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Estudio de Tránsito Analizando el Impacto de la Urbanización Monte Sierra, Carr. PR-349, Km. 1.1, barrio Quebrada Grande, Mayagüez, Puerto Rico

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Resumen ejecutivo

Este informe resume los resultados de un estudio de tránsito realizado en el kilómetro 1.1 de la carretera PR-349 así como en la intersección de esta carretera con la calle Manantiales, barrio Quebrada Grande, Mayagüez, Puerto Rico. El estudio fue realizado por la firma Pérez Berenguer y Asociados por encomienda de Urbanización Monte Sierra, Inc. El objetivo del estudio es conocer el impacto en la operación de esta facilidad vial una vez se construya la urbanización Monte Sierra en el kilómetro 1.1 de la carretera PR-349, al sur de la misma. La geometría actual de la carretera PR-349, en el lugar donde se propone el acceso al proyecto consta de un carril en cada dirección, sin paseos. El pavimento de la carretera es flexible. En el lugar donde esta carretera, bajo el nombre de calle Liceo, se interseca con la calle Manantiales, ambas vías constan de un carril en cada dirección, sin paseos, con superficie asfáltica. En este punto, tanto la calle Liceo como la calle Manantiales tienen aceras a ambos lados, dado que la intersección se encuentra en la zona urbana de Mayagüez, no así el tramo de la carretera PR-349 que sale hacia el este. Se evaluó tanto la carretera en el lugar donde se construirá el acceso al proyecto como la intersección indicada ya que se verán directamente impactadas. Al presente no existen problemas en el kilómetro 1.1 de la carretera PR-349 pero la intersección analizada no puede manejar adecuadamente el flujo vehicular que la utiliza,

por ese motivo hay que hacerle ajustes. Estos ajustes consisten en construir un carril adicional en los accesos por el sur y por el este, así como aumentar el ancho del acceso por el norte. También se tiene que semaforizar dicha intersección. Se tiene que cerrar el acceso a la intersección por la calle Fortunet, ubicada al oeste, ya que aunque no se utiliza por el tránsito vehicular, si puede causar un problema de seguridad en el lugar. La intersección que se construya en el kilómetro 1.1 de la carretera PR-349 debe cumplir con todas las guías y reglamentación vigente. Especial atención se le debe prestar a la visibilidad que tengan los conductores en el sitio, ya que en el lugar la carretera PR-349 tiene varias curvas. Estas recomendaciones asegurarán que las facilidades analizadas tengan capacidad suficiente para manejar el tránsito que las utilizará a corto, mediano y largo plazo. Es importante notar que los ajustes a la intersección de la carretera PR-349 con la calle Manantiales son necesarios independientemente de si se construye la urbanización Monte Sierra o no, por lo que los mismos no se deben, necesariamente, al impacto que producirá el proyecto. Una vez se hayan implantado las recomendaciones aquí formuladas las vías del área manejarán adecuadamente el flujo adicional que producirá la urbanización.

Introducción y Objetivo

Este estudio de tránsito se condujo para conocer el impacto que tendrá el propuesto proyecto residencial urbanización Monte Sierra, a construirse en la carretera PR-349, Km. 1.1, barrio Quebrada Grande, Mayagüez, Puerto Rico (figuras 1 y 2). Urbanización Monte Sierra, Inc. encomendó a Pérez Berenguer y Asociados la realización del estudio. En este estudio se analizó la condición existente en la carretera PR-349, Km. 1.1 (figura 3) así como la intersección de esta carretera con la calle Manantiales, en la zona urbana de Mayagüez (figura 4). Se obtuvieron las horas pico en el lugar indicado y el flujo vehicular durante estas horas para un día laborable típico. Con estos datos se realizaron análisis operacionales para conocer como la carretera PR-349 y su intersección con la calle Manantiales le sirven a los conductores al momento del conteo. Se realizó una proyección del flujo vehicular al año 2006, suponiendo que para ese año esté finalizado el proyecto propuesto, para tener una idea de cual será su efecto en las facilidades viales cercanas. También se hizo una proyección del flujo vehicular al año 2026 para tener una idea de como operarán las facilidades analizadas a largo plazo. Los resultados permitieron formular recomendaciones relacionadas a las medidas que se deben implantar en las facilidades viales existentes para que las mismas operen satisfactoriamente a corto, mediano y largo plazo.

Antecedentes

Este estudio se realizó debido a la necesidad de conocer el efecto de la urbanización Monte Sierra en la vía adyacente. Así se pudieron recomendar medidas a tomar para que ésta se mantenga operando de forma satisfactoria a corto, mediano y largo plazo. La urbanización Monte Sierra será un proyecto de interés social que se ubicará al sur del kilómetro 1.1 de la carretera PR-349, por donde será su acceso. La urbanización constará de 302 residencias individuales, en solares con una cabida aproximada de 350 metros y una área comercial (figura 5). El proyecto producirá un incremento en el tránsito existente en el área. Por esto es importante conocer como operará la carretera en el futuro.

Se realizaron conteos vehiculares en los dos carriles de la carretera PR-349, así como en todos los carriles de su intersección con la calle Manantiales, para determinar cuales son las horas pico en un día laborable típico y cuantos vehículos los utilizan durante esas horas, así como durante todo el día. Con los datos que surgieron de los conteos, una vez reducidos, se realizó el análisis operacional para la condición actual. Se proyectaron estos datos al momento en el que se espera que la urbanización Monte Sierra esté ocupada, se sumaron los que generará este proyecto y se efectuó el respectivo análisis operacional. Así se pudo tener una idea de como operarán estas facilidades en el momento indicado.

Aunque varias calles intersecan a la carretera PR-349 entre el lugar donde se propone el acceso a la urbanización Monte Sierra y la intersección con la calle Manantiales, estas no se analizaron por la siguiente razón. Estas calles son calles locales que son utilizadas principalmente por los residentes de las mismas, quienes componen las comunidades La Mineral y Felices Días, entre otras. Muchas de estas calles no tienen salida, por lo que personas que no vayan para las mismas no las utilizan. Debido a que la intersección más inmediata de mayor importancia que utilizarán los viajes que generará el proyecto propuesto será la de la carretera PR-349 con la calle Manantiales y ya que estos vehículos harán cualquiera de los movimientos permitidos en esa intersección, la misma fue evaluada en este estudio.

La carretera PR-349 comienza en su intersección con la carretera PR-105 y termina en el Centro Residencial de Oportunidades Educativas de Mayagüez (CROEM), ubicado en el barrio Rosario de este municipio. La carretera fue construida hace más de sesenta años para servir a la Base de Proyectiles Teledirigidos del Ejército de los Estados Unidos, la cual ubicaba en el lugar donde hoy radica la instalación educativa CROEM. En el punto donde comienza la carretera la misma se llama calle Liceo y la carretera PR-105 lleva el nombre de calle McKinley. La calle Liceo continúa hacia el sur hasta intersecar con la calle Manantiales donde la carretera PR-349 continua hacia el este. De aquí en adelante la

carretera sirve como el acceso principal a las comunidades La Mineral, Quintas de San Francisco y al Cerro de las Mesas. Este tramo también sirve a instalaciones tales como el Hospital Bella Vista, el centro ESPIBI para tratamiento a personas con impedimentos y la escuela CROEM, entre otros, así como instalaciones privadas de radio y telecomunicaciones. Desde la calle Manantiales hasta el final, la carretera PR-349 es mayormente montañosa. Poco antes del fin, la carretera se interseca con la carretera PR-3349, la cual permite a los usuarios llegar hasta la parte sur del municipio de Mayagüez, al Poblado Rosario en el municipio de San Germán o al municipio de Hormigueros. El proyecto residencial que aquí se evalúa estará ubicado en el tramo de la carretera PR-349 entre La Mineral y el área residencial del Cerro de las Mesas. Dada la cantidad de instalaciones y áreas residenciales que existen en la carretera PR-349, la misma es bastante utilizada para las características que tiene.

La geometría de la carretera PR-349 se compone mayormente de un carril en cada dirección con un ancho de alrededor de 3.00 metros, sin paseos, con superficie asfáltica. En algunos lugares, especialmente en el tramo frente al proyecto propuesto, el ancho de los carriles es mayor, pudiendo sobrepasar los cuatro metros cada uno. Esto se debe a que en ese sector la carretera PR-349 tiene muchas curvas y se han hecho mejoras geométricas para incrementar la visibilidad y la seguridad de los usuarios.

Alcance del estudio

Con este estudio se pretende principalmente conocer el impacto en el tránsito y operación de la carretera PR-349 debido a la influencia directa de la urbanización Monte Sierra. Como resultado del análisis se obtuvieron las horas pico en un día laborable típico, el volumen y el Nivel de Servicio actual y futuro durante esas horas, al que operan y operarán las intersecciones evaluadas. También se calculó el tránsito promedio diario en el kilómetro 1.1 de la carretera PR-349 y en su intersección con la calle Manantiales. Se realizaron conteos por períodos 24 horas consecutivas los días martes, 9 y 16 de noviembre de 2004. Tanto el día previo como el posterior al período de 24 horas tenían que ser laborables para tomar en cuenta la información obtenida. Este tipo de estudio permite conocer si las facilidades viales existentes y futuras son adecuadas para manejar satisfactoriamente el volumen vehicular al que se verán sujetas o si hay que modificarlas. Todo el volumen vehicular fue captado por un equipo electrónico de conteo de alta precisión ("Countcard"), lo que permite obtener datos de buena calidad. Esto, junto con la constante supervisión del consultor encargado del estudio permitió tener resultados confiables.

Métodología utilizada

El método utilizado para obtener los datos fue el conteo y clasificación de vehículos según el movimiento que estos efectuaron. Se instalaron detectores electrónicos ("Countcard") en el pavimento de cada carril de la carretera y la intersección analizadas. Estos detectores se programan de antemano para que realicen las lecturas durante el período y en los intervalos deseados. Por medio de ondas magnéticas, el detector capta cuando un vehículo pasa sobre él, lo registra y lo clasifica. Luego esa información se pasa directamente a la computadora, utilizando el programa CDM 20/30 (**Nu-Metrics, 1994**), de donde se obtienen los informes relacionados. Este equipo ofrece resultados exactos y asegura que se utilizaron datos reales para efectuar los análisis pertinentes.

El conteo de flujo vehicular se efectuó por períodos de 24 horas desde la medianoche de martes hasta la medianoche de miércoles. Para asegurar la validez de los datos, las 24 horas previas y posteriores al conteo fueron laborables. Al hacer los conteos de la forma como se describe aquí se obtiene un flujo vehicular más estable y se consiguen datos de mejor calidad. Cuando el objetivo del estudio de tránsito es captar el comportamiento del flujo vehicular en cierto lugar en un día laborable y la instalación propuesta no altere el comportamiento de los conductores, de forma que días no laborables se conviertan en

días críticos, se recomienda realizar el trabajo de campo en los días martes, miércoles o jueves (**Robertson et. al., 1994**) (**Homburger et. al., 1992**). Los conteos se dividieron en intervalos de 15 minutos durante todo el día, para tener una idea más exacta de las variaciones del tránsito a través de los períodos estudiados.

Se utilizó la hoja de cálculo electrónica *Quattro Pro* versión 9.0.0.588 (**Corel Corporation Limited, 1999a**) para reducir los datos de flujo vehicular y hacer un análisis matemático de los mismos. Para hacer el análisis operacional de las facilidades viales se utilizaron las guías que aparecen en la referencia *Highway Capacity Manual* (**TRB, 2000**), parte de las que se ilustran en el Apéndice 2. Esta referencia describe las guías para el análisis operacional de facilidades de transportación que actualmente utilizan las agencias encargadas de la operación y mantenimiento de las carreteras en Puerto Rico y Estados Unidos. Para facilitar este análisis se utilizó el programa de computadora *Highway Capacity Software*, versión 4.1e (**University of Florida, 2003**) el cual resume electrónicamente los procedimientos para el análisis sugeridos en el *Highway Capacity Manual*. Con las referencias *Trip Generation* (**ITE, 2003a, b**) y *Trip Generation Handbook* (**ITE, 2001**) se estimaron cuantos viajes adicionales se espera que genere el proyecto. El informe se redactó utilizando el procesador de palabras *WordPerfect* versión 9.0.0.588 (**Corel Corporation Limited, 1999b**). Algunas de

las ilustraciones que aparecen en el informe se realizaron con el programa AutoCad 2000 (**Autodesk, 1999**). Utilizando estas referencias y herramientas se realizó el estudio de impacto en el tránsito resumido en este informe.

Datos recopilados

Los datos recopilados durante los conteos fueron el flujo vehicular y el porciento de camiones en las dos direcciones en el kilómetro 1.1 de la carretera PR-349 y en todos los movimientos de la intersección de esta carretera con la calle Manantiales. Para poder analizar los datos de forma tal que se pueda conocer el flujo vehicular que realiza los movimientos en y fuera de las horas pico, estos se identificaron utilizando una nomenclatura de dos letras donde la primera indica desde donde transita el vehículo y la segunda se refiere sobre hacia donde va. Por ejemplo, N-E se refiere al movimiento que hacen los vehículos que transitan de norte a este. Un esquema describiendo los movimientos arriba mencionados se puede apreciar en la figura 6. Una vez se recopilaron y redujeron los datos se procedió a realizar el análisis de los mismos.

Análisis realizados

Los datos de flujo vehicular se recopilaron en períodos de 15 minutos, tal y como aparecen en el Apéndice 3. Con estos datos se produjeron gráficas que permiten visualizar el comportamiento del flujo vehicular en el lugar durante un día laborable típico. Estas gráficas permiten también efectuar comparaciones en los datos de flujo para cada movimiento entre diferentes períodos del día. Se calcularon parámetros que describen el comportamiento del flujo vehicular en una facilidad vial tales como:

- Identificación de las horas pico del día
- Flujo vehicular durante las horas pico
- Factor de la hora pico (PHF, por sus siglas en inglés)
- Porciento del flujo vehicular durante las horas pico compuesto por camiones
- Nivel de servicio (LOS, por sus siglas en inglés) al que operan las facilidades analizadas durante las horas críticas del día
- Estimado del tránsito promedio diario en el lugar.

Estos valores se calcularon para la situación al momento del conteo, se proyectaron para la situación al momento de ocupada la urbanización Monte Sierra y para la situación 20 años después. Los valores para estos parámetros se calcularon utilizando procedimientos y fórmulas aceptados (**TRB, 2000**) (**University of Florida, 2003**) (**Khisty, 1990**) (**Robertson et. al., 1994**).

El método utilizado en este estudio para conocer el funcionamiento de las facilidades viales es el que se sugiere en la referencia *Highway Capacity Manual* (**TRB, 2000**). El mismo describe las condiciones en una facilidad de transportación utilizando el concepto de Nivel de Servicio. Este concepto es una medida cualitativa que describe las condiciones operacionales dentro de una corriente de tránsito basado en medidas de servicio tales como velocidad, tiempo de viaje, libertad de manejo, interrupciones del tránsito, comodidad y conveniencia.

Se han definido seis LOS para cada tipo de facilidad donde existe un procedimiento de análisis establecido. Estos se designan con las letras desde la A a la F, siendo A la que representa las mejores condiciones operacionales y F las peores. Cada LOS representa un rango de condiciones operacionales. Las agencias encargadas del mantenimiento y operación de las carreteras generalmente buscan que las facilidades de transportación operen a un LOS D o mejor.

En intersecciones sin semáforo con dispositivos para el control del tránsito en los accesos menores el Nivel de Servicio no se define para la intersección completa. El LOS se determina basado en la demora por control medida o calculada para cada movimiento en conflicto con otro. En los accesos a intersecciones regulados por señales de "Pare", como en el caso aquí evaluado, el tiempo de detención no tiene sentido porque en vez de haber colas

más o menos estables, existen filas de vehículos avanzando y parándose intermitentemente. En ellas se ha utilizado más bien la "demora por control". Las demoras por control son calculadas en segundos por vehículo y representan el tiempo desde que el conductor llega al final de la cola hasta que sale de la linea de "Pare" de la intersección (**Radelat, 2003**). Este parámetro mide la dificultad que tiene el conductor que llega a la intersección por la calle menor para determinar la brecha entre vehículos de la calle mayor que le permita hacer el movimiento para salir. Mientras más altos sean los valores de densidad, menos brechas adecuadas existirán, incrementandose la dificultad para que los conductores de la calle menor puedan salir de la intersección.

En carreteras de dos carriles el Nivel de Servicio depende de la clasificación de la carretera. Este tipo de carretera se divide en Clase I para las que se transita a velocidades relativamente altas y Clase II para las que se transita a velocidades relativamente bajas (**TRB, 2000**). En el caso aquí analizado la carretera es de Clase II ya que pasa sobre terreno montañoso y sirve como una ruta escénica. En carreteras Clase II la movilidad es menos crítica y el LOS se define en términos del porciento del tiempo de seguimiento ("time-spent-following"). Esta medida toma en cuenta el tiempo que un conductor ha transitado por la carretera siguiendo a otro vehículo. En carreteras de Clase II los conductores toleran niveles más altos del porciento del tiempo de

seguimiento que en una facilidad de Clase I, porque usualmente las facilidades Clase II sirven viajes más cortos y con diferentes propósitos.

En intersecciones semaforizadas el LOS se define en términos de demora por control promedio por vehículo para cada grupo de carriles. Esta se agrega por acceso a la intersección y para la intersección completa. La demora por control promedio es una medida de la incomodidad del conductor, su frustración, consumo de combustible y tiempo de viaje perdido. La medida cuantifica la demora por deceleración inicial, tiempo en el que el conductor se mueve en la cola, demora detenido y demora en la aceleración final. La demora experimentada por un motorista se debe a una serie de factores que relacionan el control, la geometría, el tránsito y los incidentes.

Otro concepto que se utilizó en el análisis de los datos fue el Factor de la Hora Pico. Este factor es el resultado de la división del flujo vehicular total durante la hora pico por cuatro veces el flujo del período de 15 minutos pico durante esta hora. El valor de este factor tiene que estar entre 0.25 y 1.00, donde el primer valor corresponde a que todo el flujo vehicular de la hora pico pasó durante uno de los períodos de 15 minutos que lo componen. El segundo valor representa cuando el flujo vehicular en los cuatro períodos de 15 minutos que componen la hora pico es igual. El rango normal de valores está entre 0.70 y 0.98. Valores

menores de 0.70 representan un mayor grado de variación en el flujo durante la hora pico. El PHF es una medida descriptiva de las características de generación de viajes (**McShane, et.al., 1998**). Una descripción más detallada de estas guías y conceptos aparece en el Apéndice 2.

Hallazgos Obtenidos

En la figura 7 se puede apreciar el comportamiento del flujo vehicular en un día laborable típico en el kilómetro 1.1 de la carretera PR-349. Se observa que el volumen comienza a aumentar a eso de las 4:15 de la madrugada hasta las 8:15 de la mañana, cuando se registra un pico. El flujo vehicular baja hasta las 9:00 de la mañana, desde donde se mantiene constante hasta las 11:45 de la mañana, cuando comienza a aumentar nuevamente hasta las 3:00 de la tarde, cuando se registra otro pico. Desde aquí en adelante comienza a disminuir hasta llegar a un mínimo durante la madrugada. Los picos registrados responden a los generadores existentes en el sector. Aunque el tránsito en ambas direcciones es casi similar durante todo el día, durante el pico de la mañana el tránsito hacia el oeste es un poco mayor. Esto se debe a las personas que salen de las residencias ubicadas en el Cerro de las Mesas hacia sus trabajos, estudios o a realizar gestiones. Durante el pico de la tarde el tránsito hacia el este es un poco mayor al que va en dirección contraria debido a que a media tarde una cantidad considerable de personas llegan hasta el Hospital Bella Vista a visitar a familiares y conocidos que estén recluídos en la institución. De hecho, el tránsito que genera el Hospital Bella Vista es uno constante durante todo el día, debido a los diferentes servicios que allí se ofrecen. Por esa razón, el volumen vehicular es similar en ambas direcciones durante todo el día. El que el

patrón del flujo vehicular se vea influenciado por el patrón de generación de viajes del Hospital Bella Vista demuestra que este es el principal generador en el sector.

En la figura 8 se puede apreciar el comportamiento del flujo vehicular en un día laborable típico en la intersección de la carretera PR-349 (calle Liceo en su tramo hacia el norte) con la calle Manantiales. Se observa que el volumen comienza a aumentar a eso de las 4:15 de la madrugada hasta las 7:30 de la mañana, cuando se registra un pico. El flujo vehicular baja hasta las 10:00 de la mañana, cuando comienza a aumentar nuevamente hasta las 3:30 de la tarde, cuando se registra otro pico. Desde aquí comienza a disminuir hasta llegar a un mínimo durante la madrugada. El pico de la mañana se debe principalmente al tránsito de los movimientos S-E, E-S y N-S. Estos movimientos los producen las personas que van hacia generadores ubicados en el Cerro de las Mesas, tales como el Hospital Bella Vista y el centro ESPIBI, a personas que salen de las áreas residenciales ubicadas a lo largo de la carretera PR-349 y de personas residentes de otras áreas de la ciudad o de municipios aledaños que transitan por la intersección para llegar al centro urbano de Mayagüez, tanto para trabajar como para estudiar o a realizar gestiones. Durante la hora pico de la tarde los movimientos E-S, N-E y S-E son los que más influyen en el patrón del flujo vehicular de la intersección analizada. Estos movimientos los realizan las personas que salen

de los generadores ubicados en el Cerro de las Mesas así como las personas que residen en alguna comunidad ubicada a lo largo de la carretera PR-349 y que regresan a sus viviendas después de trabajar, estudiar o realizar gestiones. La mayor influencia en el tránsito de esta intersección la ejercen las comunidades ubicadas a lo largo de la carretera PR-349 así como el Hospital Bella Vista y otros generadores más pequeños ubicados en el Cerro de las Mesas.

El patrón del comportamiento descrito en las figuras 7 y 8 no debe cambiar con la construcción de la urbanización Monte Sierra ya que el actual se debe en gran medida a proyectos residenciales y otros generadores ubicados a lo largo de la carretera PR-349. El cambio que se podría observar es que se carguen más los movimientos desde y hacia el este.

Las horas pico en los lugares evaluados se registraron entre **7:00** y **8:00** de la mañana y entre **2:45** y **3:45** de la tarde. La primer hora pico responde principalmente a personas viajando de su residencia a su trabajo, estudios o gestiones así como personas que van a citas médicas en el Hospital Bella Vista o en el centro ESPIBI mientras que la segunda hora pico se debe a las personas que salieron del hospital o del centro de rehabilitación o que regresan a sus viviendas.

El tránsito que utiliza el kilómetro 1.1 de la carretera PR-349 durante la hora pico de la mañana es **174** vehículos por hora y durante la hora crítica de la tarde es **181** vehículos por hora. La

intersección de la carretera PR-349 con la calle Manantiales es utilizada por **1348** vehículos durante la hora pico de la mañana y **1368** vehículos durante la hora pico de la tarde. Los volúmenes que se registran en la intersección son relativamente altos para sus características y se deben a la función que tiene, descrita anteriormente. El flujo por movimiento durante las horas pico es el que se utilizó para realizar el análisis operacional de las facilidades viales indicadas. Estos volúmenes junto con el PHF para cada movimiento aparecen en las figuras 9 y 10.

El patrón de flujo vehicular que se registra en las facilidades viales aquí analizada demuestra que son utilizadas principalmente por los llamados "conmuters". Estas son las personas que realizan prácticamente el mismo viaje todos los días desde su casa, al trabajo, a estudios o a realizar alguna gestión y regresan a la casa. El hecho se verifica al observar a la hora que aumenta o disminuye drásticamente el flujo vehicular. Se puede decir que los "conmuters" forman el grupo principal entre los usuarios de la facilidad analizada.

En el kilómetro 1.1 de la carretera PR-349 el PHF para la hora pico de la mañana es **0.89** mientras que durante la hora pico de la tarde es **0.72.** En la intersección de la carretera PR-349 con la calle Manantiales se registra un PHF de **0.93** para la hora pico de la mañana y de **0.85** para la hora pico de la tarde. Los valores registrados indican que el flujo vehicular durante las horas pico

es uniforme y continuo, por lo tanto, las facilidades viales analizadas tienen demanda continua en esos períodos de tiempo.

Tabla 1. Nivel de Servicio y Demora por Control para cada acceso de la intersección de la carretera PR-349 y la calle Manantiales. La Demora por Control aparece entre paréntesis, en segundos por vehículo

Hora Pico	Acceso por el Este	Acceso por el Sur	Acceso por el Norte
AM	A (7.8)	F (221.7)	F
PM	A (8.4)	F (64.6)	F

Basado en estos datos para las horas pico del día se realizaron análisis operacionales del tramo de la carretera PR-349 frente a donde se propone el proyecto, y de la intersección de esta carretera con la calle Manantiales, ya que se verán directamente impactados por la urbanización propuesta. Se utilizó el programa de computadora HCS, versión 4.1e (**University of Florida, 2003**), que se basa en el método del Manual de Capacidad de Carreteras (HCM, por sus siglas en inglés) (**TRB, 2000**). Por medio de este procedimiento se determina a que Nivel de Servicio (LOS, por sus siglas en inglés) opera la sección. Los resultados obtenidos indican que durante ambas horas pico el LOS de la carretera PR-349 frente al proyecto propuesto es A pero que en la intersección de la carretera PR-349 con la calle Manantiales hay problemas. En la tabla 1 se observa que, aunque la llegada a la intersección desde el Cerro de las Mesas y, por lo tanto, del predio donde se propone el proyecto, opera de forma excelente, los otros dos accesos a la misma no tienen la capacidad para manejar la demanda. De hecho,

las demoras que experimentan los usuarios que llegan a la intersección por el norte, durante ambas horas pico, son tan altas que el programa no las estima. Estos resultados demuestran que en esta intersección existen serios problemas de capacidad actualmente y que debe mejorarse la misma, independientemente de si se construye el proyecto o no. En las figuras 11 y 12 se ilustran gráficamente los Niveles de Servicio obtenidos en el análisis para las dos facilidades indicadas. Los cómputos de este análisis operacional se ilustran en el Apéndice 4.

Basado en el análisis de los datos obtenidos en el conteo realizado se estimó la cantidad promedio de vehículos que durante un período de 24 horas utilizan las facilidades estudiadas. Se estimó que unos **1765** vehículos transitan por el kilómetro 1.1 de la carretera PR-349 durante un día laborable típico y **15115** vehículos transitan por la intersección de esta carretera con la calle Manantiales. Estos resultados indican que, aunque el tránsito que produce la carretera PR-349 en su tramo desde la zona urbana hacia el Cerro de las Mesas impacta directamente a la intersección aquí evaluada, la mayoría del transito que la utiliza proviene del tránsito del centro urbano de Mayagüez así como del que se produce en las comunidades ubicadas entre los dos puntos aquí evaluados.

Se calculó la razón a la que el tránsito que utiliza las facilidades analizadas ha crecido a través de los años. Debido a que no existen suficientes datos históricos del tránsito promedio

diario de esta carretera se supuso que el tránsito en las facilidades evaluadas crecerá a la misma razón que los vehículos registrados en el municipio de Mayagüez. Se hizo esta suposición debido a la obvia relación entre los vehículos registrados en un municipio y el volumen vehicular que utiliza sus vías. El factor de crecimiento obtenido se utilizó para proyectar los datos de flujo vehicular al futuro. Los datos históricos de vehículos registrados en Mayagüez se obtuvieron de informes del Departamento de Transportación y Obras Públicas.

Tabla 2. Vehículos Registrados en el Municipio de Mayagüez

Año	Vehículos Registrados
1989	45439
1990	47196
1991	45586
1992	46572
1993	48872
1994	52962
1995	55357
1996	56861
1997	58330
1998	61350
1999	61509
2000	52142
2001	52631
2002	53202

Fuente: Departamento de Transportación y Obras Públicas

Con los datos históricos se desarrolló una ecuación de regresión lineal. Una vez se obtuvo esta ecuación se proyectó la cantidad de vehículos registrados en Mayagüez al año 2026 y se determinó el factor de crecimiento (i) utilizando la *ley de interés compuesto*[1] (**Sepúlveda, et.al., 1984**). Ésta se utiliza en la Ingeniería de Tránsito para determinar el factor de crecimiento. Se obtuvo un factor de crecimiento (i) de **1.4%** anual. Este valor de i es aceptable y es cónsono con el comportamiento del entorno ya que hace algún tiempo no se construyen grandes generadores en el sector. Debido a las características topográficas del área, aparte del proyecto propuesto, no se espera que el patrón de desarrollo cambie en los próximos años. Por esos motivos, este valor para i se utilizó para proyectar el tránsito a mediano y a largo plazo.

Utilizando el valor de i obtenido y con la ley de interés compuesto se proyectaron los valores actuales de tránsito vehicular en las facilidades analizadas para ambas horas pico. Esta proyección se realizó para el año en el que se espere que el proyecto propuesto esté ocupado. A estos datos se le sumaron los viajes que se espera que genere la urbanización Monte Sierra. Se supuso que las horas pico del día y los valores del Factor de la

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[1] $F = P(1+i)^n$

donde:

F = valor futuro
P = valor presente
i = factor de crecimiento
n = número de períodos (años)

Hora Pico se mantendrán iguales porque no se espera que las características del lugar cambien. Estos supuestos se basan en el análisis del patrón del flujo vehicular resumido anteriormente, en la naturaleza del proyecto propuesto y en las características del entorno. El análisis para la proyección de los datos aparece en el Apéndice 5. Los datos de tránsito obtenidos para el año 2006 fueron proyectados al año 2026 utilizando el mismo procedimiento.

El estimado del volumen que producirá el proyecto propuesto se realizó utilizando el procedimiento sugerido por el Instituto de Ingenieros de Transportación (ITE, por sus siglas en inglés). El procedimiento aparece en la séptima edición de la publicación *Trip Generation* (**ITE, 2003a,b**). Los datos utilizados para esta publicación fueron tomados desde los años sesenta hasta la década del 2000 a través de los Estados Unidos y Canadá. La publicación divide los terrenos según su uso. Para este caso se utilizaron los usos codificados 210 ("*Single-Familiy Detached Housing*") y 820 ("*Shopping Center*") por esta referencia. Estos usos se basan en los que se han propuesto para el proyecto. Un esquema de los usos propuestos en el proyecto se ilustra en la figura 5.

Según *Trip Generation* el uso de terreno 210 incluye todas las unidades sencillas de vivienda unifamiliares, en lotes separados. Un ejemplo típico de este uso de terreno es una subdivisión suburbana o una urbanización. Este uso de terreno incluye información para una amplia variedad de unidades con diferentes

tamaños, rangos de precios, localizaciones y edades. Como se espera, las unidades más grandes, más costosas o más lejos de los centros urbanos tienen una razón de generación de viajes más alta que las más pequeñas, menos costosas o más cercanas al centro urbano. Otros factores como, por ejemplo, la localización geográfica y el tipo de desarrollo adyacente y colindante, pueden tener un efecto en la generación de viajes del lugar. En este estudio se utilizó este uso de terreno para conocer cuantos viajes generará la parte residencial de la urbanización Monte Sierra durante las horas pico.

Este uso de terreno tiene la razón de generación de viajes mayor por unidad de vivienda entre todos los usos residenciales porque son las unidades más grandes en tamaño y tienen más residentes y vehículos por unidad que otros usos de terreno residenciales. Las unidades residenciales de este uso de terreno generalmente están localizadas más lejos de grandes centros comerciales, áreas de empleo y de otras instalaciones que atraen viajes, al compararlas con otros usos residenciales. También este uso de terreno generalmente tiene menos modos de transportación alternos disponibles porque típicamente no es tan denso como otros usos de terreno residenciales. Las horas pico de este generador típicamente coinciden con las horas pico del tránsito de las calles adyacentes.

Trip Generation define el uso de terreno 820 como un grupo integrado de establecimientos comerciales que es de un solo dueño, planificado, desarrollado y administrado como una unidad. La composición del centro comercial depende de su área de mercado en términos de tamaño, localización y tipo de tienda. Un centro comercial también provee suficientes facilidades de estacionamiento en el lugar para servir su propia demanda. En este estudio se utilizaron los datos de este uso de terreno para conocer la generación de viajes del solar que se propone para uso comercial en la urbanización Monte Sierra.

Como se puede apreciar, las descripciones y supuestos con los que se desarrolló la información que provee la referencia *Trip Generation* coinciden con las características que tendrá el proyecto aquí evaluado. Por este motivo es adecuado aplicar la información que provee la referencia a los análisis realizados y resumidos en este informe. La cantidad de viajes que generará el proyecto propuesto y como se distribuirán por las intersecciones analizadas se resume en la figura 13.

Utilizando los procedimientos y supuestos descritos en los últimos párrafos se obtuvieron datos acerca de la cantidad de vehículos que utilizarán las intersecciones analizadas durante las horas pico en los años 2006 y 2026. Estos datos se resumen en las figuras 14 a la 17. Para obtener estos datos se sumaron los volúmenes actuales proyectados y los que generará la urbanización

Monte Sierra. El patrón del viajes en las horas pico será similar al actual. Los detalles sobre como se calcularon los viajes que generará el proyecto propuesto, como se asignaron y como se proyectaron los volúmenes aparecen en el Apéndice 6 del informe.

Debido a que la intersección de la carretera PR-349 con la calle Manantiales no funciona adecuadamente, se evaluó cambiar los dispositivos de control del tránsito en la misma así como la geometría. Para recomendar los dispositivos de control del tránsito a utilizar, antes de realizar el análisis operacional de la intersección, se evaluó utilizando como guías las justificaciones[2] que aparecen en el Manual de Dispositivos de Control del Tránsito Uniformes (MUTCD, por sus siglas en inglés) (**ATSSA/ITE/AASHTO, 2003**). Al aplicar estas justificaciones a la situación que se experimenta en la intersección se tiene una recomendación acerca de si la intersección debería estar controlada por un sistema de semáforos o por otros dispositivos. Las justificaciones aparecen en el Apéndice 7 de este informe. Al aplicar las justificaciones relacionadas al tránsito durante ocho horas del día, cuatro horas del día y durante la hora pico se obtuvo que es recomendable la instalación de un sistema de semáforos en el lugar. Esta evaluación, con los datos futuros, también se hizo para la intersección que se formará al construir el

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

acceso a la urbanización Monte Sierra. De la misma se obtuvo que no es necesario instalar un sistema de semáforos allí.

Del análisis operacional para la condición actual se obtuvo que la capacidad de la intersección está por debajo de la demanda. Por esta razón también se propone que se añada un carril a los accesos a la intersección por el sur y por el este. También se debe ampliar el carril que llega al lugar por el norte. De esta manera se aumenta la capacidad de la intersección. Además de estos ajustes se debe modificar la intersección de manera que la carretera PR-349 llegue a la calle Manantiales a un ángulo más cercano a los 90 grados. El ángulo al que actualmente se encuentran ambas vías hacen de esta intersección una peligrosa ya que hay áreas donde la visibilidad no es la adecuada, lo que se incrementa con las pendientes pronunciadas que existen en los tres accesos. Esto también aplica, por las mismas razones, a la intersección que se formará en la entrada al proyecto propuesto. Tanto lo indicado en este párrafo como en el anterior debe ser implantado en la intersección de la carretera PR-349 con la calle Manantiales independientemente de si se construye el proyecto propuesto o no, ya que la situación actual no se debe al mismo.

En la figura 18 se resumen los resultados del análisis operacional de la entrada al proyecto propuesto para ambas horas pico para el escenario durante el año 2006 así como en la figura 19 se ilustran los resultados correspondientes para la intersección de

la carretera PR-349 con la calle Manantiales. En las figuras 20 y 21 se ilustran los resultados correspondientes para ambos escenarios durante ambas horas pico para el año 2026. En los análisis operacionales para ambas fechas se supusieron los cambios en la geometría propuesta y en los dispositivos de control del tránsito, indicados en los pasados párrafos, para asegurar el mejor funcionamiento de las vías.

Tabla 3. Nivel de Servicio y Demora por Control que se experimentarán durante las horas pico, en el año 2006, en las intersecciones evaluadas en este estudio. Las Demoras por Control aparecen entre paréntesis, expresadas en segundos por vehículo

Hora Pico	Acceso por el Sur	Acceso por el Norte	Acceso por el Este	Acceso por el Oeste	Total
Intersección Carretera PR-349 y Calle Núm. 1					
AM	B (11.9)		A (7.7)		
PM	C (17.3)		A (8.5)		
Intersección Carretera PR-349 (Calle Liceo) y Calle Manantiales					
AM	A (7.4)	B (12.0)	C (23.3)		B (14.3)
PM	B (15.7)	E (67.1)	E (63.1)		D (51.2)

De acuerdo con los resultados obtenidos, resumidos en la tabla 3, las facilidades viales modificadas podrán manejar aceptablemente el volumen vehicular para el año 2006. Se observa que solamente, durante la hora pico de la tarde, los accesos por la carretera PR-349 operarán a capacidad en su intersección con la calle Manantiales. Aun así, la intersección en su totalidad operarán de forma satisfactoria. El acceso a la urbanización Monte Sierra operará muy bien durante ambas horas pico del año 2006. Estos resultados se ilustran gráficamente en las figuras 18 y 19.

Tabla 4. Nivel de Servicio y Demora por Control que se experimentarán durante las horas pico, en el año 2026, en las intersecciones evaluadas en este estudio. Las Demoras por Control aparecen entre paréntesis, expresadas en segundos por vehículo

Hora Pico	Acceso por el Sur	Acceso por el Norte	Acceso por el Este	Acceso por el Oeste	Total
Intersección Carretera PR-349 y Calle Núm. 1					
AM	B (14.4)		A (7.8)		
PM	E (38.1)		A (9.2)		
Intersección Carretera PR-349 (Calle Liceo) y Calle Manantiales					
AM	B (12.0)	D (53.3)	D (50.4)		D (37.3)
PM	B (12.9)	E (77.8)	E (75.3)		E (58.4)

Al efectuar el análisis operacional para el escenario correspondiente al año 2026 se obtuvo que las facilidades, aunque experimentarán una disminución en la calidad del servicio brindan, podrán manejar el flujo vehicular que las utiliza ya que la demoras que sufrirán los usuarios aun serán tolerables. Se observa que el caso más crítico será en la intersección de la carretera PR-349 con la calle Manantiales durante la hora pico de la tarde. En este momento la intersección operará a capacidad. Durante el resto del día los usuarios de ambas intersecciones experimentarán demoras menores.

De acuerdo a los resultados obtenidos, las facilidades viales, con los ajustes aquí indicados, podrán manejar el tránsito que utilizará las vías impactadas por la urbanización Monte Sierra tanto desde el momento en el que se ocupe el proyecto como veinte años después. Los cómputos para efectuar los análisis operacionales aparecen en el Apéndice 4. De acuerdo con estos

resultados, la geometría y dispositivos de control del tránsito existentes en la intersección de la carretera PR-349 con la calle Manantiales debe modificarse para que puedan manejar adecuadamente el tránsito vehicular que la utiliza. Por otro lado, la intersección propuesta para entrar a la urbanización Monte Sierra debe modificarse de manera que el ángulo que se forme entre la carretera PR-349 y la calle Núm. 1 se acerque, en lo posible, a 90 grados. Los resultados indican que el área analizada opera actualmente de forma deficiente y operará de forma adecuada a mediano y largo plazo haciendo los cambios aquí propuestos. Se debe tener en consideración que las mejoras a la intersección de la carretera PR-349 con la calle Manantiales son necesarias, independientemente de si se construye el proyecto propuesto o no. Por esta razón los desarrolladores de la urbanización Monte Sierra solamente deben costear la intersección de acceso a su proyecto y las mejoras que haya que hacer a la carretera PR-349 exclusivamente en este lugar.

Conclusiones

La urbanización Monte Sierra proveerá al municipio de Mayagüez con 302 nuevas unidades de vivienda. Existiendo una necesidad de vivienda en el área oeste, el proyecto ayudará a que varias familias no tengan que mudarse a otros municipios, produciendo beneficios para la economía de Mayagüez. Este proyecto se convertirá en el límite de la zona urbana de Mayagüez, integrándose a la comunidad existente en el área. Esto permitirá que los residentes de la urbanización satisfagan sus necesidades en el centro urbano de Mayagüez, aunque el proyecto contará con un área comercial que pondrá a disposición de los residentes del lugar servicios y artículos de primera necesidad de forma inmediata. La carretera PR-349 podrá manejar el tránsito que la utilizará una vez se haya construido el proyecto propuesto, aunque será necesario modificar su intersección con la calle Manantiales para poder seguir sirviendo a los usuarios, independientemente de si se construye la urbanización Monte Sierra o no. Se deben implantar las mejoras a la geometría y dispositivos de control del tránsito aquí recomendados. Al día de hoy existen serios problemas de capacidad en esta intersección.

De acuerdo a los resultados obtenidos se observó que las facilidades viales analizadas registran un patrón de flujo vehicular durante un día laborable típico de acuerdo a su función y al lugar donde se encuentran. La urbanización Monte Sierra es

cónsona con el desarrollo residencial existente en el área, por lo que su construcción no debe cambiar los patrones vehiculares que se registran.

Recomendaciones

La urbanización Monte Sierra traerá una cantidad adicional de vehículos al lugar que será manejada adecuadamente por la carretera PR-349. Solamente habrían problemas en su intersección con la calle Manantiales si no se hacen cambios en la geometría y dispositivos de control del tránsito existentes. De manera inmediata se debe construir un carril adicional en los accesos por el este y por el sur a esta intersección, así como el acceso por el norte debe ser ampliado de manera que se haga cómodo tanto el transitar hacia el sur como maniobrar hacia el este. En esta intersección existe un pequeño acceso por el oeste, conocido como calle Fortunet. Esta calle no es utilizada debido a que es muy incómoda para transitar en vehículo aunque sí se podría hacer. Debido a que no se utiliza, la misma no fue considerada en los análisis aquí realizados. Ya que podría ser posible que alguien la utilizara, se recomienda que se cierre su acceso a la intersección analizada. Cerrar este acceso no significa que se vaya a incomunicar a los residentes del lugar ya que esta calle también tiene acceso cercano por la calle Salud. De esta manera se incrementa la capacidad de la intersección y se aumenta la seguridad para los usuarios de la misma.

La intersección para accesar al proyecto propuesto debe ser rediseñada de manera que la calle Núm. 1 de la urbanización llegue a la carretera PR-349 a un ángulo más cercano a los 90 grados.

Actualmente se propone que este ángulo esté cerca de los 45 grados, lo que podría traer problemas de visibilidad y de seguridad a los usuarios. Aparte de esto, la geometría puede ser la propuesta ya que no se vislumbra que se necesite más de un carril en cada acceso. El control de tránsito en esta intersección debe consistir de un rótulo de "Pare" ubicado de manera que se detengan los vehículos que salgan de la urbanización. También se debe ajustar la intersección de la carretera PR-349 con la calle Manantiales de forma que la primera llegue a un ángulo recto a la segunda.

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, 2001**) (**AATSA/ITE/AASHTO, 2003**) (**ACT, 1996**). Por ser algunas de estas mejoras necesarias de manera inmediata, se le debe requerir a los desarrolladores de la urbanización Monte Sierra costear solamente la construcción de la intersección que le dará acceso a su proyecto. Las otras deben ser costeadas por el Departamento de Transportación y Obras Públicas y/o la Autoridad de Carreteras y Transportación. En las figuras 18 a la 21 se ilustra, de forma esquemática, las mejoras aquí propuestas.

Con las recomendaciones aquí resumidas, el segmento de carretera analizado deberá operar bien a mediano y largo plazo,

tomando en consideración la construcción de la urbanización Monte Sierra. La mejora en el funcionamiento a corto plazo dependerá de la rapidez con la que se implanten las recomendaciones aquí formuladas.

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 y/o dueño. 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 Urbanización Monte Sierra, propuesto por Urbanización Monte Sierra Inc., para la carretera PR-349, Km. 1.1, barrio Quebrada Grande, Mayagüez, 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

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Apéndices

Apéndice 1. Figuras

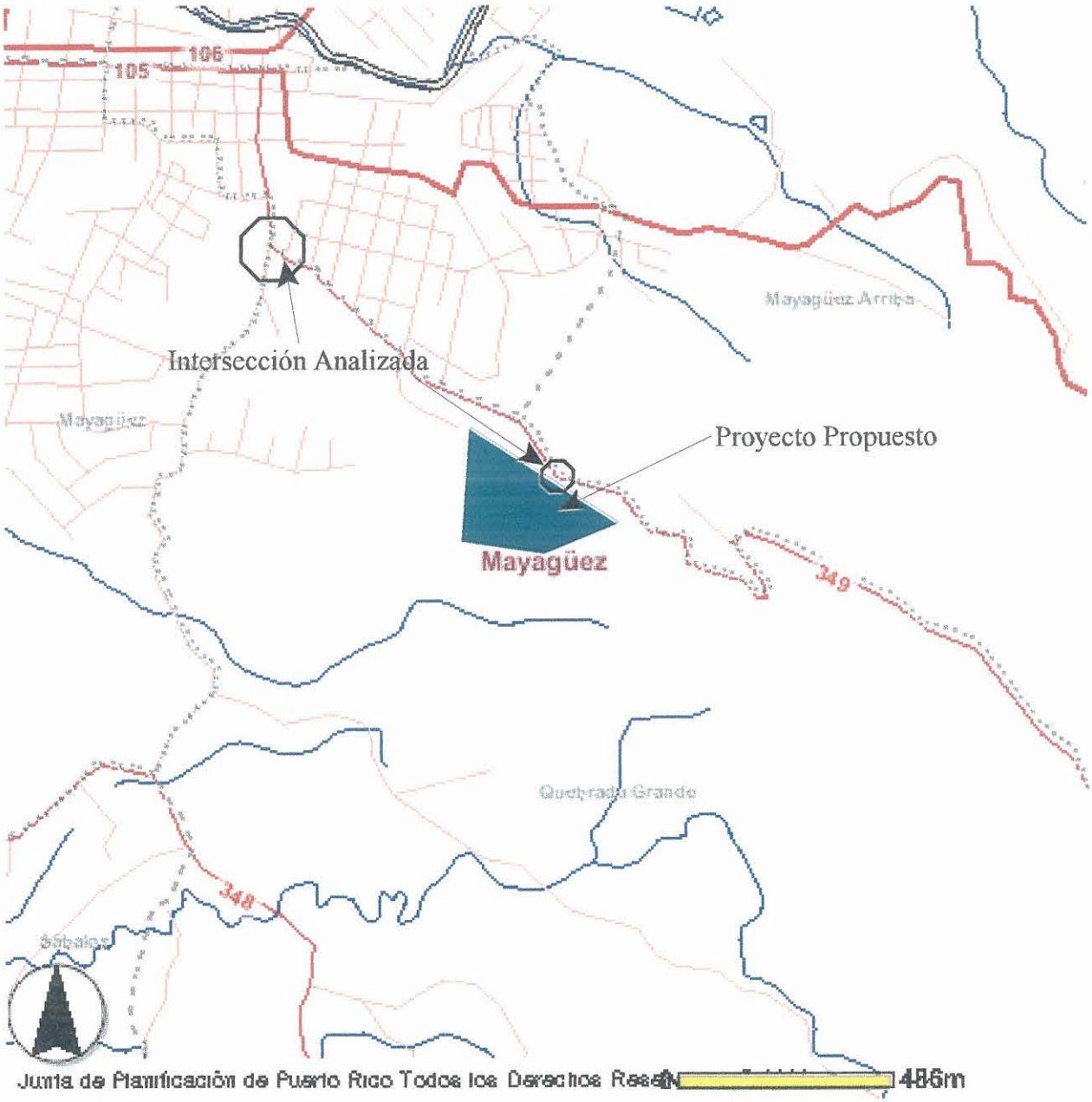


Figura 1. Mapa de carreteras del área.

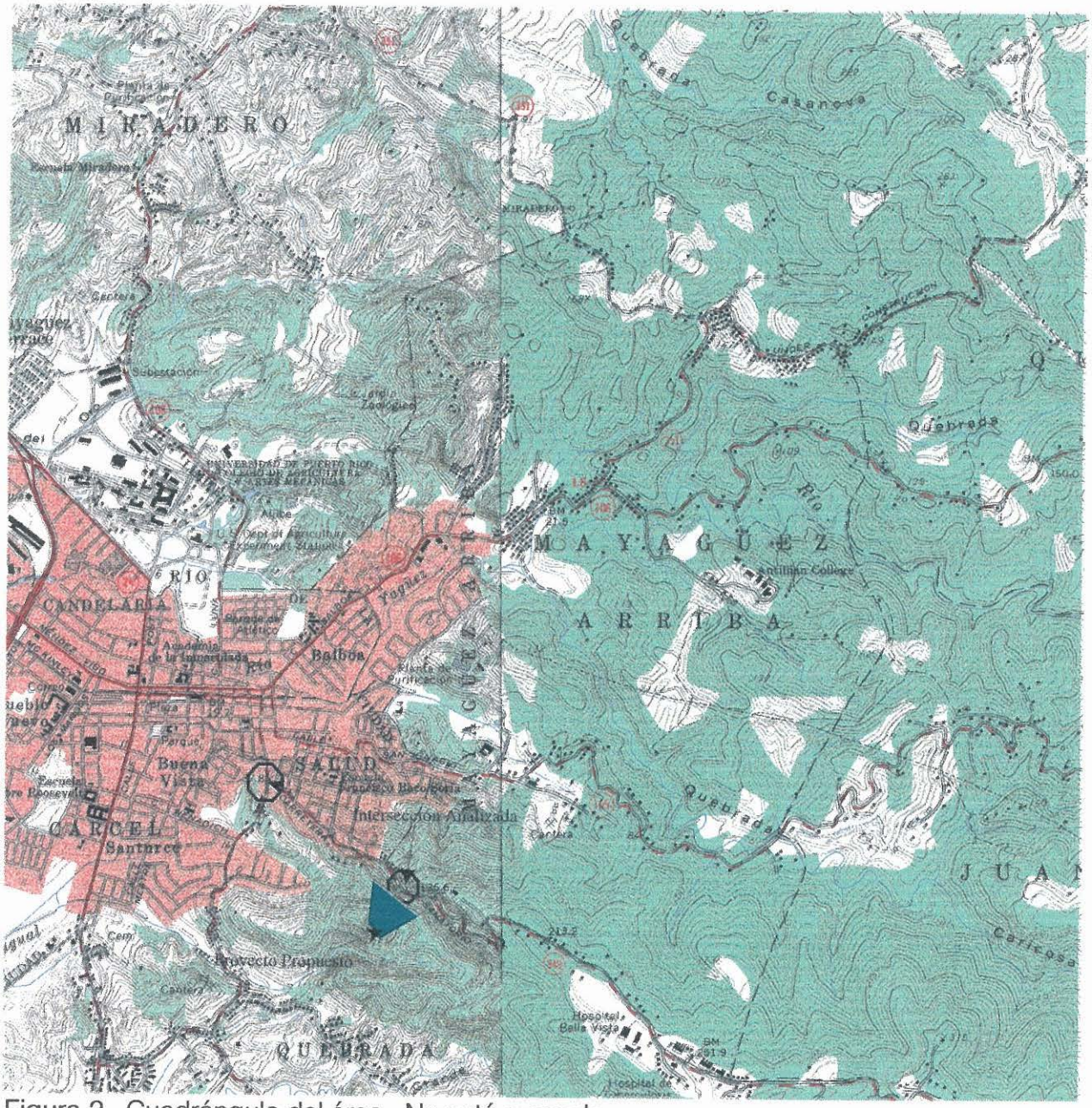


Figura 2. Cuadrángulo del área. No está a escala

Figura 3. Carretera PR-349, kilómetro 1.1, vista desde el este. La urbanización Monte Sierra se propone en el predio que se observa a la izquierda de la carretera

Figura 4. Intersección de la carretera PR-349 y la calle Manantiales, vista desde el este. La calle Manantiales sale hacia la izquierda, la calle Liceo (PR-349) sale hacia la derecha y la carretera PR-349 sale hacia la parte de abajo de la figura. De frente se puede observar la calle Fortunet, la cual no es utilizada por el tránsito vehicular

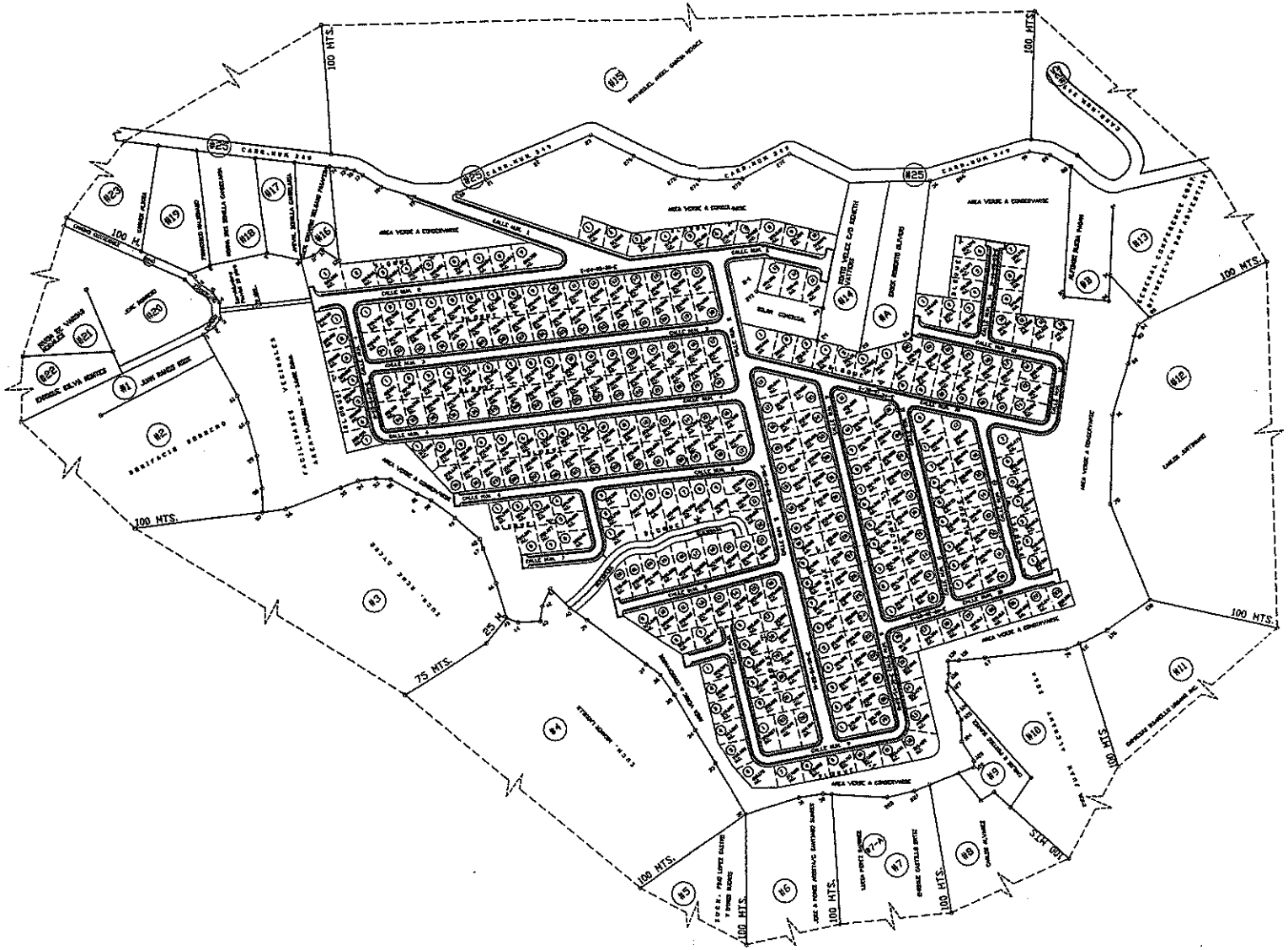


Figura 5. Planta con el proyecto propuesto

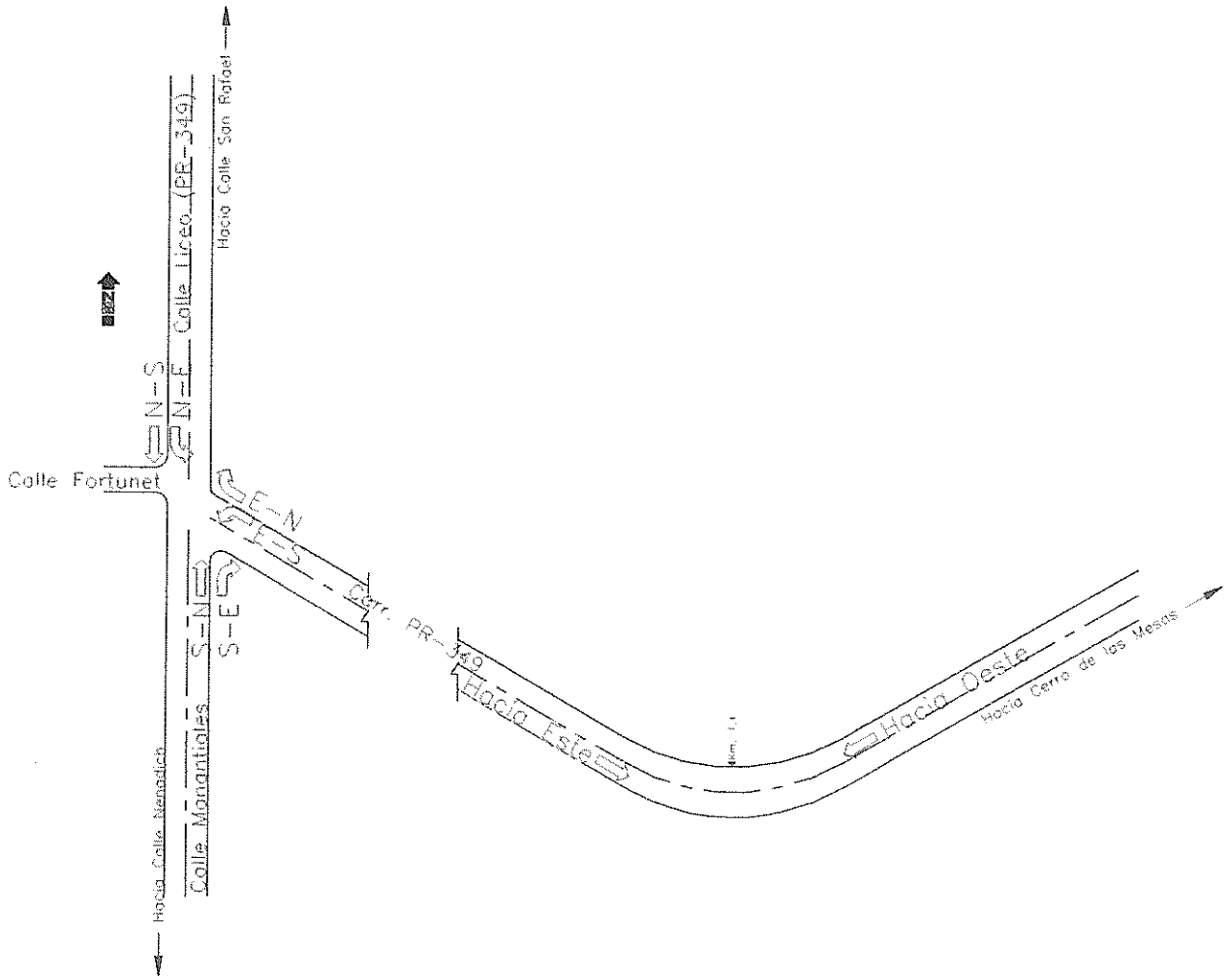


Figura 6. Movimientos analizados en este estudio. Dibujo no está a escala

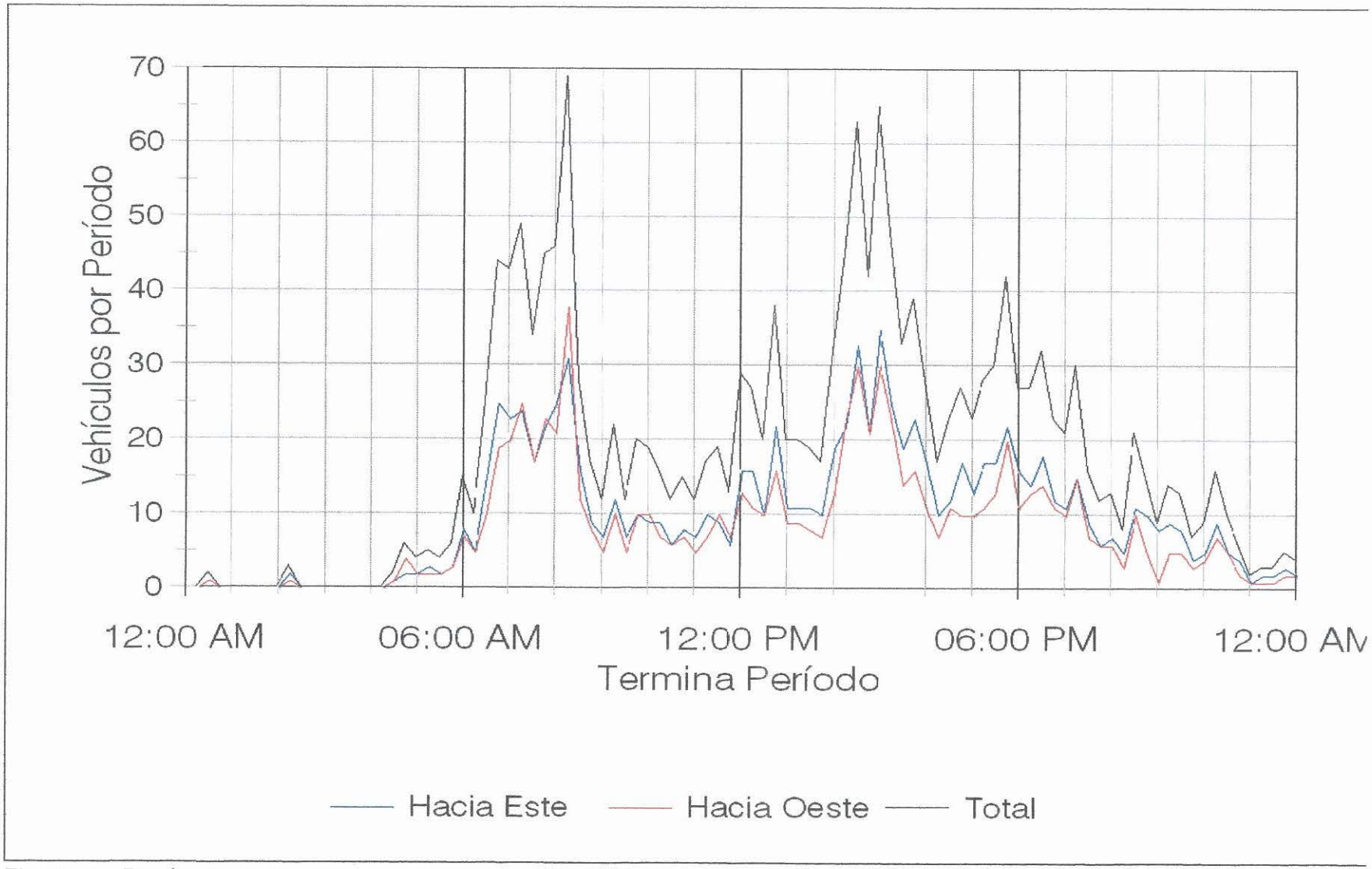


Figura 7. Patrón del flujo vehicular actual, por movimiento y total, durante un día laborable típico, en el kilómetro 1.1 de la carretera PR-349

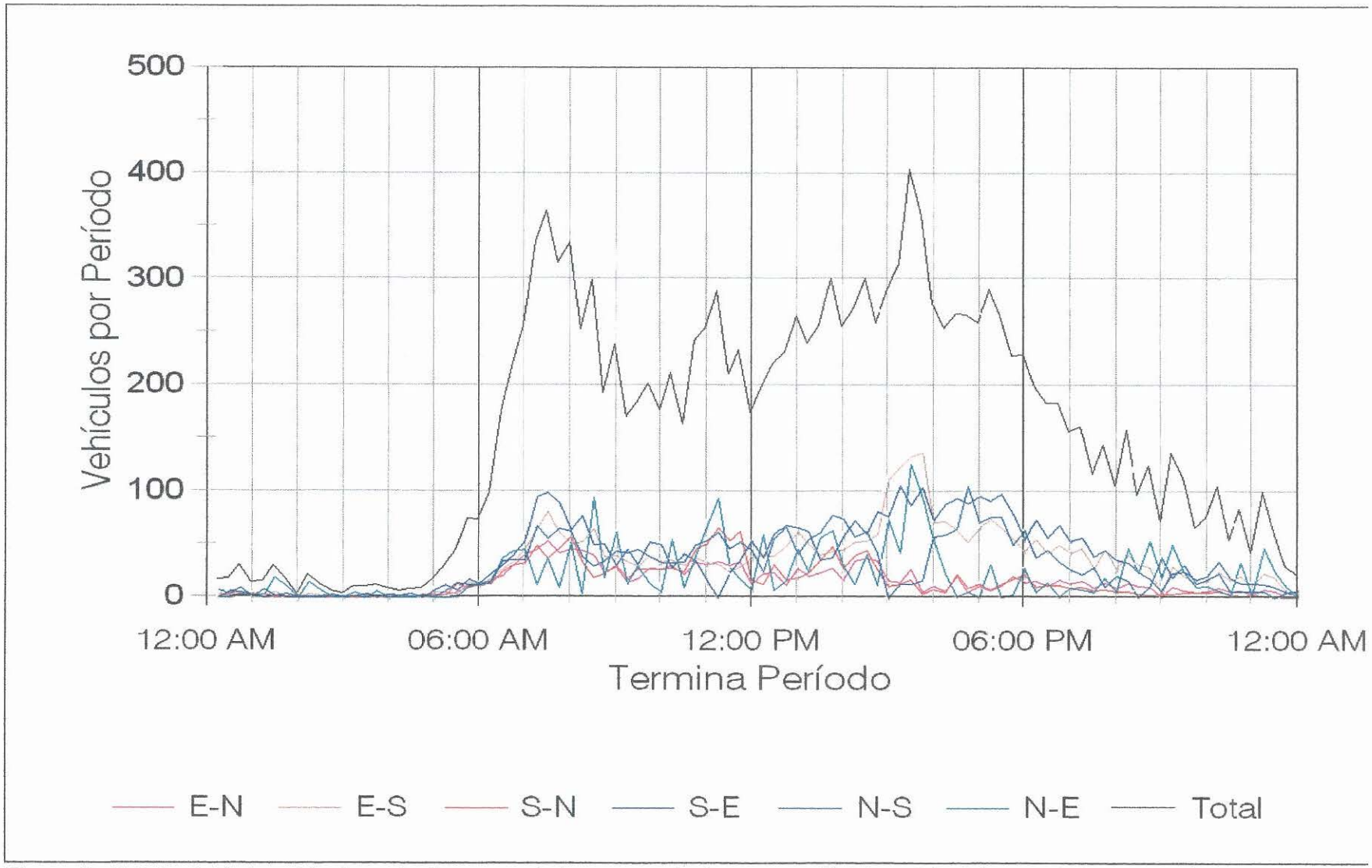


Figura 8. Patrón del flujo vehicular actual, por movimiento y total, durante un día laborable típico, en la intersección de la carretera PR-349 y la calle Manantiales

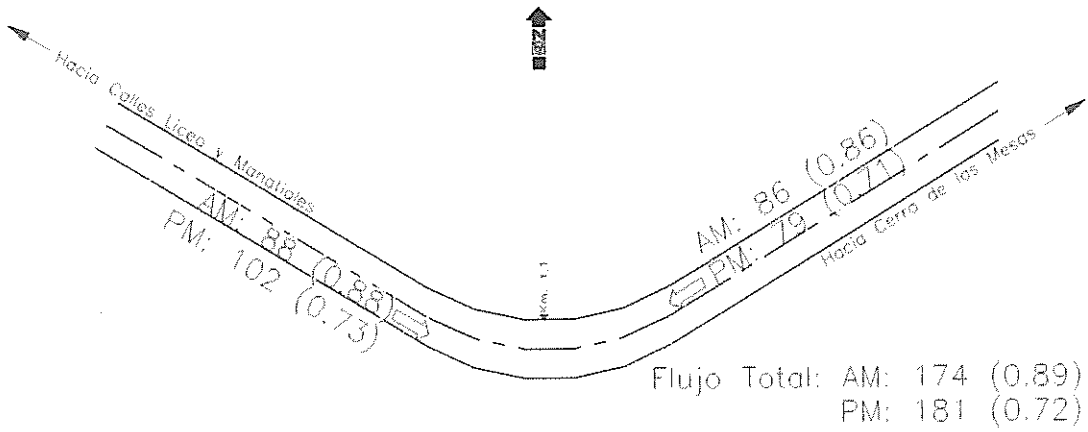


Figura 9. Flujo Vehicular y Factor de la Hora Pico actual, por movimiento y total, para ambas horas pico, en el kilómetro 1.1 de la carretera PR-349. El Factor de la Hora Pico aparece entre paréntesis

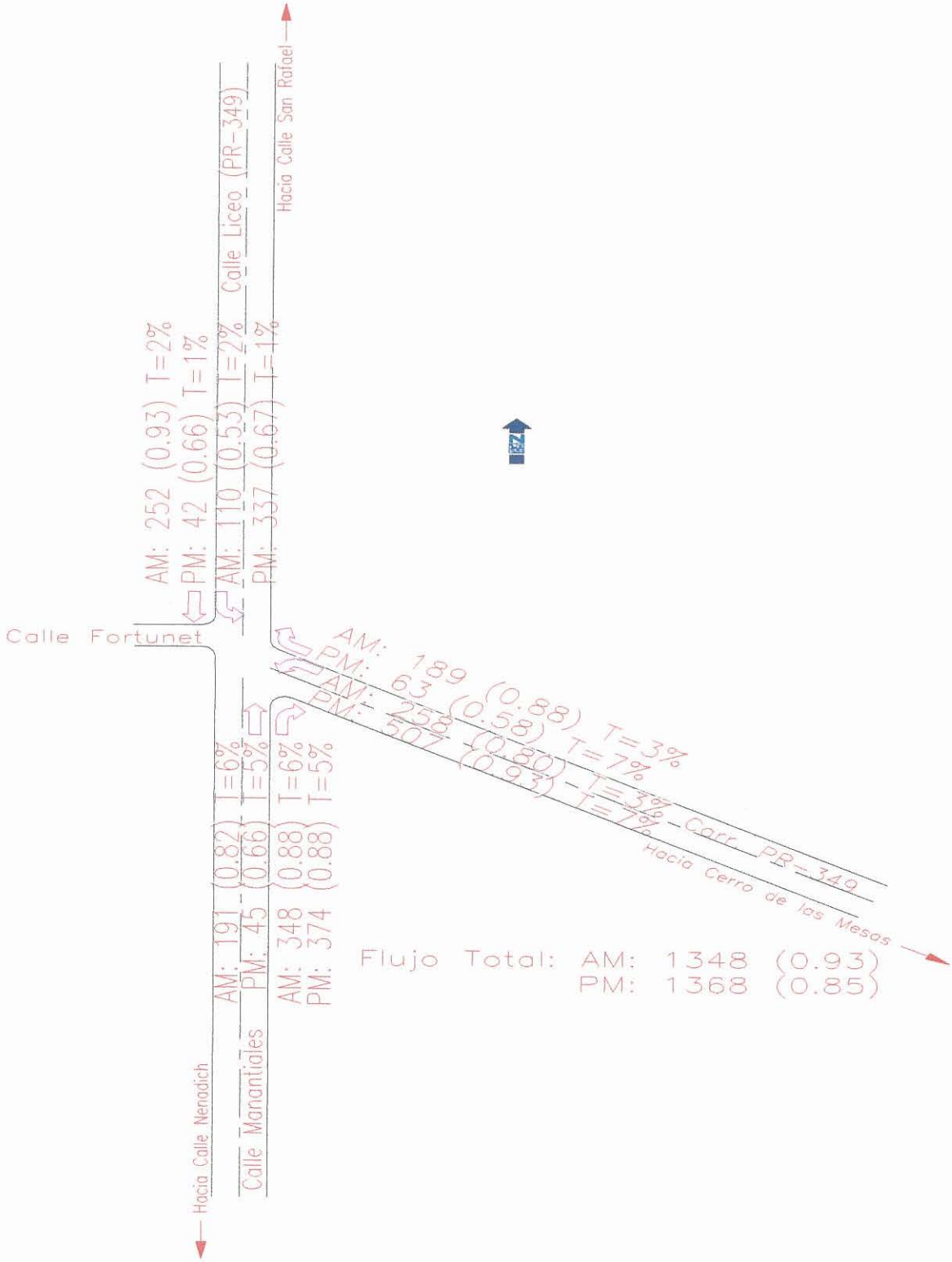


Figura 10. Flujo Vehicular, Factor de la Hora Pico y Porciento de Camiones actual, por movimiento y total, para ambas horas pico, en la intersección de la carretera PR-349 y la calle Manantiales. El Factor de la Hora Pico aparece entre paréntesis

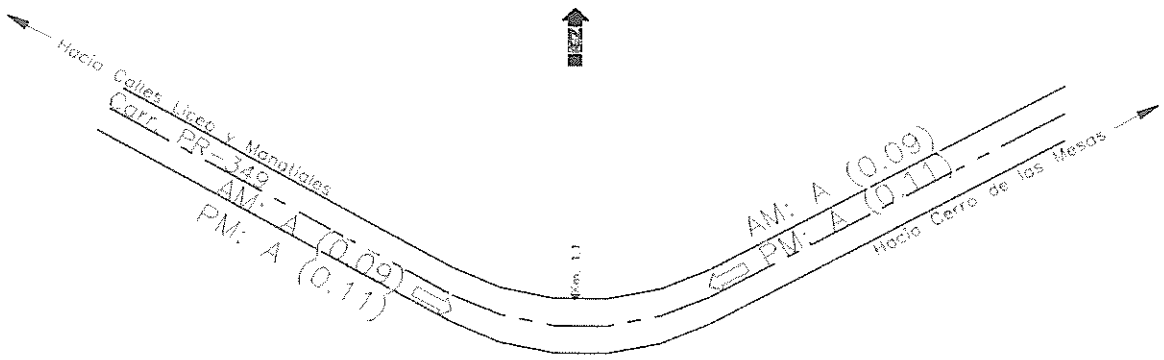


Figura 11. Nivel de Servicio y Razón de Volumen a Capacidad (v/c) actual, durante ambas horas pico, en el kilómetro 1.1 de la carretera PR-349

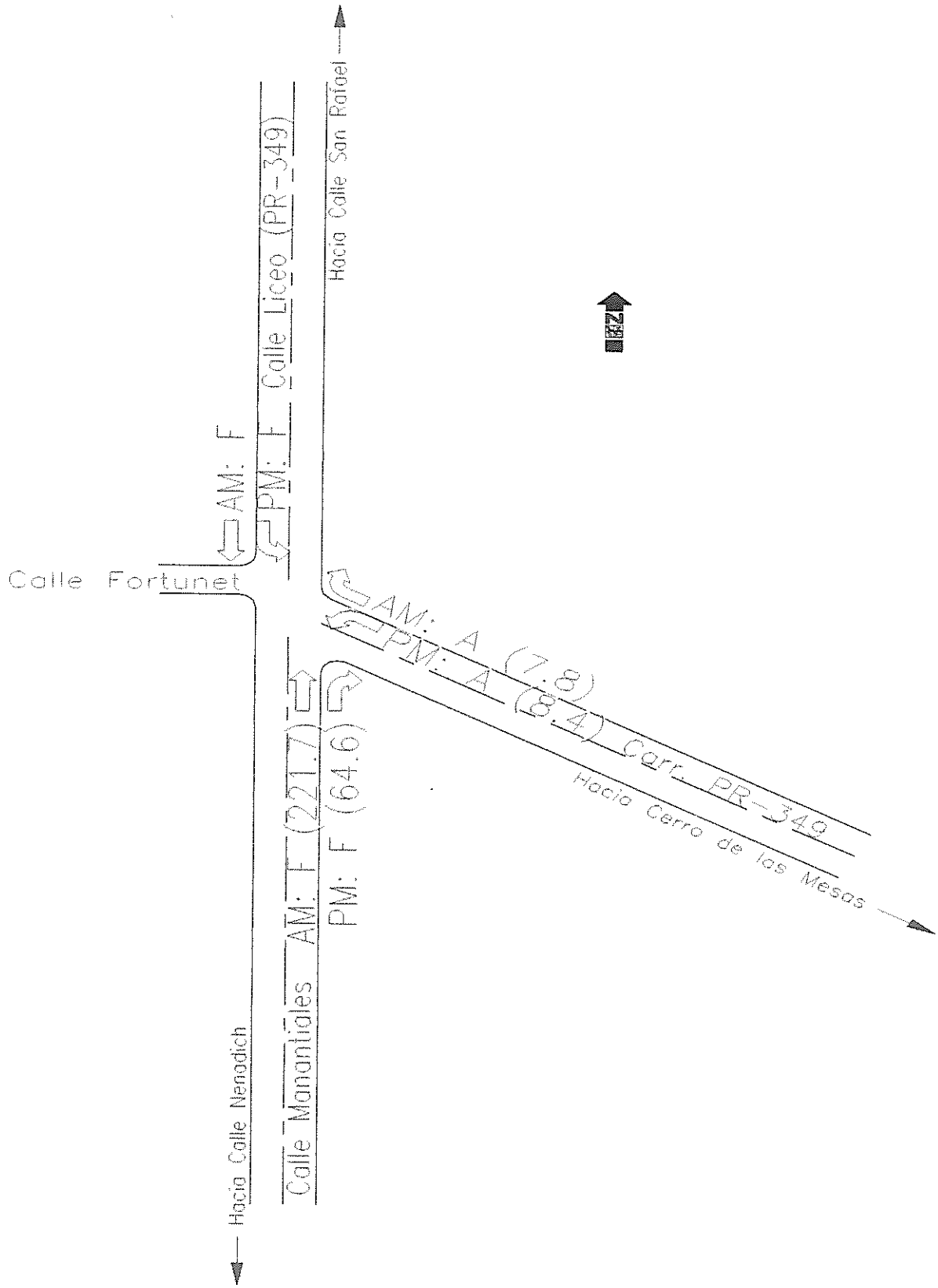


Figura 12. Nivel de Servicio y Demora por Control actual, para ambas horas pico, para cada acceso a la intersección de la carretera PR-349 y la calle Manantiales

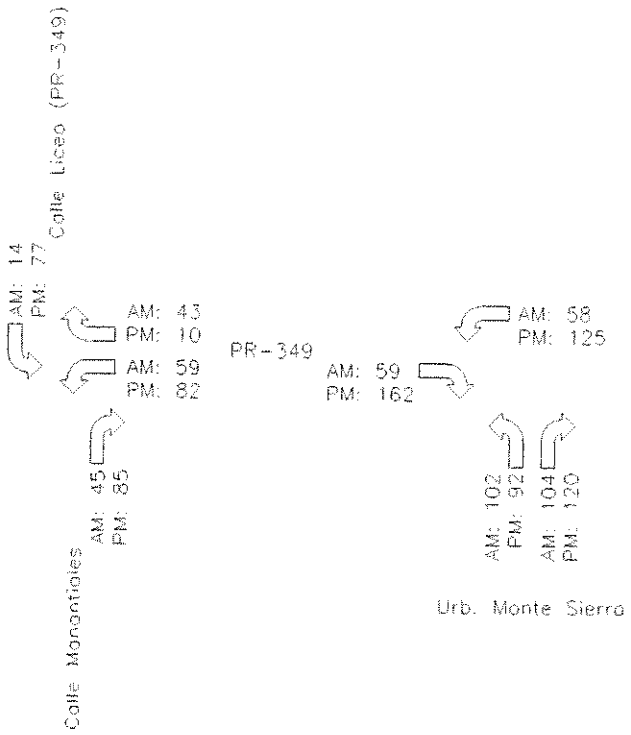


Figura 13. Generación y Distribución de los viajes del proyecto propuesto

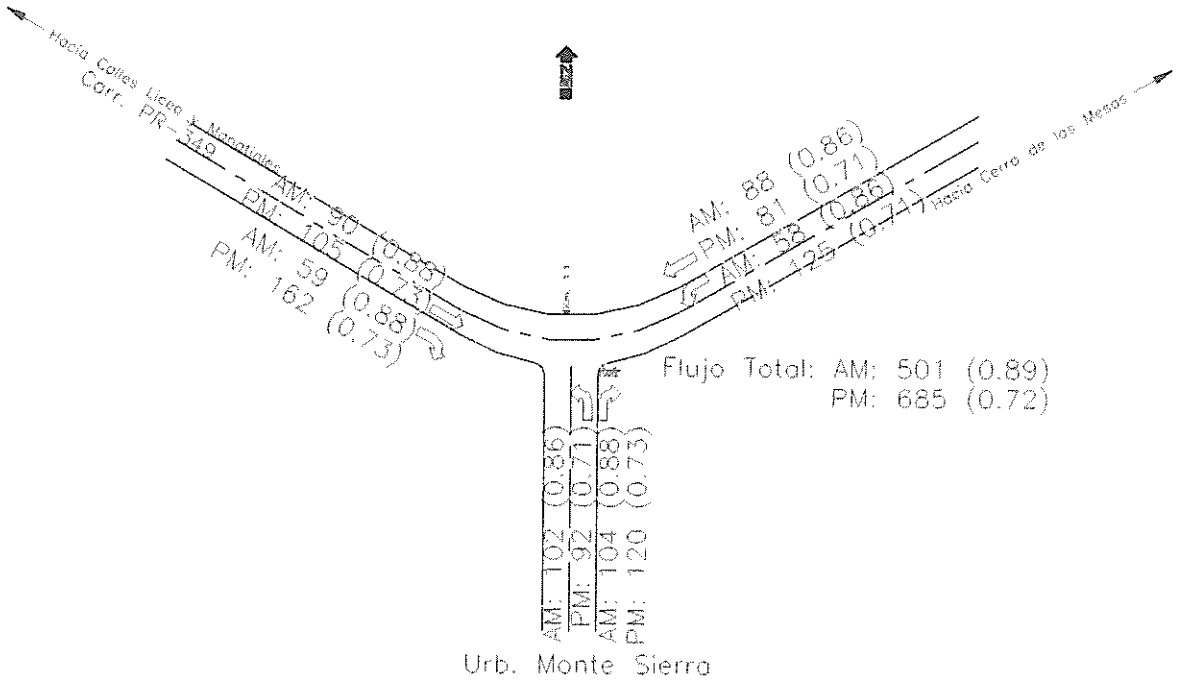


Figura 14. Flujo Vehicular y Factor de la Hora Pico para el año 2006, para ambas horas pico, por movimiento y total, en la futura intersección de la carretera PR-349 y la calle Núm. 1 de la urbanización Monte Sierra. El Factor de la Hora Pico se ilustra entre paréntesis. Se ilustra la ubicación esquemática del rótulo de "PARE" recomendado

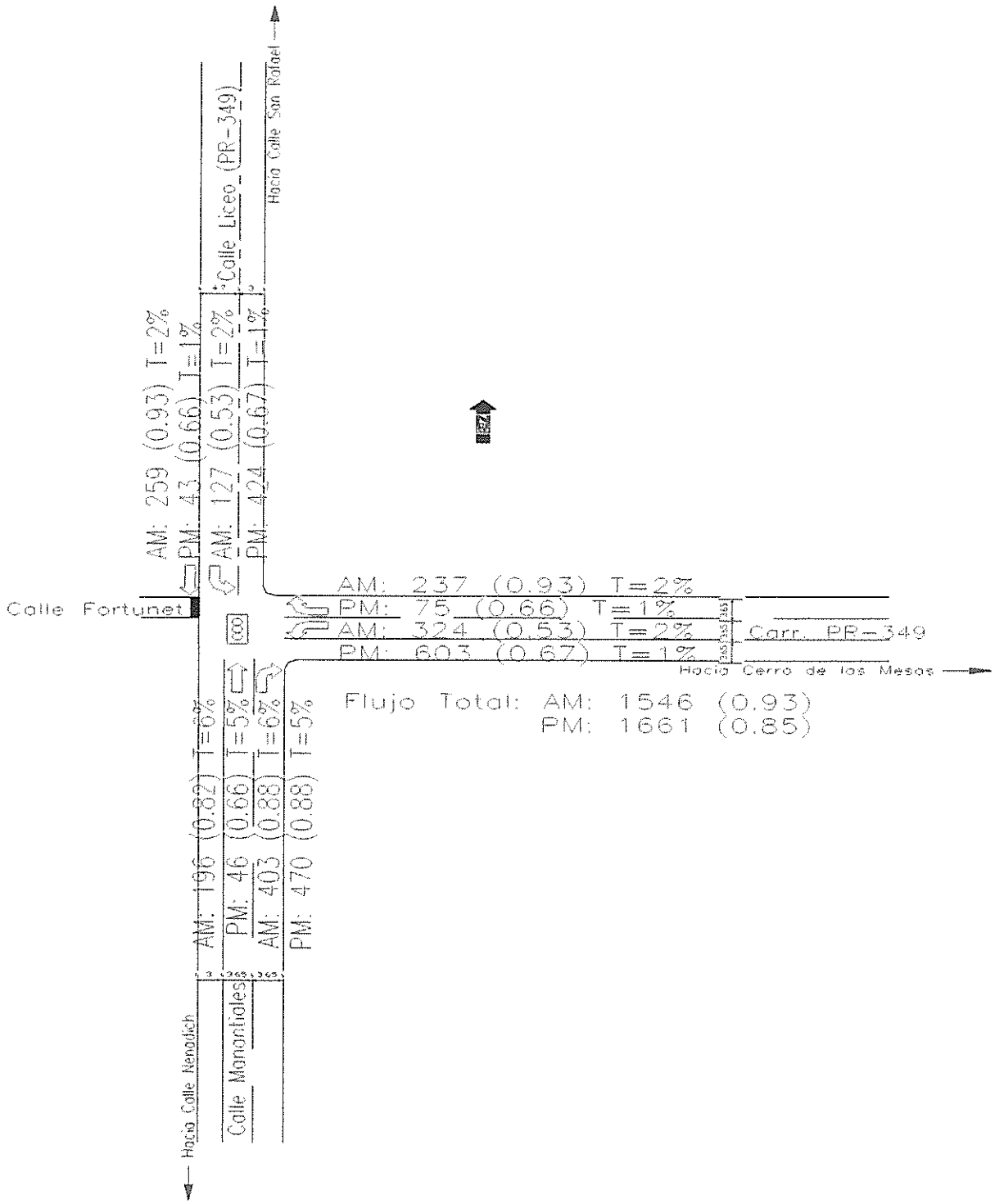


Figura 15. Flujo Vehicular, Factor de la Hora Pico y Porciento de Camiones para el año 2006, para ambas horas pico, por movimiento y total, en la intersección de la carretera PR-349 y la calle Manantiales. El Factor de la Hora Pico se ilustra entre paréntesis. Se ilustra la geometría y el control de tránsito recomendado para la intersección así como el cierre de la calle Fortunet

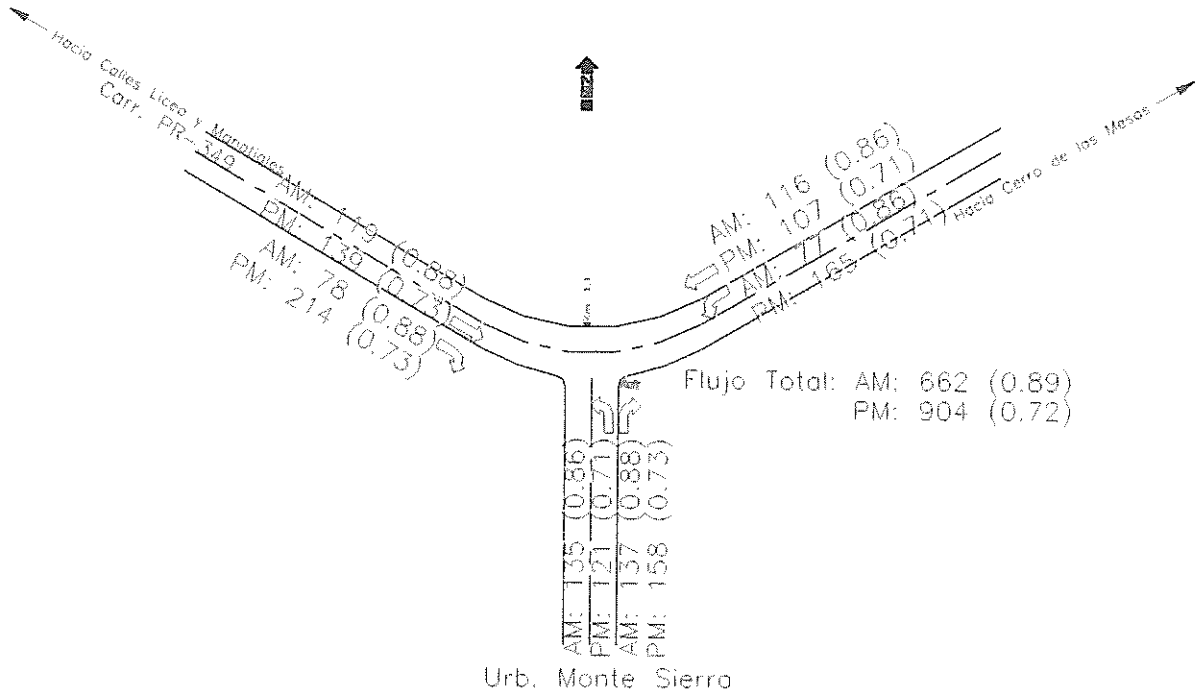


Figura 16. Flujo Vehicular y Factor de la Hora Pico para el año 2026, para ambas horas pico, por movimiento y total, en la futura intersección de la carretera PR-349 y la calle Núm. 1 de la urbanización Monte Sierra. El Factor de la Hora Pico aparece entre paréntesis

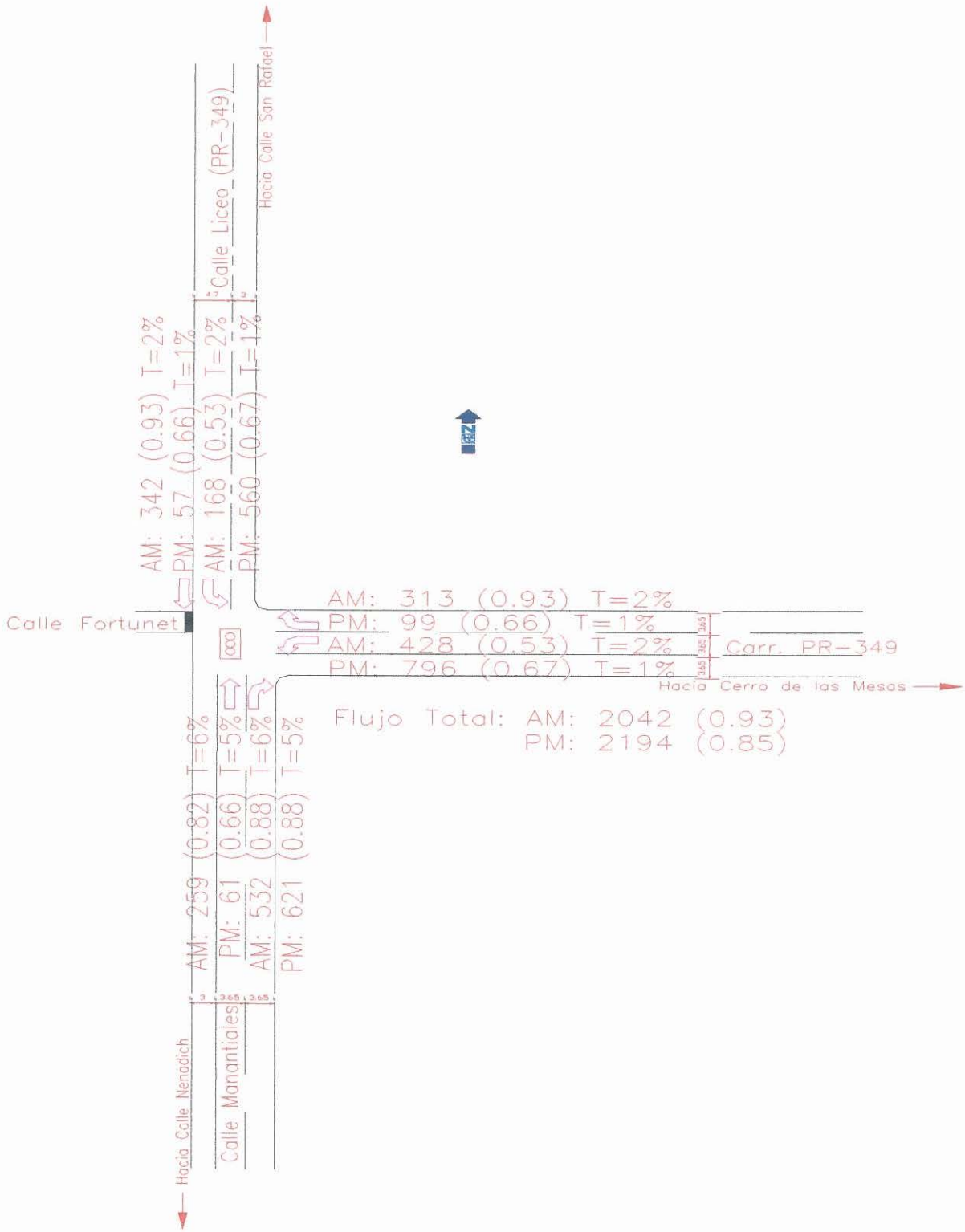


Figura 17. Flujo Vehicular, Factor de la Hora Pico y Porciento de Camiones para el año 2026, para ambas horas pico, por movimiento y total, en la intersección de la carretera PR-349 y la calle Manantiales

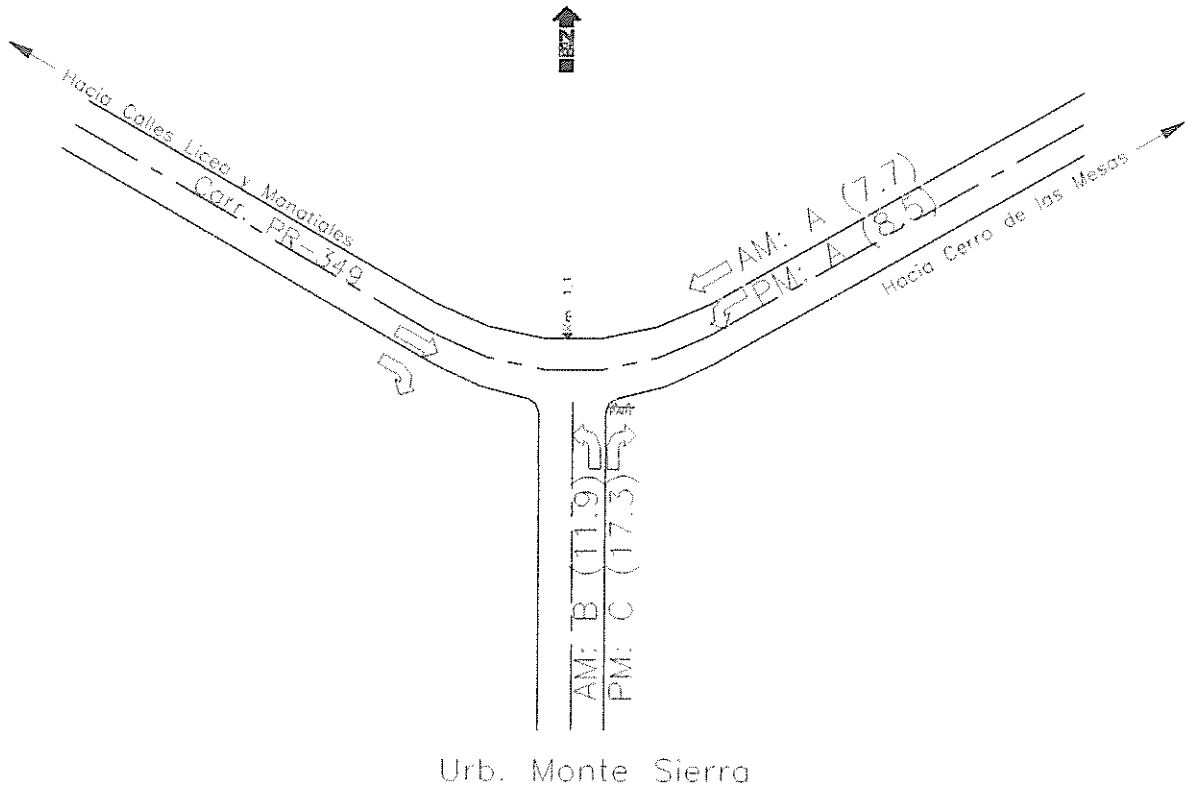


Figura 18. Nivel de Servicio y Demora por Control que se experimentará durante ambas horas pico, en el año 2006, en los accesos de la intersección de la carretera PR-349 y la calle Núm. 1 de la urbanización Monte Sierra. La demora por control aparece entre paréntesis

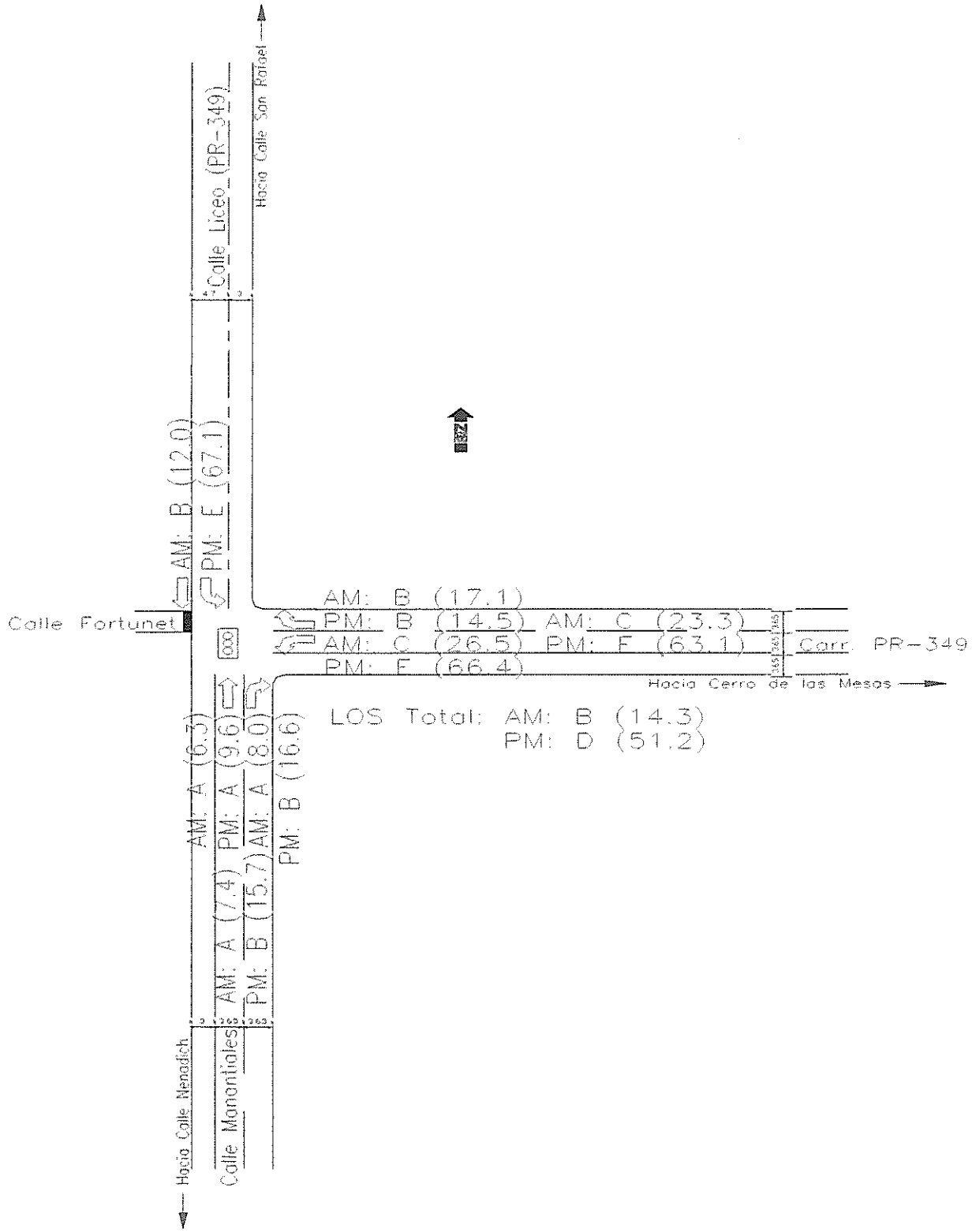


Figura 19. Nivel de Servicio y Demora por control que se experimentará durante ambas horas pico, para el año 2006, por movimiento, por acceso y total, en la intersección de la carretera PR-349 y la calle Manantiales. La Demora por Control aparece entre paréntesis

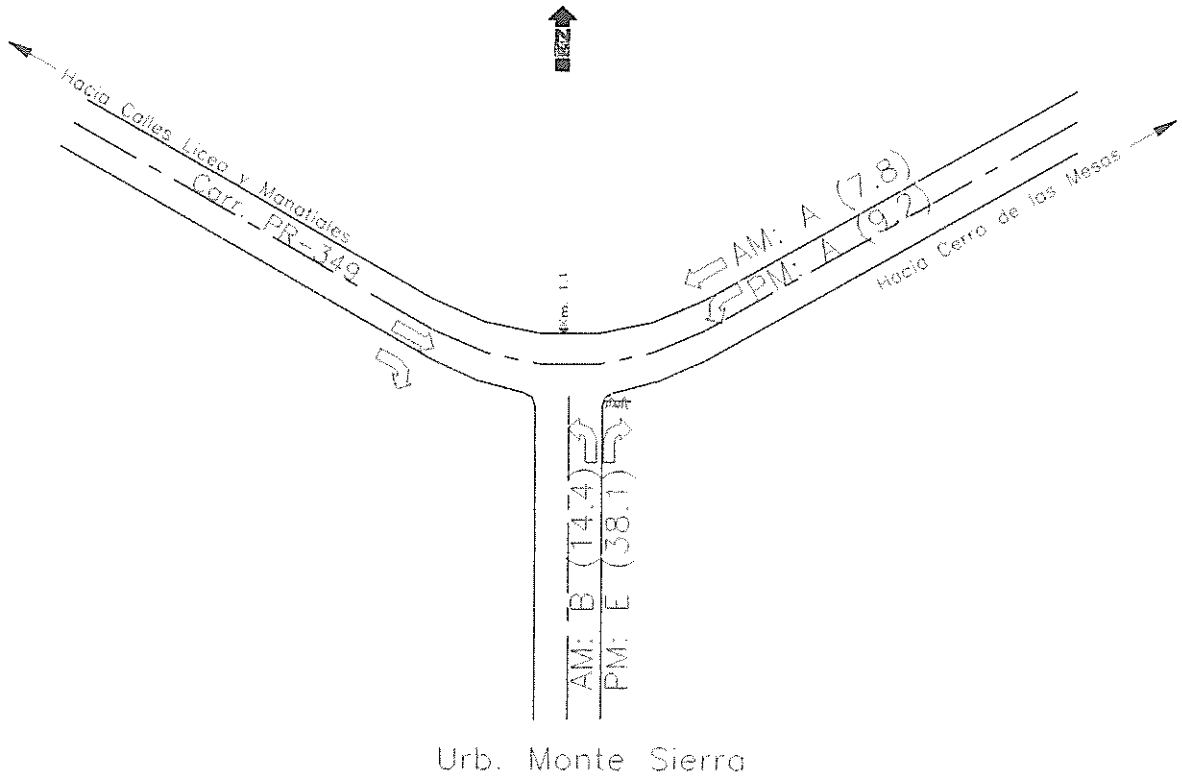


Figura 20. Nivel de Servicio y Demora por Control que se experimentará durante ambas horas pico, para el año 2026, en los accesos de la intersección de la carretera PR-349 con la calle Núm. 1 de la urbanización Monte Sierra. La Demora por Control aparece entre paréntesis

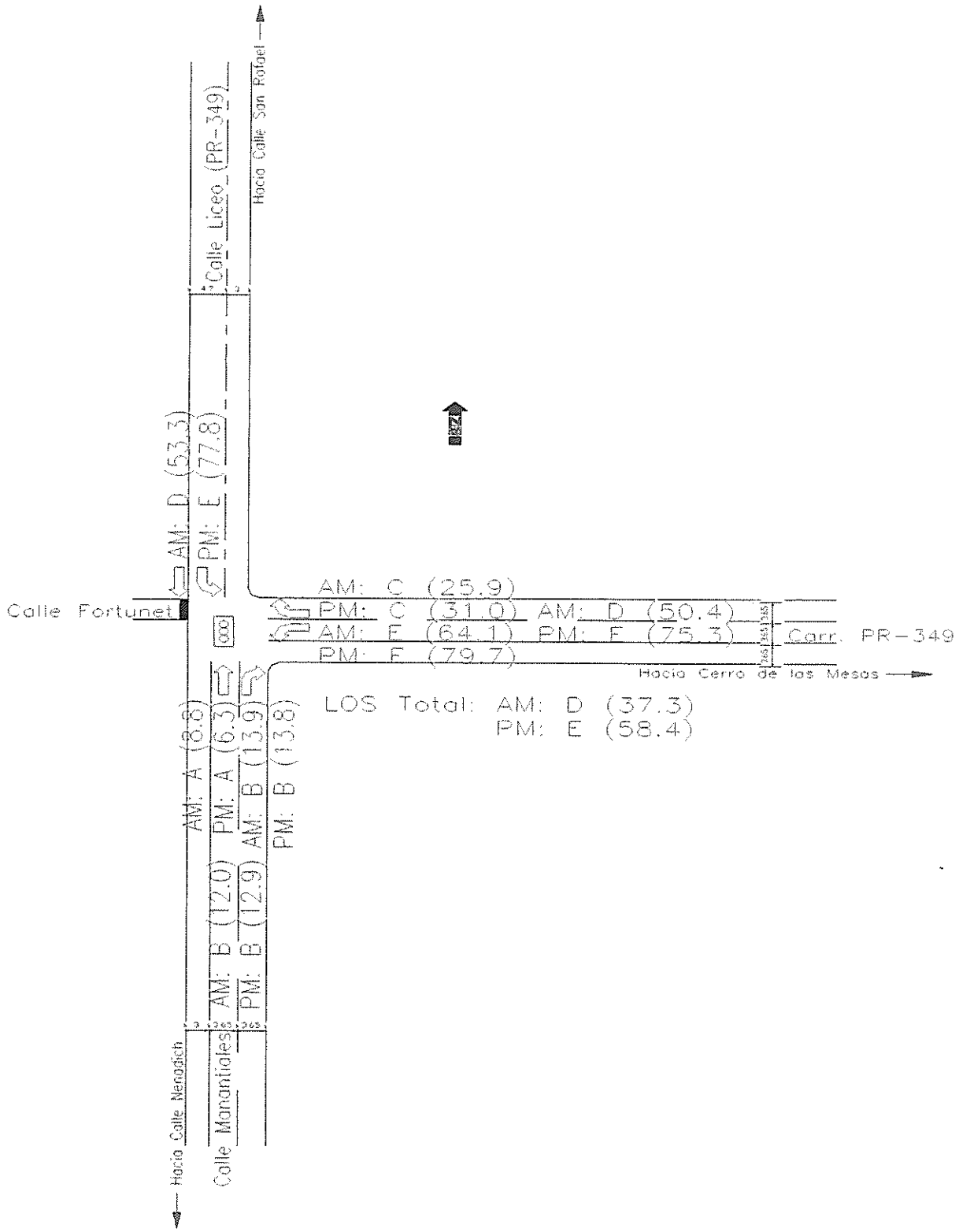


Figura 21. Nivel de Servicio y Demora por Control que se experimentará durante ambas horas pico, para el año 2026, por movimiento, acceso y total, en la intersección de la carretera PR-349 y la calle Manantiales. La Demora por Control aparece entre paréntesis

Apéndice 2. Guías utilizadas en el análisis operacional (TRB, 2000)

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.

Lane group defined

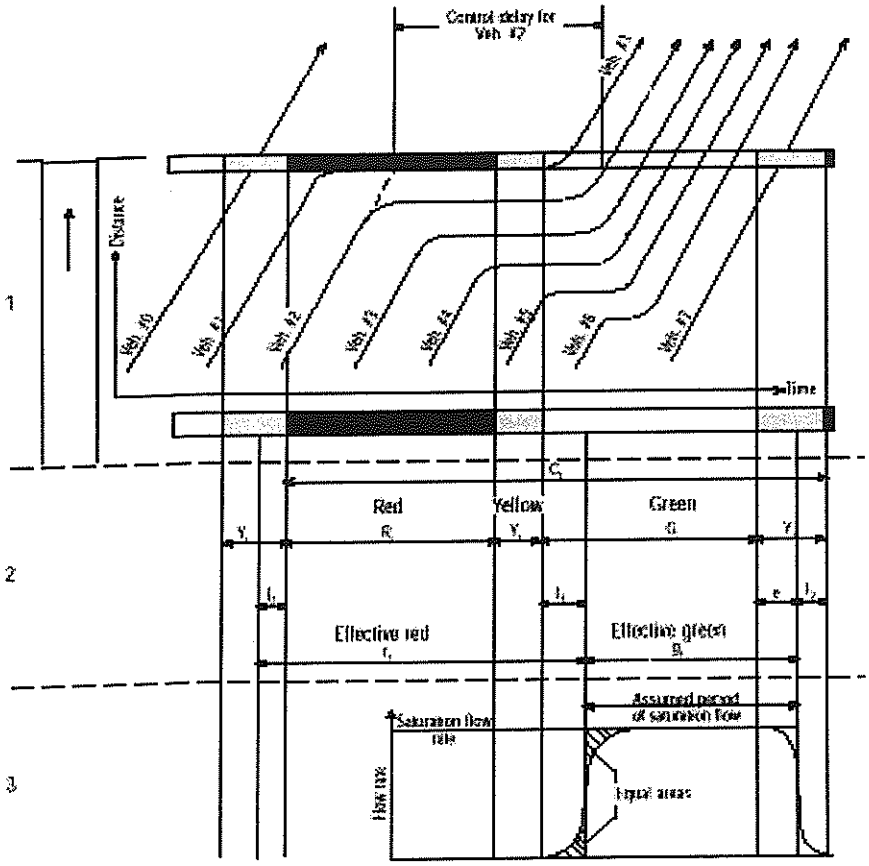
In analyzing a signalized intersection, the physical unit of analysis is the lane group. A lane group consists of one or more lanes on an intersection approach. The outputs from application of the method in this manual are reported on the basis of each lane group.

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 defined. 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	l_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

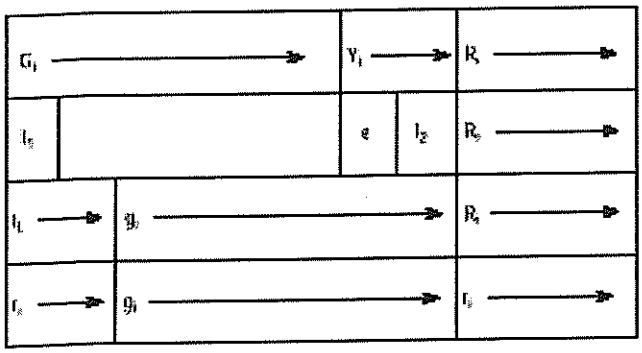
At the beginning of green, the start-up losses are called start-up lost time (l_1). At the beginning of the yellow, when vehicles tend to continue to enter the intersection for a short period of time, an extension of effective green (e) is experienced. When this extension of green has been exhausted, 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.

$$g_i = G_i + Y_i - t_L \qquad [10\text{-}2]$$

$$r_i = R_i + t_L \qquad [10\text{-}3]$$

A lost time is applied each time a movement is started

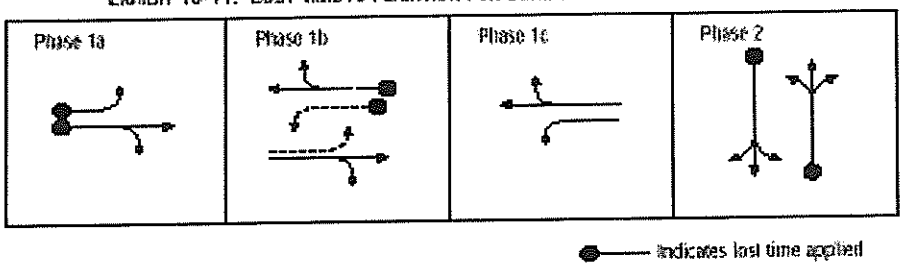
The simplified concept of applying all of the lost time at the beginning of a movement makes it easier to analyze more complex signalization involving protected-plus-permitted left-turn phasing. As a general rule, a lost time, t_L, is applied each time a movement is started. Thus, where a given movement 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) movements 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 geometries of the intersection, including the number and width of lanes, grades, and lane use allocations (including parking lanes). Signalization conditions include a full definition 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 service 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 E 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

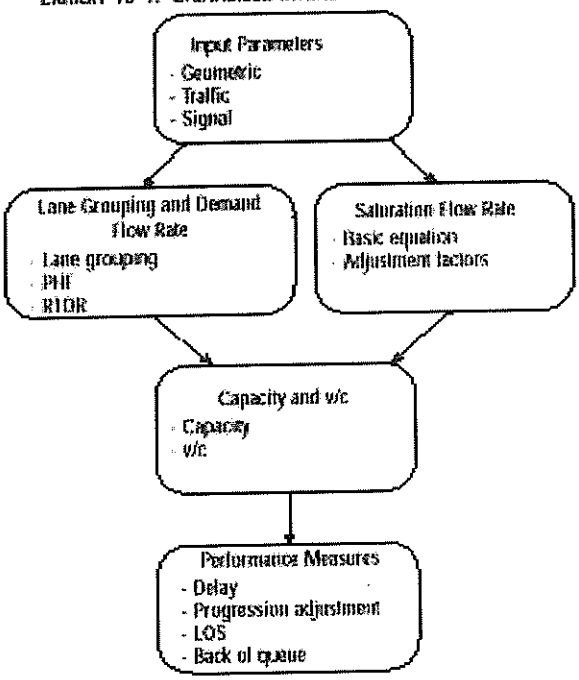
The methodology does not take into account the potential impact of downstream congestion on intersection operation. Nor does the methodology detect and adjust for the impacts of turn-pocket overflows on through traffic and intersection operation.

II. METHODOLOGY

Exhibit 16-1 shows the input and the basic computation 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

utilization, 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	B	C	D	E
	Service Volumes (veh/h)				
1	N/A	130	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 pc/h/ln. 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. Protected 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 the 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.

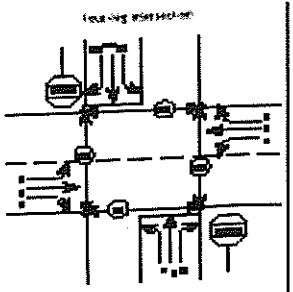
CHARACTERISTICS OF TWSC INTERSECTIONS

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, 6, 8, 15, 16
2	1, 9, 13, 14, 9, 12
3	8, 11
4	7, 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 order 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

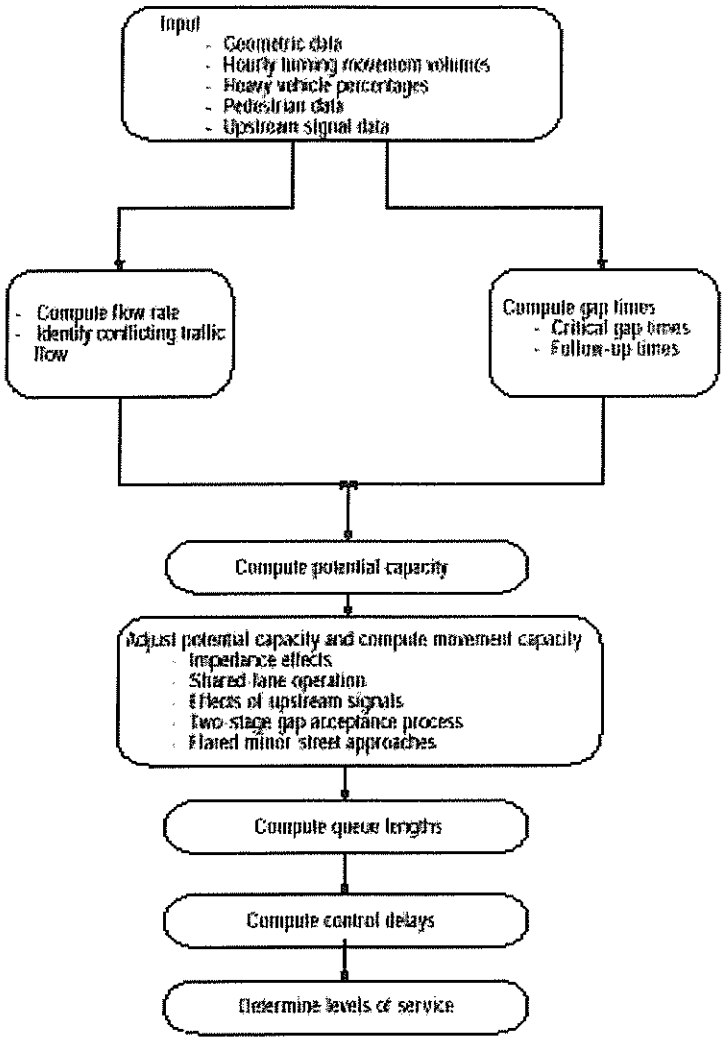


EXHIBIT 17-2. LEVEL-OF-SERVICE CRITERIA FOR TWSC INTERSECTIONS

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 nonrandom 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 time), 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.

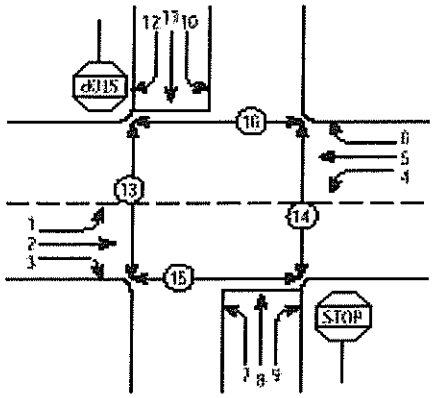
Movements of Rank 1 (denoted by the subscript i) include through traffic on the major street and right-turning traffic from the major street. Movements of Rank 2 (subordinate to 1 and denoted by the subscript j) include left-turning traffic from the major street and right-turning traffic onto the major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

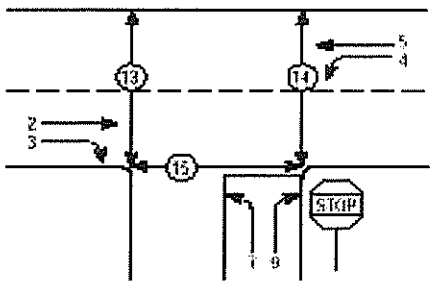
EXHIBIT 17-3. TRAFFIC STREAMS AT A TWSC INTERSECTION

Four-leg intersection



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

T intersection



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 turn). 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 inhibit 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 on movements of higher rank.

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

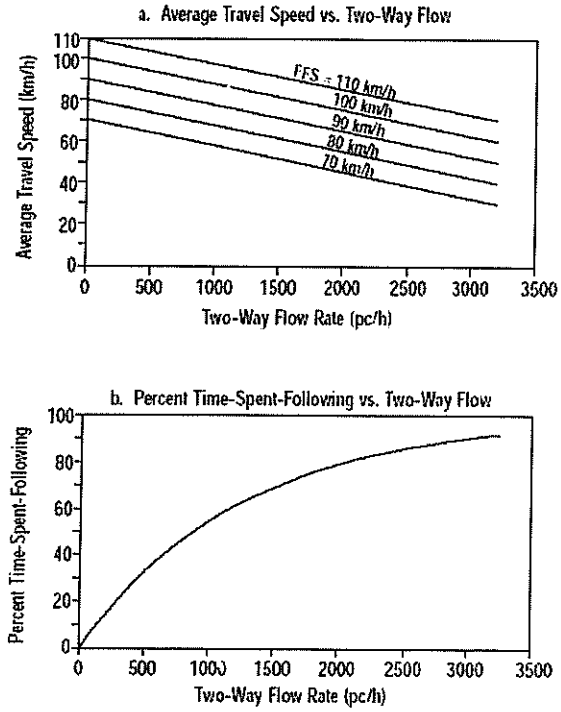
Exhibit 12-6 shows the relationship of flow rate, average travel speed, and percent time-spent-following for base conditions on an extended two-way facility (7).

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

characteristics are described by the average percentage of the highway with no-passing zones in either direction. Roadway cross-section data include lane width and usable shoulder width. Geometric data and design speed are considered in estimating the FFS.

Analysis of two-way flow usually is performed on extended lengths; but directional analysis is applied to relatively short, uniform segments

EXHIBIT 12-6. SPEED-FLOW AND PERCENT TIME-SPENT-FOLLOWING FLOW RELATIONSHIPS FOR TWO-WAY SEGMENTS WITH BASE CONDITIONS

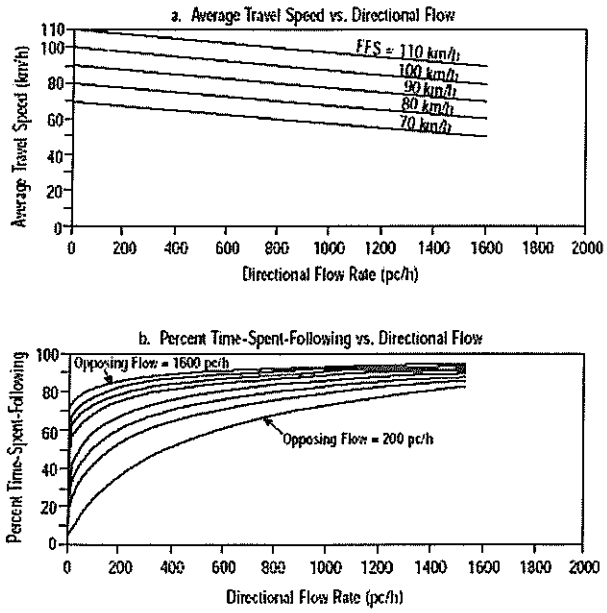


Two-way segments and directional segments

Two-lane highways can be analyzed either as two-way segments obtaining traffic performance measures for both directions of travel combined, or as directional segments, with each direction of travel considered separately. Separate analysis by direction is appropriate for steep grades and for segments with passing lanes.

Exhibit 12-7 illustrates the relationship of flow rate, average travel speed, and percent time-spent-following for base conditions of a directional segment of a two-way facility (*7*). These relationships are conceptually analogous to the relationships for the two-way segments in Exhibit 12-6; however, the relationships for directional segments incorporate the effect of the opposing flow rate on the average travel speed and percent time-spent-following. In Exhibit 12-7(a), the y-intercept represents the FFS in the analysis direction, incorporating the effect of the demand flow rate in the opposing direction. Exhibit 12-7(b) graphs the relationships between directional flow rate and percent time-spent-following when opposing flow rates range from 200 to 1,600 pc/h.

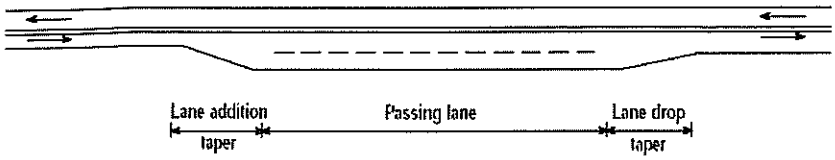
EXHIBIT 12-7. SPEED-FLOW AND PERCENT TIME-SPENT-FOLLOWING FLOW RELATIONSHIPS FOR DIRECTIONAL SEGMENTS WITH BASE CONDITIONS



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.

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-sper following, and average trave 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 I and 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.

EXHIBIT 20-3. LOS CRITERIA (GRAPHICAL) FOR TWO-LANE HIGHWAYS IN CLASS I

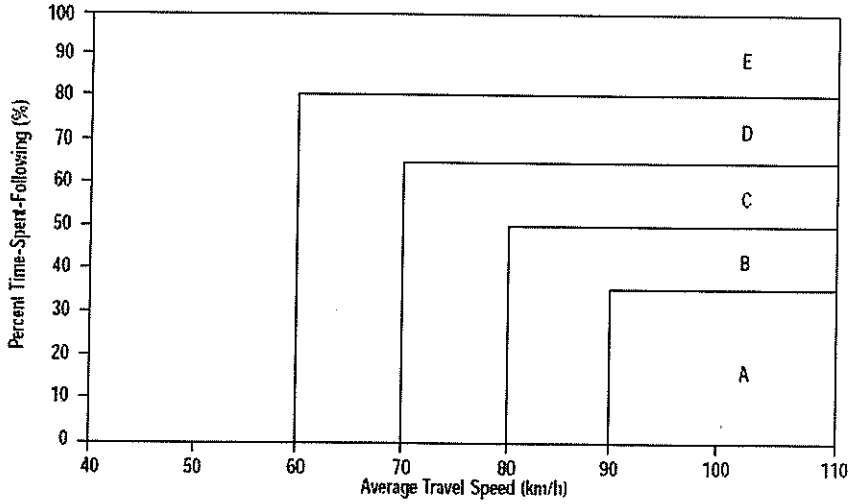


EXHIBIT 20-4. LOS CRITERIA FOR TWO-LANE HIGHWAYS IN CLASS II

LOS	Percent Time-Spent-Following
A	≤ 40
B	> 40–55
C	> 55–70
D	> 70–85
E	> 85

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

Free-flow speed occurs at two-way flows of 200 pc/h or less

Determining Free-Flow Speed

A key step in the assessment of the LOS of a two-lane highway is to determine the free-flow speed (FFS). The FFS is measured using the mean speed of traffic under low flow conditions (up to two-way flows of 200 pc/h). If field measurements must be made with two-way flow rates of more than 200 pc/h, a volume adjustment must be made in determining FFS. This volume adjustment is discussed below.

Two general methods can be used to determine the FFS for a two-lane highway: field measurement and estimation with the guidelines provided in this chapter. The field-measurement procedure assists in gathering these data directly or incorporating the measurements into a speed monitoring program. However, field measurements are not necessary for an operational analysis—the FFS can be estimated from field data and user knowledge of conditions on the highway.

Field Measurement

The FFS of a highway can be determined directly from a speed study conducted in the field. No adjustments are made to the field-measured data. The speed study should be conducted at a representative location within the highway segment being evaluated: for example, a site on a short upgrade should not be selected within a segment that is generally level. Any speed measurement technique acceptable for other types of traffic engineering speed studies may be used. The field study should be conducted in periods of low traffic flow (up to a two-way flow of 200 pc/h) and should measure the speeds of all vehicles or of a systematic sampling (e.g., of every 10th vehicle). A representative

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.

Calculating a peak-hour factor

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:

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

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

Speed parameters

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 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:

Average running speed—A traffic stream measure based on the observation of vehicle travel times traversing a section of highway of known length. It is the length of the segment divided by the average running time of vehicles to traverse the segment. Running time includes only time that vehicles are in motion.

Average travel speed

Average travel speed—A traffic stream measure based on travel time observed on a known length of highway. It is the length of the segment divided by the average travel time of vehicles traversing the segment, including all stopped delay times. It is also a space mean speed.

Space mean speed

Space mean speed—A statistical term denoting an average speed based on the average travel time of vehicles to traverse a segment of roadway. It is called a space mean speed because the average travel time weights the average to the time each vehicle spends in the defined roadway segment or space.

Time mean speed

Time mean speed—The arithmetic average of speeds of vehicles observed passing a point on a highway; also referred to as the average spot speed. The individual speeds of vehicles passing a point are recorded and averaged arithmetically.

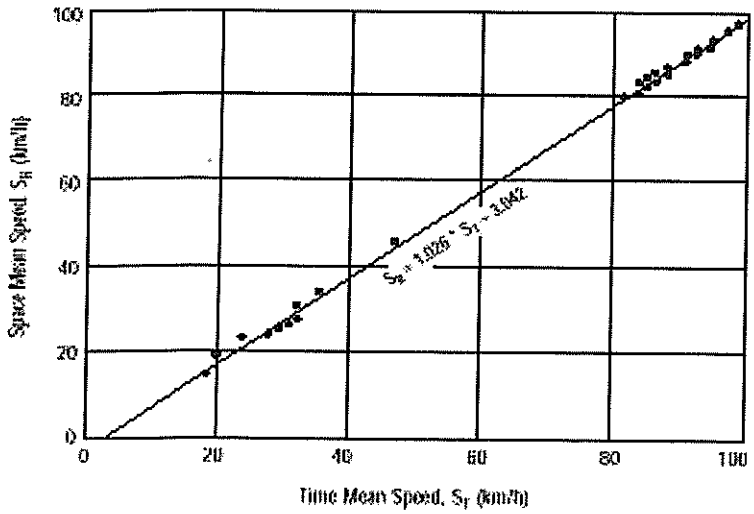
Free-flow speed

Free-flow speed—The average speed of vehicles on a given facility, measured under low-volume conditions, when drivers tend to drive at their desired speed and are not constrained by control delay.

For most of the procedures using speed as a measure of effectiveness in this manual, average travel speed is the defining parameter. For uninterrupted-flow facilities not operating at level of service (LOS) F, the average travel speed is equal to the average running speed.

Exhibit 7-1 shows a typical relationship between time mean and space mean speeds. Space mean speed is always less than time mean speed, but the difference decreases as the absolute value of speed increases. Based on the statistical analysis of observed data, this relationship is useful because time mean speeds often are easier to measure in the field than space mean speeds.

EXHIBIT 7-1. TYPICAL RELATIONSHIP BETWEEN TIME MEAN AND SPACE MEAN SPEED



Source: Drake et al. (7).

It is possible to calculate both time mean and space mean speeds from a sample of individual vehicle speeds. For example, three vehicles are recorded with speeds of 40, 60, and 80 km/h. The time to traverse 1 km is 1.5 min, 1.0 min, and 0.75 min, respectively. The time mean speed is 60 km/h, calculated as (40 + 60 + 80)/3. The space mean speed is 55.4 km/h, calculated as 4(60)/3 ÷ (1.5 + 1.0 + 0.75)].

For capacity analysis, speeds are best measured by observing travel times over a known length of highway. For uninterrupted-flow facilities operating in the range of stable flow, the length may be as short as 50 to 100 m for ease of observation.

As measures of effectiveness, speed criteria must recognize driver expectations and roadway function. For example, a driver expects a higher speed on a freeway than on an urban street. Lower free-flow speeds are tolerable on a roadway with more severe horizontal and vertical alignment, since drivers are not comfortable driving at high speeds. LOS criteria reflect these expectations.

DENSITY

Density is the number of vehicles (or pedestrians) occupying a given length of a lane or roadway at a particular instant. For the computations in this manual, density is averaged over time and is usually expressed as vehicles per kilometer (veh/km) or passenger cars per kilometer (pc/km).

Direct measurement of density in the field is difficult, requiring a vantage point for photographing, videotaping, or observing significant lengths of highway. Density can be computed, however, from the average travel speed and flow rate, which are measured more easily. Equation 7-5 is used for undersaturated traffic conditions.

Computing density

$$D = \frac{v}{S} \qquad (7\text{-}5)$$

where

v = flow rate (veh/h),
S = average travel speed (km/h), and
D = density (veh/km).

A highway segment with a rate of flow of 1,000 veh/h and an average travel speed of 50 km/h would have a density of

$$D = \frac{1000 \text{ veh/h}}{50 \text{ km/h}} = 20 \text{ veh/km}$$

Density is a critical parameter for uninterrupted flow facilities because it characterizes the quality of traffic operations. It describes the proximity of vehicles to one another and reflects the freedom to maneuver within the traffic stream.

Roadway occupancy is frequently used as a surrogate for density in control systems because it is easier to measure. Occupancy in space is the proportion of roadway length covered by vehicles, and occupancy in time identifies the proportion of time a roadway cross section is occupied by vehicles.

HEADWAY AND SPACING

Spacing is the distance between successive vehicles in a traffic stream, measured from the same point on each vehicle (e.g., front bumper, rear axle, etc.). Headway is the time between successive vehicles as they pass a point on a lane or roadway, also measured from the same point on each vehicle.

These characteristics are microscopic, since they relate to individual pairs of vehicles within the traffic stream. Within any traffic stream, both the spacing and the headway of individual vehicles are distributed over a range of values, generally related to the speed of the traffic stream and prevailing conditions. In the aggregate, these microscopic parameters relate to the macroscopic flow parameters of density and flow rate.

Spacing is a distance, measured in meters. It can be determined directly by measuring the distance between common points on successive vehicles at a particular instant. This generally requires complex aerial photographic techniques, so that spacing usually derives from other direct measurements. Headway, in contrast, can be easily measured with stopwatch observations as vehicles pass a point on the roadway.

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 60-s 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 de la carretera PR-349 (calle Liceo) y la calle Manantiales

Termina Período	E-N	E-S	S-N	S-E	N-S	N-E	Total
12:15 AM	4	4	1	0	7	0	16
12:30 AM	1	2	4	6	4	0	17
12:45 AM	3	4	5	5	9	4	30
01:00 AM	4	2	3	2	1	2	14
01:15 AM	0	1	3	0	8	3	15
01:30 AM	1	5	0	2	2	19	29
01:45 AM	1	1	1	0	4	10	17
02:00 AM	0	0	2	0	0	0	2
02:15 AM	0	4	0	1	0	15	20
02:30 AM	2	2	0	1	0	7	12
02:45 AM	0	1	0	3	1	0	5
03:00 AM	1	1	0	0	1	0	3
03:15 AM	0	2	0	1	1	5	9
03:30 AM	3	2	0	4	0	0	9
03:45 AM	1	1	1	0	2	6	11
04:00 AM	3	2	0	3	0	0	8
04:15 AM	0	1	0	2	2	0	5
04:30 AM	1	1	0	4	1	0	7
04:45 AM	2	0	3	0	2	1	8
05:00 AM	3	5	1	7	1	0	17
05:15 AM	3	5	3	12	6	0	29
05:30 AM	5	4	13	7	14	1	44
05:45 AM	11	15	9	17	12	10	74
06:00 AM	11	11	12	13	12	14	73
06:15 AM	16	18	13	22	14	15	98
06:30 AM	20	27	25	31	35	37	175
06:45 AM	31	34	31	40	36	44	216
07:00 AM	41	46	33	54	36	46	256
07:15 AM	46	65	49	95	68	12	335
07:30 AM	54	81	37	99	56	37	364
07:45 AM	43	61	47	90	65	9	315
08:00 AM	46	51	58	64	63	52	334
08:15 AM	45	53	34	77	39	4	252
08:30 AM	40	65	19	50	30	95	299
08:45 AM	25	39	23	51	35	19	192
09:00 AM	29	40	31	31	44	62	237
09:15 AM	15	34	18	46	43	13	169
09:30 AM	18	30	27	32	46	30	183
09:45 AM	28	40	27	53	39	14	201
10:00 AM	27	34	27	50	33	5	176
10:15 AM	31	36	28	28	33	55	211
10:30 AM	22	31	25	42	34	9	163
10:45 AM	33	37	46	35	50	40	241
11:00 AM	32	33	51	17	54	66	253
11:15 AM	34	32	66	0	62	94	288
11:30 AM	31	25	54	23	46	29	208

11:45 AM	34	29	63	35	53	18	232
12:00 PM	13	38	16	54	45	8	174
12:15 PM	23	41	13	38	25	59	199
12:30 PM	24	40	32	60	56	7	219
12:45 PM	12	49	17	69	67	16	230
01:00 PM	28	63	19	66	42	46	264
01:15 PM	21	51	24	63	55	25	239
01:30 PM	24	41	36	36	62	57	256
01:45 PM	28	44	48	38	78	64	300
02:00 PM	17	46	28	58	75	31	255
02:15 PM	35	53	40	73	58	13	272
02:30 PM	37	54	45	60	63	41	300
02:45 PM	35	60	26	82	43	12	258
03:00 PM	16	112	11	77	0	74	290
03:15 PM	15	124	13	106	13	43	314
03:30 PM	27	134	17	87	13	126	404
03:45 PM	5	137	4	104	16	94	360
04:00 PM	11	71	8	74	58	55	277
04:15 PM	6	72	5	38	59	23	253
04:30 PM	21	64	22	94	65	1	267
04:45 PM	5	53	9	89	105	5	266
05:00 PM	11	66	13	96	71	1	258
05:15 PM	7	75	8	92	76	32	290
05:30 PM	11	66	13	98	76	1	265
05:45 PM	20	56	18	80	50	3	227
06:00 PM	15	44	22	55	64	28	228
06:15 PM	16	55	10	74	38	5	198
06:30 PM	11	43	12	57	45	15	183
06:45 PM	17	50	12	69	34	1	183
07:00 PM	14	43	9	54	27	9	156
07:15 PM	16	47	10	57	22	8	160
07:30 PM	9	27	7	39	29	5	116
07:45 PM	15	46	8	45	10	19	143
08:00 PM	9	28	6	36	20	6	105
08:15 PM	14	41	6	33	16	47	157
08:30 PM	11	32	4	25	1	24	97
08:45 PM	10	29	3	18	10	54	124
09:00 PM	1	3	4	6	38	19	71
09:15 PM	10	30	4	23	19	50	136
09:30 PM	7	21	5	25	31	24	113
09:45 PM	5	14	5	17	14	10	65
10:00 PM	6	16	5	20	17	11	75
10:15 PM	9	25	6	34	23	7	104
10:30 PM	6	16	4	19	6	3	54
10:45 PM	7	21	2	14	6	33	83
11:00 PM	4	12	3	14	6	3	42
11:15 PM	8	23	2	13	6	47	99
11:30 PM	6	19	2	10	0	25	62
11:45 PM	2	6	1	5	5	10	29
12:00 PM	2	6	1	7	3	2	21
Total	1483	3324	1531	3706	2865	2204	15113

Carretera PR-349, Km. 1.1

Termina Período	Hacia Este	Hacia Oeste	Total
12:15 AM	0	0	0
12:30 AM	1	1	2
12:45 AM	0	0	0
01:00 AM	0	0	0
01:15 AM	0	0	0
01:30 AM	0	0	0
01:45 AM	0	0	0
02:00 AM	0	0	0
02:15 AM	2	1	3
02:30 AM	0	0	0
02:45 AM	0	0	0
03:00 AM	0	0	0
03:15 AM	0	0	0
03:30 AM	0	0	0
03:45 AM	0	0	0
04:00 AM	0	0	0
04:15 AM	0	0	0
04:30 AM	1	1	2
04:45 AM	2	4	6
05:00 AM	2	2	4
05:15 AM	3	2	5
05:30 AM	2	2	4
05:45 AM	3	3	6
06:00 AM	8	7	15
06:15 AM	5	5	10
06:30 AM	15	10	25
06:45 AM	25	19	44
07:00 AM	23	20	43
07:15 AM	24	25	49
07:30 AM	17	17	34
07:45 AM	22	23	45
08:00 AM	25	21	46
08:15 AM	31	38	69
08:30 AM	16	12	28
08:45 AM	9	8	17
09:00 AM	7	5	12
09:15 AM	12	10	22
09:30 AM	7	5	12
09:45 AM	10	10	20
10:00 AM	9	10	19
10:15 AM	9	7	16
10:30 AM	6	6	12
10:45 AM	8	7	15
11:00 AM	7	5	12
11:15 AM	10	7	17
11:30 AM	9	10	19
11:45 AM	6	7	13

12:00 PM	16	13	29
12:15 PM	16	11	27
12:30 PM	10	10	20
12:45 PM	22	16	38
01:00 PM	11	9	20
01:15 PM	11	9	20
01:30 PM	11	8	19
01:45 PM	10	7	17
02:00 PM	19	13	32
02:15 PM	22	23	45
02:30 PM	33	30	63
02:45 PM	21	21	42
03:00 PM	35	30	65
03:15 PM	25	23	48
03:30 PM	19	14	33
03:45 PM	23	16	39
04:00 PM	17	11	28
04:15 PM	10	7	17
04:30 PM	12	11	23
04:45 PM	17	10	27
05:00 PM	13	10	23
05:15 PM	17	11	28
05:30 PM	17	13	30
05:45 PM	22	20	42
06:00 PM	16	11	27
06:15 PM	14	13	27
06:30 PM	18	14	32
06:45 PM	12	11	23
07:00 PM	11	10	21
07:15 PM	15	15	30
07:30 PM	9	7	16
07:45 PM	6	6	12
08:00 PM	7	6	13
08:15 PM	5	3	8
08:30 PM	11	10	21
08:45 PM	10	5	15
09:00 PM	8	1	9
09:15 PM	9	5	14
09:30 PM	8	5	13
09:45 PM	4	3	7
10:00 PM	5	4	9
10:15 PM	9	7	16
10:30 PM	5	5	10
10:45 PM	4	2	6
11:00 PM	1	1	2
11:15 PM	2	1	3
11:30 PM	2	1	3
11:45 PM	3	2	5
12:00 PM	2	2	4
Total	961	806	1767

Apéndice 4. Cómputos para el análisis operacional

TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*1*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Mayagüez*
Date Performed	*11/24/2004*	Analysis Year	*2004*
Analysis Time Period	*7:00 - 8:00 AM*		
Project Description *Estudio de Impacto en el Tránsito Urb. Monte Sierra*			
East/West Street: *PR-349*		North/South Street: *Liceo (PR-349)/Manantiales*	
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)	*0*	*0*	*0*	*258*	*0*	*189*
Peak-hour factor, PHF	*1.00*	*1.00*	*1.00*	*0.80*	*1.00*	*0.88*
Hourly Flow Rate (veh/h)	*0*	*0*	*0*	*322*	*0*	*214*
Proportion of heavy vehicles, P_{HV}	*0*	--	--	*3*	--	--
Median type	*Undivided*					
RT Channelized?			*0*			*0*
Lanes	*0*	*0*	*0*	*0*	*1*	*0*
Configuration				*LTR*		
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*	*191*	*348*	*110*	*252*	*0*
Peak-hour factor, PHF	*1.00*	*0.82*	*0.88*	*0.53*	*0.93*	*1.00*
Hourly Flow Rate (veh/h)	*0*	*232*	*395*	*207*	*270*	*0*
Proportion of heavy vehicles, P_{HV}	*0*	*6*	*6*	*2*	*2*	*0*

Percent grade (%)	5			5		
Flared approach		N			N	
Storage		0			0	
RT Channelized?			0			0
Lanes	0	1	0	0	1	0
Configuration			TR	LT		

Control Delay, Queue Length, Level of Service

Approach	EB	WB	Northbound			Southbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LTR			TR	LT		
Volume, v (vph)		322			627	477		
Capacity, c_m (vph)		1617			445	0		
v/c ratio		0.20			1.41			
Queue length (95%)		0.74			30.47			
Control Delay (s/veh)		7.8			221.7			
LOS		A			F	F		
Approach delay (s/veh)	–	–	221.7					
Approach LOS	–	–	F					

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HCS2000: Two-Lane Highways Release 4.1d

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Two-Way Two-Lane Highway Segment Analysis

Analyst	José L. Pérez Berenguer
Agency/Co.	Pérez Berenguer y Asociados
Date Performed	11/24/2004
Analysis Time Period	7:00 - 8:00 AM
Highway	PR-349
From/To	Mayagüez - Las Mesas
Jurisdiction	Mayagüez
Analysis Year	2004
Description	Estudio de Impacto en el Tránsito, Urb. Monte Sierra

Input Data

Highway class	Class 2				
Shoulder width	1.0	m	Peak-hour factor, PHF	0.87	
Lane width	3.6	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		174	veh/h		
Directional split	51 / 49		%		

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,	1.000	
Two-way flow rate,(note-1) vp	282	pc/h
Highest directional split proportion (note-2)	144	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	70.0	km/h
Adj. for lane and shoulder width, fLS	4.2	km/h
Adj. for access points, fA	0.7	km/h
Free-flow speed, FFS	65.1	km/h
Adjustment for no-passing zones, fnp	0.0	km/h
Average travel speed, ATS	61.6	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	1.000	
Two-way flow rate,(note-1) vp	260	pc/h
Highest directional split proportion (note-2)	133	
Base percent time-spent-following, BPTSF	20.4	%
Adj.for directional distribution and no-passing zones, fd/np	0.1	
Percent time-spent-following, PTSF	20.6	%

Level of Service and Other Performance Measures

Level of service, LOS	A	
Volume to capacity ratio, v/c	0.09	
Peak 15-min vehicle-kilometers of travel, VkmT15	50	veh-km
Peak-hour vehicle-kilometers of travel, VkmT60	174	veh-km
Peak 15-min total travel time, TT15	0.8	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	*1*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Mayagüez*
Date Performed	*11/24/2004*	Analysis Year	*2004*
Analysis Time Period	*2:45 - 3:45 PM*		
Project Description *Estudio de Impacto en el Tránsito Urb. Monte Sierra*			
East/West Street: *PR-349*		North/South Street: *Liceo (PR-349)/Manantiales*	
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)	*0*	*0*	*0*	*507*	*0*	*63*
Peak-hour factor, PHF	*1.00*	*1.00*	*1.00*	*0.93*	*1.00*	*0.58*
Hourly Flow Rate (veh/h)	*0*	*0*	*0*	*545*	*0*	*108*
Proportion of heavy vehicles, P_{HV}	*0*	--	--	*3*	--	--
Median type	*Undivided*					
RT Channelized?			*0*			*0*
Lanes	*0*	*0*	*0*	*0*	*1*	*0*
Configuration				*LTR*		
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*	*45*	*374*	*337*	*42*	*0*
Peak-hour factor, PHF	*1.00*	*0.66*	*0.88*	*0.67*	*0.66*	*1.00*
Hourly Flow Rate (veh/h)	*0*	*68*	*425*	*502*	*63*	*0*
Proportion of heavy vehicles, P_{HV}	*0*	*6*	*6*	*2*	*2*	*0*

Percent grade (%)	5			5		
Flared approach		N			N	
Storage		0			0	
RT Channelized?			0			0
Lanes	0	1	0	0	1	0
Configuration			TR	LT		

Control Delay, Queue Length, Level of Service								
Approach	EB	WB	Northbound			Southbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LTR			TR	LT		
Volume, v (vph)		545			493	565		
Capacity, c_m (vph)		1617			502	32		
v/c ratio		0.34			0.98	17.66		
Queue length (95%)		1.51			13.05	69.67		
Control Delay (s/veh)		8.4			64.6	7730		
LOS		A			F	F		
Approach delay (s/veh)	–	–	64.6			7730		
Approach LOS	–	–	F			F		

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Two-Way Two-Lane Highway Segment Analysis

Analyst	José L. Pérez Berenguer
Agency/Co.	Pérez Berenguer y Asociados
Date Performed	11/24/2004
Analysis Time Period	2:45 - 3:45 PM
Highway	PR-349
From/To	Mayagüez - Las Mesas
Jurisdiction	Mayagüez
Analysis Year	2004
Description	Estudio de Impacto en el Tránsito, Urb. Monte Sierra

Input Data

Highway class	Class 2				
Shoulder width	1.0	m	Peak-hour factor, PHF	0.72	
Lane width	3.6	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	181		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,	1.000	
Two-way flow rate,(note-1) vp	354	pc/h
Highest directional split proportion (note-2)	198	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	70.0	km/h
Adj. for lane and shoulder width, fLS	4.2	km/h
Adj. for access points, fA	0.7	km/h
Free-flow speed, FFS	65.1	km/h
Adjustment for no-passing zones, fnp	0.0	km/h
Average travel speed, ATS	60.7	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	1.000	
Two-way flow rate,(note-1) vp	326	pc/h
Highest directional split proportion (note-2)	183	
Base percent time-spent-following, BPTSF	24.9	%
Adj.for directional distribution and no-passing zones, fd/np	0.5	
Percent time-spent-following, PTSF	25.5	%

Level of Service and Other Performance Measures

Level of service, LOS	A	
Volume to capacity ratio, v/c	0.11	
Peak 15-min vehicle-kilometers of travel, VkmT15	63	veh-km
Peak-hour vehicle-kilometers of travel, VkmT60	181	veh-km
Peak 15-min total travel time, TT15	1.0	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.

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PLANNING ANALYSIS

Analyst:	José L. Pérez Berenguer
Intersection:	1
Agency/Co.:	Pérez Berenguer y Asociados
Area Type:	CBD or Similar
Date Performed:	11/26/2004
Jurisdiction:	Mayagüez
Analysis Time Period:	7:00 - 8:00 AM
Analysis Year:	2006

Project ID: Estudio de Impacto en el Tránsito Urb. Monte Sierra

East/West Street	North/South Street
PR-349	Liceo (PR-349)/Manantiales

VOLUME DATA

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
Num. Lanes	0	0	0	1	0	1	0	1	1	0	1	0
Volume				324		237	0	196	403	127	259	0
Parking					N			N			N	
Coord.					N			N			N	
LT Treat.				?			?			?		

Peak hour factor: 0.93 Area Type: CBD or Similar

LANE VOLUME WORKSHEET

	EAST BOUND	WEST BOUND	NORTH BOUND	SOUTH BOUND
LEFT TURN MOVEMENT				
1. LT volume		324	0	127
2. Opposing mainline volume		0	259	599
3. Number of exclusive LT lanes		1	0	0
Cross Product [2] * [1]		0	0	76073
Left Lane Configuration (E=Excl, S=Shrd):		E	S	S
Left Turn Treatment Type:		P	U	P
4. LT adjustment factor		0.950		
5. LT lane vol		341	0	127
RIGHT TURN MOVEMENT				
Right Lane Configuration (E=Excl, S=Shrd)		E	E	S
6. RT volume		237	403	0
7. Exclusive lanes		1	1	0
8. RT adjustment factor		0.850	0.850	0.850
9. Exclusive RT lane volume		279	474	
10. Shared lane vol				0
THROUGH MOVEMENT				
11. Thru volume		0	196	259

12. Parking adjustment factor	1.00	1.00	1.00
13. No. of thru lanes including shared	0	1	1
14. Total approach volume	0	196	259
15. Prop. of left turns in lane group	0.00	0.00	0.33
16. Left turn equivalence		1.80	
17. LT adj. factor:		1.000	1.000
18. Through lane volume	0	196	259
19. Critical lane volume	279	474	259

Left Turn Check (if [16] > 3.5)

20. Permitted left turn sneaker capacity: 60
7200/Cmax

SIGNAL OPERATIONS WORKSHEET

		EAST BOUND	WEST BOUND	NORTH BOUND	SOUTH BOUND
Phase Plan Selection from Lane Volume Worksheet					
Critical through-RT vol: [19]			279	474	259
LT lane vol: [5]			341	0	127
Left turn protection: (P/U/N)			P	U	P
Dominant left turn: (Indicate by '<')					
Selection Criteria based on the	Plan 1:	U	U	U	U
specified left turn protection	Plan 2a:	U	P	U	P
	Plan 2b:	P	U	P	U
< Indicates the dominant left turn	Plan 3a:	<P	P	<P	P
for each opposing pair	Plan 3b:	P	<P	P	<P
	Plan 4:	N	N	N	N
Phase plan selected (1 to 4)			4		2a

Min. cycle (Cmin) 60 Max. cycle (Cmax) 120

Timing Plan	Value	EAST-WEST Ph 1	EAST-WEST Ph 2	EAST-WEST Ph 3	NORTH-SOUTH Ph 1	NORTH-SOUTH Ph 2	NORTH-SOUTH Ph 3
Movement codes		ETL	WTL		STL	NST	
Critical phase vol [CV]		0	341	0	127	474	0
Critical sum [CS]	942						
CBD adjustment [CBD]	0.900						
Reference sum [RS]	1431						
Lost time/phase [PL]		4	4	0	4	4	0
Lost time/cycle [TL]	16						
Cycle length [CYC]	60.0						
Phase time		0.0	19.9	0.0	9.9	26.1	0.0
Critical v/c Ratio [Xcm]	0.90						
Status	Near capacity						

HCS2000™™ DETAILED REPORT

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*1*
Agency or Co.	*Pérez Berenguer y Asociados*	Area Type	*CBD or Similar*
Date Performed	*11/26/2004*	Jurisdiction	*Mayagüez*
Time Period	*7:00 - 8:00 AM*	Analysis Year	*2006*
		Project ID	*Estudio de Impacto en el Tránsito Urb. Monte Sierra*

Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of lanes, N_1	*0*	*0*	*0*	*1*	*0*	*1*	*0*	*1*	*1*	*0*	*1*	*0*
Lane group				*L*		*R*		*T*	*R*		*LT*	
Volume, V (vph)				*324*		*237*		*196*	*403*	*127*	*259*	
% Heavy vehicles, %HV				*3*		*3*		*6*	*6*	*2*	*2*	
Peak-hour factor, PHF				*0.80*		*0.88*		*0.82*	*0.88*	*0.53*	*0.93*	
Pretimed (P) or actuated (A)				*A*		*A*		*A*	*A*	*A*	*A*	
Start-up lost time, l_1				*2.0*		*2.0*		*2.0*	*2.0*		*2.0*	
Extension of effective green, e				*2.0*		*2.0*		*2.0*	*2.0*		*2.0*	
Arrival type, AT				*3*		*3*		*3*	*3*		*3*	
Unit extension, UE				*3.0*		*3.0*		*3.0*	*3.0*		*3.0*	
Filtering/metering, I				*1.000*	*1.000*	*1.000*		*1.000*	*1.000*		*1.000*	
Initial unmet demand, Q_b				*0.0*		*0.0*		*0.0*	*0.0*		*0.0*	
Ped / Bike / RTOR volumes	*0*			*0*		*50*	*0*		*50*			
Lane width				*3.7*		*3.7*		*3.7*	*3.7*		*4.7*	
Parking / Grade / Parking	*N*		*N*	*N*	*-3*	*N*	*N*	*5*	*N*	*N*	*5*	*N*
Parking maneuvers, N_m												
Buses stopping, N_B				*0*		*0*		*0*	*0*		*0*	
Min. time for pedestrians, G_p	*3.2*			*3.2*			*3.2*					

Phasing	WB Only	02	03	04	NS Perm	06	07	08
Timing	G = *19.1*	G =	G =	G =	G = *34.9*	G =	G =	G =
	Y = *3*	Y =	Y =	Y =	Y = *3*	Y =	Y =	Y =
Duration of Analysis, T = *0.25*						Cycle Length, C = *60.0*		

Lane Group Capacity, Control Delay, and LOS Determination

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Adjusted flow rate, v				*405*		*213*		*239*	*401*		*518*	
Lane group capacity, c				*515*		*461*		*925*	*786*		*735*	
v/c ratio, X				*0.79*		*0.46*		*0.26*	*0.51*		*0.70*	
Total green ratio, g/C				*0.32*		*0.32*		*0.58*	*0.58*		*0.58*	
Uniform delay, d_1				*18.6*		*16.3*		*6.2*	*7.5*		*8.9*	
Progression factor, PF				*1.000*		*1.000*		*1.000*	*1.000*		*1.000*	
Delay calibration, k				*0.33*		*0.11*		*0.11*	*0.12*		*0.27*	
Incremental delay, d_2				*7.9*		*0.7*		*0.1*	*0.6*		*3.1*	
Initial queue delay, d_3												
Control delay				*26.5*		*17.1*		*6.3*	*8.0*		*12.0*	
Lane group LOS				*C*		*B*		*A*	*A*		*B*	
Approach delay				*23.3*			*7.4*			*12.0*		
Approach LOS				*C*			*A*			*B*		
Intersection delay	*14.3*			X_c = *0.73*			Intersection LOS			*B*		

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Version 4.1e

TWO-WAY STOP CONTROL SUMMARY

General Information		Site Information	
Analyst	*José L. Pérez Berenguer*	Intersection	*2*
Agency/Co.	*Pérez Berenguer y Asoc.*	Jurisdiction	*Mayagüez*
Date Performed	*11/26/2004*	Analysis Year	*2006*
Analysis Time Period	*7:00 - 8:00 AM*		
Project Description *Estudio de Impacto en el Tránsito, Urb. Monte Sierra*			
East/West Street: *PR-349*		North/South Street: *Calle Núm. 1*	
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)	*0*	*90*	*59*	*58*	*88*	*0*
Peak-hour factor, PHF	*1.00*	*0.88*	*0.88*	*0.86*	*0.86*	*1.00*
Hourly Flow Rate (veh/h)	*0*	*102*	*67*	*67*	*102*	*0*
Proportion of heavy vehicles, P_{HV}	*0*	–	–	*0*	–	–
Median type	*Raised curb*					
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)	*102*	*0*	*104*	*0*	*0*	*0*
Peak-hour factor, PHF	*0.86*	*1.00*	*0.88*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate (veh/h)	*118*	*0*	*118*	*0*	*0*	*0*
Proportion of heavy vehicles, P_{HV}	*0*	*0*	*0*	*0*	*0*	*0*
Percent grade (%)	*4*			*0*		

Flared approach		N			N	
Storage		0			0	
RT Channelized?			0			0
Lanes	0	1	0	0	0	0
Configuration		LTR				

Control Delay, Queue Length, Level of Service								
Approach	EB	WB	Northbound			Southbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LT		LTR				
Volume, v (vph)		67		236				
Capacity, c_m (vph)		1421		758				
v/c ratio		0.05		0.31				
Queue length (95%)		0.15		1.33				
Control Delay (s/veh)		7.7		11.9				
LOS		A		B				
Approach delay (s/veh)	--	--	11.9					
Approach LOS	--	--	B					

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PLANNING ANALYSIS

Analyst:	José L. Pérez Berenguer
Intersection:	1
Agency/Co.:	Pérez Berenguer y Asociados
Area Type:	CBD or Similar
Date Performed:	11/26/2004
Jurisdiction:	Mayagüez
Analysis Time Period:	2:45 - 3:45 PM
Analysis Year:	2006

Project ID: Estudio de Impacto en el Tránsito Urb. Monte Sierra

East/West Street	North/South Street
PR-349	Liceo (PR-349)/Manantiales

VOLUME DATA

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
Num. Lanes	0	0	0	1	0	1	0	1	1	0	1	0
Volume				603		75	0	46	470	424	43	0
Parking					N			N			N	
Coord.					N			N			N	
LT Treat.				?			?			?		

Peak hour factor: 0.85 Area Type: CBD or Similar

LANE VOLUME WORKSHEET

	EAST BOUND	WEST BOUND	NORTH BOUND	SOUTH BOUND
LEFT TURN MOVEMENT				
1. LT volume		603	0	424
2. Opposing mainline volume		0	43	516
3. Number of exclusive LT lanes		1	0	0
Cross Product [2] * [1]		0	0	218784
Left Lane Configuration (E=Excl, S=Shrd):		E	S	S
Left Turn Treatment Type:		P	U	P
4. LT adjustment factor		0.950		
5. LT lane vol		635	0	424
RIGHT TURN MOVEMENT				
Right Lane Configuration (E=Excl, S=Shrd)		E	E	S
6. RT volume		75	470	0
7. Exclusive lanes		1	1	0
8. RT adjustment factor		0.850	0.850	0.850
9. Exclusive RT lane volume		88	553	
10. Shared lane vol				0
THROUGH MOVEMENT				
11. Thru volume		0	46	43

12. Parking adjustment factor	1.00	1.00	1.00
13. No. of thru lanes including shared	0	1	1
14. Total approach volume	0	46	43
15. Prop. of left turns in lane group	0.00	0.00	0.91
16. Left turn equivalence		1.44	
17. LT adj. factor:		1.000	1.000
18. Through lane volume	0	46	43
19. Critical lane volume	88	553	43
Left Turn Check (if [16] > 3.5)			
20. Permitted left turn sneaker capacity: 7200/Cmax		60	

SIGNAL OPERATIONS WORKSHEET

		EAST BOUND	WEST BOUND	NORTH BOUND	SOUTH BOUND
Phase Plan Selection from Lane Volume Worksheet			88	553	43
Critical through-RT vol: [19]			635	0	424
LT lane vol: [5]			P	U	P
Left turn protection: (P/U/N)					
Dominant left turn: (Indicate by '<')					
Selection Criteria based on the	Plan 1:	U	U	U	U
specified left turn protection	Plan 2a:	U	P	U	P
	Plan 2b:	P	U	P	U
< Indicates the dominant left turn	Plan 3a:	<P	P	<P	P
for each opposing pair	Plan 3b:	P	<P	P	<P
	Plan 4:	N	N	N	N
Phase plan selected (1 to 4)			4		2a

Min. cycle (Cmin) 60 Max. cycle (Cmax) 120

Timing Plan	Value	EAST-WEST			NORTH-SOUTH		
		Ph 1	Ph 2	Ph 3	Ph 1	Ph 2	Ph 3
Movement codes		ETL	WTL		STL	NST	
Critical phase vol [CV]		0	635	0	424	553	0
Critical sum [CS]	1612						
CBD adjustment [CBD]	0.900						
Reference sum [RS]	1308						
Lost time/phase [PL]		4	4	0	4	4	0
Lost time/cycle [TL]	16						
Cycle length [CYC]	120.0						
Phase time		0.0	45.0	0.0	31.4	39.7	0.0
Critical v/c Ratio [Xcm]	1.42						
Status	Over capacity						

HCS2000™™ DETAILED REPORT

General Information		Site Information	
Analyst	José L. Pérez Berenguer	Intersection	1
Agency or Co.	Pérez Berenguer y Asociados	Area Type	CBD or Similar
Date Performed	11/26/2004	Jurisdiction	Mayagüez
Time Period	2:45 - 3:45 PM	Analysis Year	2006
		Project ID	Estudio de Impacto en el Tránsito, Urb. Monte Sierra

Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of lanes, N_1	0	0	0	1	0	1	0	1	1	0	1	0
Lane group				L		R		T	R		LT	
Volume, V (vph)				603		75		46	470	424	43	
% Heavy vehicles, %HV				7		7		5	5	1	1	
Peak-hour factor, PHF				0.93		0.58		0.66	0.88	0.67	0.66	
Pretimed (P) or actuated (A)				A		A		A	A	A	A	
Start-up lost time, l_1				2.0		2.0		2.0	2.0		2.0	
Extension of effective green, e				2.0		2.0		2.0	2.0		2.0	
Arrival type, AT				3		3		3	3		3	
Unit extension, UE				3.0		3.0		3.0	3.0		3.0	
Filtering/metering, I				1.000	1.000	1.000		1.000	1.000		1.000	
Initial unmet demand, Q_b				0.0		0.0		0.0	0.0		0.0	
Ped / Bike / RTOR volumes	0			0		50	0		50			
Lane width				3.7		3.7		3.7	3.7		4.7	
Parking / Grade / Parking	N		N	N	-3	N	N	5	N	N	5	N
Parking maneuvers, N_m												
Buses stopping, N_B				0		0		0	0		0	
Min. time for pedestrians, G_p	3.2			3.2			3.2					

Phasing	WB Only	02	03	04	NS Perm	06	07	08
Timing	G = *32.0*	G =	G =	G =	G = *41.0*	G =	G =	G =
	Y = *3*	Y =	Y =	Y =	Y = *3*	Y =	Y =	Y =
Duration of Analysis, T = *0.25*						Cycle Length, C = *79.0*		

Lane Group Capacity, Control Delay, and LOS Determination

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Adjusted flow rate, v				*648*		*43*		*70*	*477*		*698*	
Lane group capacity, c				*631*		*565*		*833*	*708*		*666*	
v/c ratio, X				*1.03*		*0.08*		*0.08*	*0.67*		*1.05*	
Total green ratio, g/C				*0.41*		*0.41*		*0.52*	*0.52*		*0.52*	
Uniform delay, d_1				*23.5*		*14.4*		*9.6*	*14.1*		*19.0*	
Progression factor, PF				*1.000*		*1.000*		*1.000*	*1.000*		*1.000*	
Delay calibration, k				*0.50*		*0.11*		*0.11*	*0.25*		*0.50*	
Incremental delay, d_2				*42.9*		*0.1*		*0.0*	*2.5*		*48.1*	
Initial queue delay, d_3												
Control delay				*66.4*		*14.5*		*9.6*	*16.6*		*67.1*	
Lane group LOS				*E*		*B*		*A*	*B*		*E*	
Approach delay				*63.1*			*15.7*			*67.1*		
Approach LOS				*E*			*B*			*E*		
Intersection delay	*51.2*			X_c = *1.04*			Intersection LOS			*D*		

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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	*Mayagüez*
Date Performed	*11/26/2004*	Analysis Year	*2006*
Analysis Time Period	*2:45 - 3:45 PM*		
Project Description *Estudio de Impacto en el Tránsito, Urb. Monte Sierra*			
East/West Street: *PR-349*		North/South Street: *Calle Núm. 1*	
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)	*0*	*105*	*162*	*125*	*81*	*0*
Peak-hour factor, PHF	*1.00*	*0.73*	*0.73*	*0.71*	*0.71*	*1.00*
Hourly Flow Rate (veh/h)	*0*	*143*	*221*	*176*	*114*	*0*
Proportion of heavy vehicles, P_{HV}	*0*	–	–	*0*	–	–
Median type	*Raised curb*					
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)	*92*	*0*	*120*	*0*	*0*	*0*
Peak-hour factor, PHF	*0.71*	*1.00*	*0.73*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate (veh/h)	*129*	*0*	*164*	*0*	*0*	*0*
Proportion of heavy vehicles, P_{HV}	*0*	*0*	*0*	*0*	*0*	*0*
Percent grade (%)	*4*			*0*		

Flared approach		N			N	
Storage		0			0	
RT Channelized?			0			0
Lanes	0	1	0	0	0	0
Configuration		LTR				

Control Delay, Queue Length, Level of Service								
Approach	EB	WB	Northbound			Southbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LT		LTR				
Volume, v (vph)		176		293				
Capacity, c_m (vph)		1206		581				
v/c ratio		0.15		0.50				
Queue length (95%)		0.51		2.83				
Control Delay (s/veh)		8.5		17.3				
LOS		A		C				
Approach delay (s/veh)	–	–	17.3					
Approach LOS	–	–	C					

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PLANNING ANALYSIS

Analyst:	José L. Pérez Berenguer
Intersection:	1
Agency/Co.:	Pérez Berenguer y Asociados
Area Type:	CBD or Similar
Date Performed:	11/26/2004
Jurisdiction:	Mayagüez
Analysis Time Period:	7:00 - 8:00 AM
Analysis Year:	2026

Project ID: Estudio de Impacto en el Tránsito Urb. Monte Sierra

East/West Street	North/South Street
PR-349	Liceo (PR-349)/Manantiales

VOLUME DATA

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
Num. Lanes	0	0	0	1	0	1	0	1	1	0	1	0
Volume				428		313	0	259	532	168	342	0
Parking					N			N			N	
Coord.					N			N			N	
LT Treat.				?			?			?		

Peak hour factor: 0.93 Area Type: CBD or Similar

LANE VOLUME WORKSHEET

	EAST BOUND	WEST BOUND	NORTH BOUND	SOUTH BOUND
LEFT TURN MOVEMENT				
1. LT volume		428	0	168
2. Opposing mainline volume		0	342	791
3. Number of exclusive LT lanes		1	0	0
Cross Product [2] * [1]		0	0	132888
Left Lane Configuration (E=Excl, S=Shrd):		E	S	S
Left Turn Treatment Type:		P	U	P
4. LT adjustment factor		0.950		
5. LT lane vol		451	0	168
RIGHT TURN MOVEMENT				
Right Lane Configuration (E=Excl, S=Shrd)		E	E	S
6. RT volume		313	532	0
7. Exclusive lanes		1	1	0
8. RT adjustment factor		0.850	0.850	0.850
9. Exclusive RT lane volume		368	626	
10. Shared lane vol				0
THROUGH MOVEMENT				
11. Thru volume		0	259	342

12. Parking adjustment factor	1.00	1.00	1.00
13. No. of thru lanes including shared	0	1	1
14. Total approach volume	0	259	342
15. Prop. of left turns in lane group	0.00	0.00	0.33
16. Left turn equivalence		1.96	
17. LT adj. factor:		1.000	1.000
18. Through lane volume	0	259	342
19. Critical lane volume	368	626	342
Left Turn Check (if [16] > 3.5)			
20. Permitted left turn sneaker capacity: 7200/Cmax		60	

SIGNAL OPERATIONS WORKSHEET

Phase Plan Selection from Lane Volume Worksheet		EAST BOUND	WEST BOUND	NORTH BOUND	SOUTH BOUND
Critical through-RT vol: [19]			368	626	342
LT lane vol: [5]			451	0	168
Left turn protection: (P/U/N)			P	U	P
Dominant left turn: (Indicate by '<')					
Selection Criteria based on the	Plan 1:	U	U	U	U
specified left turn protection	Plan 2a:	U	P	U	P
	Plan 2b:	P	U	P	U
< Indicates the dominant left turn	Plan 3a:	<P	P	<P	P
for each opposing pair	Plan 3b:	P	<P	P	<P
	Plan 4:	N	N	N	N
Phase plan selected (1 to 4)			4		2a

Min. cycle (Cmin) 60 Max. cycle (Cmax) 120

Timing Plan	Value	EAST-WEST Ph 1	EAST-WEST Ph 2	EAST-WEST Ph 3	NORTH-SOUTH Ph 1	NORTH-SOUTH Ph 2	NORTH-SOUTH Ph 3
Movement codes		ETL	WTL		STL	NST	
Critical phase vol [CV]		0	451	0	168	626	0
Critical sum [CS]	1245						
CBD adjustment [CBD]	0.900						
Reference sum [RS]	1431						
Lost time/phase [PL]		4	4	0	4	4	0
Lost time/cycle [TL]	16						
Cycle length [CYC]	120.0						
Phase time		0.0	41.7	0.0	18.0	56.3	0.0
Critical v/c Ratio [Xcm]	1.00						
Status	Over capacity						

HCS2000™ DETAILED REPORT

General Information		Site Information	
Analyst	José L. Pérez Berenguer	Intersection	1
Agency or Co.	Pérez Berenguer y Asociados	Area Type	CBD or Similar
Date Performed	11/26/2004	Jurisdiction	Mayagüez
Time Period	7:00 - 8:00 AM	Analysis Year	2026
		Project ID	Estudio de Impacto en el Tránsito, Urb. Monte Sierra

Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of lanes, N_1	0	0	0	1	0	1	0	1	1	0	1	0
Lane group				L		R		T	R		LT	
Volume, V (vph)				428		313		259	532	168	342	
% Heavy vehicles, %HV				3		3		6	6	2	2	
Peak-hour factor, PHF				0.80		0.88		0.82	0.88	0.53	0.93	
Pretimed (P) or actuated (A)				A		A		A	A	A	A	
Start-up lost time, l_1				2.0		2.0		2.0	2.0		2.0	
Extension of effective green, e				2.0		2.0		2.0	2.0		2.0	
Arrival type, AT				3		3		3	3		3	
Unit extension, UE				3.0		3.0		3.0	3.0		3.0	
Filtering/metering, I				1.000	1.000	1.000		1.000	1.000		1.000	
Initial unmet demand, Q_b				0.0		0.0		0.0	0.0		0.0	
Ped / Bike / RTOR volumes	0			0		50	0		50			
Lane width				3.7		3.7		3.7	3.7		4.7	
Parking / Grade / Parking	N		N	N	-3	N	N	5	N	N	5	N
Parking maneuvers, N_m												
Buses stopping, N_B				0		0		0	0		0	
Min. time for pedestrians, G_p	3.2			3.2			3.2					

Phasing	WB Only	02	03	04	NS Perm	06	07	08
Timing	G = *28.0*	G =	G =	G =	G = *50.0*	G =	G =	G =
	Y = *3*	Y =	Y =	Y =	Y = *3*	Y =	Y =	Y =
Duration of Analysis, T = *0.25*						Cycle Length, C = *84.0*		

Lane Group Capacity, Control Delay, and LOS Determination

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Adjusted flow rate, v				*535*		*299*		*316*	*548*		*685*	
Lane group capacity, c				*540*		*483*		*946*	*805*		*680*	
v/c ratio, X				*0.99*		*0.62*		*0.33*	*0.68*		*1.01*	
Total green ratio, g/C				*0.33*		*0.33*		*0.60*	*0.60*		*0.60*	
Uniform delay, d_1				*27.9*		*23.5*		*8.6*	*11.6*		*17.0*	
Progression factor, PF				*1.000*		*1.000*		*1.000*	*1.000*		*1.000*	
Delay calibration, k				*0.49*		*0.20*		*0.11*	*0.25*		*0.50*	
Incremental delay, d_2				*36.2*		*2.4*		*0.2*	*2.4*		*36.3*	
Initial queue delay, d_3												
Control delay				*64.1*		*25.9*		*8.8*	*13.9*		*53.3*	
Lane group LOS				*E*		*C*		*A*	*B*		*D*	
Approach delay				*50.4*			*12.0*			*53.3*		
Approach LOS				*D*			*B*			*D*		
Intersection delay	*37.3*			X_c = *1.00*			Intersection LOS			*D*		

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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	*Mayagüez*
Date Performed	*11/26/2004*	Analysis Year	*2026*
Analysis Time Period	*7:00 - 8:00 AM*		

Project Description *Estudio de Impacto en el Tránsito, Urb. Monte Sierra*

East/West Street: *PR-349*	North/South Street: *Calle Núm. 1*
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)	*0*	*119*	*78*	*77*	*116*	*0*
Peak-hour factor, PHF	*1.00*	*0.88*	*0.88*	*0.86*	*0.86*	*1.00*
Hourly Flow Rate (veh/h)	*0*	*135*	*88*	*89*	*134*	*0*
Proportion of heavy vehicles, P_{HV}	*0*	--	--	*0*	--	--
Median type	*Raised curb*					
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)	*135*	*0*	*137*	*0*	*0*	*0*
Peak-hour factor, PHF	*0.86*	*1.00*	*0.88*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate (veh/h)	*156*	*0*	*155*	*0*	*0*	*0*
Proportion of heavy vehicles, P_{HV}	*0*	*0*	*0*	*0*	*0*	*0*
Percent grade (%)	*4*			*0*		

Flared approach		N			N	
Storage		0			0	
RT Channelized?			0			0
Lanes	0	1	0	0	0	0
Configuration		LTR				

Control Delay, Queue Length, Level of Service

Approach	EB	WB	Northbound			Southbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LT		LTR				
Volume, v (vph)		89		311				
Capacity, c_m (vph)		1358		690				
v/c ratio		0.07		0.45				
Queue length (95%)		0.21		2.35				
Control Delay (s/veh)		7.8		14.4				
LOS		A		B				
Approach delay (s/veh)	--	--	14.4					
Approach LOS	--	--	B					

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PLANNING ANALYSIS

Analyst:	José L. Pérez Berenguer
Intersection:	1
Agency/Co.:	Pérez Berenguer y Asociados
Area Type:	CBD or Similar
Date Performed:	11/26/2004
Jurisdiction:	Mayagüez
Analysis Time Period:	2:45 - 3:45 PM
Analysis Year:	2026

Project ID: Estudio de Impacto en el Tránsito Urb. Monte Sierra

East/West Street	North/South Street
PR-349	Liceo (PR-349)/Manantiales

VOLUME DATA

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
Num. Lanes	0	0	0	1	0	1	0	1	1	0	1	0
Volume				796		99	0	61	621	560	57	0
Parking					N			N			N	
Coord.					N			N			N	
LT Treat.				?			?			?		

Peak hour factor: 0.85 Area Type: CBD or Similar

LANE VOLUME WORKSHEET

	EAST BOUND	WEST BOUND	NORTH BOUND	SOUTH BOUND
LEFT TURN MOVEMENT				
1. LT volume		796	0	560
2. Opposing mainline volume		0	57	682
3. Number of exclusive LT lanes		1	0	0
Cross Product [2] * [1]		0	0	381920
Left Lane Configuration (E=Excl, S=Shrd):		E	S	S
Left Turn Treatment Type:		P	U	P
4. LT adjustment factor		0.950		
5. LT lane vol		838	0	560
RIGHT TURN MOVEMENT				
Right Lane Configuration (E=Excl, S=Shrd)		E	E	S
6. RT volume		99	621	0
7. Exclusive lanes		1	1	0
8. RT adjustment factor		0.850	0.850	0.850
9. Exclusive RT lane volume		116	731	
10. Shared lane vol				0
THROUGH MOVEMENT				
11. Thru volume		0	61	57
12. Parking adjustment factor		1.00	1.00	1.00

13. No. of thru lanes including shared	0	1	1
14. Total approach volume	0	61	57
15. Prop. of left turns in lane group	0.00	0.00	0.91
16. Left turn equivalence		1.46	
17. LT adj. factor:		1.000	1.000
18. Through lane volume	0	61	57
19. Critical lane volume	116	731	57

Left Turn Check (if [16] > 3.5)

20. Permitted left turn sneaker capacity: 60
7200/Cmax

SIGNAL OPERATIONS WORKSHEET

Phase Plan Selection from Lane Volume Worksheet		EAST BOUND	WEST BOUND	NORTH BOUND	SOUTH BOUND
Critical through-RT vol: [19]			116	731	57
LT lane vol: [5]			838	0	560
Left turn protection: (P/U/N)			P	U	P
Dominant left turn: (Indicate by '<')					
Selection Criteria based on the	Plan 1:	U	U	U	U
specified left turn protection	Plan 2a:	U	P	U	P
	Plan 2b:	P	U	P	U
< Indicates the dominant left turn	Plan 3a:	<P	P	<P	P
for each opposing pair	Plan 3b:	P	<P	P	<P
	Plan 4:	N	N	N	N
Phase plan selected (1 to 4)			4		2a

Min. cycle (Cmin) 60 Max. cycle (Cmax) 120

Timing Plan	Value	EAST-WEST Ph 1	EAST-WEST Ph 2	EAST-WEST Ph 3	NORTH-SOUTH Ph 1	NORTH-SOUTH Ph 2	NORTH-SOUTH Ph 3
Movement codes		ETL	WTL		STL	NST	
Critical phase vol [CV]		0	838	0	560	731	0
Critical sum [CS]	2129						
CBD adjustment [CBD]	0.900						
Reference sum [RS]	1308						
Lost time/phase [PL]		4	4	0	4	4	0
Lost time/cycle [TL]	16						
Cycle length [CYC]	120.0						
Phase time		0.0	44.9	0.0	31.4	39.7	0.0
Critical v/c Ratio [Xcm]	1.88						
Status	Over capacity						

HCS2000™™ DETAILED REPORT

General Information		Site Information	
Analyst	José L. Pérez Berenguer	Intersection	1
Agency or Co.	Pérez Berenguer y Asociados	Area Type	CBD or Similar
Date Performed	11/26/2004	Jurisdiction	Mayagüez
Time Period	2:45 - 3:45 PM	Analysis Year	2026
		Project ID	Estudio de Impacto en el Tránsito Urb. Monte Sierra

Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of lanes, N_1	0	0	0	2	0	0	0	1	1	0	1	0
Lane group				L	LR			TR	R		LT	
Volume, V (vph)				796		99		61	621	560	57	
% Heavy vehicles, %HV				7		7		5	5	1	1	
Peak-hour factor, PHF				0.93		0.58		0.66	0.88	0.67	0.66	
Pretimed (P) or actuated (A)				A		A		A	A	A	A	
Start-up lost time, l_1				2.0	2.0			2.0	2.0		2.0	
Extension of effective green, e				2.0	2.0			2.0	2.0		2.0	
Arrival type, AT				3	3			3	3		3	
Unit extension, UE				3.0	3.0			3.0	3.0		3.0	
Filtering/metering, I				1.000	1.000			1.000	1.000		1.000	
Initial unmet demand, Q_b				0.0	0.0			0.0	0.0		0.0	
Ped / Bike / RTOR volumes	0			0		50	0		50			
Lane width				3.7	3.7			3.7	3.7		4.7	
Parking / Grade / Parking	N		N	N	-3	N	N	5	N	N	5	N
Parking maneuvers, N_m												
Buses stopping, N_B				0	0			0	0		0	
Min. time for pedestrians, G_p	3.2			3.2			3.2					

Phasing	01	WB Only	03	04	NS Perm	06	07	08
Timing	G =	G = *30.0*	G =	G =	G = *73.5*	G =	G =	G =
	Y =	Y = *3*	Y =	Y =	Y = *3*	Y =	Y =	Y =
Duration of Analysis, T = *0.25*						Cycle Length, C = *109.5*		

Lane Group Capacity, Control Delay, and LOS Determination

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Adjusted flow rate, v				*856*	*84*			*92*	*649*		*922*	
Lane group capacity, c				*829*	*382*			*1078*	*916*		*843*	
v/c ratio, X				*1.03*	*0.22*			*0.09*	*0.71*		*1.09*	
Total green ratio, g/C				*0.27*	*0.27*			*0.67*	*0.67*		*0.67*	
Uniform delay, d_1				*39.8*	*30.7*			*6.3*	*11.3*		*18.0*	
Progression factor, PF				*1.000*	*1.000*			*1.000*	*1.000*		*1.000*	
Delay calibration, k				*0.50*	*0.11*			*0.11*	*0.27*		*0.50*	
Incremental delay, d_2				*39.9*	*0.3*			*0.0*	*2.6*		*59.8*	
Initial queue delay, d_3												
Control delay				*79.7*	*31.0*			*6.3*	*13.8*		*77.8*	
Lane group LOS				*E*	*C*			*A*	*B*		*E*	
Approach delay				*75.3*			*12.9*			*77.8*		
Approach LOS				*E*			*B*			*E*		
Intersection delay	*58.4*			X_c = *1.08*			Intersection LOS			*E*		

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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	*Mayagüez*
Date Performed	*11/26/2004*	Analysis Year	*2026*
Analysis Time Period	*2:45 - 3:45 PM*		
Project Description *Estudio de Impacto en el Tránsito*			
East/West Street: *PR-349*		North/South Street: *Calle Núm. 1*	
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)	*0*	*139*	*214*	*165*	*107*	*0*
Peak-hour factor, PHF	*1.00*	*0.73*	*0.73*	*0.71*	*0.71*	*1.00*
Hourly Flow Rate (veh/h)	*0*	*190*	*293*	*232*	*150*	*0*
Proportion of heavy vehicles, P_{HV}	*0*	–	–	*0*	–	–
Median type	*Raised curb*					
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)	*121*	*0*	*158*	*0*	*0*	*0*
Peak-hour factor, PHF	*0.71*	*1.00*	*0.73*	*1.00*	*1.00*	*1.00*
Hourly Flow Rate (veh/h)	*170*	*0*	*216*	*0*	*0*	*0*
Proportion of heavy vehicles, P_{HV}	*0*	*0*	*0*	*0*	*0*	*0*
Percent grade (%)	*4*			*0*		

Flared approach		N			N	
Storage		0			0	
RT Channelized?			0			0
Lanes	0	1	0	0	0	0
Configuration		LTR				

Control Delay, Queue Length, Level of Service								
Approach	EB	WB	Northbound			Southbound		
Movement	1	4	7	8	9	10	11	12
Lane Configuration		LT		LTR				
Volume, v (vph)		232		386				
Capacity, c_m (vph)		1090		474				
v/c ratio		0.21		0.81				
Queue length (95%)		0.81		7.73				
Control Delay (s/veh)		9.2		38.1				
LOS		A		E				
Approach delay (s/veh)	--	--	38.1					
Approach LOS	--	--	E					

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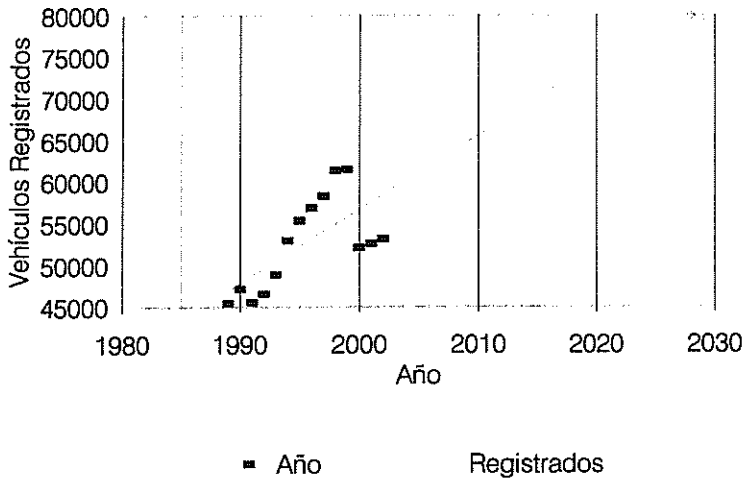
Apéndice 5. Determinación del Factor de Crecimiento del Tránsito Promedio Diario

Proyección de Vehículos Registrados, Mayagüez, Puerto Rico

Año	Vehículos Registrados	Vehículos Proyectados
1989	45439	46940
1990	47196	47828
1991	45586	48717
1992	46572	49605
1993	48872	50494
1994	52962	51382
1995	55357	52271
1996	56861	53159
1997	58330	54048
1998	61350	54936
1999	61509	55825
2000	52142	56713
2001	52631	57602
2002	53202	58490
2004		60267
2006		62044
2026		79814

Regression Output:	
Constant	-172027585711716
Std Err of Y Est	423637609772689
R Squared	454717797545048
No. of Observations	14
Degrees of Freedom	12
X Coefficient(s)	888494505481629
Std Err of Coef.	280869000328634



Ley de Interés Compuesto:

$$F = P(1+i)^n$$

$$\frac{F}{P} = (1+i)^n$$

$$1 + i = \sqrt[n]{\frac{F}{P}}$$

$$i = \sqrt[n]{\frac{F}{P}} - 1$$

$$i = \sqrt[37]{\frac{79814}{46940}} - 1$$

i = 1.01 - 1 = **1.4% anual**

Apéndice 6. Generación, Asignación y Proyección de Viajes

GENERACIÓN DE VIAJES

Con el propósito de determinar el flujo vehicular que generará la urbanización Monte Sierra se utilizó la referencia *Trip Generation* (**ITE, 2003a,b**). Esta referencia define una serie de usos de terreno e indica el flujo vehicular que se espera que cada uno genere, basado en estudios realizados previamente.

La urbanización Monte Sierra se compondrá de 302 unidades de vivienda de interés social ubicadas en solares individuales con cabida de 350 metros cuadrados, aproximadamente. También existirá un predio para uso comercial con cabida de 2150 metros cuadrados, aproximadamente. Para estimar el flujo que generará la parte residencial de este proyecto se utilizó el uso de terreno 210 ("*Single-Family Detached Housing*"), según definido por *Trip Generation* (**ITE, 2003a**). El estimado del volumen en las horas pico se realizó basado en el número de unidades residenciales propuestas.

Se utilizó el uso de terreno 820 ("*Shopping Center*") (**ITE, 2003b**) para estimar los viajes que generará la parte comercial del proyecto, basado en el área para alquiler, en pie cuadrado. Como no se tiene aun cuales van a ser las características del área comercial, se supuso que se utilizarán parámetros de un distrito de zonificación C-1 (Comercial Local), según definidos por la Junta de Planificación (**ELA, 2000**). Se usaron estos parámetros, aunque esta

no es la zonificación del predio debido a que un proyecto comercial en el lugar será básicamente para servir a los residentes del área. Ya que los usos permitidos en el distrito de zonificación C-1 se ajustan a este supuesto, se utilizaron sus parámetros para estimar cual podría ser la situación más crítica que se podría dar en el lugar y utilizar este escenario en el análisis. De acuerdo al Reglamento de Planificación Número 4, se puede desarrollar un proyecto comercial que tenga parámetros de diseño correspondientes a C-1 de manera que se provea un 210% del área del solar como área de piso. Por lo tanto, la situación más crítica que se podría dar en este solar sería que se desarrollara un proyecto comercial con un área para alquiler de 48000 pies cuadrados, escenario que se utilizó para el análisis.

AJUSTE POR VIAJES "PASS-BY"

Es reconocido que los viajes generados por centros comerciales y otros establecimientos (bancos, farmacias, restaurantes de comida rápida, etc.) poseen características diferentes a los viajes generados por otros tipos de facilidades. Una proporción significativa de los viajes generados por centros comerciales son simplemente viajes desviados del tránsito actual en el sistema de carreteras adyacente. Estos viajes se conocen como "pass-by".

Debido a que en el área existirá un centro comercial, los viajes generados por estos deben ser ajustados por viajes "pass-by". El *Trip Generation Handbook* (**ITE, 2001**) incluye estadísticas

para estimar los porcientos de viajes "pass-by" en función del área de alquiler comercial. De esta referencia, el porciento promedio de viajes "pass-by" para un área comercial de 48,000 pies cuadrados es 48.2%. Por lo tanto, se supuso que 48.2% de los viajes generados por el área comercial del desarrollo propuesto serán viajes que actualmente transitan por el lugar. Los resultados de la generación de viajes realizada, ajustada por viajes "pass-by" se resumen en la tabla A6-1.

Tabla A6-1. Flujo Vehicular que Generará la Urbanización Monte Sierra

Hora Pico	Uso de Terreno	Viajes Generados (veh/hr)		
		Entrando	Saliendo	Total
AM	210	55	166	221
	820	62	40	102
PM	210	190	107	297
	820	277	299	576
	820 (ajustado)	187	202	389

DISTRIBUCION DE VIAJES

Una vez determinado el flujo vehicular que generará el proyecto propuesto, se procedió a distribuir la nueva demanda de vehículos. Esta distribución se realizó basado en la proporción direccional obtenida en los conteos realizados recientemente en las cercanías del proyecto propuesto, durante un día laborable típico. En la figura 13 se ilustran los viajes que se generarán y como se distribuyeron por la red de carreteras analizada.

PROYECCION DEL TRANSITO

Para proyectar los conteos de flujo vehicular actuales hacia el futuro, se supuso que el volumen vehicular en el área de análisis crecerá a una razón similar que el número de vehículos registrados en el municipio de Mayagüez. Se obtuvieron datos históricos de vehículos registrados, recopilados por el Departamento de Transportación y Obras Públicas. Con estos datos se llevó a cabo una regresión lineal para determinar la curva que mejor se les ajusta. Los resultados se detallan en el Apéndice 5. Utilizando la fórmula de interés compuesto y el crecimiento anual obtenido, se proyectaron los datos actuales al año 2006. En las figuras 14 y 15 se ilustran los datos de flujo vehicular proyectados a dicho año. De forma similar se hizo la proyección de los datos de flujo vehicular durante las horas pico al año 2026 y los resultados aparecen en las figuras 16 y 17.

Apéndice 7. Justificaciones para la Evaluación de la Instalación de Sistemas de Semáforos (ATSSA/ITE/AASHTO, 2003)

CHAPTER 4C. TRAFFIC CONTROL SIGNAL NEEDS STUDIES

Section 4C.01 Studies and Factors for Justifying Traffic Control Signals

Standard:

An engineering study of traffic conditions, pedestrian characteristics, and physical characteristics of the location shall be performed to determine whether installation of a traffic control signal is justified at a particular location.

The investigation of the need for a traffic control signal shall include an analysis of the applicable factors contained in the following traffic signal warrants and other factors related to existing operation and safety at the study location:

Warrant 1, Eight-Hour Vehicular Volume.
Warrant 2, Four-Hour Vehicular Volume.
Warrant 3, Peak Hour.
Warrant 4, Pedestrian Volume.
Warrant 5, School Crossing.
Warrant 6, Coordinated Signal System.
Warrant 7, Crash Experience.
Warrant 8, Roadway Network.

The satisfaction of a traffic signal warrant or warrants shall not in itself require the installation of a traffic control signal.

Support:

Sections 8D.07 and 10D.05 contain information regarding the use of traffic control signals instead of gates and/or flashing light signals at highway-railroad grade crossings and highway-light rail transit grade crossings, respectively.

Guidance:

A traffic control signal should not be installed unless one or more of the factors described in this Chapter are met.

A traffic control signal should not be installed unless an engineering study indicates that installing a traffic control signal will improve the overall safety and/or operation of the intersection.

A traffic control signal should not be installed if it will seriously disrupt progressive traffic flow.

The study should consider the effects of the right-turn vehicles from the minor-street approaches. Engineering judgment should be used to determine what, if any, portion of the right-turn traffic is subtracted from the minor-street traffic count when evaluating the count against the above signal warrants.

Engineering judgment should also be used in applying various traffic signal warrants to cases where approaches consist of one lane plus one left-turn or right-turn lane. The site-specific traffic characteristics dictate whether an approach should be considered as one lane or two lanes. For example, for an approach with one lane for through and right-turning traffic plus a left-turn lane, engineering judgment could indicate that it should be considered a one-lane approach if the traffic using the left-turn lane is minor. In such a case, the total traffic volume approaching the intersection should be applied against the signal warrants as a one-lane approach. The approach should be considered two lanes if approximately half of the traffic on the approach turns left and the left-turn lane is of sufficient length to accommodate all left-turn vehicles.

Similar engineering judgment and rationale should be applied to a street approach with one lane plus a right-turn lane. In this case, the degree of conflict of minor-street right-turn traffic with traffic on the major street should be considered. Thus, right-turn traffic should not be included in the minor-street volume if the movement enters the major street with minimal conflict. The approach should be evaluated as a one-lane approach with only the traffic volume in the through/left-turn lane considered.

At a location that is under development or construction and where it is not possible to obtain a traffic count that would represent future traffic conditions, hourly volumes should be estimated as part of an engineering study for comparison with traffic signal warrants. Except for locations where the engineering study uses the satisfaction of Warrant 8 to justify a signal, a traffic control signal installed under projected conditions should have an engineering study done within 1 year of putting the signal into stop-and-go operation to determine if the signal is justified. If not justified, the signal should be taken out of stop-and-go operation or removed.

For signal warrant analysis, a location with a wide median, even if the median width is greater than 9 m (30 ft), should be considered as one intersection.

Option:

At an intersection with a high volume of left-turn traffic from the major street, the signal warrant analysis may be performed in a manner that considers the higher of the major-street left-turn volumes as the "minor-street" volume and the corresponding single direction of opposing traffic on the major street as the "major-street" volume.

For signal warrant analysis, bicyclists may be counted as either vehicles or pedestrians.

Support:

When performing a signal warrant analysis, bicyclists riding in the street with other vehicular traffic are usually counted as vehicles and bicyclists who are clearly using pedestrian facilities are usually counted as pedestrians.

Option:

Engineering study data may include the following:

A. The number of vehicles entering the intersection in each hour from each approach during 12 hours of an average day. It is desirable that the hours selected contain the greatest percentage of the 24-hour traffic volume.
B. Vehicular volumes for each traffic movement from each approach, classified by vehicle type (heavy trucks, passenger cars and light trucks, public-transit vehicles, and, in some locations, bicycles), during each 15-minute period of the 2 hours in the morning and 2 hours in the afternoon during which total traffic entering the intersection is greatest.
C. Pedestrian volume counts on each crosswalk during the same periods as the vehicular counts in Item B above and during hours of highest pedestrian volume. Where young, elderly, and/or persons with physical or visual disabilities need special consideration, the pedestrians and their crossing times may be classified by general observation.
D. Information about nearby facilities and activity centers that serve the young, elderly, and/or persons with disabilities, including requests from persons with disabilities for accessible crossing improvements at the location under study. These persons might not be adequately reflected in the pedestrian volume count if the absence of a signal restrains their mobility.
E. The posted or statutory speed limit or the 85th-percentile speed on the uncontrolled approaches to the location.
F. A condition diagram showing details of the physical layout, including such features as intersection geometrics, channelization, grades, sight-distance restrictions, transit stops and routes, parking conditions, pavement markings, roadway lighting, driveways, nearby railroad crossings, distance to nearest traffic control signals, utility poles and fixtures, and adjacent land use.
G. A collision diagram showing crash experience by type, location, direction of movement, severity, weather, time of day, date, and day of week for at least 1 year.

The following data, which are desirable for a more precise understanding of the operation of the intersection, may be obtained during the periods specified in Item B of the preceding paragraph:

A. Vehicle-hours of stopped time delay determined separately for each approach.
B. The number and distribution of acceptable gaps in vehicular traffic on the major street for entrance from the minor street.
C. The posted or statutory speed limit or the 85th-percentile speed on controlled approaches at a point near to the intersection but unaffected by the control.
D. Pedestrian delay time for at least two 30-minute peak pedestrian delay periods of an average weekday or like periods of a Saturday or Sunday.
E. Queue length on stop-controlled approaches.

Section 4C.02 Warrant 1, Eight-Hour Vehicular Volume

Support:

The Minimum Vehicular Volume, Condition A, is intended for application at locations where a large volume of intersecting traffic is the principal reason to consider installing a traffic control signal.

The Interruption of Continuous Traffic, Condition B, is intended for application at locations where Condition A is not satisfied and where the traffic volume on a major street is so heavy that traffic on a minor intersecting street suffers excessive delay or conflict in entering or crossing the major street.

It is intended that Warrant 1 be treated as a single warrant. If Condition A is satisfied, then the criteria for Warrant 1 is satisfied and Condition B and the combination of Conditions A and B are not needed. Similarly, if Condition B is satisfied, then the criteria for Warrant 1 is satisfied and the combination of Conditions A and B is not needed.

Table 4C-1. Warrant 1, Eight-Hour Vehicular Volume

Condition A—Minimum Vehicular Volume									
Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total of both approaches)				Vehicles per hour on higher-volume minor-street approach (one direction only)			
Major Street	Minor Street	100%[a]	80%[b]	70%[c]	56%[d]	100%[a]	80%[b]	70%[c]	56%[d]
1	1	500	400	350	280	150	120	105	84
2 or more	1	600	480	420	336	150	120	105	84
2 or more	2 or more	600	480	420	336	200	160	140	112
1	2 or more	500	400	350	280	200	160	140	112

Condition B—Interruption of Continuous Traffic									
Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total of both approaches)				Vehicles per hour on higher-volume minor-street approach (one direction only)			
Major Street	Minor Street	100%[a]	80%[b]	70%[c]	56%[d]	100%[a]	80%[b]	70%[c]	56%[d]
1	1	750	600	525	420	75	60	53	42
2 or more	1	900	720	630	504	75	60	53	42
2 or more	2 or more	900	720	630	504	100	80	70	56
1	2 or more	750	600	525	420	100	80	70	56

[a] Basic minimum hourly volume.
[b] Used for combination of Conditions A and B after adequate trial of other remedial measures.
[c] May be used when the major-street speed exceeds 70 km/h or exceeds 40 mph or in an isolated community with a population of less than 10,000.
[d] May be used for combination of Conditions A and B after adequate trial of other remedial measures when the major-street speed exceeds 70 km/h or exceeds 40 mph or in an isolated community with a population of less than 10,000.

Standard:

The need for a traffic control signal shall be considered if an engineering study finds that one of the following conditions exist for each of any 8 hours of an average day:

A. The vehicles per hour given in both of the 100 percent columns of Condition A in Table 4C-1 exist on the major-street and the higher-volume minor-street approaches, respectively, to the intersection; or

B. The vehicles per hour given in both of the 100 percent columns of Condition B in Table 4C-1 exist on the major-street and the higher-volume minor-street approaches, respectively, to the intersection.

In applying each condition the major-street and minor-street volumes shall be for the same 8 hours. On the minor street, the higher volume shall not be required to be on the same approach during each of these 8 hours.

Option:

If the posted or statutory speed limit or the 85th-percentile speed on the major street exceeds 70 km/h or exceeds 40 mph, or if the intersection lies within the built-up area of an isolated community having a population of less than 10,000, the traffic volumes in the 70 percent columns in Table 4C-1 may be used in place of the 100 percent columns.

Guidance:

The combination of Conditions A and B is intended for application at locations where Condition A is not satisfied and Condition B is not satisfied and should be applied only after an adequate trial of other alternatives that could cause less delay and inconvenience to traffic has failed to solve the traffic problems.

Standard:

The need for a traffic control signal shall be considered if an engineering study finds that both of the following conditions exist for each of any 8 hours of an average day:

A. The vehicles per hour given in both of the 80 percent columns of Condition A in Table 4C-1 exist on the major-street and the higher-volume minor-street approaches, respectively, to the intersection; and

B. The vehicles per hour given in both of the 80 percent columns of Condition B in Table 4C-1 exist on the major-street and the higher-volume minor-street approaches, respectively, to the intersection.

These major-street and minor-street volumes shall be for the same 8 hours for each condition; however, the 8 hours satisfied in Condition A shall not be required to be the same 8 hours satisfied in Condition B. On the minor street, the higher volume shall not be required to be on the same approach during each of the 8 hours.

Option:

If the posted or statutory speed limit or the 85th-percentile speed on the major street exceeds 70 km/h or exceeds 40 mph, or if the intersection lies within the built-up area of an isolated community having a population of less than 10,000, the traffic volumes in the 56 percent columns in Table 4C-1 may be used in place of the 80 percent columns.

Section 4C.03 Warrant 2, Four-Hour Vehicular Volume

Support:

The Four-Hour Vehicular Volume signal warrant conditions are intended to be applied where the volume of intersecting traffic is the principal reason to consider installing a traffic control signal.

Standard:

The need for a traffic control signal shall be considered if an engineering study finds that, for each of any 4 hours of an average day, the plotted points representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-volume minor-street approach (one direction only) all fall above the applicable curve in Figure 4C-1 for the existing combination of approach lanes. On the minor street, the higher volume shall not be required to be on the same approach during each of these 4 hours.

Option:

If the posted or statutory speed limit or the 85th-percentile speed on the major street exceeds 70 km/h or exceeds 40 mph or if the intersection lies within the built-up area of an isolated community having a population of less than 10,000, Figure 4C-2 may be used in place of Figure 4C-1.

Section 4C.04 Warrant 3, Peak Hour

Support:

The Peak Hour signal warrant is intended for use at a location where traffic conditions are such that for a minimum of 1 hour of an average day, the minor-street traffic suffers undue delay when entering or crossing the major street.

Standard:

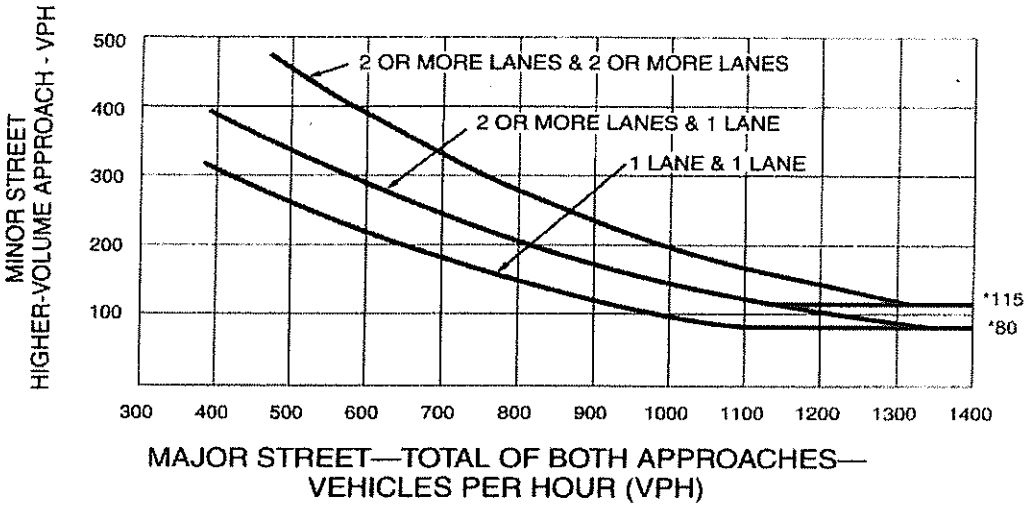
This signal warrant shall be applied only in unusual cases, such as office complexes, manufacturing plants, industrial complexes, or high-occupancy vehicle facilities that attract or discharge large numbers of vehicles over a short time.

The need for a traffic control signal shall be considered if an engineering study finds that the criteria in either of the following two categories are met:

A. If all three of the following conditions exist for the same 1 hour (any four consecutive 15-minute periods) of an average day:

1. The total stopped time delay experienced by the traffic on one minor-street approach (one direction only) controlled by a STOP sign equals or exceeds: 4 vehicle-hours for a one-lane approach; or 5 vehicle-hours for a two-lane approach, and

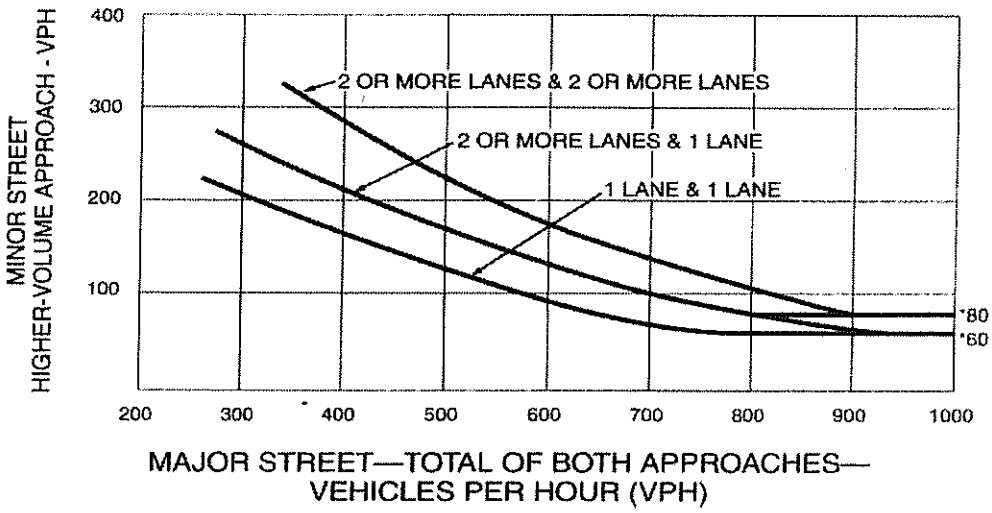
Figure 4C-1. Warrant 2, Four-Hour Vehicular Volume



*Note: 115 vph applies as the lower threshold volume for a minor-street approach with two or more lanes and 80 vph applies as the lower threshold volume for a minor-street approach with one lane.

Figure 4C-2. Warrant 2, Four-Hour Vehicular Volume (70% Factor)

(COMMUNITY LESS THAN 10,000 POPULATION OR ABOVE 70 km/h OR ABOVE 40 mph ON MAJOR STREET)



*Note: 80 vph applies as the lower threshold volume for a minor-street approach with two or more lanes and 60 vph applies as the lower threshold volume for a minor-street approach with one lane.

2. **The volume on the same minor-street approach (one direction only) equals or exceeds 100 vehicles per hour for one moving lane of traffic or 150 vehicles per hour for two moving lanes, and**
3. **The total entering volume serviced during the hour equals or exceeds 650 vehicles per hour for intersections with three approaches or 800 vehicles per hour for intersections with four or more approaches.**

B. **The plotted point representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-volume minor-street approach (one direction only) for 1 hour (any four consecutive 15-minute periods) of an average day falls above the applicable curve in Figure 4C-3 for the existing combination of approach lanes.**

Option:

If the posted or statutory speed limit or the 85th-percentile speed on the major street exceeds 70 km/h or exceeds 40 mph, or if the intersection lies within the built-up area of an isolated community having a population of less than 10,000, Figure 4C-4 may be used in place of Figure 4C-3 to satisfy the criteria in the second category of the Standard.

Section 4C.05 Warrant 4, Pedestrian Volume

Support:

The Pedestrian Volume signal warrant is intended for application where the traffic volume on a major street is so heavy that pedestrians experience excessive delay in crossing the major street.

Standard:

The need for a traffic control signal at an intersection or midblock crossing shall be considered if an engineering study finds that both of the following criteria are met:

A. **The pedestrian volume crossing the major street at an intersection or midblock location during an average day is 100 or more for each of any 4 hours or 190 or more during any 1 hour; and**
B. **There are fewer than 60 gaps per hour in the traffic stream of adequate length to allow pedestrians to cross during the same period when the pedestrian volume criterion is satisfied. Where there is a divided street having a median of sufficient width for pedestrians to wait, the requirement applies separately to each direction of vehicular traffic.**

The Pedestrian Volume signal warrant shall not be applied at locations where the distance to the nearest traffic control signal along the major street is less than 90 m (300 ft), unless the proposed traffic control signal will not restrict the progressive movement of traffic.

If this warrant is met and a traffic control signal is justified by an engineering study, the traffic control signal shall be equipped with pedestrian signal heads conforming to requirements set forth in Chapter 4E.

Guidance:

If this warrant is met and a traffic control signal is justified by an engineering study, then:

A. If at an intersection, the traffic control signal should be traffic-actuated and should include pedestrian detectors.
B. If at a nonintersection crossing, the traffic control signal should be pedestrian-actuated, parking and other sight obstructions should be prohibited for at least 30 m (100 ft) in advance of and at least 6.1 m (20 ft) beyond the crosswalk, and the installation should include suitable standard signs and pavement markings.
C. Furthermore, if installed within a signal system, the traffic control signal should be coordinated.

Option:

The criterion for the pedestrian volume crossing the major roadway may be reduced as much as 50 percent if the average crossing speed of pedestrians is less than 1.2 m/sec (4 ft/sec).

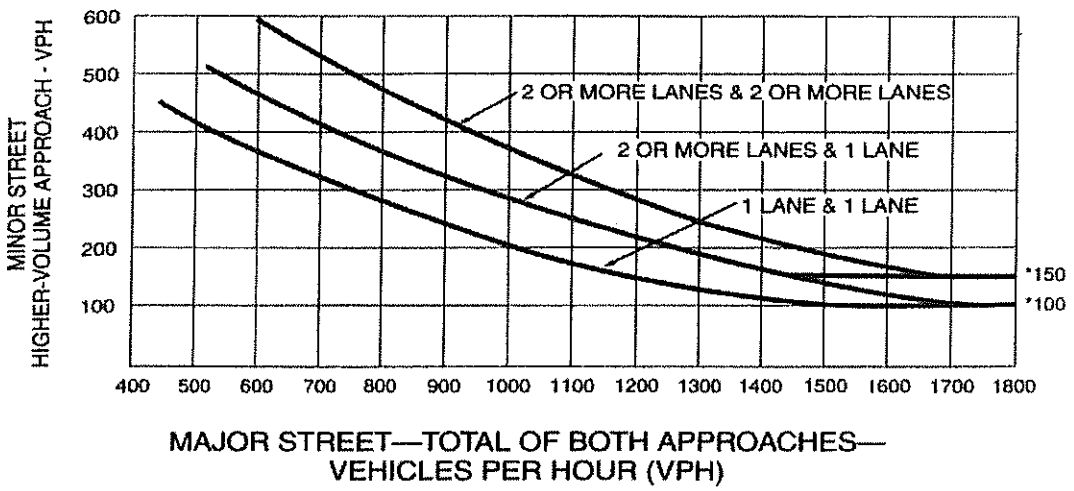
A traffic control signal may not be needed at the study location if adjacent coordinated traffic control signals consistently provide gaps of adequate length for pedestrians to cross the street, even if the rate of gap occurrence is less than one per minute.

Section 4C.06 Warrant 5, School Crossing

Support:

The School Crossing signal warrant is intended for application where the fact that school children cross the major street is the principal reason to consider installing a traffic control signal.

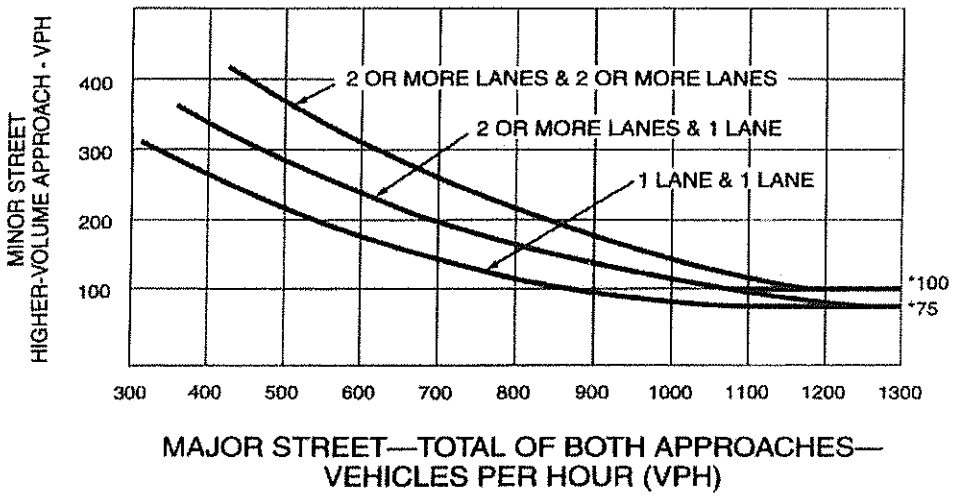
Figure 4C-3. Warrant 3, Peak Hour



*Note: 150 vph applies as the lower threshold volume for a minor-street approach with two or more lanes and 100 vph applies as the lower threshold volume for a minor-street approach with one lane.

Figure 4C-4. Warrant 3, Peak Hour (70% Factor)

(COMMUNITY LESS THAN 10,000 POPULATION OR ABOVE 70 km/h OR ABOVE 40 mph ON MAJOR STREET)



*Note: 100 vph applies as the lower threshold volume for a minor-street approach with two or more lanes and 75 vph applies as the lower threshold volume for a minor-street approach with one lane.

Standard:

The need for a traffic control signal shall be considered when an engineering study of the frequency and adequacy of gaps in the vehicular traffic stream as related to the number and size of groups of school children at an established school crossing across the major street shows that the number of adequate gaps in the traffic stream during the period when the children are using the crossing is less than the number of minutes in the same period (see Section 7A.03) and there are a minimum of 20 students during the highest crossing hour.

Before a decision is made to install a traffic control signal, consideration shall be given to the implementation of other remedial measures, such as warning signs and flashers, school speed zones, school crossing guards, or a grade-separated crossing.

The School Crossing signal warrant shall not be applied at locations where the distance to the nearest traffic control signal along the major street is less than 90 m (300 ft), unless the proposed traffic control signal will not restrict the progressive movement of traffic.

Guidance:

If this warrant is met and a traffic control signal is justified by an engineering study, then:

A. If at an intersection, the traffic control signal should be traffic-actuated and should include pedestrian detectors.
B. If at a nonintersection crossing, the traffic control signal should be pedestrian-actuated, parking and other sight obstructions should be prohibited for at least 30 m (100 ft) in advance of and at least 6.1 m (20 ft) beyond the crosswalk, and the installation should include suitable standard signs and pavement markings.
C. Furthermore, if installed within a signal system, the traffic control signal should be coordinated.

Section 4C.07 Warrant 6, Coordinated Signal System

Support:

Progressive movement in a coordinated signal system sometimes necessitates installing traffic control signals at intersections where they would not otherwise be needed in order to maintain proper platooning of vehicles.

Standard:

The need for a traffic control signal shall be considered if an engineering study finds that one of the following criteria is met:

A. On a one-way street or a street that has traffic predominantly in one direction, the adjacent traffic control signals are so far apart that they do not provide the necessary degree of vehicular platooning.
B. On a two-way street, adjacent traffic control signals do not provide the necessary degree of platooning and the proposed and adjacent traffic control signals will collectively provide a progressive operation.

Guidance:

The Coordinated Signal System signal warrant should not be applied where the resultant spacing of traffic control signals would be less than 300 m (1,000 ft).

Section 4C.08 Warrant 7, Crash Experience

Support:

The Crash Experience signal warrant conditions are intended for application where the severity and frequency of crashes are the principal reasons to consider installing a traffic control signal.

Standard:

The need for a traffic control signal shall be considered if an engineering study finds that all of the following criteria are met:

A. Adequate trial of alternatives with satisfactory observance and enforcement has failed to reduce the crash frequency; and
B. Five or more reported crashes, of types susceptible to correction by a traffic control signal, have occurred within a 12-month period, each crash involving personal injury or property damage apparently exceeding the applicable requirements for a reportable crash; and
C. For each of any 8 hours of an average day, the vehicles per hour (vph) given in both of the 80 percent columns of Condition A in Table 4C-1 (see Section 4C.02), or the vph in both of the 80 percent columns of Condition B in Table 4C-1 exists on the major-street and the higher-volume minor-street approach, respectively, to the intersection, or the volume of pedestrian traffic is not

less than 80 percent of the requirements specified in the Pedestrian Volume warrant. These major-street and minor-street volumes shall be for the same 8 hours. On the minor street, the higher volume shall not be required to be on the same approach during each of the 8 hours.

Option:

If the posted or statutory speed limit or the 85th-percentile speed on the major street exceeds 70 km/h or exceeds 40 mph, or if the intersection lies within the built-up area of an isolated community having a population of less than 10,000, the traffic volumes in the 56 percent columns in Table 4C-1 may be used in place of the 80 percent columns.

Section 4C.09 Warrant 8, Roadway Network

Support:

Installing a traffic control signal at some intersections might be justified to encourage concentration and organization of traffic flow on a roadway network.

Standard:

The need for a traffic control signal shall be considered if an engineering study finds that the common intersection of two or more major routes meets one or both of the following criteria:

A. The intersection has a total existing, or immediately projected, entering volume of at least 1,000 vehicles per hour during the peak hour of a typical weekday and has 5-year projected traffic volumes, based on an engineering study, that meet one or more of Warrants 1, 2, and 3 during an average weekday; or

B. The intersection has a total existing or immediately projected entering volume of at least 1,000 vehicles per hour for each of any 5 hours of a nonnormal business day (Saturday or Sunday).

A major route as used in this signal warrant shall have one or more of the following characteristics:

A. It is part of the street or highway system that serves as the principal roadway network for through traffic flow; or

B. It includes rural or suburban highways outside, entering, or traversing a City; or

C. It appears as a major route on an official plan, such as a major street plan in an urban area traffic and transportation study.

Apéndice 8. Curriculum Vitæ del consultor

José Luis Pérez Berenguer, M.S.C.E., P.E., P.T.O.E.
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Mayagüez, Puerto Rico 00681-6512
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PBAsoc@centennialpr.net

CURRICULUM VITÆ

EDUCATION:

JANUARY 1992 TO MAY 1994
Master of Science in Civil Engineering, Pavement Design and Transportation. University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

AUGUST 1986 TO DECEMBER 1991
Bachelor of Science in Civil Engineering. University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

PROFESSIONAL LICENSES AND CERTIFICATIONS:

Professional Engineer - License 12673 P.E.

Professional Transportation Operations Engineer - Certificate Number 840

Certified Worksite Traffic Supervisor - Certification 54159

RESEARCH PROJECTS:

JULY 1998 TO FEBRUARY 1999
Análisis del Diseño de los Accesos, Facilidades de Transferencia Intermodal, Estacionamientos y "Kiss and Ride" en las Estaciones del Tren Urbano. Research supervisor of the undergraduate student Joel A. Martínez Vega for the UPR/MIT Tren Urbano Internship Program - coordinated by the Civil Infrastructure Research Center. Poster and presentation

JULY 1997 TO MAY 1998
Analisis de los Planes de Mantenimiento de Tránsito a Implementarse Durante la Construcción del Tren Urbano. Research supervisor of the undergraduate student Jorge L. Ramos Ortiz for the UPR/MIT Tren Urbano Internship Program - coordinated by the Civil Infrastructure Research Center. Written report, poster and presentation

SEPTEMBER 1996 TO MAY 1998
El Impacto Futuro en el Patrón de Flujo Vehicular en las Intersecciones Adyacentes al Corredor del Tren Urbano. Research supervisor of the undergraduate student Diego Torres for the UPR/MIT Tren Urbano Internship Program - coordinated by the Civil Infrastructure Research Center. Written report, poster and presentation

MARCH 1996 TO FEBRUARY 1998
Measurement and Analysis of Pavement Condition and Improvement of Existing Roughness Mathematical Models Applied to the Puerto Rico Highway Network. Principal Investigator, supported by the Puerto Rico Department of Transportation and Public Works - coordinated by the Civil Infrastructure Research Center. Written report poster and presentation

JUNE 1996 TO AUGUST 1996
Medidas y Análisis de Rugosidad en el Pavimento en la Red de Carreteras de Puerto Rico. Research supervisor of the undergraduate student Jorge L. Ramos Ortiz, sponsored by the Puerto Rico Alliance for Minority Participation. Written report, poster and presentation

AUGUST 1995 TO MAY 1996
Análisis Operacional de un Sistema de Transportación sobre Rieles; Plan de Servicio Propuesto para el Tren Urbano. Research supervisor of the graduate student Ginger M. Rossy Robles for the UPR/MIT Tren Urbano Internship Program - coordinated by the Civil Infrastructure Research Center. Written report and presentation

MAY 1992 TO MAY 1994
Develop of the thesis ***Análisis de datos de Rugosidad y Desarrollo de Modelos para Estimar la Tendencia del Deterioro de Pavimentos.*** Requirement to obtain a Master Degree in Pavement Design and Transportation Engineering Area. University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico. Written report and oral exam

PAPERS:

SEPTEMBER 1996
Replicabilidad de Medidas de Rugosidad del Pavimento Utilizando el equipo ARAN. Published. IX Congreso Panamericano de Ingeniería de Tránsito y Transportes, La Habana, Cuba

JULY 1996
Analysis of Roughness Measurements for Delineating Homogeneous Units for Pavement Management Application. Co-author, submitted for consideration to the Transportation Research Board 76th Annual Meeting

JUNE 1994
Un Nuevo Enfoque en el Análisis de Datos de Rugosidad para un Sistema de Gerencia de Pavimentos. Published. VIII Congreso Panamericano de Ingenería de Tránsito y Transportes, Ciudad de Mexico, Mexico

PRESENTATIONS:

DECEMBER 2, 1996 TO DECEMBER 6, 1996
Replicabilidad de Medidas de Rugosidad del Pavimento Utilizando el equipo ARAN. IX Congreso Panamericano de Ingeniería de Tránsito y Transportes, Capitolio Nacional, La Habana, Cuba

OCTOBER 24, 1996
Measurement and Analysis of Pavement Condition and Improvement of Existing Roughness Mathematical Models Applied to the Puerto Rico Highway Network. Civil Engineering Department, University of Puerto Rico - Mayagüez Campus. Sponsored by the Civil Infrastructure Research Center

APRIL 1994
Análisis de datos de Rugosidad y Desarrollo de Modelos para Estimar la Tendencia del Deterioro de Pavimentos. 29th ACS Junior Technical Meeting, University of Puerto Rico - Mayagüez Campus. Sponsored by Puerto Rico Alliance for Minority Participation.

WORK EXPERIENCE:

JULY 1994 TO PRESENT
Pérez Berenguer y Asociados; Principal, Transportation and Civil Engineer

AUGUST 1994 TO DECEMBER 1998
University of Puerto Rico - Mayagüez Campus, Civil Engineering Department; Professor of the courses:

- Highway Location and Curve Design
- Highway Engineering
- Undergraduate Research
- Seminar of Civil Engineering

OCTOBER 1992
Traffic Study in Lomas Verdes Ave. and Ramírez de Arellano Ave. at Guaynabo, Puerto Rico. Dr. Sergio L. González Quevedo - Consultant; Technical support

JANUARY 1992 TO MAY 1992
University of Puerto Rico - Mayagüez Campus; Teacher Assistant of Dr. José F. Lluch García, Construction Management Course

MARCH 1991 TO JANUARY 1992
Office for the Improvement of the Public Schools, Department of Education, Commonwealth of Puerto Rico; Engineer's Assistant, Field Inspector

DECEMBER 1990 TO JANUARY 1991
Office for the Improvement of the Public Schools, Department of Education, Commonwealth of Puerto Rico; Engineer's Assistant, Field Inspector

AUGUST 1990 TO DECEMBER 1990
University of Puerto Rico - Mayagüez Campus; Assistant of Dr. Milton R. Martínez Delgado (R.I.P.) in the development of Structural Analysis Computer Programs

GRADUATE COURSES TAKEN:

Analysis and Design of Public Transportation Systems
Urban Transportation Planning
Traffic Engineering I
Pavement Design
Advanced Engineering Economics
Bituminous Mix Design and Construction Techniques I
Bituminous Mix Design and Construction Techniques II
Pavement Management
Applied Soil Mechanics
Operations Research
Geosynthetics in Civil Engineering
Design of Intelligent Traffic-Control Systems

CONTINUING EDUCATION COURSES TAKEN

AUGUST 2004 TO PRESENT
Theory of Map Projection - Evi de la Rosa Ricciardi, M.S.C.E., University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

SEPTEMBER 24, 2004
The Process of Forced Expropriation: Important Aspects for Engineers and Land Surveyors - Johnny Colón Díaz, P.E., Polytechnic University of Puerto Rico, San Juan, Puerto Rico

JANUARY 2004 TO MAY 2003
Physical Geodesy - Julio C. Ríos, M.S.L.S., University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

AUGUST 2003 TO DECEMBER 2003
Adjustment Computations I - Julio C. Ríos, M.S.L.S., University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

JANUARY 2003 TO MAY 2003
Geodetic Astronomy - Julio C. Ríos, M.S.L.S., University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

AUGUST 2002 TO DECEMBER 2002
Special Surveys - José L. Flores Malavé, M.S.L.S., University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

JANUARY 2002 TO MAY 2002
Topographic Drawing - José L. Flores Malavé, M.S.L.S., University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

AUGUST 2001 TO DECEMBER 2001
Astronomy I - Leszekan Nowakowski, Ph.D., University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

OCTOBER 13, 2001
Professional Traffic Operations Engineer Certification Exam Refresher Course - Benjamín Colucci Ríos, Ph.D., Institute of Engineers and Surveyors of Puerto Rico, San Juan, Puerto Rico

JULY 31, 2001
Addressing Lead-Based Paint Hazards During Renovation, Remodeling and Rehabilitation in Federally Owned and Assisted Housing -Benjamín Cintrón, Quantech - Fort Buchanan, Guaynabo, Puerto Rico. Sponsored by HUD

JANUARY 2001 TO MAY 2001
Photogrammetry I - José L. Flores Malavé, M.S.L.S., University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

AUGUST 2000 TO DECEMBER 2000
Elements of Optics and Photography in Surveying and Photogrammetry - Linda L. Vélez Rodríguez, B.S.C.E., M.S.L.S., University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

JANUARY 2000 TO MAY 2000
Geodesy I - José L. Flores Malavé, M.S.L.S., University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

JULY 14-16, 1998
Worksite Traffic Supervisor Training Course - James A. Brandon, Jr., P.E., American Traffic Safety Services Association. PGA Mariott Hotel, Palm Beach Gardens, Florida

JULY 1998
Public Transportation in Boston - Nigel Wilson, Ph.D. and Kenneth Kruckemeyer, Ph.D., MIT. Massachussetts Institute of Technology, Cambridge, Massachussetts

JULY 1997
Reglamentación y Procedimientos para el Desarrollo de Terreno y Permisos de Construcción en Puerto Rico - José F. Lluch García, Ph.D., P.E., Institute of Engineers and Surveyors of Puerto Rico (CIAPR). University of Puerto Rico-Mayagüez Campus, Mayagüez, Puerto Rico

JULY 1997
Public Transportation in Boston - Nigel Wilson, Ph.D. and Kenneth Kruckemeyer, Ph.D., MIT. Massachussetts Institute of Technology, Cambridge, Massachussetts

FEBRUARY 8, 1996
Standard Drawings of the Puerto Rico Highway and Transportation Authority - Isabel Cristina Victoria Jaramillo, M.S.C.E., PRHTA - Institute of Engineers and Surveyors of Puerto Rico (CIAPR), Hato Rey, Puerto Rico

SEPTEMBER 1995 TO DECEMBER 1995
Autocad I - Surveyor José Vigo, University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

JULY 1995
Public Transportation in Boston - Nigel Wilson, Ph.D. and Kenneth Kruckemeyer, Ph.D., MIT. Massachussetts Institute of Technology, Cambridge, Massachussetts

MEMBERSHIP IN PROFESSIONAL AND COMMUNITY ORGANIZATIONS:

Institute of Engineers and Surveyors of Puerto Rico (CIAPR) - License 12673 P.E.

Institute of Civil Engineers of Puerto Rico

Member - Institute of Transportation Engineers

Associate Member - American Society of Civil Engineers

Miembro - Centro Cultural Salvador Brau, Cabo Rojo

NOVEMBER 1998 TO NOVEMBER 2000
President, Asociación de Residentes de Urbanización y Extensión La Concepción, Inc., Cabo Rojo, Puerto Rico

NOVEMBER 1994 TO DECEMBER 1998
Tren Urbano Internship Program - coordinated by the Civil Infrastructure Research Center, University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

1995 TO 1998
Member of the University of Puerto Rico evaluating committee for grant the Dwight D. Eisenhower Scholarship for Transportation Engineering studies

NOVEMBER 1994 TO OCTOBER 1997
Faculty Advisor - Institute of Transportation Engineers, Student Chapter, University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

NOVEMBER 1995 TO AUGUST 1996
Member of the Academic Affairs Committee of the Civil Engineering Department, University of Puerto Rico - Mayagüez Campus, Mayagüez, Puerto Rico

WORK DONE BY THE OFFICE IN RELATED FIELDS:

MAY 2003 TO PRESENT
Maintenance of Traffic Plans for projects to be constructed in 27 municipalities as part of the Urban Cores Improvements Program, for the Directorate of Urbanism, Department of Transportation and Public Works

DECEMBER 2004
Traffic Study to analyze the impact of Monte Sierra housing development in the adjacent area, to be located at the State Highway PR-349, Km. 1.1, Quebrada Grande ward, Mayagüez, Puerto Rico, for Urbanización Monte Sierra, Inc.

OCTOBER 2004
Traffic Study (written report and comparison to public hearings) to analyze the impact of Frailes Centro shopping center in the adjacent area, to be located at the State Highway PR-873, Km. 0.8, Tortugo ward, San Juan, Puerto Rico, for Best Properties

OCTOBER 2004
Evaluation of the Traffic Impact Study (written report and comparison to public hearings) submitted by the proponents of the Expansion of the Actual Facilities of SER de Puerto Rico, located at 500 Báez street, Pérez Morris Housing Development, Universidad ward, San Juan, Puerto Rico, for Asociación de Residentes y Propietarios de la Urbanización Pérez Morris, Inc.

MAY 2003 TO OCTOBER 2004
Maintenance of Traffic Plan for the Reconstruction of Intersection number 5 (Junction of the State Highways PR-1, PR-25, PR-26 and PR-35), Santurce ward, San Juan, Puerto Rico for Orval E. Sifontes, A.I.A.

OCTOBER 2 AND 10, 2004
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

AUGUST 2004
Evaluation of the impact in the traffic of the adjacent area of Caribbean Medical Testing Center, located at 300 Clemson street, University Gardens Housing Development, Hato Rey Sur ward, San Juan, Puerto Rico, for Farmacias El Amal

AUGUST 2004
Evaluation of the impact in the traffic of the adjacent area of Terrazas del Turabo Housing Development, to be located at Reina de las Flores road, Rincón ward, Gurabo, Puerto Rico, for Terrazas del Turabo

MAY 2004
Traffic Count to determine the Average Daily Traffic at McKinley and Méndez Vigo streets, immediately at west of State Highway PR-2, Marina Meridional ward, Mayagüez, Puerto Rico, for Mr. Eduardo Ruiz Valentín

MAY 2004
Traffic Count to determine the Average Daily Traffic at the intersection of State Highway PR-2 and Carlos Chardón street, Río ward, Mayagüez, Puerto Rico, for Mr. Eduardo Ruiz Valentín

MAY 2004
Consultant hired by the opposite part in the procedures for a Necessity and Convenience Certificate for a Walgreens Drug Store proposed at Boulevard street, Pueblo ward, Bayamón, Puerto Rico, for Farmacias El Amal

MAY 2004
Traffic Study to analyze the impact of Laurel Sur Housing Development in the adjacent area, to be located at the State Highway PR-506, Km. 1.7, Coto Laurel ward, Ponce, Puerto Rico, for D.I.S., Inc.

MARCH 2004
Feasibility Study to Evaluate the Public Transportation System of the Municipality of Dorado, Puerto Rico - Project FTA - P.R. 80-X011 - Task 22-01, for the Municipality of Dorado

MARCH 6 AND 20, 2004
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

DECEMBER 2003
Traffic Study to analyze the impact of Plaza La Cumbre project in the adjacent area, to be located at the State Highway PR-199, Km. 4.0, Cupey ward, San Juan, Puerto Rico, for Álvarez-Díaz Group, P.S.C.

OCTOBER 2003
Report with the results of an evaluation of the future street network of Multeado Estrella urban development, Playa ward, Ponce, Puerto Rico, for Administración de Terrenos

SEPTEMBER 2003
Expert witness report of an accident occurred on September 27, 1997 in 435 Street, Villa Carolina Housing Development, Carolina, Puerto Rico, for Mrs. Sonia Carballo Fernandez

SEPTEMBER 13 AND 27, 2003
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

MARCH 29 AND APRIL 5, 2003
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

MARCH 2003
Expert witness report analyzing the Maintenance of Traffic Plan approved for the projects AC-014206 and AC-14207 (State Highway PR-142 Quebrada Arenas ward, Toa Alta, Puerto Rico), for American International Insurance Company

OCTOBER 5 AND 19, 2002
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

SEPTEMBER 2002
Composed Pavement Design for a frontage road of the State Highway PR-199, Río Piedras, Puerto Rico, for A.S. Consulting Engineers

SEPTEMBER 2002
Traffic Study to analyze the impact of the operation of Antonio S. Paoli Fine Arts Center in the adjacent area, to be located at Los Caobos Avenue, Bucaná Ward, Ponce, Puerto Rico, for Urban Venture Group, P.S.C.

JUNE 2002
Traffic Study to analyze the impact of a proposed housing development in the State Highway PR-4466, Km. 0.8, Bajura Ward, Isabela, Puerto Rico, for Mr. Hauger Martín Hau

APRIL 2002
Traffic Study to analyze the impact of the expansion of the Panamerican Terminal in the adjacent area, located at Lindbergh Street interior, Isla Grande sector, Santurce Ward, San Juan, Puerto Rico, for Royal Caribbean and RAY Architects Engineers

APRIL 6 AND 13, 2002
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

MARCH 2002
Traffic Study to determine the Average Daily Traffic at the State Highway PR-54, Km. 3.0, Guayama, Puerto Rico, for Mr. Oscar Rodríguez Crespo

JANUARY 2002
Traffic Study to analyze the impact of the expansion of the Winnie's Active Learning Kids School in the adjacent area (including depositions at public audiences), located in the State Highway PR-104, Km. 1.9, Algarrobos Ward, Mayagüez, Puerto Rico, for WALKS

NOVEMBER 2001
Traffic Study to analyze the impact of Apartamentos Marazul housing and commercial development in the State Highway PR-187, Km. 9.7, Medianía Baja Ward, Loíza, Puerto Rico, for Ossam Construction

NOVEMBER 2001
Traffic Study to analyze the impact of Paraíso de Mayagüez housing development in the adjacent roads, Sábalos ward, Mayagüez, Puerto Rico, for Pulte Homes Caribbean

SEPTEMBER 29 AND OCTOBER 6, 2001
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

FEBRUARY 2001 TO JULY 2001
Inspector of the Project "Widening of the Bridge over Maricao River", located at the State Highway PR-120, Km. 11.30, Maricao, Puerto Rico, for Ingeniar Engineering Solutions, P.S.C.

JULY 2001
Traffic Study to analyze the impact of Garden State Commercial Center project in the State Highway PR-100, Kms. 3.3 to 4.6, Guanajibo and Miradero wards, Cabo Rojo, Puerto Rico, for Mr. Guisseppe Marino Damiani

JUNE 2001
Traffic Study to determine the Average Daily Traffic at the State Highway PR-980, Km. 1.1, Florida ward, San Lorenzo, Puerto Rico, for Mr. Rafael Martínez Santana

MARCH 28 AND APRIL 2, 2001
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

FEBRUARY 3, 2001
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Interamerican University of Puerto Rico at Arecibo

JANUARY 2001
Traffic Study to determine the Average Daily Traffic at the junction of the State Highways PR-3 and PR-901, Juan Martín ward, Yabucoa, Puerto Rico, for Farmacias El Amal

DECEMBER 2000
Traffic Study to determine the Average Daily Traffic at the junction of the State Highways PR-14 and PR-721 (including depositions at public audiences), Pueblo Norte and Pueblo Sur wards, Aibonito, Puerto Rico, for Farmacias El Amal

OCTOBER 2000
Traffic Study to determine the Average Daily Traffic at the State Highway PR-1, Km. 88.1, Lapa ward, Salinas, Puerto Rico, for Mr. Bernardo Olmeda Morales

SEPTEMBER 30, 2000
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

SEPTEMBER 2000
Traffic Study to analyze the impact of Haciendas El Monte project in the State Highway PR-2, Kms. 173.2 to 174.7, Cain Alto ward, San Germán, Puerto Rico, for Paseos de San Antonio Development Corporation

AUGUST 2000
Expert witness report with the results of a traffic study to analyze the impact of Percon Industrial park at the junctions of the State Highway PR-14 with the future Caribe Principal Street, State Highway PR-52, Puerto Viejo road and the future Contreras Avenue, Playa ward, Ponce, Puerto Rico, for Percon Development Corporation

MAY 2000
Traffic Count to determine the Average Daily Traffic at Ave. Del Pescador, La Parguera ward, Lajas, Puerto Rico, for Carlyle Benavent, M.D.

APRIL 1st, 2000
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

MARCH 2000
Expert witness report based on the results of an evaluation about the possible access for the project Percon Industrial Park, Playa ward, Ponce, Puerto Rico, for Percon Development Corporation

FEBRUARY 2000
Traffic Study to determine the Average Daily Traffic at the State Highway PR-348, Km. 8.8, Rosario Bajo ward, San Germán, Puerto Rico, for Félix J. Rodríguez Arroyo

DECEMBER 1999
Traffic Study to determine the impact of an extension of the PRTC facilities at the junctions of the State Highway PR-5 with the access of the Puerto Nuevo Distribution Center and the 11th street of Las Vegas Housing Development, Palmas ward, Cataño, Puerto Rico, for Puerto Rico Telephone Company

NOVEMBER 1999
Traffic Study to determine the Average Daily Traffic at the junction of the State Highways PR-921 and PR-9922, Collores ward, Las Piedras, Puerto Rico, for Mr. Carlos Beltrán Rodríguez

OCTOBER 16, 1999
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

OCTOBER 1999
Traffic Study to determine the Average Daily Traffic at the junction of the State Highways PR-189, PR-9919 and Hormazábal street, Pueblo Norte ward, Juncos, Puerto Rico, for Mr. Carlos L. Beltrán Rodríguez

JULY 1997 TO JULY 1999
Maintenance of Traffic Plan for the Bayamón Alignment Section Contract-Tren Urbano (project AC-500007), for ICA-Miramar Metro San Juan, Corp.

MAY 1999
Expert witness report of an accident occurred on June 21, 1998 in the junction of the State Highway PR-823 and the access to the construction project of the future PR-142, Quebrada Arenas ward, Toa Alta, Puerto Rico, for Mr. Ismael Díaz Santos

MAY 1999
Expert witness report of an accident occurred on May 27, 1996 in the State Highways PR-335, Km. 11.0, Indios ward, Guayanilla, Puerto Rico, for Mr. Frank A. Quiñones Martínez

MAY 1999
Pavement design for projects AC-000136, AC-100202 and AC-100203 (Mejoras a la PR-1 desde PR-735 a PR-170), Cayey, Puerto Rico, for A.S. Consulting Engineers

MARCH 13, 1999
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

FEBRUARY 1999
Traffic Count to determine the Average Daily Traffic at the State Highway PR-111, Km. 0.4 at Aguadilla, Puerto Rico, for Fidel De Frías, M.D.

NOVEMBER 1998
Expert witness report of an accident occurred on June 19, 1994 in the junction of the state highways PR-2 and PR-114, Sábalos ward, Mayagüez, Puerto Rico, for Eng. Pablo Crespo Pérez

OCTOBER 1998
Expert witness report of an accident occurred on December 6, 1997 in state highway PR-307, Km. 8.9, Boquerón ward, Cabo Rojo, Puerto Rico, for Mrs. Anabelle Rodríguez Sepúlveda

OCTOBER 1998
Report with the results of an evaluation about the proposed alternatives for the project AC-030108 (Conector PR-100 Intersección PR-101 a PR-301), Boquerón ward, Cabo Rojo, Puerto Rico, for Dr. Ricardo Ramírez Ramírez

OCTOBER 17, 1998
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

MAY 1998
Traffic Study to analize the impact of Isla Azul housing development, proposed at the State Highway PR-472; km. 2.7, Bejucos ward, Isabela, Puerto Rico, for Mr. Luis A. Pérez Villanueva

MAY 1998
Traffic Impact Study and preliminar design of access for the Parque de Diversiones de Luquillo (including depositions at public audiences), proposed at the State Highway PR-3, kms. 33.0 to 35.0, Mata de Plátano ward, Luquillo, Puerto Rico, for Fernando L. Rodríguez, P.E. & Associates

MARCH 1998
Evaluation of the Haul Route used by Betteroads Asphalt Corp. when operates the extraction area at Cercado and Cidra wards, Añasco, Puerto Rico, for Betteroads Asphalt Corp.

FEBRUARY 28, 1998
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

JANUARY 1998
Traffic Study to determine the impact of Garden State Commercial Center at the junction of the State Highways PR-100 and PR-311, Miradero and Guanajibo wards, Cabo Rojo, Puerto Rico, for Mr. Guisseppe Marino Damiani

DECEMBER 1997
Traffic Study to determine the impact of an extension of S.E.S.O. school at the junction of their access and the Cuba road, Miradero ward, Mayagüez, Puerto Rico, for Southwestern Educational Society

NOVEMBER 1997
Rigid Pavement Design for Project AC-006608 (State Highway PR-66, Carolina - Canóvanas, Puerto Rico), for Behar-Ybarra Associates

OCTOBER 4, 1997
Instructor - Review for the Professional Civil Engineer Exam, sponsored by the Institute of Engineers and Surveyors of Puerto Rico

AUGUST 1997
Traffic Count to determine the Average Daily Traffic at the State Highway PR-128, Km. 2.0, Yauco, Puerto Rico, for PCO Associates

APRIL 1997
Traffic Study to determine the Average Daily Traffic at the junction of the State Highways PR-102 and PR-308, La Garita Sector, Miradero ward, Cabo Rojo, Puerto Rico, for Chaibén J. Fas Alzamora, Esq.

AUGUST 1996
Quality tests of the asphalt concrete used on the parking of the Municipal Drop-Off Center, for the Municipality of Las Marías

JULY 1996
Traffic Study (written report and comparison to public hearings) to determine the Average Daily Traffic at the State Highway PR-3, Km. 16.3, Canóvanas, Puerto Rico, for Moravarcy Engineering Group.

JULY 1996
Traffic Study to determine the Average Daily Traffic at the State Highway PR-128, Km. 2.0, Yauco, Puerto Rico, for Moravarcy Engineering Group

MAY 1996
Pavement Design for the project AC-053603, Rehabilitation of State Highway PR-536, Santa Isabel, Puerto Rico, for A.S. Consulting Engineers

MARCH 1996
Origin and Destiny Study for the preliminary design of the future interchange of the State Highways PR-148 and PR-828, Bayamón and Toa Alta, Puerto Rico, for CSA Architects and Engineers

FEBRUARY 1996
Traffic Study to determine the Average Daily Traffic at the State Highway PR-110, Km. 9.7, Moca, Puerto Rico, for Moravarcy Engineering Group

NOVEMBER 1995
Traffic Study to determine the Average Daily Traffic at the junction of the State Highways PR-2 and PR-402, Añasco, Puerto Rico, for Moravarcy Engineering Group

AUGUST 1995
Pavement Design for preliminary design of State Highway PR-66 (Río Piedras-Loiza), for Lebrón Associates

JULY 1995
Pavement design for project AC-001732 (Mejoras inmediatas a carretera PR-17), Jesús T. Piñero Avenue, Hato Rey, Puerto Rico, for Lebrón Associates

JULY 1995
Pavement design for Piers N 1/2 & O. San Juan, San Juan, Puerto Rico, for EBP Design Group Consulting Engineers

MARCH 1995
Traffic Study to determine the Average Daily Traffic at the State Highway PR-111, Km. 4.0, Moca, Puerto Rico, for Moravar Engineering Group

MARCH 1995
Traffic Study to determine the Average Daily Traffic at the State Highway PR-420, Km. 5.2, Moca, Puerto Rico, for Moravar Engineering Group

FEBRUARY 1995
Traffic Study to determine the Average Daily Traffic at the State Highway PR-115, Km. 26.6, Aguada, Puerto Rico, for Moravar Engineering Group

SEPTEMBER 1994
Structural evaluation of the streets of Montesol Housing Development for the Municipal Legislature of Cabo Rojo