations, which might not agree with the functional classification. For example, an intercity route that passes through rugged mountainous terrain might be described as Class II instead of Class I if motorists recognize that a high-speed route is not feasible in that corridor.

The LOS for Class I highways on which efficient mobility is paramount is defined in terms of both percent time-spent-following and average travel speed. On Class II highways, mobility is less critical, and LOS is defined only in terms of percent time-spent-following. Drivers generally tolerate higher levels of percent time-spent-following on a Class II facility than on a Class I facility, because Class II highways usually serve shorter trips and different trip purposes.

For Class I highways, two criteria define LOS: percent time-spent-following and average travel speed. For Class II highways, LOS is based only on percent time-spent-following.

BASE CONDITIONS

The base conditions for a two-lane highway are the absence of restrictive geometric, traffic, or environmental factors. Base conditions are not the same as typical or default conditions. The methodology in Chapter 20 accounts for the effects of geometric, traffic, or environmental conditions that are more restrictive than the base conditions. The base conditions include

- Lane widths greater than or equal to 3.6 m;
- Clear shoulders wider than or equal to 1.8 m;
- No no-passing zones;
- All passenger cars;
- No impediments to through traffic, such as traffic control or turning vehicles; and
- Level terrain.

For the analysis of two-way flow (i.e., both directions), a 50/50 directional split of traffic is also considered a base condition. Most directional distribution on rural two-lane highways ranges from 50/50 to 70/30. On recreational routes, the directional distribution may be as high as 80/20 or more during holiday or other peak periods. Some variation in speed and percent time-spent-following occurs with changing directional distribution and volume. For directional analysis (i.e., separate analysis of each direction), directional distribution is not a base condition.

Traffic can operate ideally only if lanes and shoulders are wide enough not to constrain speeds. Lane and shoulder widths less than the base values of 3.6 m and 1.8 m, respectively, are likely to reduce speeds and may increase percent time-spent-following.

The frequency of no-passing zones is used to characterize roadway design and to analyze expected traffic conditions along a two-lane highway. A no-passing zone is any zone marked for no passing or any section of road with a passing sight distance of 300 m or less. The average percentage of no-passing zones in both directions along a section is used for the analysis of two-way flow. The percentage of no-passing zones for a particular direction of travel is used in directional analysis.

No-passing zone

The sight distance value of 300 m is equivalent to that used by the Manual on Uniform Traffic Control Devices for passing and no-passing zones on highways with an 85th-percentile speed of 90 km/h.

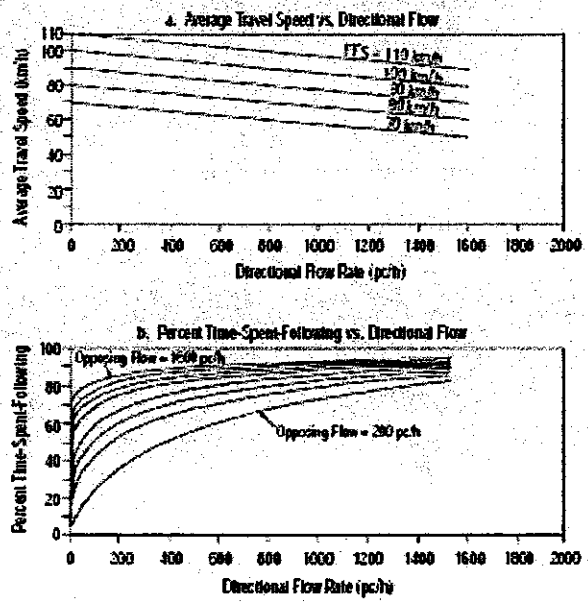
No-passing zones typically range from 20 to 50 percent of a rural two-lane highway. Values approaching 100 percent can be found on sections of winding, mountainous roads. No-passing zones have a greater effect in mountainous terrain than in level or rolling terrain. Heavy platoon formation along a highway section also can cause greater-than-expected operational problems on an adjacent downstream section with restricted passing opportunities.

BASIC RELATIONSHIPS

Exhibit 12-6 shows the relationship of flow rate, average travel speed, and percent time-spent-following for base conditions on an extended two-way facility (7).

Highway geometric features include a general description of longitudinal section characteristics and specific roadway cross-section information. Longitudinal section

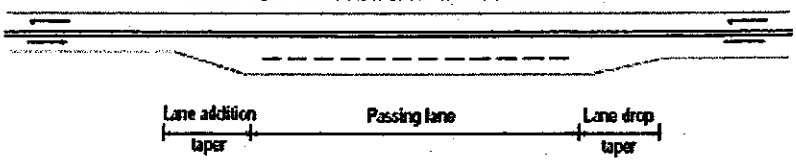
EXHIBIT 12-7. SPEED-FLOW AND PERCENT TIME-SPENT-FOLLOWING FLOW RELATIONSHIPS FOR DIRECTIONAL SEGMENTS WITH BASE CONDITIONS



PASSING LANES ON TWO-LANE HIGHWAYS

A passing lane is a lane added in one direction of travel on a conventional two-lane highway to improve opportunities for passing. The addition of a passing lane to a two-lane highway provides a three-lane cross section with two lanes in one direction of travel and one lane in the other. Exhibit 12-8 illustrates a typical passing lane on a two-lane highway. Depending on local practice, traffic in the opposing direction may be prohibited from passing or may be permitted to pass if adequate sight distance is available, as shown in Exhibit 12-8. On some two-lane highways, passing lanes are provided intermittently or at intervals for each direction of travel. On other highways, added passing lanes alternate continuously between the two directions of travel. Passing lanes also can be provided in both directions of travel at the same location, resulting in a short section of four-lane undivided highway with improved passing opportunities in both directions.

EXHIBIT 12-8. PLAN VIEW OF A TYPICAL PASSING LANE



LOS

The primary measures of service quality for Class I two-lane highways are percent time-spent-following and average travel speed. For Class II two-lane highways, service

quality is based only on percent time-spent-following. LOS criteria are defined for peak 15-min flow periods and are intended for application to segments of significant length.

LOS A describes the highest quality of traffic service, when motorists are able to travel at their desired speed. Without strict enforcement, this highest quality would result in average speeds of 90 km/h or more on two-lane highways in Class I. The passing frequency required to maintain these speeds has not reached a demanding level, so that passing demand is well below passing capacity, and platoons of three or more vehicles are rare. Drivers are delayed no more than 35 percent of their travel time by slow-moving vehicles. A maximum flow rate of 490 pc/h total in both directions may be achieved with base conditions. On Class II highways, speeds may fall below 90 km/h, but motorists will not be delayed in platoons for more than 40 percent of their travel time.

LOS B characterizes traffic flow with speeds of 80 km/h or slightly higher on level-terrain Class I highways. The demand for passing to maintain desired speeds becomes significant and approximates the passing capacity at the lower boundary of LOS B. Drivers are delayed in platoons up to 50 percent of the time. Service flow rates of 780 pc/h total in both directions can be achieved under base conditions. Above this flow rate, the number of platoons increases dramatically. On Class II highways, speeds may fall below 80 km/h, but motorists will not be delayed in platoons for more than 55 percent of their travel time.

LOS C describes further increases in flow, resulting in noticeable increases in platoon formation, platoon size, and frequency of passing impediments. The average speed still exceeds 70 km/h on level-terrain Class I highways, even though unrestricted passing demand exceeds passing capacity. At higher volumes the chaining of platoons and significant reductions in passing capacity occur. Although traffic flow is stable, it is susceptible to congestion due to turning traffic and slow-moving vehicles. Percent time-spent-following may reach 65 percent. A service flow rate of up to 1,190 pc/h total in both directions can be accommodated under base conditions. On Class II highways, speeds may fall below 70 km/h, but motorists will not be delayed in platoons for more than 70 percent of their travel time.

LOS D describes unstable traffic flow. The two opposing traffic streams begin to operate separately at higher volume levels, as passing becomes extremely difficult. Passing demand is high, but passing capacity approaches zero. Mean platoon sizes of 5 to 10 vehicles are common, although speeds of 60 km/h still can be maintained under base conditions on Class I highways. The proportion of no-passing zones along the roadway section usually has little influence on passing. Turning vehicles and roadside distractions cause major shock waves in the traffic stream. Motorists are delayed in platoons for nearly 80 percent of their travel time. Maximum service flow rates of 1,830 pc/h total in both directions can be maintained under base conditions. On Class II highways, speeds may fall below 60 km/h, but in no case will motorists be delayed in platoons for more than 85 percent of their travel time.

At LOS E, traffic flow conditions have a percent time-spent-following greater than 80 percent on Class I highways and greater than 85 percent on Class II. Even under base conditions, speeds may drop below 60 km/h. Average travel speeds on highways with less than base conditions will be slower, even down to 40 km/h on sustained upgrades. Passing is virtually impossible at LOS E, and platooning becomes intense, as slower vehicles or other interruptions are encountered.

The highest volume attainable under LOS E defines the capacity of the highway, generally 3,200 pc/h total in both directions. Operating conditions at capacity are unstable and difficult to predict. Traffic operations seldom reach near capacity on rural highways, primarily because of a lack of demand.

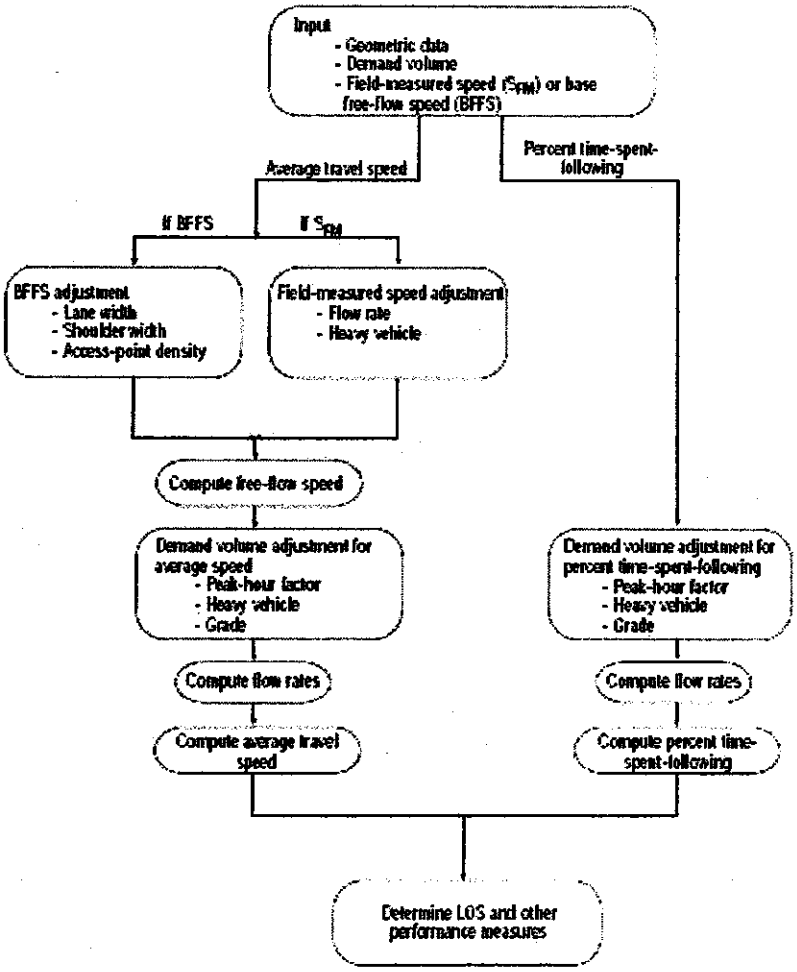
LOS F represents heavily congested flow with traffic demand exceeding capacity. Volumes are lower than capacity and speeds are highly variable.

intersections at spacings of 3.2 km or less can be evaluated with the methodology of Chapter 15, "Urban Streets."

II. METHODOLOGY

The following discussion presents estimates of two-lane highway capacity, defines the LOS for two-lane highways, and documents the methodology for operational and for planning applications. Exhibit 20-1 summarizes the basic methodology for two-lane highways.

EXHIBIT 20-1. TWO-LANE HIGHWAY METHODOLOGY



CAPACITY

The capacity of a two-lane highway is 1,700 pc/h for each direction of travel. The capacity is nearly independent of the directional distribution of traffic on the facility, except that for extended lengths of two-lane highway, the capacity will not exceed 3,200 pc/h for both directions of travel combined. For short lengths of two-lane highway—such as tunnels or bridges—a capacity of 3,200 to 3,400 pc/h for both directions of travel combined may be attained but cannot be expected for an extended length.

Capacity = 1,700 pc/h for each direction, and 3,200 for both directions combined

LEVELS OF SERVICE

The service measures for a two-lane highway are defined in Chapter 12, "Highway Concepts." On Class I highways, efficient mobility is paramount, and LOS is defined in terms of both percent time-spent-following and average travel speed. On Class II highways, mobility is less critical, and LOS is defined only in terms of percent time-spent-following, without consideration of average travel speed. Drivers will tolerate higher levels of percent time-spent-following on a Class II facility than on a Class I facility, because Class II facilities usually serve shorter trips and different trip purposes.

For definitions of the service measures for two-lane highways, percent time-spent-following, and average travel speed, see Chapter 12, "Highway Concepts"

LOS criteria for two-lane highways in Classes I and II are presented in Exhibits 20-2, 20-3, and 20-4. Exhibit 20-2 reflects the maximum values of percent time-spent-following and average travel speed for each LOS for Class I highways. A segment of a Class I highway must meet the criteria for both the percent time-spent-following and the average travel speed shown in Exhibit 20-2 to be classified in any particular LOS. Exhibit 20-3 illustrates the LOS criteria for Class I highways. For example, a Class I two-lane highway with percent time-spent-following equal to 45 percent and an average travel speed of 65 km/h would be classified as LOS D based on Exhibit 20-2. However, a Class II highway with the same conditions would be classified as LOS B based on Exhibit 20-4. The difference between these LOS assessments represents the difference in motorist expectations for Class I and II facilities.

For definitions of Class II and I highways, also see Chapter 12

The LOS criteria in Exhibits 20-2 through 20-4 apply to all types of two-lane highways, including extended two-way segments, extended directional segments, specific upgrades, and specific downgrades.

TWO-WAY SEGMENTS

The two-way segment methodology estimates measures of traffic operation along a section of highway, based on terrain, geometric design, and traffic conditions. Terrain is classified as level or rolling, as described below. Mountainous terrain is addressed in the operational analysis of specific upgrades and downgrades, presented below. This methodology typically is applied to highway sections of at least 3.0 km.

Traffic data needed to apply the two-way segment methodology include the two-way hourly volume, a peak-hour factor (PHF), and the directional distribution of traffic flow. The PHF may be computed from field data, or appropriate default values may be selected from the tabulated values presented in Chapter 12. Traffic data also include the proportion of trucks and recreational vehicles (RVs) in the traffic stream. The operational analysis of extended two-way segments for a two-lane highway involves several steps, described in the following sections.

EXHIBIT 20-2. LOS CRITERIA FOR TWO-LANE HIGHWAYS IN CLASS I

LOS	Percent Time-Spent-Following	Average Travel Speed (km/h)
A	≤ 35	> 90
B	> 35–50	> 80–90
C	> 50–65	> 70–80
D	> 65–80	> 60–70
E	> 80	≤ 60

Note:
LOS F applies whenever the flow rate exceeds the segment capacity.

BPTSF = base percent time-spent-following for both directions of travel combined (use Equation 20-7), and

f_{np} = adjustment for the combined effect of the directional distribution of traffic and of the percentage of no-passing zones on percent time-spent-following.

$$BPTSF = 100\left(1 - e^{-0.000879v_p}\right) \quad (20\text{-}7)$$

An adjustment representing the combined effect of directional distribution of traffic and percentage of no-passing zones (f_{np}) is presented in Exhibit 20-11.

EXHIBIT 20-11. ADJUSTMENT (f_{np}) FOR EFFECT OF NO-PASSING ZONES ON AVERAGE TRAVEL SPEED ON TWO-WAY SEGMENTS

	Reduction in Average Travel Speed (km/h)					
	No-Passing Zones (%)					
Two-Way Demand Flow Rate, v_p (pc/h)	0	20	40	60	80	100
0	0.0	0.0	0.0	0.0	0.0	0.0
200	0.0	1.0	2.3	3.8	4.2	5.6
400	0.0	2.7	4.3	5.7	6.3	7.3
600	0.0	2.5	3.8	4.9	5.5	6.2
800	0.0	2.2	3.1	3.9	4.3	4.9
1000	0.0	1.8	2.5	3.2	3.6	4.2
1200	0.0	1.3	2.0	2.6	3.0	3.4
1400	0.0	0.9	1.4	1.9	2.3	2.7
1600	0.0	0.9	1.3	1.7	2.1	2.4
1800	0.0	0.8	1.1	1.6	1.8	2.1
2000	0.0	0.8	1.0	1.4	1.6	1.9
2200	0.0	0.8	1.0	1.4	1.5	1.7
2400	0.0	0.8	1.0	1.3	1.5	1.7
2600	0.0	0.8	1.0	1.3	1.4	1.6
2800	0.0	0.8	1.0	1.2	1.3	1.4
3000	0.0	0.8	0.9	1.1	1.1	1.3
3200	0.0	0.8	0.9	1.0	1.0	1.1

Determining LOS

The first step in determining LOS is to compare the passenger-car equivalent flow rate (v_p) to the two-way capacity of 3,200 pc/h. If v_p is greater than the capacity, then the roadway is oversaturated and the LOS is F. Similarly, if the demand flow rate in either direction of travel—as determined from the two-way flow rate and the directional split—is greater than 1,700 pc/h, then the roadway is oversaturated and the LOS is F. In LOS F, percent time-spent-following is nearly 100 percent and speeds are highly variable and difficult to estimate.

When a segment of a Class I facility has a demand less than its capacity, the LOS is determined by locating a point on Exhibit 20-3 that corresponds to the estimated percent time-spent-following and average travel speed. If a segment of a Class II facility has a demand less than its capacity, the LOS is determined by comparing the percent time-spent-following with the criteria in Exhibit 20-4. The analysis should include the LOS and the estimated values of percent time-spent-following and average travel speed. Although average travel speed is not considered in the LOS determination for a Class II highway, the estimate may be useful in evaluating the quality of service of two-lane highway facilities, highway networks, or systems including the segment.

I. INTRODUCTION

This chapter introduces capacity and quality-of-service concepts for highway facilities with points of access that are not fully controlled. This chapter can be used in conjunction with Chapter 20, which provides a methodology for two-lane highways, and Chapter 21, which provides a methodology for multilane highways.

II. MULTILANE HIGHWAYS

Multilane highways generally have posted speed limits of 60 to 90 km/h. They usually have a total of four or six lanes, counting both directions, often with medians or two-way left-turn lanes (TWLTL); however, they also may be undivided.

Traffic signals spaced at 3.0 km or less create urban street conditions

Multilane highways typically are located in suburban communities, leading into central cities, or along high-volume rural corridors connecting two cities or two significant activities that generate a substantial number of daily trips. Such highways often have traffic signals; but traffic signals spaced at 3.0 km or less typically create urban street conditions.

Traffic volumes on multilane highways vary but might range from 15,000 to 40,000 veh/day. In some cases, volumes as high as 100,000 veh/day have been observed when access across the median is restricted and when all major crossings are grade separated.

Illustrations 12-1 through 12-4 show typical multilane highways.

ILLUSTRATION 12-1. Divided multilane highway in a rural environment.

ILLUSTRATION 12-2. Divided multilane highway in a suburban environment.

ILLUSTRATION 12-3. Undivided multilane highway in a rural environment.

ILLUSTRATION 12-4. Undivided multilane highway in a suburban environment.

MULTILANE HIGHWAY CAPACITY

Multilane highways in suburban and rural settings have different operational characteristics from freeways, urban streets, and two-lane highways. Most notably, multilane highways are not completely access controlled—they can have at-grade intersections and occasional traffic signals.

Multilane highways can be similar to freeways or can approach urban street conditions

Friction caused by opposing vehicles on undivided highways and the access to roadside development contribute to a different operational setting from that of freeways. Multilane highways range from the uninterrupted flow of freeways to the flow conditions on urban streets, which are frequently interrupted by signals.

The capacity of a multilane highway is the maximum sustained hourly flow rate at which vehicles reasonably can be expected to traverse a uniform segment under prevailing roadway and traffic conditions.

FREE-FLOW SPEED

Free-flow speed (FFS) is the speed of traffic at low volume and low density. It is the speed at which drivers feel comfortable traveling under the physical, environmental, and traffic-control conditions on an uncongested section of multilane highway. Free-flow speeds will be lower on sections of highway with restricted vertical or horizontal alignments. FFS tend to be lower when posted speed limits are lower. The importance of

The posted speed limit correlates significantly with the speed at which vehicles move along the highway. When no other estimate is available, the FFS can be calculated based on the posted speed limit.

Vehicular speeds, and the proportion of vehicles exceeding the speed limit, are affected by speed enforcement. However, several studies (5–9) have found that enforcement effects are limited, both temporally and spatially. The effect of enforcement depends on its type and duration. In general, a stationary enforcement activity affects no more than a 13 to 15 km of roadway; moreover, its effect decays exponentially downstream. Nonetheless, the speeds at the site may be affected for up to 2 to 3 days afterwards. If the roadway is located in a community that regularly enforces speed limits, local measurements can be used to calibrate the relationship between 85th-percentile speed and FFS. These measurements should be taken, therefore, when the average anticipated enforcement efforts are under way.

FACTORS AFFECTING FLOW RATE

The volume estimate is adjusted by factors relating to both the composition and fluctuation of traffic, so that all roadways can be compared with an equivalent measure of passenger cars per hour per lane (pc/h/ln).

Peak-hour factor

The basis for traffic volumes is a 15-min peak-period flow during a peak hour of the day. Therefore, volumes in two time periods are required: a peak-hour volume and the flow rate within the peak 15 min of the peak hour. The hourly volume divided by the peak-hour factor (PHF) yields a flow rate. The PHF is the ratio of total hourly volume to four times the highest 15-min volume within the peak hour.

Heavy Vehicles

The second adjustment to the volume relates to heavy vehicles—the number of trucks, buses, and recreational vehicles (RVs) are converted into an equivalent number of passenger cars. Two categories of heavy vehicles are used: trucks and RVs. For analysis purposes, buses on multilane highways are categorized as trucks.

Converting heavy vehicles to equivalent passenger cars is especially important when analyzing sections of highway with grades. On level terrain and for near-capacity conditions, trucks, buses, and RVs tend to operate like passenger cars, so that the equivalency factor approaches 1.

Driver Population

The base conditions for multilane highway flow include a driver population primarily of commuters. Studies have shown that commuter and noncommuter driver populations do not display the same characteristics. Capacities for recreational traffic can be up to 20 percent lower than for commuter traffic on the same highway; however, the FFS does not seem to be similarly affected. If this possible effect of driver population is taken into account, locally derived data should be obtained and used carefully, according to the methodology for multilane highways.

LOS

A multilane highway is characterized by three performance measures:

- Density, in terms of passenger cars per kilometer per lane;
- Speed, in terms of mean passenger car speed; and
- Volume to capacity ratio.

Each of these measures indicates how well the highway accommodates traffic flow.

Density defines LOS for multilane highways

Density is the assigned primary performance measure for estimating LOS. The three measures of speed, density, and flow or volume are interrelated. If the values of two of these measures are known, the remaining measure can be computed.

LOS A describes completely free-flow conditions. The operation of vehicles is virtually unaffected by the presence of other vehicles, and operations are constrained only

by the geometric features of the highway and by driver preferences. Maneuverability within the traffic stream is good. Minor disruptions to flow are easily absorbed without a change in travel speed.

LOS B also indicates free flow, although the presence of other vehicles becomes noticeable. Average travel speeds are the same as in LOS A, but drivers have slightly less freedom to maneuver. Minor disruptions are still easily absorbed, although local deterioration in LOS will be more obvious.

In LOS C, the influence of traffic density on operations becomes marked. The ability to maneuver within the traffic stream is clearly affected by other vehicles. On multilane highways with an FFS above 90 km/h, the travel speeds reduce somewhat. Minor disruptions can cause serious local deterioration in service, and queues will form behind any significant traffic disruption.

At LOS D, the ability to maneuver is severely restricted due to traffic congestion. Travel speed is reduced by the increasing volume. Only minor disruptions can be absorbed without extensive queues forming and the service deteriorating.

LOS E represents operations at or near capacity, an unstable level. The densities vary, depending on the FFS. Vehicles are operating with the minimum spacing for maintaining uniform flow. Disruptions cannot be dissipated readily, often causing queues to form and service to deteriorate to LOS F. For the majority of multilane highways with FFS between 70 and 100 km/h, passenger-car mean speeds at capacity range from 68 to 88 km/h but are highly variable and unpredictable.

LOS F represents forced or breakdown flow. It occurs either when vehicles arrive at a rate greater than the rate at which they are discharged or when the forecast demand exceeds the computed capacity of a planned facility. Although operations at these points—and on sections immediately downstream—appear to be at capacity, queues form behind these breakdowns. Operations within queues are highly unstable, with vehicles experiencing brief periods of movement followed by stoppages. Travel speeds within queues are generally less than 48 km/h. Note that the term LOS F may be used to characterize both the point of the breakdown and the operating condition within the queue.

For a detailed discussion of flow breakdown on uninterrupted-flow facilities, see Chapter 13, "Freeway Concepts"

Although the point of breakdown causes the queue to form, operations within the queue generally are not related to deficiencies along the highway segment.

REQUIRED INPUT DATA AND ESTIMATED VALUES

Exhibit 12-3 lists the default values that may be used for input parameters in the absence of local data. However, taking field measurements for use as inputs to an analysis is the most reliable means of generating parameter values. Only when this is not feasible should default values be considered.

Lane Width and Lateral Clearance

Field inspection, aerial photos, as-built plans, and local highway operating agency policies are sources of information for existing facilities. The standard lane width for new highway construction in the United States is 3.6 m. The standard shoulder width is 1.8 m (6). These standards may be reduced to accommodate special historical, environmental, or topographical constraints. In Canada, the standard lane width is 3.7 m.

Default values of 3.6 m for lane width and 1.8 m for lateral clearance may be used in the absence of field data or local data. If lane widths vary within a segment, either the segment should be split into subsegments of uniform width or the distance-weighted average of the lane widths should be computed and used to determine the effects on FFS

I. INTRODUCTION

The procedures in this chapter are used to analyze the capacity, level of service (LOS), lane requirements, and impacts of traffic and design features of rural and suburban multilane highways.

The methodology in this chapter is based on the results of a National Cooperative Highway Research study (*1*). The study used additional references in developing the original methodology (*2–6*), which subsequently has been updated (*7*).

For background and concepts, see Chapter 12, "Highway Concepts"

BASE CONDITIONS FOR MULTILANE HIGHWAYS

The procedures in this chapter determine the reduction in travel speed that occurs for less-than-base conditions. Under base conditions, the full speed and capacity of a multilane highway are achieved. These conditions include good weather, good visibility, and no incidents or accidents.

Studies of the flow characteristics of multilane highways have defined base conditions for developing flow relationships and adjustments to speed. The base conditions for multilane highways are as follows:

- 3.6-m minimum lane widths;
- 3.6-m minimum total lateral clearance in the direction of travel—this represents the total lateral clearances from the edge of the traveled lanes to obstructions along the edge of the road and in the median (in computations, lateral clearances greater than 1.8 m are considered in computations to be equal to 1.8 m);
- Only passenger cars in the traffic stream;
- No direct access points along the roadway;
- A divided highway; and
- Free-flow speed (FFS) higher than 100 km/h.

These base conditions represent the highest operating level of multilane rural and suburban highways.

LIMITATIONS OF THE METHODOLOGY

The methodology in this chapter does not take into account the following conditions:

- Transitory blockages caused by construction, accidents, or railroad crossings;
- Interference caused by parking on the shoulders (such as in the vicinity of a country store, flea market, or tourist attraction);
- Three-lane cross sections;
- The effect of lane drops and additions at beginning or end of segments;
- Possible queuing delays when transitions from a multilane segment into a two-lane segment are neglected;
- Differences between median barriers and two-way left-turn lanes; and
- FFS below 70 km/h or above 100 km/h.

II. METHODOLOGY

The methodology described in this chapter is intended for analysis of uninterrupted-flow highway segments. Chapter 15 presents the methodology for analyzing urban streets that have one or more of the following characteristics:

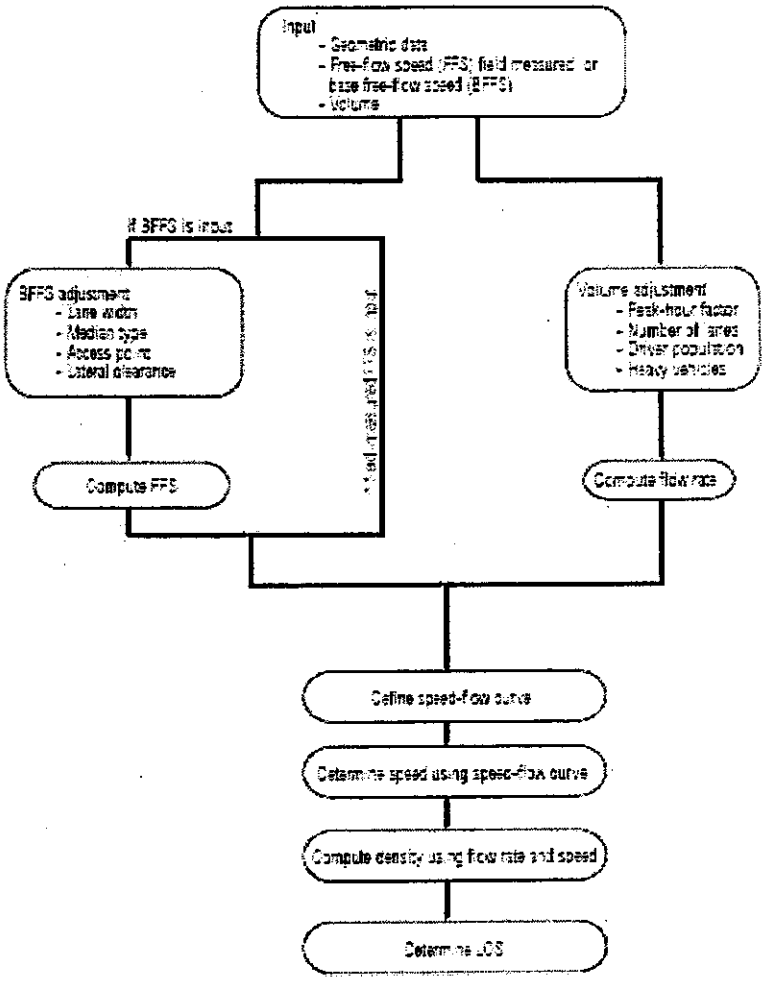
Methodology applies to signal spacing greater than 3.0 km

- Flow significantly influenced by other signals (i.e., a signal spacing less than or equal to 3.0 km).
- Significant presence of on-street parking,
- Presence of bus stops that have significant use, or
- Significant pedestrian activity.

Exhibit 21-1 illustrates the inputs and the basic computational order for the method described in this chapter. The primary output is LOS.

Uninterrupted-flow facilities that allow access solely through a system of on-ramps and off-ramps from grade separations or service roads are considered freeways and should be evaluated using the methodology presented in Chapter 23.

EXHIBIT 21-1. MULTILANE HIGHWAY METHODOLOGY



LOS

Although speed is a major concern of drivers, freedom to maneuver within the traffic stream and the proximity to other vehicles are also important. LOS criteria are listed in Exhibit 21-2. The criteria are based on the typical speed-flow and density-flow relationships shown in Exhibits 12-1 and 12-2. Exhibit 21-3 shows LOS boundaries as sloped lines, each corresponding to a constant value of density.

EXHIBIT 21-2. LOS CRITERIA FOR MULTILANE HIGHWAYS

		LOS				
Free-Flow Speed	Criteria	A	B	C	D	E
100 km/h	Maximum density (pc/km/ln)	7	11	16	22	28
	Average speed (km/h)	100.0	100.0	99.4	91.5	88.0
	Maximum volume to capacity ratio (v/c)	0.32	0.50	0.72	0.92	1.00
	Maximum service flow rate (pc/h/ln)	700	1100	1575	2015	2200
90 km/h	Maximum density (pc/km/ln)	7	11	16	22	28
	Average speed (km/h)	90.0	90.0	89.8	84.7	80.8
	Maximum v/c	0.30	0.47	0.68	0.89	1.00
	Maximum service flow rate (pc/h/ln)	630	990	1435	1860	2100
80 km/h	Maximum density (pc/km/ln)	7	11	16	22	27
	Average speed (km/h)	80.0	80.0	80.0	77.6	74.1
	Maximum v/c	0.29	0.44	0.64	0.85	1.00
	Maximum service flow rate (pc/h/ln)	560	880	1280	1705	2000
70 km/h	Maximum density (pc/km/ln)	7	11	16	22	29
	Average speed (km/h)	70.0	70.0	70.0	59.6	57.9
	Maximum v/c	0.25	0.41	0.59	0.81	1.00
	Maximum service flow rate (pc/h/ln)	490	770	1120	1550	1900

Notes:
The exact mathematical relationship between density and volume to capacity ratio (v/c) has not always been maintained at LOS boundaries because of the use of rounded values. Density is the primary determinant of LOS. LOS F is characterized by highly unstable and variable traffic flow. Prediction of accurate flow rate, density, and speed at LOS F is difficult.

The LOS criteria reflect the shape of the speed-flow and density-flow curves, particularly as speed remains relatively constant across LOS A to D but is reduced as capacity is approached. For FFS of 100, 90, 80, and 70 km/h, Exhibit 21-2 gives the average speed, the maximum value of v/c, the maximum density, and the corresponding maximum service flow rate for each LOS.

As with other LOS criteria, the maximum service flow rates in Exhibit 21-2 are stated in terms of flow rate based on the peak 15-min volume. Demand or forecast hourly volumes generally are divided by the peak-hour factor (PHF) to reflect a maximum hourly flow rate before comparison with the criteria of Exhibit 21-2. Using the basic speed-flow curves (see Exhibit 21-3), the relationships between LOS, flow, and speed can be analyzed.

DETERMINING FFS

FFS is measured using the mean speed of passenger cars operating in low-to-moderate flow conditions (up to 1,400 pc/h/ln). Mean speed is virtually constant across this range of flow rates. Field measurement and estimation with guidelines provided in this chapter are methods that can be used to determine FFS.

The field measurement procedure is for those who prefer to gather data directly or to incorporate the measurements into a speed-monitoring program. However, field measurements are not necessary to apply the method.

The FFS of a highway can be determined directly from a speed study conducted in the field. If field-measured data are used, no adjustments need to be made to FFS. The speed study should be conducted along a reasonable length of highway within the segment under evaluation; for example, an upgrade should not be selected within a site that is generally level. Any speed measurement technique acceptable for other types of traffic engineering speed studies can be used.

The field study should be conducted in the more stable regime of low-to-moderate flow conditions (up to 1,400 pc/h/ln). If the speed study must be conducted at a flow rate of more than 1,400 pc/h/ln, the FFS can be found by using the model speed-flow curve, assuming that data on traffic volumes are recorded at the same time.

FFS occurs at flow rates of 1,400 pc/h/ln

EXHIBIT 21-11. PASSENGER-CAR EQUIVALENTS FOR TRUCKS ON DOWNGRADES

Downgrade (%)	Length (km)	E_T			
		Percentage of Trucks			
		5	10	15	20
< 4	All	1.5	1.5	1.5	1.5
4–5	≤ 5.4	1.5	1.5	1.5	1.5
4–5	> 5.4	2.0	2.0	2.0	1.5
> 5–6	≤ 5.4	1.5	1.5	1.5	1.5
> 5–6	> 5.4	3.5	4.0	4.0	3.0
> 6	≤ 5.4	1.5	1.5	1.5	1.5
> 6	> 5.4	7.5	5.0	5.5	4.5

Driver Population Factor

The adjustment factor f_p reflects the effect weekend recreational and perhaps even midday drivers have on the facility. The values for f_p range from 0.85 to 1.00. Typically, the analyst should select 1.00, which reflects weekday commuter traffic (i.e., users familiar with the highway), unless there is sufficient evidence that a lesser value, reflecting more recreational or weekend traffic characteristics, should be applied. When greater accuracy is needed, comparative field studies of weekday and weekend traffic flow and speeds are recommended.

DETERMINING LOS

The LOS on a multilane highway can be determined directly from Exhibit 21-3 on the basis of the FFS and the service flow rate (v_p) in pc/h/ln. The procedure is as follows:

- Step 1. Define and segment the highway as appropriate.
- Step 2. On the basis of the measured or estimated FFS, construct an appropriate speed-flow curve of the same shape as the typical curves shown in Exhibit 21-3. The curve should intercept the y-axis at the FFS.
- Step 3. Based on the flow rate v_p, read up to the FFS curve identified in Step 2 and determine the average passenger-car speed and LOS corresponding to that point.
- Step 4. Determine the density of flow according to Equation 21-5.

$$D = \frac{v_p}{S} \tag{21-5}$$

where

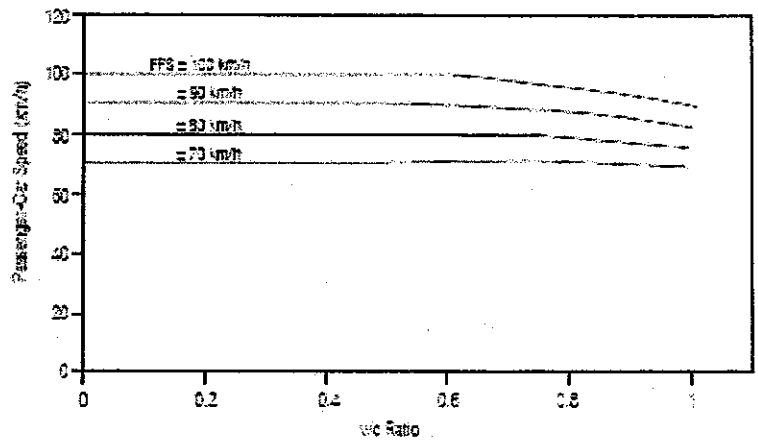
D = density (pc/km/ln),
v_p = flow rate (pc/h/ln), and
S = average passenger-car travel speed (km/h).

The LOS also can be determined by comparing the computed density with the density ranges provided in Exhibit 21-2.

SENSITIVITY OF RESULTS TO INPUT VARIABLES

Exhibit 21-12 shows the impact of v/c ratios on passenger-car speed for multilane highways. Note that speed is insensitive to demand until demand is at least 70 percent of capacity; also note that the mean speed on lower-speed segments is not sensitive to demand until the demand reaches at least 90 percent of capacity.

EXHIBIT 21-12. EFFECT OF v/c RATIO ON MEAN SPEED



III. APPLICATIONS

For guidelines on required inputs and estimated values, see Chapter 12, "Highway Concepts"

The methodology of this chapter can be used to analyze the capacity and LOS of multilane highways. The analyst must address two fundamental questions. First, the primary output must be identified. Primary outputs typically solved for in a variety of applications include LOS, number of lanes required (N), and flow rate achievable (v_p). Performance measures related to density (D) and speed (S) are also achievable but are considered secondary outputs.

Second, the analyst must identify the default values or estimated values for use in the analysis. Basically, the analyst has three sources of input data:

1. Default values found in this manual;
2. Estimates and locally derived default values developed by the user; and
3. Values derived from field measurements and observation.

For each of the input variables, a value must be supplied to calculate the outputs, both primary and secondary.

A common application of the method is to compute the LOS of an existing segment or of a changed segment in the near term or distant future. This type of application is often termed operational, and its primary output is LOS, with secondary outputs for density and speed. Another application is to check the adequacy or to recommend the required number of lanes for a multilane highway given the volume or flow rate and LOS goal. This is termed a design application since its primary output is the number of lanes required to serve the assumed conditions. Other outputs from this application include speed and density. Finally, the achievable flow rate v_p can be calculated as a primary output. This analysis requires stating a LOS goal and a number of lanes as inputs. This analysis typically estimates the point at which a flow rate will cause the highway to operate at an unacceptable LOS.

Another general type of analysis can be termed planning. These analyses use estimates, *Highway Capacity Manual* (HCM) default values, and local default values as inputs in the calculation. As outputs, LOS, number of lanes, or flow rate can be determined, along with the secondary outputs of density and speed. The difference between planning analysis and operational or design analysis is that most or all of the input values in planning come from estimates or default values, but the operational and design analyses tend to use field measurements or known values for most or all of the

input variables. Note that for each of the analyses, FFS, either measured or estimated, is required as an input for the computation.

SEGMENTING THE HIGHWAY

The procedures described in this chapter are best applied to homogeneous segments of roadway, for which the variables affecting travel speeds are constant. Therefore, it is often necessary for the analyst to divide a section of highway into separate segments for analysis. The following conditions generally necessitate segmenting the highway:

- A change in the basic number of travel lanes along the highway,
- A change in the median treatment along the highway,
- A change of grade of 2 percent or more or a constant upgrade over 1220 m,
- The presence of a traffic signal or a stop sign along the multilane highway,
- A significant change in the density of access points,
- A change in speed limits, and
- The presence of a bottleneck condition.

In general, when segmenting a highway for analysis, the minimum length of a study segment should be 760 m. Also, the limits of study segments should be no closer than 0.4 km to a signalized intersection. The procedures in this chapter are based on average conditions observed over an extended highway segment with generally consistent physical characteristics.

Study segments should be at least 760 m long and 0.4 km from a signal.

COMPUTATIONAL STEPS

The multilane highways worksheet for computations is shown in Exhibit 21-13. For all applications, the analyst provides general information and site information.

Operational (LOS) analysis

For operational (LOS) analysis, all speed and flow data are entered as inputs. Equivalent flow is then computed with the aid of the exhibits for passenger-car equivalencies. FFS is estimated by adjusting a base FFS. Finally, LOS is determined by entering (with v_p) the speed-flow graph at the top of the worksheet and intersecting the specific curve that has been selected or constructed for the highway segment.

This point of intersection identifies the LOS and, on the vertical axis of the graph, the estimated speed S. If the analyst requires a value for density D, it is calculated as v_p/S.

Design (N) analysis

The key to design analysis for number of lanes N is establishing an hourly volume. All information, with the exception of number of lanes, can be entered in the flow input and speed input portion of the worksheet (see Exhibit 21-13). An FFS, either computed or measured directly, is entered on the worksheet. The appropriate curve representing the FFS is established on the graph. The required or desired LOS is also entered. Then, the analyst assumes N and computes flow v_p with the aid of the exhibits for passenger-car equivalencies. LOS is determined by entering the speed-flow graph with v_p at the top of the worksheet. The derived LOS is compared with the desired LOS. This process is then repeated, adding one lane to the previously assumed number of lanes, until the determined LOS matches or is better than the desired LOS. Density is calculated using v_p and S.

Design (v_p) analysis

The objective of design analysis for flow rate v_p is to estimate the flow rate in pc/h/ln given a set of traffic, roadway, and FFS conditions. A desired LOS is entered on the worksheet. Then, the FFS of the segment is established using either the base FFS and the four adjustment factors or an FFS measured in the field. Once this segment speed-flow curve is established, the analyst can determine what flow rate is achievable with the given LOS. This would be considered the maximum flow rate achievable or allowable for the given level. Also directly available from the graph is the average passenger-car speed. Finally, if required, a value for density can be calculated, using v_p and S.

PLANNING APPLICATIONS

Planning (LOS), Planning (v_p) and Planning (N) applications

The three planning applications—planning for LOS, flow rate v_p, and number of lanes N—correspond directly to the procedures described for operations and design. The

I. INTRODUCTION

Three basic variables—volume or flow rate, speed, and density—can be used to describe traffic on any roadway. In this manual, volume or traffic flow is a parameter common to both uninterrupted- and interrupted-flow facilities, but speed and density apply primarily to uninterrupted flow. Some parameters related to flow rate, such as spacing and headway, also are used for both types of facilities; other parameters, such as saturation flow or gap, are specific to interrupted flow.

II. UNINTERRUPTED FLOW

VOLUME AND FLOW RATE

Basic concepts for uninterrupted-flow facilities: volume, flow rate, speed, density, headway, and capacity

Volume and flow rate are two measures that quantify the amount of traffic passing a point on a lane or roadway during a given time interval. These terms are defined as follows:

- Volume—the total number of vehicles that pass over a given point or section of a lane or roadway during a given time interval; volumes can be expressed in terms of annual, daily, hourly, or subhourly periods.
- Flow rate—the equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval of less than 1 h, usually 15 min.

Volume and flow are variables that quantify demand, that is, the number of vehicle occupants or drivers (usually expressed as the number of vehicles) who desire to use a given facility during a specific time period. Congestion can influence demand, and observed volumes sometimes reflect capacity constraints rather than true demand.

The distinction between volume and flow rate is important. Volume is the number of vehicles observed or predicted to pass a point during a time interval. Flow rate represents the number of vehicles passing a point during a time interval less than 1 h, but expressed as an equivalent hourly rate. A flow rate is the number of vehicles observed in a subhourly period, divided by the time (in hours) of the observation. For example, a volume of 100 vehicles observed in a 15-min period implies a flow rate of 100 veh/0.25 h or 400 veh/h.

Volume and flow rate can be illustrated by the volumes observed for four consecutive 15-min periods. The four counts are 1,000, 1,200, 1,100, and 1,000. The total volume for the hour is the sum of these counts, or 4,300 veh. The flow rate, however, varies for each 15-min period. During the 15-min period of maximum flow, the flow rate is 1,200 veh/0.25 h, or 4,800 veh/h. Note that 4,800 vehicles do not pass the observation point during the study hour, but they do pass at that rate for 15 min.

Consideration of peak flow rates is important in capacity analysis. If the capacity of the segment of highway studied is 4,500 veh/h, capacity would be exceeded during the peak 15-min period of flow, when vehicles arrive at a rate of 4,800 veh/h, even though volume is less than capacity during the full hour. This is a serious problem, because dissipating a breakdown of capacity can extend congestion for up to several hours.

Peak flow rates and hourly volumes produce the peak-hour factor (PHF), the ratio of total hourly volume to the peak flow rate within the hour, computed by Equation 7-1:

Calculating a peak-hour factor

$$PHF = \frac{\text{Hourly volume}}{\text{Peak flow rate (within the hour)}} \qquad (7\text{-}1)$$

If 15-min periods are used, the PHF may be computed by Equation 7-2:

$$PHF = \frac{V}{4 \times V_{15}} \qquad (7\text{-}2)$$

where

PHF = peak-hour factor,
V = hourly volume (veh/h), and
V_{15} = volume during the peak 15 min of the peak hour (veh/15 min).

When the PHF is known, it can convert a peak-hour volume to a peak flow rate, as in Equation 7-3:

$$v = \frac{V}{PHF} \qquad (7\text{-}3)$$

where

v = flow rate for a peak 15-min period (veh/h),
V = peak-hour volume (veh/h), and
PHF = peak-hour factor.

Equation 7-3 does not need to be used to estimate peak flow rates if traffic counts are available; however, the chosen count interval must identify the maximum 15-min flow period. The rate then can be computed directly as 4 times the maximum 15-min count. When flow rates in terms of vehicles are known, a conversion to a flow rate in terms of passenger car equivalents (pce) can be computed using the PHF and the heavy vehicle factor.

SPEED

Although traffic volumes provide a method of quantifying capacity values, speed (or its reciprocal of travel time) is an important measure of the quality of the traffic service provided to the motorist. It is an important measure of effectiveness defining levels of service for many types of facilities, such as rural two-lane highways, urban streets, freeway weaving segments, and others.

Speed parameters

Speed is defined as a rate of motion expressed as distance per unit of time, generally as kilometers per hour (km/h). In characterizing the speed of a traffic stream, a representative value must be used, because a broad distribution of individual speeds is observable in the traffic stream. In this manual, average travel speed is used as the speed measure because it is easily computed from observation of individual vehicles within the traffic stream and is the most statistically relevant measure in relationships with other variables. Average travel speed is computed by dividing the length of the highway, street section, or segment under consideration by the average travel time of the vehicles traversing it. If travel times $t_1, t_2, t_3, \ldots, t_n$ (in hours) are measured for n vehicles traversing a segment of length L, the average travel speed is computed using Equation 7-4.

$$S = \frac{nL}{\sum_{i=1}^{n} t_i} = \frac{L}{\frac{1}{n}\sum_{i=1}^{n} t_i} = \frac{L}{t_a} \qquad (7\text{-}4)$$

where

S = average travel speed (km/h),
L = length of the highway segment (km),
t_i = travel time of the ith vehicle to traverse the section (h),
n = number of travel times observed, and
t_a = $\frac{1}{n}\sum_{i=1}^{n} t_i$ = average travel time over L (h).

The travel times in this computation include stopped delays due to fixed interruptions or traffic congestion. They are total travel times to traverse the defined roadway length.

Several different speed parameters can be applied to a traffic stream. These include the following:

vehicles on the roadway that has the right-of-way at an unsignalized intersection. Gap acceptance describes the completion of a vehicle's movement into a gap.

The capacity of a minor street approach depends on two factors:

- The distribution of available gaps in the major-street traffic stream, and
- The gap sizes required by minor-street drivers to execute their desired movements.

Critical gap and gap acceptance

The distribution of available gaps in the major-street traffic stream depends on the total volume on the street, its directional distribution, the number of lanes on the major street, and the degree and type of platooning in the traffic stream. The gap sizes required by the minor-street drivers depend on the type of maneuver (left, through, right), the number of lanes on the major street, the speed of major-street traffic, sight distances, the length of time the minor-street vehicle has been waiting, and driver characteristics (eyesight, reaction time, age, etc.). The critical gap is the minimum time interval between the front bumpers of two successive vehicles in the major traffic stream that will allow the entry of one minor-street vehicle. When more than one minor-street vehicle uses one major-street gap, the time headway between the two minor-street vehicles is called follow-up time. In general, the follow-up time is shorter than the critical gap. Roundabouts operate similarly to two-way stop-controlled intersections. In roundabouts, however, entering drivers scan only one stream of traffic—the circulating stream—for an acceptable gap.

At an all-way stop-controlled intersection, all drivers must come to a complete stop. The decision to proceed is based in part on the rules of the road, which suggest that the driver on the right has the right-of-way; it also is a function of the traffic condition on the other approaches. The departure headway for the subject approach is defined as the time between the departure of one vehicle and that of the next behind it. A departure headway is considered a saturation headway if the second vehicle stops behind the first at the stop line. If there is traffic on one approach only, vehicles can depart as rapidly as the drivers can safely accelerate into and clear the intersection. If traffic is present on other approaches, the saturation headway on the subject approach will increase, depending on the degree of conflict between vehicles.

As at signalized intersections, the front axles of two consecutive vehicles are the reference points for determining the saturation headways of the vehicles departing from the stop line of two-way and all-way stop-controlled intersection approaches. In measuring the unobstructed flow of vehicles on the major roadway at a two-way stop-controlled intersection, the reference points normally are the front bumpers.

SPEED

For interrupted-flow conditions, delay rather than speed is the primary measure of operations. However, speed measures similar to those for uninterrupted flow are helpful in determining the added travel time due to deceleration, movement in queues, and acceleration of vehicles passing through an intersection.

DELAY

Control delay

Delay is a critical performance measure on interrupted-flow facilities. There are several types of delay, but in this manual, control delay is the principal service measure for evaluating LOS at signalized and unsignalized intersections. Although the definition of control delay is the same for signalized and unsignalized intersections, its application, including LOS threshold values, differs.

Control delay involves movements at slower speeds and stops on intersection approaches, as vehicles move up in the queue or slow down upstream of an intersection. Drivers frequently reduce speed when a downstream signal is red or there is a queue at the downstream intersection approach. Control delay requires the determination of a realistic average speed for each roadway segment. Any estimate of the average travel speed on urban streets implies the effects of control delay.

At two-way stop-controlled and all-way stop-controlled intersections, control delay is the total elapsed time from a vehicle joining the queue until its departure from the stopped position at the head of the queue. The control delay also includes the time required to decelerate to a stop and to accelerate to the free-flow speed.

SATURATION FLOW RATE AND LOST TIME

Computing saturation flow rate and lost time

Saturation flow rate is defined as the flow rate per lane at which vehicles can pass through a signalized intersection. By definition, it is computed by Equation 7-9:

$$s = \frac{3600}{h} \qquad (7\text{-}9)$$

where

s = saturation flow rate (veh/h), and
h = saturation headway (s).

The saturation flow rate represents the number of vehicles per hour per lane that can pass through a signalized intersection if the green signal was available for the full hour, the flow of vehicles was never halted, and there were no large headways.

Each time a flow is stopped, it must start again, with the first four vehicles experiencing the start-up reaction and acceleration headways shown in Exhibit 7-3. In this exhibit, the first four vehicles in the queue encounter headways longer than the saturation headway, h. The increments, t_i, are called start-up lost times. The total start-up lost time for the vehicles is the sum of the increments, computed using Equation 7-10:

$$l_1 = \sum_{i=1}^{N} t_i \qquad (7\text{-}10)$$

where

l_1 = total start-up lost time (s),
t_i = lost time for ith vehicle in queue (s), and
N = last vehicle in queue.

Each stop of a stream of vehicles is another source of lost time. When one stream of vehicles stops, safety requires some clearance time before a conflicting stream of traffic is allowed to enter the intersection. This interval when no vehicles use the intersection is called clearance lost time, l_2.

In practice, signal cycles provide for this clearance through change intervals, which can include yellow or all-red indications or both. Drivers generally cannot observe this entire interval but can use the intersection during some portion of it. The clearance lost time, l_2, is the portion of this change interval not used by drivers.

The relationship between saturation flow rate and lost times is a critical one. For any given lane or movement, vehicles use the intersection at the saturation flow rate for a period equal to the available green time plus the change interval minus the start-up and clearance lost times. Because lost time is experienced with each start and stop of a movement, the total amount of time lost over an hour is related to the signal timing. For instance, if a signal has a fifty-cycle length, it will start and stop each movement 60 times per hour, and the total lost time per movement will be $60(l_1 + l_2)$.

Lost time affects capacity and delay. It might appear that the capacity of an intersection would increase with increased cycle length. But this is offset somewhat by the observation that the saturation headway, h, can increase if the length of a continuous green indication increases. Other intersection features, such as turning lanes, also can offset the reduction in capacity due to short cycles. Longer cycle lengths increase the number of vehicles in the queues and can cause the left-turn lane to overflow, reducing capacity by blocking the through-lanes.

Apéndice 3. Datos de Flujo Vehicular Tomados Durante el Conteo

Intersección PR-941 y PR-9941

Termina Período	S-N	S-E	N-S	N-E	E-S	E-N	Total
12:15 AM	1	0	1	0	0	0	2
12:30 AM	1	0	0	0	1	0	2
12:45 AM	1	0	0	0	0	0	1
01:00 AM	1	0	0	0	1	0	2
01:15 AM	2	0	1	0	2	0	5
01:30 AM	1	0	0	0	0	0	1
01:45 AM	2	0	1	0	1	1	5
02:00 AM	0	0	1	0	0	0	1
02:15 AM	1	0	1	0	0	0	2
02:30 AM	0	0	0	0	0	0	0
02:45 AM	0	0	1	0	0	0	1
03:00 AM	0	0	0	0	0	0	0
03:15 AM	1	0	0	0	0	0	1
03:30 AM	0	0	1	0	0	0	1
03:45 AM	0	0	1	0	1	0	2
04:00 AM	1	0	1	0	0	0	2
04:15 AM	0	0	4	0	0	0	4
04:30 AM	1	0	1	0	3	0	5
04:45 AM	0	0	1	0	4	0	5
05:00 AM	2	0	5	0	0	0	7
05:15 AM	5	0	9	0	0	1	15
05:30 AM	6	0	7	0	2	2	17
05:45 AM	6	0	10	0	3	3	22
06:00 AM	11	0	12	0	3	3	29
06:15 AM	3	0	17	0	4	2	26
06:30 AM	13	0	15	0	4	4	36
06:45 AM	5	0	12	0	4	3	24
07:00 AM	11	0	23	0	4	4	42
07:15 AM	7	0	15	0	3	3	28
07:30 AM	21	0	34	0	0	4	59
07:45 AM	24	0	34	0	0	0	58
08:00 AM	6	2	38	0	0	0	46
08:15 AM	2	3	31	3	0	0	39
08:30 AM	10	1	26	1	0	0	38
08:45 AM	16	0	27	0	4	0	47
09:00 AM	16	1	17	1	0	0	35
09:15 AM	14	0	24	0	0	5	43
09:30 AM	14	1	26	1	0	0	42
09:45 AM	14	0	25	1	0	0	40
10:00 AM	11	0	9	0	1	2	23
10:15 AM	11	0	12	0	5	4	32
10:30 AM	15	1	19	2	0	0	37
10:45 AM	17	0	23	0	0	0	40
11:00 AM	10	2	16	3	0	0	31
11:15 AM	19	0	20	2	0	1	42
11:30 AM	14	1	22	0	0	0	37
11:45 AM	10	2	19	0	0	0	31

12:00 PM	18	0	19	1	0	0	38
12:15 PM	9	2	18	1	0	0	30
12:30 PM	24	0	15	0	9	0	48
12:45 PM	13	1	18	1	0	0	33
01:00 PM	5	3	23	4	0	0	35
01:15 PM	2	3	23	3	0	0	31
01:30 PM	23	0	13	2	0	0	38
01:45 PM	17	1	18	0	1	0	37
02:00 PM	11	0	15	0	0	0	26
02:15 PM	15	1	13	1	0	0	30
02:30 PM	14	0	12	3	0	0	29
02:45 PM	15	2	19	2	0	0	38
03:00 PM	8	3	17	5	0	0	33
03:15 PM	20	1	17	1	0	0	39
03:30 PM	23	0	21	0	2	0	46
03:45 PM	23	2	12	3	0	0	40
04:00 PM	29	0	22	1	0	0	52
04:15 PM	32	0	33	1	0	2	68
04:30 PM	20	1	23	0	0	0	44
04:45 PM	22	0	21	0	4	0	47
05:00 PM	9	3	19	6	0	0	37
05:15 PM	4	3	25	5	0	0	37
05:30 PM	9	3	11	7	0	0	30
05:45 PM	29	1	19	4	0	0	53
06:00 PM	21	1	20	5	0	0	47
06:15 PM	23	1	25	0	0	0	49
06:30 PM	12	3	27	4	0	0	46
06:45 PM	5	2	16	4	0	0	27
07:00 PM	24	1	19	3	0	0	47
07:15 PM	24	2	29	6	0	0	61
07:30 PM	13	1	23	4	0	0	41
07:45 PM	12	2	22	4	0	0	40
08:00 PM	14	0	13	0	0	0	27
08:15 PM	5	1	13	4	0	0	23
08:30 PM	7	2	19	3	0	0	31
08:45 PM	17	0	12	0	1	0	30
09:00 PM	10	0	21	0	0	0	31
09:15 PM	10	2	19	3	0	0	34
09:30 PM	14	1	20	1	0	0	36
09:45 PM	18	1	7	1	0	0	27
10:00 PM	17	0	5	0	1	1	24
10:15 PM	9	0	3	0	0	2	14
10:30 PM	11	0	8	0	1	1	21
10:45 PM	13	0	1	0	2	0	16
11:00 PM	8	0	1	1	0	0	10
11:15 PM	3	0	5	0	0	0	8
11:30 PM	3	0	2	0	2	3	10
11:45 PM	7	0	2	0	0	0	9
12:00 PM	6	0	4	0	1	1	12
Total	1026	64	1344	108	74	52	2668

Intersección PR-941 y Camino Félix Díaz

Termina Período	O-S	O-N	N-O	N-S	S-O	S-N	Total
12:15 AM	0	0	0	1	0	1	2
12:30 AM	0	0	0	2	1	1	4
12:45 AM	0	0	0	1	0	0	1
01:00 AM	0	0	1	0	1	1	3
01:15 AM	0	0	1	1	0	0	2
01:30 AM	0	0	0	0	1	3	4
01:45 AM	0	0	1	1	0	1	3
02:00 AM	0	0	1	0	1	0	2
02:15 AM	0	0	0	1	0	0	1
02:30 AM	0	0	0	1	0	0	1
02:45 AM	0	0	0	1	1	0	2
03:00 AM	0	0	0	0	1	0	1
03:15 AM	0	0	0	1	0	0	1
03:30 AM	0	0	0	1	1	0	2
03:45 AM	0	0	0	2	1	1	4
04:00 AM	0	0	0	3	0	1	4
04:15 AM	0	0	1	2	1	0	4
04:30 AM	0	0	0	4	1	2	7
04:45 AM	0	0	0	4	1	2	7
05:00 AM	0	0	0	7	1	2	10
05:15 AM	0	0	0	12	1	5	18
05:30 AM	0	0	0	13	0	4	17
05:45 AM	0	0	0	25	0	1	26
06:00 AM	0	0	0	36	0	10	46
06:15 AM	0	0	0	29	0	10	39
06:30 AM	0	0	0	41	0	12	53
06:45 AM	1	0	0	34	0	14	49
07:00 AM	0	0	0	39	2	12	53
07:15 AM	0	0	0	44	0	16	60
07:30 AM	0	0	0	62	0	15	77
07:45 AM	0	0	0	45	0	16	61
08:00 AM	0	0	0	47	0	27	74
08:15 AM	1	0	2	29	0	22	54
08:30 AM	0	0	0	29	0	30	59
08:45 AM	0	0	2	27	0	22	51
09:00 AM	1	0	0	28	0	17	46
09:15 AM	0	0	0	41	0	13	54
09:30 AM	0	0	0	29	0	13	42
09:45 AM	0	0	2	22	0	14	38
10:00 AM	0	0	0	15	2	12	29
10:15 AM	1	1	1	16	0	20	39
10:30 AM	1	1	0	19	0	11	32
10:45 AM	1	2	2	12	0	22	39
11:00 AM	0	0	0	17	0	12	29
11:15 AM	1	1	0	26	2	19	49
11:30 AM	1	1	0	25	0	23	50
11:45 AM	0	0	2	12	0	25	39
12:00 PM	0	0	2	14	0	38	54

12:15 PM	1	0	0	20	0	22	43
12:30 PM	0	0	2	20	2	25	49
12:45 PM	1	0	0	29	0	27	57
01:00 PM	0	0	2	21	0	34	57
01:15 PM	2	1	2	21	0	32	58
01:30 PM	2	2	0	19	2	22	47
01:45 PM	0	0	0	24	0	29	53
02:00 PM	1	0	0	32	0	31	64
02:15 PM	1	0	0	18	0	31	50
02:30 PM	1	0	0	20	0	32	53
02:45 PM	1	0	0	18	0	30	49
03:00 PM	3	1	0	29	0	27	60
03:15 PM	1	0	0	31	0	36	68
03:30 PM	1	1	2	16	0	57	77
03:45 PM	2	1	0	33	0	43	79
04:00 PM	0	0	0	22	2	30	54
04:15 PM	1	1	0	15	0	55	72
04:30 PM	0	0	0	32	0	34	66
04:45 PM	1	0	0	26	0	37	64
05:00 PM	0	0	0	25	0	47	72
05:15 PM	1	0	2	23	0	39	65
05:30 PM	0	0	0	29	0	37	66
05:45 PM	0	0	0	27	1	30	58
06:00 PM	0	1	2	18	0	29	50
06:15 PM	1	0	0	25	0	29	55
06:30 PM	1	0	2	24	0	33	60
06:45 PM	1	0	0	28	0	20	49
07:00 PM	1	1	0	22	0	28	52
07:15 PM	0	0	0	33	2	26	61
07:30 PM	2	1	0	34	0	31	68
07:45 PM	1	0	0	23	0	19	43
08:00 PM	1	1	0	20	0	34	56
08:15 PM	2	1	0	17	0	24	44
08:30 PM	1	1	1	12	0	24	39
08:45 PM	1	1	1	12	0	15	30
09:00 PM	0	1	1	6	0	16	24
09:15 PM	0	1	1	10	0	22	34
09:30 PM	0	1	1	13	0	25	40
09:45 PM	0	0	0	12	0	13	25
10:00 PM	0	1	1	5	1	16	24
10:15 PM	1	2	1	1	0	19	24
10:30 PM	0	1	0	9	1	10	21
10:45 PM	0	0	0	14	0	11	25
11:00 PM	0	0	0	3	1	10	14
11:15 PM	0	0	0	7	0	9	16
11:30 PM	0	0	1	1	1	5	8
11:45 PM	0	0	1	0	1	7	9
12:00 PM	0	0	1	3	1	6	11
Total	41	27	42	1723	34	1738	3605

Intersección PR-941 y PR-942

Termina Período	S-O	S-E	O-S	O-E	E-S	E-O	Total
12:15 AM	1	2	0	1	3	0	7
12:30 AM	2	2	2	0	0	0	6
12:45 AM	0	0	0	1	3	0	4
01:00 AM	1	1	0	0	1	1	4
01:15 AM	0	1	1	0	0	0	2
01:30 AM	3	1	0	0	0	1	5
01:45 AM	1	0	1	0	1	0	3
02:00 AM	1	0	0	0	0	0	1
02:15 AM	0	0	1	0	0	0	1
02:30 AM	0	1	1	0	3	0	5
02:45 AM	0	1	0	1	2	1	5
03:00 AM	0	1	0	0	1	1	3
03:15 AM	0	1	1	0	2	0	4
03:30 AM	1	0	1	0	1	0	3
03:45 AM	1	0	1	1	2	1	6
04:00 AM	1	1	3	0	1	0	6
04:15 AM	1	0	2	0	3	0	6
04:30 AM	2	1	4	0	4	1	12
04:45 AM	2	1	5	0	4	1	13
05:00 AM	2	1	6	1	9	1	20
05:15 AM	2	0	12	0	7	4	25
05:30 AM	1	1	11	2	14	3	32
05:45 AM	0	0	24	1	11	1	37
06:00 AM	10	2	36	0	10	0	58
06:15 AM	10	2	29	0	15	0	56
06:30 AM	9	6	38	3	14	3	73
06:45 AM	14	2	35	0	16	0	67
07:00 AM	14	2	39	0	17	0	72
07:15 AM	16	4	44	0	24	0	88
07:30 AM	11	4	61	1	19	4	100
07:45 AM	16	5	45	0	17	0	83
08:00 AM	24	4	47	0	17	3	95
08:15 AM	18	6	29	1	15	4	73
08:30 AM	30	2	29	0	13	0	74
08:45 AM	18	8	26	1	6	4	63
09:00 AM	17	2	29	0	14	0	62
09:15 AM	13	4	41	0	8	0	66
09:30 AM	13	7	29	0	5	0	54
09:45 AM	14	1	22	0	9	0	46
10:00 AM	11	6	13	2	7	3	42
10:15 AM	20	3	17	0	11	0	51
10:30 AM	11	10	20	0	8	0	49
10:45 AM	22	5	13	0	11	0	51
11:00 AM	10	7	16	1	13	2	49
11:15 AM	21	3	27	0	1	0	52
11:30 AM	19	8	24	2	6	4	63
11:45 AM	25	5	12	0	10	0	52

12:00 PM	37	5	14	0	8	1	65
12:15 PM	21	10	21	0	9	1	62
12:30 PM	27	6	20	0	5	0	58
12:45 PM	22	9	28	2	11	5	77
01:00 PM	32	7	21	0	4	2	66
01:15 PM	27	7	22	1	9	5	71
01:30 PM	18	14	17	4	15	6	74
01:45 PM	29	8	24	0	12	0	73
02:00 PM	30	7	33	0	15	1	86
02:15 PM	31	7	19	0	9	0	66
02:30 PM	32	8	21	0	7	0	68
02:45 PM	27	9	18	1	9	3	67
03:00 PM	21	14	29	3	14	6	87
03:15 PM	36	13	32	0	7	0	88
03:30 PM	52	13	16	1	18	5	105
03:45 PM	38	7	34	1	14	5	99
04:00 PM	32	12	22	0	6	0	72
04:15 PM	50	15	15	1	9	5	95
04:30 PM	33	14	31	1	11	1	91
04:45 PM	32	9	24	3	7	5	80
05:00 PM	41	16	23	2	13	6	101
05:15 PM	39	11	24	0	7	0	81
05:30 PM	33	14	28	1	6	4	86
05:45 PM	30	10	26	1	16	1	84
06:00 PM	29	13	18	0	6	0	66
06:15 PM	29	8	26	0	8	0	71
06:30 PM	28	8	24	1	10	5	76
06:45 PM	17	12	26	3	5	3	66
07:00 PM	19	17	14	9	20	9	88
07:15 PM	21	15	27	6	23	7	99
07:30 PM	21	13	29	7	13	10	93
07:45 PM	17	15	22	2	15	2	73
08:00 PM	34	7	21	0	11	0	73
08:15 PM	23	6	19	0	13	1	62
08:30 PM	20	7	11	2	12	4	56
08:45 PM	14	8	12	1	3	1	39
09:00 PM	11	9	3	3	14	5	45
09:15 PM	21	7	10	0	4	1	43
09:30 PM	25	4	13	0	7	0	49
09:45 PM	13	9	12	0	3	0	37
10:00 PM	17	9	5	0	17	0	48
10:15 PM	19	12	2	0	10	0	43
10:30 PM	7	9	6	3	11	3	39
10:45 PM	10	7	14	0	6	1	38
11:00 PM	11	4	3	0	5	0	23
11:15 PM	9	2	7	0	4	0	22
11:30 PM	6	4	1	0	2	0	13
11:45 PM	8	4	0	0	8	0	20
12:00 PM	3	1	0	3	7	4	18
Total	1610	579	1684	81	836	161	4951

Intersección PR-942 y PR-9942

Termina Período	S-N	S-E	E-S	E-N	N-S	N-E	Total
12:15 AM	3	0	0	1	3	0	7
12:30 AM	2	0	0	1	0	0	3
12:45 AM	1	0	0	0	2	0	3
01:00 AM	1	0	1	1	1	2	6
01:15 AM	0	1	0	1	0	1	3
01:30 AM	0	1	0	1	0	0	2
01:45 AM	0	0	0	0	0	0	0
02:00 AM	0	0	0	0	0	0	0
02:15 AM	0	0	0	0	0	0	0
02:30 AM	1	0	0	0	0	0	1
02:45 AM	1	1	1	0	0	0	3
03:00 AM	1	0	0	0	0	0	1
03:15 AM	1	0	0	0	0	0	1
03:30 AM	0	0	0	0	1	1	2
03:45 AM	1	0	0	0	1	0	2
04:00 AM	1	0	0	0	1	0	2
04:15 AM	0	0	0	0	0	0	0
04:30 AM	1	0	1	0	2	0	4
04:45 AM	1	0	0	0	1	0	2
05:00 AM	2	0	0	0	3	0	5
05:15 AM	0	0	1	1	8	0	10
05:30 AM	3	0	1	0	12	0	16
05:45 AM	1	0	0	0	5	0	6
06:00 AM	2	0	1	0	9	0	12
06:15 AM	2	0	2	0	8	0	12
06:30 AM	9	0	1	1	10	0	21
06:45 AM	2	0	2	0	11	0	15
07:00 AM	2	0	3	2	11	0	18
07:15 AM	4	0	1	0	16	0	21
07:30 AM	5	0	1	1	22	1	30
07:45 AM	5	0	1	1	16	2	25
08:00 AM	4	0	2	1	14	0	21
08:15 AM	7	0	1	0	17	0	25
08:30 AM	2	0	2	1	10	0	15
08:45 AM	9	0	2	2	8	6	27
09:00 AM	2	0	3	2	11	4	22
09:15 AM	3	1	3	1	5	6	19
09:30 AM	7	0	1	2	4	5	19
09:45 AM	1	0	1	1	8	0	11
10:00 AM	8	0	1	1	9	0	19
10:15 AM	3	0	2	2	6	0	13
10:30 AM	10	0	1	2	5	0	18
10:45 AM	5	0	3	3	8	2	21
11:00 AM	8	0	2	1	13	2	26
11:15 AM	1	2	1	5	0	9	18
11:30 AM	10	0	1	2	9	3	25
11:45 AM	5	0	3	3	7	2	20
12:00 PM	5	0	3	5	6	2	21

12:15 PM	10	0	1	1	7	0	19
12:30 PM	6	0	1	2	4	7	20
12:45 PM	11	0	2	2	9	0	24
01:00 PM	7	0	2	4	4	9	26
01:15 PM	8	0	3	2	8	0	21
01:30 PM	10	8	6	3	13	0	40
01:45 PM	8	0	3	3	9	0	23
02:00 PM	5	2	2	0	8	0	17
02:15 PM	5	2	2	2	7	1	19
02:30 PM	8	0	1	4	6	1	20
02:45 PM	10	0	2	3	9	0	24
03:00 PM	16	1	2	2	9	0	30
03:15 PM	13	0	1	4	6	9	33
03:30 PM	14	0	3	4	5	0	26
03:45 PM	8	0	4	3	11	0	26
04:00 PM	12	0	1	2	5	5	25
04:15 PM	12	4	1	2	6	0	25
04:30 PM	15	0	3	5	9	0	32
04:45 PM	9	3	3	4	9	7	35
05:00 PM	15	3	2	2	13	0	35
05:15 PM	11	0	2	7	5	3	28
05:30 PM	12	3	2	3	8	5	33
05:45 PM	11	0	4	3	5	0	23
06:00 PM	9	4	2	6	4	11	36
06:15 PM	8	0	2	3	6	9	28
06:30 PM	7	2	5	4	10	1	29
06:45 PM	17	2	2	5	6	11	43
07:00 PM	22	4	5	5	18	0	54
07:15 PM	16	5	5	4	16	0	46
07:30 PM	19	1	2	1	18	0	41
07:45 PM	15	2	1	2	13	0	33
08:00 PM	7	0	2	4	9	5	27
08:15 PM	6	0	2	2	3	0	13
08:30 PM	9	0	1	1	5	0	16
08:45 PM	9	0	0	1	4	0	14
09:00 PM	12	0	2	1	9	0	24
09:15 PM	7	0	1	2	4	5	19
09:30 PM	4	0	3	5	4	2	18
09:45 PM	9	0	1	5	2	8	25
10:00 PM	9	0	2	1	12	0	24
10:15 PM	7	5	5	7	5	1	30
10:30 PM	12	0	1	1	4	0	18
10:45 PM	6	1	1	2	3	0	13
11:00 PM	4	0	1	4	4	1	14
11:15 PM	0	2	0	0	1	0	3
11:30 PM	4	0	0	0	0	0	4
11:45 PM	4	0	1	0	0	0	5
12:00 PM	4	0	0	0	1	0	5
Total	604	60	149	178	619	149	1759

Intersección PR-942 y PR-9943

Termina Período	S-N	S-O	N-S	N-O	O-N	O-S	Total
12:15 AM	1	3	0	0	0	0	4
12:30 AM	1	2	0	0	0	0	3
12:45 AM	0	1	0	0	0	1	2
01:00 AM	0	2	0	0	0	0	2
01:15 AM	0	1	0	0	0	0	1
01:30 AM	0	1	0	0	0	0	1
01:45 AM	0	0	0	0	0	0	0
02:00 AM	0	0	0	0	0	0	0
02:15 AM	0	0	0	0	0	0	0
02:30 AM	0	1	0	0	0	0	1
02:45 AM	0	1	0	0	0	0	1
03:00 AM	0	1	0	0	0	0	1
03:15 AM	1	0	0	0	0	0	1
03:30 AM	0	0	0	0	0	1	1
03:45 AM	0	1	0	0	0	0	1
04:00 AM	0	1	0	0	0	0	1
04:15 AM	0	0	0	1	0	0	1
04:30 AM	1	0	2	0	0	0	3
04:45 AM	1	0	1	0	0	0	2
05:00 AM	2	0	1	0	0	2	5
05:15 AM	1	0	6	0	0	0	7
05:30 AM	2	1	6	0	0	1	10
05:45 AM	1	0	4	0	0	1	6
06:00 AM	2	0	6	0	0	0	8
06:15 AM	2	0	5	0	0	0	7
06:30 AM	10	0	5	0	0	3	18
06:45 AM	0	2	2	0	0	3	7
07:00 AM	4	0	9	0	1	2	16
07:15 AM	4	0	3	0	0	2	9
07:30 AM	3	3	3	0	0	1	10
07:45 AM	0	6	3	0	0	0	9
08:00 AM	2	3	4	0	0	1	10
08:15 AM	5	2	4	0	0	1	12
08:30 AM	3	0	10	1	0	0	14
08:45 AM	2	9	1	0	0	3	15
09:00 AM	4	0	6	0	0	3	13
09:15 AM	4	0	3	0	0	0	7
09:30 AM	9	0	6	0	0	0	15
09:45 AM	2	0	8	0	1	0	11
10:00 AM	8	1	8	0	0	1	18
10:15 AM	4	1	4	0	0	2	11
10:30 AM	6	6	2	0	0	3	17
10:45 AM	5	3	5	0	0	2	15
11:00 AM	3	6	3	0	0	5	17
11:15 AM	5	1	6	0	0	0	12
11:30 AM	3	9	3	0	0	0	15
11:45 AM	3	5	2	0	0	1	11
12:00 PM	10	0	5	0	0	2	17

12:15 PM	6	5	5	0	0	1	17
12:30 PM	8	0	4	0	0	2	14
12:45 PM	6	7	8	0	0	1	22
01:00 PM	2	9	2	0	0	4	17
01:15 PM	6	4	6	0	0	2	18
01:30 PM	5	8	4	0	0	3	20
01:45 PM	5	6	3	0	0	0	14
02:00 PM	3	2	4	0	0	0	9
02:15 PM	5	2	5	0	0	1	13
02:30 PM	5	7	2	0	0	0	14
02:45 PM	3	10	2	0	0	3	18
03:00 PM	5	13	0	0	0	1	19
03:15 PM	2	15	4	0	0	6	27
03:30 PM	5	13	0	0	0	4	22
03:45 PM	2	9	4	0	0	2	17
04:00 PM	4	10	2	0	0	2	18
04:15 PM	3	11	1	0	0	3	18
04:30 PM	13	7	3	0	0	0	23
04:45 PM	6	7	3	0	0	0	16
05:00 PM	8	9	1	0	0	3	21
05:15 PM	13	5	3	0	0	3	24
05:30 PM	8	7	6	0	0	3	24
05:45 PM	14	0	5	1	0	0	20
06:00 PM	8	7	5	0	0	1	21
06:15 PM	7	4	2	0	0	3	16
06:30 PM	2	9	8	0	0	3	22
06:45 PM	7	15	7	0	0	7	36
07:00 PM	10	17	8	0	0	1	36
07:15 PM	11	9	5	0	0	2	27
07:30 PM	5	15	6	0	0	8	34
07:45 PM	9	8	5	0	0	0	22
08:00 PM	11	0	7	0	0	3	21
08:15 PM	8	0	3	1	1	0	13
08:30 PM	1	9	3	0	0	2	15
08:45 PM	5	5	4	2	0	0	16
09:00 PM	6	7	1	0	0	2	16
09:15 PM	4	5	2	0	0	1	12
09:30 PM	5	4	2	0	0	1	12
09:45 PM	8	6	4	0	0	0	18
10:00 PM	6	4	1	0	0	4	15
10:15 PM	2	12	2	0	0	3	19
10:30 PM	7	6	1	0	0	0	14
10:45 PM	2	6	0	0	0	0	8
11:00 PM	2	6	0	0	0	1	9
11:15 PM	0	0	1	0	0	0	1
11:30 PM	1	3	0	0	0	0	4
11:45 PM	3	1	0	0	0	0	4
12:00 PM	4	0	1	1	1	0	7
Total	385	397	291	7	4	127	1211

Intersección PR-942 y PR-9941

Termina Período	E-O	E-S	O-E	O-S	S-E	S-O	Total
12:15 AM	0	1	0	0	0	0	1
12:30 AM	1	0	0	1	0	0	2
12:45 AM	0	0	0	0	0	0	0
01:00 AM	0	0	0	0	0	0	0
01:15 AM	0	0	0	0	0	0	0
01:30 AM	0	0	0	0	0	0	0
01:45 AM	0	0	0	0	0	0	0
02:00 AM	0	1	0	0	0	0	1
02:15 AM	0	0	0	0	0	0	0
02:30 AM	0	0	0	0	0	0	0
02:45 AM	0	0	0	0	0	0	0
03:00 AM	0	0	0	0	0	0	0
03:15 AM	1	0	0	0	0	0	1
03:30 AM	0	0	0	0	0	0	0
03:45 AM	0	0	0	0	0	0	0
04:00 AM	0	0	0	0	0	0	0
04:15 AM	1	0	1	1	0	0	3
04:30 AM	0	1	2	1	0	0	4
04:45 AM	0	1	1	0	0	0	2
05:00 AM	0	2	1	2	0	0	5
05:15 AM	1	2	6	0	0	0	9
05:30 AM	1	1	5	0	1	0	8
05:45 AM	3	1	3	1	1	0	9
06:00 AM	2	4	6	1	0	0	13
06:15 AM	4	0	5	0	0	0	9
06:30 AM	8	7	3	7	2	0	27
06:45 AM	0	0	1	0	1	0	2
07:00 AM	9	0	7	5	2	0	23
07:15 AM	4	1	1	5	2	0	13
07:30 AM	2	1	1	1	2	0	7
07:45 AM	0	0	3	0	0	0	3
08:00 AM	0	2	2	0	2	0	6
08:15 AM	4	1	2	2	2	0	11
08:30 AM	0	5	10	1	1	0	17
08:45 AM	1	1	0	1	1	0	4
09:00 AM	2	3	5	0	1	0	11
09:15 AM	3	1	2	2	1	0	9
09:30 AM	6	4	3	5	3	0	21
09:45 AM	9	1	7	1	1	0	19
10:00 AM	5	3	5	4	3	0	20
10:15 AM	2	2	3	3	1	0	11
10:30 AM	3	3	0	3	2	0	11
10:45 AM	4	1	3	4	2	0	14
11:00 AM	1	2	2	4	1	0	10
11:15 AM	4	1	5	2	1	0	13
11:30 AM	0	3	0	6	3	0	12
11:45 AM	2	1	1	3	1	0	8
12:00 PM	6	5	1	10	4	0	26

12:15 PM	5	1	4	4	1	0	15
12:30 PM	6	3	1	3	3	0	16
12:45 PM	4	2	6	3	2	0	17
01:00 PM	2	0	0	4	2	3	11
01:15 PM	5	1	0	4	6	0	16
01:30 PM	5	0	1	6	3	0	15
01:45 PM	4	1	0	4	3	0	12
02:00 PM	2	1	2	1	2	0	8
02:15 PM	3	2	3	4	2	0	14
02:30 PM	5	0	2	4	0	0	11
02:45 PM	2	1	1	3	1	0	8
03:00 PM	5	0	0	4	0	0	9
03:15 PM	1	1	3	0	1	0	6
03:30 PM	5	0	0	2	0	0	7
03:45 PM	2	0	4	1	0	0	7
04:00 PM	1	3	0	2	2	0	8
04:15 PM	3	0	1	1	0	1	6
04:30 PM	10	3	0	8	3	0	24
04:45 PM	6	0	3	2	0	0	11
05:00 PM	4	4	0	5	1	0	14
05:15 PM	10	3	1	4	2	0	20
05:30 PM	7	1	3	5	3	0	19
05:45 PM	9	5	3	10	3	0	30
06:00 PM	7	1	5	2	0	0	15
06:15 PM	4	3	1	4	1	0	13
06:30 PM	2	0	8	0	0	0	10
06:45 PM	4	3	3	0	4	0	14
07:00 PM	8	2	6	1	2	0	19
07:15 PM	5	6	1	8	4	0	24
07:30 PM	4	1	4	3	2	0	14
07:45 PM	5	4	1	7	4	0	21
08:00 PM	12	1	3	7	4	0	27
08:15 PM	8	2	0	6	4	0	20
08:30 PM	1	0	2	1	1	0	5
08:45 PM	5	0	5	4	1	0	15
09:00 PM	3	3	0	4	1	0	11
09:15 PM	4	0	2	2	0	1	9
09:30 PM	4	1	1	3	1	0	10
09:45 PM	7	1	2	7	2	0	19
10:00 PM	4	2	1	5	0	0	12
10:15 PM	2	0	2	2	0	0	6
10:30 PM	7	0	0	7	1	0	15
10:45 PM	1	1	0	1	0	0	3
11:00 PM	1	1	0	1	0	0	3
11:15 PM	2	2	0	2	1	0	7
11:30 PM	0	1	0	0	0	0	1
11:45 PM	2	1	0	3	0	0	6
12:00 PM	4	1	1	5	1	0	12
Total	301	131	183	245	115	5	980

Apéndice 4. Matriz del Estudio

Facilidad	Condición Actual		Condición 2020		Condición 2030	
	AM	PM	AM	PM	AM	PM
PR-941/PR-9941	9419941A	9419941P	419941A2	419941P2	419941A3	419941P3
PR-941/Félix Díaz	DíazA	DíázP	DíazA20	DíazP20	DíazA30	DíazP30
PR-941 al sur de PR-942	941AM	941PM	941AM20	941PM20	941AM30	941PM30
PR-941/PR-942	941942A	941942P	41942A20	41942P20	41942A30	41942P30
PR-942/La Aventura			AccesoA2	AccesoP2	AccesoA3	AccesoP3
PR-942 al norte de La Aventura	942AM	942PM	942AM20	942PM20	942AM30	942PM30
PR-942/PR-9942	9429942A	9429942P	429942A2	429942P2	429942A3	429942P2
PR-942/PR-9943	9429943A	9429943P	429943A2	429943P2	429943A3	429943P3
PR-942/PR-9941	9429941A	9429941P	429941A2	429941P2	429941A3	429941P3

Justificaciones

Facilidad	Condición Actual	Condición 2020	Condición 2030
PR-941/Félix Díaz		2020	2030

Apéndice 5. Cómputos para el análisis operacional

TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	2
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*7:00 - 8:00 AM*		
Project Description *Estudio de Impacto en el Tránsito La Aventura*			
East/West Street: *PR-9941*		North/South Street: *PR-941*	
Intersection Orientation: *North-South*		Study Period (hrs): *0.25*	

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)		58	2	0	121	
Peak-Hour Factor, PHF	1.00	0.60	0.25	1.00	0.80	1.00
Hourly Flow Rate, HFR (veh/h)	0	0	0	12	0	15
Percent Heavy Vehicles	0	–	–	0	–	–
Median Type	*Undivided*					
RT Channelized			0			0
Lanes	0	1	0	0	1	0
Configuration			TR	LT		
Upstream Signal		0			0	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)				3	0	7
Peak-Hour Factor, PHF	1.00	1.00	1.00	0.25	1.00	0.44
Hourly Flow Rate, HFR (veh/h)	0	151	0	0	96	8
Percent Heavy Vehicles	0	0	0	0	0	0
Percent Grade (%)	0			-6		
Flared Approach		N			N	

Storage		0			0	
RT Channelized			0			0
Lanes	0	0	0	0	1	0
Configuration					LTR	

Delay, Queue Length, and Level of Service								
Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LT		LTR				
v (veh/h)		0		27				
C (m) (veh/h)		1500		850				
v/c		0.00		0.03				
95% queue length		0.00		0.10				
Control Delay (s/veh)		7.4		9.4				
LOS		A		A				
Approach Delay (s/veh)	–	–	9.4					
Approach LOS	–	–	A					

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TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	2
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*7:00 - 8:00 AM*		

Project Description *Estudio de Impacto en el Tránsito La Aventura*

East/West Street: *Félix Díaz*	North/South Street: *PR-941*
Intersection Orientation: *North-South*	Study Period (hrs): *0.25*

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)	*0*	*74*			*198*	*0*
Peak-Hour Factor, PHF	*1.00*	*0.69*	*1.00*	*1.00*	*0.80*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*0*	*0*	*0*	*0*	*0*
Percent Heavy Vehicles	*0*	–	–	*0*	–	–
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*1*	*0*
Configuration	*LT*					*TR*
Upstream Signal		*0*			*0*	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)	*0*	*0*	*0*			
Peak-Hour Factor, PHF	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*247*	*0*	*0*	*107*	*0*
Percent Heavy Vehicles	*0*	*0*	*0*	*0*	*0*	*0*
Percent Grade (%)	*-5*			*-6*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	

RT Channelized			0			0
Lanes	0	1	0	0	0	0
Configuration		LTR				

Delay, Queue Length, and Level of Service								
Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration	LT						LTR	
v (veh/h)	0						0	
C (m) (veh/h)	1331							
v/c	0.00							
95% queue length	0.00							
Control Delay (s/veh)	7.7							
LOS	A							
Approach Delay (s/veh)	--	--						
Approach LOS	--	--						

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Two-Way Two-Lane Highway Segment Analysis

Analyst	José L. Pérez Berenguer
Agency/Co.	Pérez Berenguer y Asociados
Date Performed	8/10/2006
Analysis Time Period	7:00 - 8:00 AM
Highway	PR-941
From/To	Gurabo - Jaguas
Jurisdiction	Gurabo
Analysis Year	2006
Description	Estudio de Impacto en el Tránsito La Aventura

Input Data

Highway class	Class 2				
Shoulder width	0.0	m	Peak-hour factor, PHF	0.79	
Lane width	2.7	m	% Trucks and buses	1	%
Segment length	1.5	km	% Recreational vehicles	0	%
Terrain type	Level		% No-passing zones	0	%
Grade: Length		km	Access points/km	1	/km
Up/down		%			

Two-way hourly volume, V		358	veh/h	
Directional split	77	/	23	%

Average Travel Speed

Grade adjustment factor, fG	1.00	
PCE for trucks, ET	1.7	
PCE for RVs, ER	1.0	
Heavy-vehicle adjustment factor,	0.993	
Two-way flow rate, (note-1) vp	456	pc/h
Highest directional split proportion (note-2)	351	pc/h
Free-Flow Speed from Field Measurement:		
Field measured speed, SFM	-	km/h
Observed volume, Vf	-	veh/h
Estimated Free-Flow Speed:		
Base free-flow speed, BFFS	100.0	km/h
Adj. for lane and shoulder width, fLS	10.3	km/h
Adj. for access points, fA	0.7	km/h
Free-flow speed, FFS	89.0	km/h
Adjustment for no-passing zones, fnp	0.0	km/h
Average travel speed, ATS	83.3	km/h

Percent Time-Spent-Following		
Grade adjustment factor, fG	1.00	
PCE for trucks, ET	1.1	
PCE for RVs, ER	1.0	
Heavy-vehicle adjustment factor, fHV	0.999	
Two-way flow rate,(note-1) vp	454	pc/h
Highest directional split proportion (note-2)	350	
Base percent time-spent-following, BPTSF	32.9	%
Adj.for directional distribution and no-passing zones, fd/np	1.4	
Percent time-spent-following, PTSF	34.3	%

Level of Service and Other Performance Measures		
Level of service, LOS	A	
Volume to capacity ratio, v/c	0.14	
Peak 15-min vehicle-kilometers of travel, VkmT15	170	veh-km
Peak-hour vehicle-kilometers of travel, VkmT60	537	veh-km
Peak 15-min total travel time, TT15	2.0	veh-h

Notes:
1. If vp >= 3200 pc/h, terminate analysis-the LOS is F.
2. If highest directional split vp >= 1700 pc/h, terminate analysis-the LOS is F.

TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*3*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*7:00 - 8:00 AM*		
Project Description *Estudio de Impacto en el Tránsito La Aventura*			
East/West Street: *PR-941/PR-942*		North/South Street: *PR-941*	
Intersection Orientation: *North-South*		Study Period (hrs): *0.25*	

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)	*67*	*0*	*17*			
Peak-Hour Factor, PHF	*0.70*	*1.00*	*0.85*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*4*	*243*	*96*	*15*	*0*
Percent Heavy Vehicles	*3*	–	–	*0*	–	–
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*0*	*0*
Configuration	*LTR*					
Upstream Signal		*0*			*0*	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)		*1*	*197*	*77*	*7*	
Peak-Hour Factor, PHF	*1.00*	*0.25*	*0.81*	*0.80*	*0.44*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*0*	*0*	*95*	*0*	*19*
Percent Heavy Vehicles	*0*	*0*	*1*	*1*	*0*	*0*
Percent Grade (%)	*-1*			*-6*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	

RT Channelized			0			0
Lanes	0	1	0	0	1	0
Configuration			TR	LT		

Delay, Queue Length, and Level of Service								
Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration	LTR		LT					TR
v (veh/h)	95		111					247
C (m) (veh/h)	1617		485					1076
v/c	0.06		0.23					0.23
95% queue length	0.19		0.87					0.89
Control Delay (s/veh)	7.4		14.6					9.3
LOS	A		B					A
Approach Delay (s/veh)	–	–	14.6			9.3		
Approach LOS	–	–	B			A		

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Two-Way Two-Lane Highway Segment Analysis

Analyst	José L. Pérez Berenguer
Agency/Co.	Pérez Berenguer y Asociados
Date Performed	8/10/2006
Analysis Time Period	7:00 - 8:00 AM
Highway	PR-942
From/To	PR-941/PR-9942
Jurisdiction	Gurabo
Analysis Year	2006
Description	Estudio de Impacto en el Tránsito La Aventura

Input Data

Highway class	Class 2				
Shoulder width	0.0	m	Peak-hour factor, PHF	0.79	
Lane width	2.8	m	% Trucks and buses	1	%
Segment length	1.0	km	% Recreational vehicles	0	%
Terrain type	Rolling		% No-passing zones	0	%
Grade: Length		km	Access points/km	1	/km
Up/down		%			

Two-way hourly volume, V 91 veh/h
Directional split 80 / 20 %

Average Travel Speed

Grade adjustment factor, fG	0.71	
PCE for trucks, ET	2.5	
PCE for RVs, ER	1.1	
Heavy-vehicle adjustment factor,	0.985	
Two-way flow rate,(note-1) vp	165	pc/h
Highest directional split proportion (note-2)	132	pc/h
Free-Flow Speed from Field Measurement:		
Field measured speed, SFM	-	km/h
Observed volume, Vf	-	veh/h
Estimated Free-Flow Speed:		
Base free-flow speed, BFFS	100.0	km/h
Adj. for lane and shoulder width, fLS	10.3	km/h
Adj. for access points, fA	0.7	km/h
Free-flow speed, FFS	89.0	km/h
Adjustment for no-passing zones, fnp	0.0	km/h
Average travel speed, ATS	87.0	km/h

Percent Time-Spent-Following		
Grade adjustment factor, fG	0.77	
PCE for trucks, ET	1.8	
PCE for RVs, ER	1.0	
Heavy-vehicle adjustment factor, fHV	0.992	
Two-way flow rate,(note-1) vp	151	pc/h
Highest directional split proportion (note-2)	121	
Base percent time-spent-following, BPTSF	12.4	%
Adj.for directional distribution and no-passing zones, fd/np	5.7	
Percent time-spent-following, PTSF	18.2	%

Level of Service and Other Performance Measures		
Level of service, LOS	A	
Volume to capacity ratio, v/c	0.05	
Peak 15-min vehicle-kilometers of travel, VkmT15	29	veh-km
Peak-hour vehicle-kilometers of travel, VkmT60	91	veh-km
Peak 15-min total travel time, TT15	0.3	veh-h

Notes:
1. If vp >= 3200 pc/h, terminate analysis-the LOS is F.
2. If highest directional split vp >= 1700 pc/h, terminate analysis-the LOS is F.

TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*4*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*7:00 - 8:00 AM*		

Project Description *Estudio de Impacto en el Tránsito La Aventura*

East/West Street: *PR-9942*	North/South Street: *PR-942*
Intersection Orientation: *North-South*	Study Period (hrs): *0.25*

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)		*18*	*0*	*3*	*68*	
Peak-Hour Factor, PHF	*1.00*	*0.90*	*1.00*	*0.38*	*0.77*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*0*	*0*	*7*	*0*	*4*
Percent Heavy Vehicles	*3*	--	--	*0*	--	--
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*1*	*0*
Configuration			*TR*	*LT*		
Upstream Signal		*0*			*0*	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)				*5*	*0*	*3*
Peak-Hour Factor, PHF	*1.00*	*1.00*	*1.00*	*0.63*	*1.00*	*0.75*
Hourly Flow Rate, HFR (veh/h)	*7*	*88*	*0*	*0*	*20*	*0*
Percent Heavy Vehicles	*0*	*0*	*1*	*0*	*0*	*0*
Percent Grade (%)	*-1*			*8*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	

RT Channelized			0			0
Lanes	0	0	0	0	1	0
Configuration					LTR	

Delay, Queue Length, and Level of Service								
Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LT		LTR				
v (veh/h)		7		11				
C (m) (veh/h)		1609		935				
v/c		0.00		0.01				
95% queue length		0.01		0.04				
Control Delay (s/veh)		7.2		8.9				
LOS		A		A				
Approach Delay (s/veh)	–	–	8.9					
Approach LOS	–	–	A					

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TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*5*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*7:00 - 8:00 AM*		

Project Description *Estudio de Impacto en el Tránsito La Aventura*

East/West Street: *PR-9943*	North/South Street: *PR-942*
Intersection Orientation: *North-South*	Study Period (hrs): *0.25*

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)	*12*	*9*			*13*	*0*
Peak-Hour Factor, PHF	*0.50*	*0.56*	*1.00*	*1.00*	*0.81*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*0*	*8*	*0*	*0*	*0*
Percent Heavy Vehicles	*0*	–	–	*0*	–	–
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*1*	*0*
Configuration	*LT*					*TR*
Upstream Signal		*0*			*0*	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)	*0*	*0*	*4*			
Peak-Hour Factor, PHF	*1.00*	*1.00*	*0.50*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*16*	*0*	*24*	*16*	*0*
Percent Heavy Vehicles	*0*	*0*	*0*	*0*	*0*	*0*
Percent Grade (%)	*6*			*0*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	

RT Channelized			0			0
Lanes	0	1	0	0	0	0
Configuration		LTR				

Delay, Queue Length, and Level of Service

Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration	LT						LTR	
v (veh/h)	24						8	
C (m) (veh/h)	1615						1069	
v/c	0.01						0.01	
95% queue length	0.05						0.02	
Control Delay (s/veh)	7.3						8.4	
LOS	A						A	
Approach Delay (s/veh)	–	–				8.4		
Approach LOS	–	–				A		

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TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*6*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*7:00 - 8:00 AM*		
Project Description *Estudio de Impacto en el Tránsito La Aventura*			
East/West Street: *PR-942*		North/South Street: *PR-9941*	
Intersection Orientation: *East-West*		Study Period (hrs): *0.25*	

Vehicle Volumes and Adjustments

Major Street	Eastbound			Westbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)		*7*	*6*	*4*	*6*	
Peak-Hour Factor, PHF	*1.00*	*0.58*	*0.30*	*0.50*	*0.38*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*12*	*19*	*8*	*15*	*0*
Percent Heavy Vehicles	*0*	–	–	*0*	–	–
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*1*	*0*
Configuration			*TR*	*LT*		
Upstream Signal		*0*			*0*	

Minor Street	Northbound			Southbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)	*0*	*0*	*6*			
Peak-Hour Factor, PHF	*1.00*	*1.00*	*0.75*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*0*	*8*	*0*	*0*	*0*
Percent Heavy Vehicles	*0*	*0*	*0*	*0*	*0*	*0*
Percent Grade (%)	*10*			*0*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	

RT Channelized			0			0
Lanes	0	1	0	0	0	0
Configuration		LTR				

Delay, Queue Length, and Level of Service

Approach	Eastbound	Westbound	Northbound			Southbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LT		LTR				
v (veh/h)		8		8				
C (m) (veh/h)		1595		1061				
v/c		0.01		0.01				
95% queue length		0.02		0.02				
Control Delay (s/veh)		7.3		8.4				
LOS		A		A				
Approach Delay (s/veh)	–	–	8.4					
Approach LOS	–	–	A					

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TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*2*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*2:45 - 3:45 PM*		
Project Description *Estudio de Impacto en el Tránsito La Aventura*			
East/West Street: *PR-9941*		North/South Street: *PR-941*	
Intersection Orientation: *North-South*		Study Period (hrs): *0.25*	

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)		*74*	*6*	*9*	*67*	
Peak-Hour Factor, PHF	*1.00*	*0.80*	*0.50*	*0.45*	*0.80*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*0*	*0*	*8*	*0*	*0*
Percent Heavy Vehicles	*0*	–	–	*0*	–	–
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*1*	*0*
Configuration			*TR*	*LT*		
Upstream Signal		*0*			*0*	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)				*2*	*0*	*0*
Peak-Hour Factor, PHF	*1.00*	*1.00*	*1.00*	*0.25*	*1.00*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*20*	*83*	*0*	*0*	*92*	*12*
Percent Heavy Vehicles	*0*	*0*	*0*	*0*	*0*	*0*
Percent Grade (%)	*0*			*-6*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	

RT Channelized			0			0
Lanes	0	0	0	0	1	0
Configuration					LTR	

Delay, Queue Length, and Level of Service

Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LT		LTR				
v (veh/h)		20		8				
C (m) (veh/h)		1500		762				
v/c		0.01		0.01				
95% queue length		0.04		0.03				
Control Delay (s/veh)		7.4		9.8				
LOS		A		A				
Approach Delay (s/veh)	–	–	9.8					
Approach LOS	–	–	A					

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TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*2*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*2:45 - 3:45 PM*		

Project Description *Estudio de Impacto en el Tránsito La Aventura*

East/West Street: *Félix Díaz*	North/South Street: *PR-941*
Intersection Orientation: *North-South*	Study Period (hrs): *0.25*

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)	*0*	*163*			*109*	*2*
Peak-Hour Factor, PHF	*1.00*	*0.71*	*1.00*	*1.00*	*0.83*	*0.25*
Hourly Flow Rate, HFR (veh/h)	*4*	*0*	*12*	*0*	*0*	*0*
Percent Heavy Vehicles	*0*	--	--	*0*	--	--
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*1*	*0*
Configuration	*LT*					*TR*
Upstream Signal		*0*			*0*	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)	*3*	*0*	*7*			
Peak-Hour Factor, PHF	*0.75*	*1.00*	*0.58*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*131*	*8*	*0*	*229*	*0*
Percent Heavy Vehicles	*0*	*0*	*0*	*0*	*0*	*0*
Percent Grade (%)	*-5*			*-6*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	

RT Channelized			0			0
Lanes	0	1	0	0	0	0
Configuration		LTR				

Delay, Queue Length, and Level of Service

Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration	LT						LTR	
v (veh/h)	0						16	
C (m) (veh/h)	1457						829	
v/c	0.00						0.02	
95% queue length	0.00						0.06	
Control Delay (s/veh)	7.5						9.4	
LOS	A						A	
Approach Delay (s/veh)	–	–				9.4		
Approach LOS	–	–				A		

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Two-Way Two-Lane Highway Segment Analysis

Analyst	José L. Pérez Berenguer
Agency/Co.	Pérez Berenguer y Asociados
Date Performed	8/10/2006
Analysis Time Period	2:45 - 3:45 PM
Highway	PR-941
From/To	Gurabo - Jaguas
Jurisdiction	Gurabo
Analysis Year	2006
Description	Estudio de Impacto en el Tránsito La Aventura

Input Data

Highway class	Class 2				
Shoulder width	0.0	m	Peak-hour factor, PHF	0.80	
Lane width	2.7	m	% Trucks and buses	3	%
Segment length	1.5	km	% Recreational vehicles	0	%
Terrain type	Level		% No-passing zones	0	%
Grade: Length		km	Access points/km	1	/km
Up/down		%			

Two-way hourly volume, V	358			veh/h
Directional split	54	/	46	%

Average Travel Speed

Grade adjustment factor, fG	1.00	
PCE for trucks, ET	1.7	
PCE for RVs, ER	1.0	
Heavy-vehicle adjustment factor,	0.979	
Two-way flow rate,(note-1) vp	457	pc/h
Highest directional split proportion (note-2)	247	pc/h
Free-Flow Speed from Field Measurement:		
Field measured speed, SFM	-	km/h
Observed volume, Vf	-	veh/h
Estimated Free-Flow Speed:		
Base free-flow speed, BFFS	100.0	km/h
Adj. for lane and shoulder width, fLS	10.3	km/h
Adj. for access points, fA	0.7	km/h
Free-flow speed, FFS	89.0	km/h
Adjustment for no-passing zones, fnp	0.0	km/h
Average travel speed, ATS	83.3	km/h

Percent Time-Spent-Following

Grade adjustment factor, fG	1.00	
PCE for trucks, ET	1.1	
PCE for RVs, ER	1.0	
Heavy-vehicle adjustment factor, fHV	0.997	
Two-way flow rate,(note-1) vp	449	pc/h
Highest directional split proportion (note-2)	242	
Base percent time-spent-following, BPTSF	32.6	%
Adj.for directional distribution and no-passing zones, fd/np	0.2	
Percent time-spent-following, PTSF	32.8	%

Level of Service and Other Performance Measures

Level of service, LOS	A	
Volume to capacity ratio, v/c	0.14	
Peak 15-min vehicle-kilometers of travel, VkmT15	168	veh-km
Peak-hour vehicle-kilometers of travel, VkmT60	537	veh-km
Peak 15-min total travel time, TT15	2.0	veh-h

Notes:
1. If vp >= 3200 pc/h, terminate analysis-the LOS is F.
2. If highest directional split vp >= 1700 pc/h, terminate analysis-the LOS is F.

TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*3*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*2:45 - 3:45 PM*		
Project Description *Estudio de Impacto en el Tránsito La Aventura*			
East/West Street: *PR-941/PR-942*		North/South Street: *PR-941*	
Intersection Orientation: *North-South*		Study Period (hrs): *0.25*	

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)	*147*	*0*	*47*			
Peak-Hour Factor, PHF	*0.71*	*1.00*	*0.84*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*11*	*135*	*71*	*23*	*0*
Percent Heavy Vehicles	*2*	–	–	*0*	–	–
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*0*	*0*
Configuration	*LTR*					
Upstream Signal		*0*			*0*	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)		*5*	*111*	*53*	*16*	
Peak-Hour Factor, PHF	*1.00*	*0.42*	*0.82*	*0.74*	*0.67*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*0*	*0*	*207*	*0*	*55*
Percent Heavy Vehicles	*0*	*0*	*4*	*6*	*6*	*0*
Percent Grade (%)	*-1*			*-6*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	

RT Channelized			0			0
Lanes	0	1	0	0	1	0
Configuration			TR	LT		

Delay, Queue Length, and Level of Service

Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration	LTR		LT					TR
v (veh/h)	207		94					146
C (m) (veh/h)	1623		376					969
v/c	0.13		0.25					0.15
95% queue length	0.44		0.97					0.53
Control Delay (s/veh)	7.5		17.7					9.4
LOS	A		C					A
Approach Delay (s/veh)	–	–	17.7			9.4		
Approach LOS	–	–	C			A		

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HCS+: Two-Lane Highways Release 5.2

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Two-Way Two-Lane Highway Segment Analysis

Analyst	José L. Pérez Berenguer
Agency/Co.	Pérez Berenguer y Asociados
Date Performed	8/10/2006
Analysis Time Period	2:45 - 3:45 PM
Highway	PR-942
From/To	PR-941/PR-9942
Jurisdiction	Gurabo
Analysis Year	2006
Description	Estudio de Impacto en el Tránsito La Aventura

Input Data

Highway class	Class 2				
Shoulder width	0.0	m	Peak-hour factor, PHF	0.74	
Lane width	2.8	m	% Trucks and buses	2	%
Segment length	1.0	km	% Recreational vehicles	0	%
Terrain type	Rolling		% No-passing zones	0	%
Grade: Length		km	Access points/km	1	/km
Up/down		%			

Two-way hourly volume, V	93			veh/h
Directional split	56	/	44	%

Average Travel Speed

Grade adjustment factor, fG	0.71	
PCE for trucks, ET	2.5	
PCE for RVs, ER	1.1	
Heavy-vehicle adjustment factor,	0.971	
Two-way flow rate,(note-1) vp	182	pc/h
Highest directional split proportion (note-2)	102	pc/h
Free-Flow Speed from Field Measurement:		
Field measured speed, SFM	-	km/h
Observed volume, Vf	-	veh/h
Estimated Free-Flow Speed:		
Base free-flow speed, BFFS	100.0	km/h
Adj. for lane and shoulder width, fLS	10.3	km/h
Adj. for access points, fA	0.7	km/h
Free-flow speed, FFS	89.0	km/h
Adjustment for no-passing zones, fnp	0.0	km/h
Average travel speed, ATS	86.8	km/h

Percent Time-Spent-Following		
Grade adjustment factor, fG	0.77	
PCE for trucks, ET	1.8	
PCE for RVs, ER	1.0	
Heavy-vehicle adjustment factor, fHV	0.984	
Two-way flow rate,(note-1) vp	166	pc/h
Highest directional split proportion (note-2)	93	
Base percent time-spent-following, BPTSF	13.6	%
Adj.for directional distribution and no-passing zones, fd/np	1.1	
Percent time-spent-following, PTSF	14.6	%

Level of Service and Other Performance Measures		
Level of service, LOS	A	
Volume to capacity ratio, v/c	0.06	
Peak 15-min vehicle-kilometers of travel, VkmT15	31	veh-km
Peak-hour vehicle-kilometers of travel, VkmT60	93	veh-km
Peak 15-min total travel time, TT15	0.4	veh-h

Notes:
1. If vp >= 3200 pc/h, terminate analysis-the LOS is F.
2. If highest directional split vp >= 1700 pc/h, terminate analysis-the LOS is F.

TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*4*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*2:45 - 3:45 PM*		

Project Description *Estudio de Impacto en el Tránsito La Aventura*

East/West Street: *PR-9942*	North/South Street: *PR-942*
Intersection Orientation: *North-South*	Study Period (hrs): *0.25*

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)		51	1	9	31	
Peak-Hour Factor, PHF	1.00	0.80	0.25	0.25	0.70	1.00
Hourly Flow Rate, HFR (veh/h)	0	0	0	15	0	16
Percent Heavy Vehicles	3	–	–	0	–	–
Median Type	Undivided					
RT Channelized			0			0
Lanes	0	1	0	0	1	0
Configuration			TR	LT		
Upstream Signal		0			0	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)				10	0	13
Peak-Hour Factor, PHF	1.00	1.00	1.00	0.63	1.00	0.81
Hourly Flow Rate, HFR (veh/h)	36	44	0	0	63	4
Percent Heavy Vehicles	0	0	1	10	0	8
Percent Grade (%)	-1			8		
Flared Approach		N			N	
Storage		0			0	

RT Channelized			0			0
Lanes	0	0	0	0	1	0
Configuration					LTR	

Delay, Queue Length, and Level of Service

Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LT		LTR				
v (veh/h)		36		31				
C (m) (veh/h)		1547		868				
v/c		0.02		0.04				
95% queue length		0.07		0.11				
Control Delay (s/veh)		7.4		9.3				
LOS		A		A				
Approach Delay (s/veh)	–	–	9.3					
Approach LOS	–	–	A					

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TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*5*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*2:45 - 3:45 PM*		
Project Description *Estudio de Impacto en el Tránsito La Aventura*			
East/West Street: *PR-9943*		North/South Street: *PR-942*	
Intersection Orientation: *North-South*		Study Period (hrs): *0.25*	

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)	*50*	*14*			*8*	*0*
Peak-Hour Factor, PHF	*0.83*	*0.70*	*1.00*	*1.00*	*0.50*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*0*	*24*	*0*	*0*	*0*
Percent Heavy Vehicles	*0*	–	–	*0*	–	–
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*1*	*0*
Configuration	*LT*					*TR*
Upstream Signal		*0*			*0*	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)	*0*	*0*	*13*			
Peak-Hour Factor, PHF	*1.00*	*1.00*	*0.54*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*16*	*0*	*60*	*20*	*0*
Percent Heavy Vehicles	*0*	*0*	*0*	*0*	*0*	*0*
Percent Grade (%)	*6*			*0*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	

RT Channelized			0			0
Lanes	0	1	0	0	0	0
Configuration		LTR				

Delay, Queue Length, and Level of Service

Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration	LT						LTR	
v (veh/h)	60						24	
C (m) (veh/h)	1615						1069	
v/c	0.04						0.02	
95% queue length	0.12						0.07	
Control Delay (s/veh)	7.3						8.4	
LOS	A						A	
Approach Delay (s/veh)	–	–				8.4		
Approach LOS	–	–				A		

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TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*6*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2006*
Analysis Time Period	*2:45 - 3:45 PM*		

Project Description *Estudio de Impacto en el Tránsito La Aventura*

East/West Street: *PR-942*	North/South Street: *PR-9941*
Intersection Orientation: *East-West*	Study Period (hrs): *0.25*

Vehicle Volumes and Adjustments

Major Street	Eastbound			Westbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)		7	7	1	13	
Peak-Hour Factor, PHF	1.00	0.44	0.44	0.25	0.65	1.00
Hourly Flow Rate, HFR (veh/h)	0	15	15	4	20	0
Percent Heavy Vehicles	0	–	–	0	–	–
Median Type	Undivided					
RT Channelized			0			0
Lanes	0	1	0	0	1	0
Configuration			TR	LT		
Upstream Signal		0			0	

Minor Street	Northbound			Southbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)	0	0	1			
Peak-Hour Factor, PHF	1.00	1.00	0.25	1.00	1.00	1.00
Hourly Flow Rate, HFR (veh/h)	0	0	4	0	0	0
Percent Heavy Vehicles	0	0	0	0	0	0
Percent Grade (%)	10			0		
Flared Approach		N			N	
Storage		0			0	

RT Channelized			0			0
Lanes	0	1	0	0	0	0
Configuration		LTR				

Delay, Queue Length, and Level of Service

Approach	Eastbound	Westbound	Northbound			Southbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LT		LTR				
v (veh/h)		4		4				
C (m) (veh/h)		1596		1061				
v/c		0.00		0.00				
95% queue length		0.01		0.01				
Control Delay (s/veh)		7.3		8.4				
LOS		A		A				
Approach Delay (s/veh)	–	–	8.4					
Approach LOS	–	–	A					

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TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	2
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2020*
Analysis Time Period	*7:00 - 8:00 AM*		

Project Description *Estudio de Impacto en el Tránsito La Aventura*

East/West Street: *PR-9941*	North/South Street: *PR-941*
Intersection Orientation: *North-South*	Study Period (hrs): *0.25*

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)		*247*	*8*	*0*	*298*	
Peak-Hour Factor, PHF	*1.00*	*0.60*	*0.25*	*1.00*	*0.80*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*0*	*0*	*24*	*0*	*22*
Percent Heavy Vehicles	*0*	–	–	*0*	–	–
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*1*	*0*
Configuration			*TR*	*LT*		
Upstream Signal		*0*			*0*	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)				*6*	*0*	*10*
Peak-Hour Factor, PHF	*1.00*	*1.00*	*1.00*	*0.25*	*1.00*	*0.44*
Hourly Flow Rate, HFR (veh/h)	*0*	*372*	*0*	*0*	*411*	*32*
Percent Heavy Vehicles	*0*	*0*	*0*	*0*	*0*	*0*
Percent Grade (%)	*0*			*-6*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	

RT Channelized			0			0
Lanes	0	0	0	0	1	0
Configuration					LTR	

Delay, Queue Length, and Level of Service

Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LT		LTR				
v (veh/h)		0		46				
C (m) (veh/h)		1128		452				
v/c		0.00		0.10				
95% queue length		0.00		0.34				
Control Delay (s/veh)		8.2		13.9				
LOS		A		B				
Approach Delay (s/veh)	–	–	13.9					
Approach LOS	–	–	B					

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HCS+: MUTCD Signal Warrants Release 5.2

Analyst: José L. Pérez Berenguer
Agency: Pérez Berenguer y Asociados
Date: 8/10/2006
Project ID: La Aventura
EW Street: Félix Díaz

Intersection: 2
Jurisdiction: Gurabo
Units: U.S. Metric
Analysis Year: 2020
NS Street: PR-941

General Information

Major St. Speed (kph): 65
Nearest Signal (m): 0
Crashes per Yr: 0

Population: Not less than 10000
Coordinated Signal System: N

School Crossing

Students in Highest Hour: 0
Adequate Gaps in Period: 0
Minutes in Period: 0

Roadway Network

Two Major Routes: 0
Weekend Count: 0
5-yr Growth Factor: 10

Geometry and Traffic

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
No. Lanes	1	0	1	0	0	0	1	1	0	0	1	0
LaneUsage	L		R				L	T			TR	

Results

Warrant 1: Eight-Hour Vehicular Volume [ ]
1 A. Minimum Vehicular Volumes [ ]
1 B. Interruption of Continuous Traffic [ ]
1 80% Vehicular --and-- Interruption Volumes [ ]

Warrant 2: Four-Hour Vehicular Volume
2 A. Four-Hour Vehicular Volumes [ ]

Warrant 3: Peak Hour [ ]
3 A. Peak-Hour Conditions [ ]
3 B. Peak-Hour Vehicular Volume Hours Met [ ]

Warrant 4: Pedestrian Volume [ ]
4 A. Pedestrian Volumes [ ]
4 B. Gaps Same Period [ ]

Warrant 5: School Crossing [ ]
5 A. Student Volumes [ ]
5 B. Gaps Same Period [ ]

Warrant 6: Coordinated Signal System
6 Degree of Platooning [ ]

Warrant 7: Crash Experience [ ]
7 A. Adequate trials of alternatives [ ]
7 B. Reported crashes [ ]
7 80% Volumes for Warrants 1A, 1B --or-- 4 [ ]

Warrant 8: Roadway Network [ ]
8 A. Weekday Volume [ ]
8 B. Weekend Volume [ ]

Summary

Hours	Major Volume	Minor Volume	Total Volume	Delay (Veh-hr)	1A 100%	1A 80%	1B 100%	1B 80%	2 100%	3A 100%	3B 100%
07-08	620	310	930	0.0	Yes	Yes	No	No	No	No	No
08-09	0	0	0	0.0	No	No	No	No	No	No	No
09-10	0	0	0	0.0	No	No	No	No	No	No	No
10-11	0	0	0	0.0	No	No	No	No	No	No	No
11-12	0	0	0	0.0	No	No	No	No	No	No	No
12-13	0	0	0	0.0	No	No	No	No	No	No	No
13-14	0	0	0	0.0	No	No	No	No	No	No	No
14-15	968	196	1164	0.0	No	Yes	Yes	Yes	No	No	No
15-16	0	0	0	0.0	No	No	No	No	No	No	No
16-17	0	0	0	0.0	No	No	No	No	No	No	No
17-18	0	0	0	0.0	No	No	No	No	No	No	No
18-19	0	0	0	0.0	No	No	No	No	No	No	No
Total	1588	506	2094		1	2	1	1	0	0	0

Traffic Volumes (vph)

Eastbound			Westbound			Northbound			Southbound		
L	T	R	L	T	R	L	T	R	L	T	R
85	0	225	0	0	0	27	186	0	0	338	69
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
59	0	137	0	0	0	195	400	0	0	237	136
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Pedestrian Volumes and Gaps (Per Hour)

Volume	Gap	Volume	Gap	Volume	Gap	Volume	Gap
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0

Delay	sec/veh	veh-hrs	sec/veh	veh-hrs	sec/veh	veh-hrs	sec/veh	veh-hrs
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*2*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2020*
Analysis Time Period	*7:00 - 8:00 AM*		
Project Description *Estudio de Impacto en el Tránsito La Aventura*			
East/West Street: *Félix Díaz*		North/South Street: *PR-941*	
Intersection Orientation: *North-South*		Study Period (hrs): *0.25*	

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)	27	186			338	69
Peak-Hour Factor, PHF	0.69	0.69	1.00	1.00	0.80	0.80
Hourly Flow Rate, HFR (veh/h)	96	0	255	0	0	0
Percent Heavy Vehicles	0	–	–	0	–	–
Median Type	Raised curb					
RT Channelized			0			0
Lanes	1	1	0	0	1	0
Configuration	L	T				TR
Upstream Signal		0			0	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)	85		225			
Peak-Hour Factor, PHF	0.88	1.00	0.88	1.00	1.00	1.00
Hourly Flow Rate, HFR (veh/h)	0	422	86	39	269	0
Percent Heavy Vehicles	0	0	0	0	0	0
Percent Grade (%)	-5			-6		
Flared Approach		N			N	
Storage		0			0	

RT Channelized			0			0
Lanes	1	0	1	0	0	0
Configuration	L		R			

Delay, Queue Length, and Level of Service

Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration	L					L		R
v (veh/h)	39					96		255
C (m) (veh/h)	1067					526		602
v/c	0.04					0.18		0.42
95% queue length	0.11					0.66		2.10
Control Delay (s/veh)	8.5					13.4		15.3
LOS	A					B		C
Approach Delay (s/veh)	–	–				14.8		
Approach LOS	–	–				B		

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OPERATIONAL ANALYSIS

Analyst:	José L. Pérez Berenguer
Agency/Co:	Pérez Berenguer y Asociados
Date:	8/10/2006
Analysis Period:	7:00 - 8:00 AM
Highway:	PR-941
From/To:	Gurabo/Jaguas
Jurisdiction:	Gurabo
Analysis Year:	2020
Project ID:	Estudio de Impacto en el Tránsito La Aventura

FREE-FLOW SPEED

Direction	1		2	
Lane width	3.6	m	3.6	m
Lateral clearance:				
Right edge	1.8	m	1.8	m
Left edge	1.8	m	1.8	m
Total lateral clearance	3.6	m	3.6	m
Access points per km	1		1	
Median type	Divided		Divided	
Free-flow speed:	Base		Base	
FFS or BFFS	100.0	km/h	100.0	km/h
Lane width adjustment, FLW	0.0	km/h	0.0	km/h
Lateral clearance adjustment, FLC	0.0	km/h	0.0	km/h
Median type adjustment, FM	0.0	km/h	0.0	km/h
Access points adjustment, FA	0.7	km/h	0.7	km/h
Free-flow speed	99.3	km/h	99.3	km/h

VOLUME

Direction	1		2	
Volume, V	223	vph	1491	vph
Peak-hour factor, PHF	0.77		0.80	
Peak 15-minute volume, v15	72		466	
Trucks and buses	2	%	1	%
Recreational vehicles	0	%	0	%
Terrain type	Level		Level	
Grade	0.00	%	0.00	%
Segment length	0.00	km	0.00	km
Number of lanes	2		2	
Driver population adjustment, fP	1.00		1.00	
Trucks and buses PCE, ET	1.5		1.5	
Recreational vehicles PCE, ER	1.2		1.2	
Heavy vehicle adjustment, fHV	0.990		0.995	
Flow rate, vp	146	pcphpl	936	pcphpl

RESULTS

Direction	1		2	
Flow rate, vp	146	pcphpl	936	pcphpl
Free-flow speed, FFS	99.3	km/h	99.3	km/h
Avg. passenger-car travel speed, S	99.3	km/h	99.3	km/h
Level of service, LOS	A		B	
Density, D	1.5	pc/km/ln	9.4	pc/km/ln

Overall results are not computed when free-flow speed is less than 70 km/h.

SHORT REPORT

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*PR-941 y PR-942*
Agency or Co.	*Pérez Berenguer y Asociados*	Area Type	*All other areas*
Date Performed	*8/8/2006*	Jurisdiction	*Gurabo*
Time Period	*7:00 - 8:00 AM*	Analysis Year	*2020*

Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of Lanes		1	1	1	1		1		1			
Lane Group		T	R	L	LT		L		R			
Volume (vph)		6	557	934	88		124		99			
% Heavy Vehicles		0	1	1	0		3		0			
PHF		0.25	0.81	0.80	0.44		0.70		0.85			
Pretimed/Actuated (P/A)		A	A	A	A		A		A			
Startup Lost Time		2.0	2.0	2.0	2.0		2.0		2.0			
Extension of Effective Green		2.0	2.0	2.0	2.0		2.0		2.0			
Arrival Type		3	3	4	4		3		3			
Unit Extension		3.0	3.0	3.0	3.0		3.0		3.0			
Ped/Bike/RTOR Volume	0	0	557	0	0		0	0	99			
Lane Width		3.7	3.7	4.3	3.7		4.3		3.7			
Parking/Grade/Parking	N	-1	N	N	-6	N	N	10	N			
Parking/Hour												
Bus Stops/Hour		0	0	0	0		0		0			
Minimum Pedestrian Time		3.2			3.2			3.2				

Phasing	EB Only	WB Only	03	04	NB Only	06	07	08
Timing	G = 20.0	G = 82.5	G =	G =	G = 25.5	G =	G =	G =
	Y = 4	Y = 4	Y =	Y =	Y = 4	Y =	Y =	Y =
Duration of Analysis (hrs) = 0.25						Cycle Length C = 140.0		

Lane Group Capacity, Control Delay, and LOS Determination												
	EB			WB			NB			SB		
Adjusted Flow Rate		24	0	1167	200		177		0			
Lane Group Capacity		276	232	1169	1166		327		283			
v/c Ratio		0.09	0.00	1.00	0.17		0.54		0.00			
Green Ratio		0.14	0.14	0.59	0.59		0.18		0.18			
Uniform Delay d_1		52.1	51.4	28.7	13.1		51.9		46.8			
Delay Factor k		0.11	0.11	0.50	0.11		0.14		0.11			
Incremental Delay d_2		0.1	0.0	25.9	0.1		1.8		0.0			
PF Factor		1.000	1.000	0.600	0.600		1.000		1.000			
Control Delay		52.2	51.4	43.1	8.0		53.8		46.8			
Lane Group LOS		D	D	D	A		D		D			
Approach Delay	52.2			37.9			53.8					
Approach LOS	D			D			D					
Intersection Delay	40.0			Intersection LOS						D		

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SHORT REPORT

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*PR-942 y Acceso a Proyecto*
Agency or Co.	*Pérez Berenguer y Asociados*	Area Type	*All other areas*
Date Performed	*8/8/2006*	Jurisdiction	*Gurabo*
Time Period	*7:00 - 8:00 AM*	Analysis Year	*2020*

Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of Lanes	1		1				1	1			1	1
Lane Group	L		R				L	T			T	R
Volume (vph)	162		754				68	37			268	318
% Heavy Vehicles	0		0				0	0			1	0
PHF	0.82		0.77				0.82	0.82			0.77	0.77
Pretimed/Actuated (P/A)	A		A				A	A			A	A
Startup Lost Time	2.0		2.0				2.0	2.0			2.0	2.0
Extension of Effective Green	2.0		2.0				2.0	2.0			2.0	2.0
Arrival Type	3		3				4	4			3	3
Unit Extension	3.0		3.0				3.0	3.0			3.0	3.0
Ped/Bike/RTOR Volume	0	0	166				0	0		0	0	318
Lane Width	3.7		3.7				3.7	3.7			3.7	3.7
Parking/Grade/Parking	N	0	N				N	6	N	N	-6	N
Parking/Hour												
Bus Stops/Hour	0		0				0	0			0	0
Minimum Pedestrian Time		3.2						3.2			3.2	

Phasing	EB Only	02	03	04	NS Perm	06	07	08
Timing	G = 47.4	G =	G =	G =	G = 15.6	G =	G =	G =
	Y = 4	Y =	Y =	Y =	Y = 4	Y =	Y =	Y =
Duration of Analysis (hrs) = 0.25						Cycle Length C = 71.0		

Lane Group Capacity, Control Delay, and LOS Determination												
	EB			WB			NB			SB		
Adjusted Flow Rate	198		764				83	45			348	0
Lane Group Capacity	1218		1090				105	409			430	370
v/c Ratio	0.16		0.70				0.79	0.11			0.81	0.00
Green Ratio	0.67		0.67				0.22	0.22			0.22	0.22
Uniform Delay d_1	4.4		7.4				26.2	22.1			26.3	21.6
Delay Factor k	0.11		0.27				0.34	0.11			0.35	0.11
Incremental Delay d_2	0.1		2.0				32.4	0.1			11.1	0.0
PF Factor	1.000		1.000				1.000	1.000			1.000	1.000
Control Delay	4.5		9.4				58.6	22.3			37.3	21.6
Lane Group LOS	A		A				E	C			D	C
Approach Delay	8.4						45.8			37.3		
Approach LOS	A						D			D		
Intersection Delay	18.7			Intersection LOS						B		

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Two-Way Two-Lane Highway Segment Analysis

Analyst	José L. Pérez Berenguer
Agency/Co.	Pérez Berenguer y Asociados
Date Performed	8/10/2006
Analysis Time Period	7:00 - 8:00 AM
Highway	PR-942
From/To	PR-941/PR-9942
Jurisdiction	Gurabo
Analysis Year	2020
Description	Estudio de Impacto en el Tránsito La Aventura

Input Data

Highway class	Class 2				
Shoulder width	0.0	m	Peak-hour factor, PHF	0.74	
Lane width	2.8	m	% Trucks and buses	0	%
Segment length	1.0	km	% Recreational vehicles	0	%
Terrain type	Rolling		% No-passing zones	0	%
Grade: Length		km	Access points/km	1	/km
Up/down		%			

Two-way hourly volume, V 698 veh/h
Directional split 68 / 32 %

Average Travel Speed

Grade adjustment factor, fG	0.93	
PCE for trucks, ET	1.9	
PCE for RVs, ER	1.1	
Heavy-vehicle adjustment factor,	1.000	
Two-way flow rate,(note-1) vp	1014	pc/h
Highest directional split proportion (note-2)	690	pc/h
Free-Flow Speed from Field Measurement:		
Field measured speed, SFM	-	km/h
Observed volume, Vf	-	veh/h
Estimated Free-Flow Speed:		
Base free-flow speed, BFFS	100.0	km/h
Adj. for lane and shoulder width, fLS	10.3	km/h
Adj. for access points, fA	0.7	km/h
Free-flow speed, FFS	89.0	km/h
Adjustment for no-passing zones, fnp	0.0	km/h
Average travel speed, ATS	76.4	km/h

Percent Time-Spent-Following

Grade adjustment factor, fG	0.94	
PCE for trucks, ET	1.5	
PCE for RVs, ER	1.0	
Heavy-vehicle adjustment factor, fHV	1.000	
Two-way flow rate,(note-1) vp	1003	pc/h
Highest directional split proportion (note-2)	682	
Base percent time-spent-following, BPTSF	58.6	%
Adj.for directional distribution and no-passing zones, fd/np	0.0	
Percent time-spent-following, PTSF	58.6	%

Level of Service and Other Performance Measures

Level of service, LOS	C	
Volume to capacity ratio, v/c	0.32	
Peak 15-min vehicle-kilometers of travel, VkmT15	236	veh-km
Peak-hour vehicle-kilometers of travel, VkmT60	698	veh-km
Peak 15-min total travel time, TT15	3.1	veh-h

Notes:
1. If vp >= 3200 pc/h, terminate analysis-the LOS is F.
2. If highest directional split vp >= 1700 pc/h, terminate analysis-the LOS is F.

SHORT REPORT

General Information		Site Information	
Analyst	José L. Pérez Berenguer	Intersection	PR-942 y PR-9942
Agency or Co.	Pérez Berenguer y Asociados	Area Type	All other areas
Date Performed	8/10/2006	Jurisdiction	Gurabo
Time Period	7:00 - 8:00 AM	Analysis Year	2020

Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of Lanes				1		0		1	1	0	1	
Lane Group				L	LR			T	R		LT	
Volume (vph)				336		4		30	195	4	137	
% Heavy Vehicles				0		0		0	0	0	1	
PHF				0.63		0.75		0.90	0.90	0.38	0.77	
Pretimed/Actuated (P/A)				A		A		A	A	A	A	
Startup Lost Time				2.0	2.0			2.0	2.0		2.0	
Extension of Effective Green				2.0	2.0			2.0	2.0		2.0	
Arrival Type				3	3			3	3		3	
Unit Extension				3.0	3.0			3.0	3.0		3.0	
Ped/Bike/RTOR Volume				0	0	1	0	0	195	0	0	
Lane Width				4.0	3.7			3.7	3.7		4.0	
Parking/Grade/Parking				N	8	N	N	-1	N	N	-6	N
Parking/Hour												
Bus Stops/Hour				0	0			0	0		0	
Minimum Pedestrian Time					3.2			3.2			3.2	

Phasing	WB Only	02	03	04	NS Perm	06	07	08
Timing	G = 36.0	G =	G =	G =	G = 16.0	G =	G =	G =
	Y = 4	Y =	Y =	Y =	Y = 4	Y =	Y =	Y =
Duration of Analysis (hrs) = 0.25						Cycle Length C = 60.0		

Lane Group Capacity, Control Delay, and LOS Determination												
	EB			WB			NB			SB		
Adjusted Flow Rate				533	4			33	0		189	
Lane Group Capacity				1086	941			515	438		534	
v/c Ratio				0.49	0.00			0.06	0.00		0.35	
Green Ratio				0.60	0.60			0.27	0.27		0.27	
Uniform Delay d_1				6.8	4.8			16.4	16.1		17.8	
Delay Factor k				0.11	0.11			0.11	0.11		0.11	
Incremental Delay d_2				0.4	0.0			0.1	0.0		0.4	
PF Factor				1.000	1.000			1.000	1.000		1.000	
Control Delay				7.2	4.8			16.5	16.1		18.2	
Lane Group LOS				A	A			B	B		B	
Approach Delay				7.1			16.5			18.2		
Approach LOS				A			B			B		
Intersection Delay	10.3			Intersection LOS						B		

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TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*5*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2020*
Analysis Time Period	*7:00 - 8:00 AM*		

Project Description *Estudio de Impacto en el Tránsito La Aventura*

East/West Street: *PR-9943*	North/South Street: *PR-942*
Intersection Orientation: *North-South*	Study Period (hrs): *0.25*

Vehicle Volumes and Adjustments

Major Street	Northbound			Southbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)	*19*	*15*			*53*	*0*
Peak-Hour Factor, PHF	*0.50*	*0.56*	*1.00*	*1.00*	*0.81*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*0*	*32*	*0*	*0*	*0*
Percent Heavy Vehicles	*0*	–	–	*0*	–	–
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*1*	*0*
Configuration	*LT*					*TR*
Upstream Signal		*0*			*0*	

Minor Street	Eastbound			Westbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)	*0*	*0*	*16*			
Peak-Hour Factor, PHF	*1.00*	*1.00*	*0.50*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*65*	*0*	*38*	*26*	*0*
Percent Heavy Vehicles	*0*	*0*	*0*	*0*	*0*	*0*
Percent Grade (%)	*6*			*0*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	

RT Channelized			0			0
Lanes	0	1	0	0	0	0
Configuration		LTR				

Delay, Queue Length, and Level of Service								
Approach	Northbound	Southbound	Westbound			Eastbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration	LT						LTR	
v (veh/h)	38						32	
C (m) (veh/h)	1550						1005	
v/c	0.02						0.03	
95% queue length	0.08						0.10	
Control Delay (s/veh)	7.4						8.7	
LOS	A						A	
Approach Delay (s/veh)	–	–				8.7		
Approach LOS	–	–				A		

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TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*6*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Gurabo*
Date Performed	*8/7/2006*	Analysis Year	*2020*
Analysis Time Period	*7:00 - 8:00 AM*		

Project Description *Estudio de Impacto en el Tránsito La Aventura*

East/West Street: *PR-942*	North/South Street: *PR-9941*
Intersection Orientation: *East-West*	Study Period (hrs): *0.25*

Vehicle Volumes and Adjustments

Major Street	Eastbound			Westbound		
Movement	1	2	3	4	5	6
	L	T	R	L	T	R
Volume (veh/h)		*28*	*8*	*6*	*10*	
Peak-Hour Factor, PHF	*1.00*	*0.58*	*0.30*	*0.50*	*0.38*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*48*	*26*	*12*	*26*	*0*
Percent Heavy Vehicles	*0*	–	–	*0*	–	–
Median Type	*Undivided*					
RT Channelized			*0*			*0*
Lanes	*0*	*1*	*0*	*0*	*1*	*0*
Configuration			*TR*	*LT*		
Upstream Signal		*0*			*0*	

Minor Street	Northbound			Southbound		
Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (veh/h)	*0*	*0*	*25*			
Peak-Hour Factor, PHF	*1.00*	*1.00*	*0.75*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate, HFR (veh/h)	*0*	*0*	*33*	*0*	*0*	*0*
Percent Heavy Vehicles	*0*	*0*	*0*	*0*	*0*	*0*
Percent Grade (%)	*10*			*0*		
Flared Approach		*N*			*N*	
Storage		*0*			*0*	